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Coastal protection with respect to climate change. Practical application on the Belgian coast

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Frequently used parameters

C_0	[kg/m³]	sediment concentration
d	[m]	water depth
D_{50}	[mm]	median grain size
f_p	[s]	peak frequency
F_G	[N]	gravity force
F_D	[N]	drag force
F	[m/s]	wave flux
g	[m/s²]	fall velocity (= 9.81 m/s²)
H_s	[m]	significant wave height
K	[m/s]	hydraulic conductivity of the bed material
L	[m]	wave length
m_b	[-]	beach slope where the waves break
P	[-]	Rouse number
Q_{lst}	[m³/s]	longshore transport rate
R	[m]	hydraulic radius
S_0	[°]	slope
T	[s]	wave period
T_p	[s]	peak period
$\overline{U}$	[m/s]	mean flow velocity
V	[m/s]	longshore current velocity
W	[J]	amount of work
z	[m]	distance above the bed
β_c	[-]	Shield's parameter
γ'	[kg/m³]	density
γ_b	[-]	breaker index (= H_b/h_b)
θ_b	[°]	wave angle at breaking
κ	[-]	Karman constant
ξ_m	[kg/(s*m)]	kinematic eddy viscosity
ρ	[kg/m³]	density
τ_0	[N/mm]	boundary stress
τ_c	[N/mm]	critical boundary stress
υ	[kg/(s*m)]	kinematic viscosity
ω_s	[m/s]	particle fall velocity

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Abstract

With 60% of the world's population inhabited in close proximity to the shoreline and the coast responsible for a big part of the world's economy and tourisms, it is of the essence to protect this coast no matter what happens. Therefore it's very valuable to know with which processes this coast is associated, leading to a lot of previous research regarding the matter. Not only research on what goes on at the coast, but also on how to stop these processes from happening offering different solutions. These solutions consist of hard measures or soft measures, with the latter having less of an impact on the environment.

In recent centuries, the climate has been changing leading to an increase of storm surges and a rise of the average sea water level. This brings about different complications to consider while designing coastal protection mechanisms. Due to this phenomenon a lot of countries have started a masterplan to counteract the increased erosion risk caused by climate change. In Belgium this meant the development of the integrated coastal safety plan, which has to make sure the Belgian coastline can endure a 1000-year storm. To make this happen, there has been a global investigation along the coastline to point out the weak points, which were in need of reinforcement.

This thesis describes one of such weak spots along the Belgian shoreline, being Wenduine, a small coastal town with a sea wall that the integrated coastal safety plan deemed insufficient. A practical application of the previous literature was conducted to check for possibilities for the reinforcement of this area. Followed by an economic analysis.

Resumen

Con el 60 % de la población mundial habitando en la cercanías de las líneas costeras y siendo la costa responsable de gran parte de la economía mundial y del turismo, es esencial proteger las costas, paso lo que pase. Por lo tanto, es de gran valor saber con que procesos se asocian la costa, llevándonos a antiguas investigaciones sobre la materia. No solo investigaciones sobre lo que sucede en la costa si no también como parar estos procesos y ofreciendo diferentes soluciones. Estas soluciones consisten en medidas de mayor o menor calibre, teniendo la última mencionada menor impacto medioambiental.

En los últimos tiempos, el clima ha ido cambiando llevándonos a uno aumento repentino de las tormentas y de nivel medio del mar. Esto causa diferentes complicaciones a considerar, teniendo un encuentra el proceso de destino del mecanismo de la protección de costas. A causa de este fenómeno muchos países han empezado un plan para contrarrestar el aumento del riesgo de la erosión causada por el cambio climático. En Bélgica, esto supone el desarrollo de un plan integrado en la seguridad de las costas, el cual tiene que asegurar una perduración de las costas de Bélgica más de 1000 anos. Para que esto suceda, ha habido una investigación global por toda la costa para señalar los puntos débiles, los cuales estaba necesitaban un refuerzo.

Esta tesis describe uno de los puntos más débiles a lo largo de la costa belga, siendo Wenduine, una pequeña ciudad costense con un dique de mar, el cual el plan integrado de la seguridad de las costas se consideró insuficiente. Una utilidad practica de la anterior literatura fue dirigida a la comprobación de la posibilidad de refuerzo del área. Seguido de un análisis economico.

1 Introduction

Coastal ocean zones are very important ecosystems with a very high productivity; dense population, exploitation of renewable and non-renewable energy sources, development of industries and spurts in recreational activities. This coastal zone consists of an area from 200m below the sea level (also known as the continental shelf) and up to 200m above the sea level on the landward side. There are a couple of different reasons to why this coastal zone is so important. A first of these reasons is that the coastal zone inhabits nearly 60% of the world’s population in a stretch of 60 km from the coast and most of the biggest cities in the world are situated near the coastline. Secondly 95% of the world’s fish-catch is originating from coastal zones, there are whole countries who are basing their economies on these fish-catches. So, coastal zones are not only very ecosystems but also have a very important economic significance for some countries as coastal zones could be used for fishing, tourism, ports, oil and gas extraction, mining of minerals and industrial development. Therefore, it’s very important to handle these zones, as it is they are the focus of expansion and diversification, with the utmost care and dedication. Proper management of the coastal zone is becoming more and more important in the present day

Knowledge of coastal processes is very important in deciding on how to think of solutions to try to preserve the coastal zones and hereby keeping intact the different functions the coastal zones offer, like fishing, navigation, transport and dispersal of pollutants, microclimate moderation, etc. Since there are so many different uses of the coastal zones it’s important to consider all these uses and where necessary conduct different researches. Apart from the human use of the shores, they are most of the times enriched with mineral deposits, that’s why protection of the shore and coastal resources are very important issues.

Proper management for the coast and its adjacent areas is only possible with adequate knowledge about the base principles and processes out of which the coast consists. These topics are coastal resources, coastal landforms, sediment transport and coastal erosion. Which are clarified in what follows.

- *Coastal resources*
  Coastal regions inhabit a large part of marine life, its lagoons, mangroves, coral reefs and shallow bays serve as shelter many oceanic species. Also, a lot of the world’s precious minerals are found along the shorelines. Fluviatile processes and sea level changes play a major part in the sedimentation of placer mineral deposits along coastlines and offshore regions. The process of forming placer mineral deposits takes a very long time, they’re derived in very low concentrations from igneous and metamorphic rocks whereas they get formed by gravity separation during sedimentary processes. Placer materials must be both dense and resistant to weathering processes. These materials contain a lot of valuable minerals like gold, tin, uranium, titanium, etc. That’s why places that contain big placer deposits get mined frequently and that could affect the coastal zone greatly. If placers in those regions get mined more than the natural supply of these deposits, then the marine environment and ecosystem could get harmed. Excessive mining of these minerals could also severely alter the coastal habitats of the oceanic species making it impossible for the species to survive. Therefore, the public mindset must change in exploring these deposits and strict

regulations are to be imposed to balance the environment degradation and sustenance.

- *Coastal landforms*
  Quantitative and qualitative studies have been conducted on the evolution of the coastal landforms using topo maps, satellite images and field data. From these data, it was found that there are some reoccurring patterns in the formation of new coastal landforms. Most of the known coastal landforms issue from longshore and cross-shore sediment transportation, which make shorelines very dynamic. Because it tends to evolve during time, it gets smaller due to erosion or gets bigger due to accretion. The dynamic behavior of these shorelines is very impractical for human developments as they want to build their buildings close to the shoreline. That's why the shoreline must be kept in the same place for a long time, coastal defense mechanisms are developed to make coasts more static.
- *Sediment transport and maintenance dredging of ports*
  The navigation in ports is function of the deepness of the navigation channels. These channels could become less deep due to siltation or across the entrance there could be formed a bar formation which would eventually block the entrance. To avoid these problems there must be a continuous dredging along these channels and entrances to maintain the minimum navigation depth. This maintenance makes up a major portion of port operation, it involves a big expenditure, hence it is necessary to fully understand the sediment transport to port planners.
  The amount of sediment transport is function of the different seasons, for example during rain seasons the rivers will discharge a lot more to the sea than usual and that's why the sediment transport will be a lot higher than normal. What further amplifies this is the storm induced turbulence in the water.
  Tracking of such sediment transport happens by radiotracer experiments, where radioactive ^{46}SC in the form of scandium glass powder was used as the tracer of the bed sediment.
- *Coastal erosion and its management*
  Depending on the season beaches erode or grow, so it's only normal that people want to stop this process to get a beach that has a more constant size. Although it must be stated that human interference is not something that could be realized over a fortnight. All different kinds of management decisions must be considered, for example if it's decided to build seawalls it must be made sure that these walls don't enhance the erosion by reducing the beach width, steepening the offshore gradient or increasing the wave heights. Also, construction of seawalls could lead to a shift of the erosion site to adjacent non-engineered areas or in the worst case the walls could even be destructed.
  Coastal protection structures are not only built to avoid erosion on beaches or other coastal landforms, but must also preserve sand dunes, mangrove vegetation, casuarina and other salt tolerant species as part of coastal zone management. That means that sand mining in the coastal regulation zone must be regulated so that the consequences of these actions don't have any negative results for the coasts.

Naturally there are different ways to preserve/protect the coast, where some of the most important (habitation and industrial activities) cities of the world are located, against the natural forces. Coastal structures as seawalls, groynes and artificial nourishment serve this purpose. One of the most important and most used marine/coastal structures are the breakwaters, these are constructed worldwide to protect the beaches and harbors, stabilize coastal inlets against high seas and silting. They reflect wave energy, break the high waves and dissipate the energy of wind generated waves as well as preventing longshore drift. Of course, there are not only advantages connected to the use of breakwaters, there are also a lot of negative points. The most important critics of breakwaters point out their massive size, high cost of construction, blocking of the view which downgrades the tourism potential of the area, prevention of onshore and offshore movement of currents, environmental destruction and repeated failure and unsatisfactory performance over all.

These grounds are the main topics on which coastal management is based. In recent decades, the whole process of coastal preservation has become more complicated, due to the undeniable global climate change. Climate change has led to global warming which initiated rise of the mean sea level and an increase of storm surges. Coastal measures that were deemed good enough to preserve coast from weathering due to erosion and winds, have become insufficient. Coastal defenses must be adapted to the consequences of climate change.

Consequences of climate change include an increase of the sea level and an increase in the occurrence of heavy storms. This could allow for big quantities of water to skip which could lead to the instability of dikes and flooding in the coastal and inland area. The economic, structural and human damage is incalculable. From this point of view there has been conducted a lot of research to heighten the safety level in coastal zone regions. Everywhere around the globe governments had to decide on the measures that had to be taken. More specifically to Belgium, this meant a development of an integrated coastal safety plan to protect the coastal area against the 1000-year storm. A big investigation was conducted to check whether the Belgian shoreline and its defence mechanisms would be able to withstand a 1000-year storm, weak points were highlighted and further examined on how to make them sufficient in terms of defence.

2 Coastal landforms

2.1 Erosional coasts

High wave energy and lack of sediment available for deposition are two causes responsible for erosional coastal landforms. In such places sediment is primarily removed from the coasts, which leads to narrow beaches that are mostly characterized by rocky shorelines that can resist the high-energy waves.

2.1.1 Sea cliffs

Considering coastal landforms, sea cliffs are the main outcome following coastal erosion. Shaped by erosion caused by wave energy and weather influences, they form very steep cliffs with heights that can range from several meters to hundreds of meters above the sea level. Cliffs that are in direct contact with the sea will suffer erosion caused by wave energy, the sea will 'attack' the cliff between the low tide height and the high tide height. This will cause the material in this place to weaken and some cracks will appear which will later to develop into a wave cut notch (figure 1).

1 Wave-cut notch

Wave-cut notches lead to the mass above those notches to become unsupported because the mass is completely undercut by the waves. From the moment when the gravitational force of that unsupported landmass becomes greater than the shear strength of the material, the landmass will collapse causing the cliff to retreat. This process will continue to repeat itself and will from a wave-cut platform only visible during the low tide. (figure 2)

2 Wave-cut platform

Benumof et al. investigated the effect of the incident wave energy on the cliff erosion and found that the material out of which the sea cliffs are composed is the dominant influence on sea cliff erosion rates and the resulting landforms produced. In a real sense, the collective findings suggest that while waves are a primary control on the timing of sea cliff erosion, material strength largely determines whether sea cliffs will be stable or, if they retreat, the rate and manner of their erosion. (Benumof et al., 2000)

2.1.2 Headlands and bays

A coastline, in direct contact with the sea, can be built up out of different materials with different hardnesses which will lead to a different rate of erosion on the different materials where the softer materials will erode more than the harder materials. The harder materials are left to stick out forming the headlands after which these will be more vulnerable to erosion as the wave energy will concentrate itself on the headlands. Formation of the headlands automatically forms a bay area that's sheltered by the headlands (figure 3). The bay areas are formed from the softer materials which have eroded faster than the headlands. Eventually this natural process will reach an equilibrium either static or dynamic. Static equilibrium to a headland-bay beach is a state when the predominant waves are breaking simultaneously around the whole bay periphery, hence littoral drift is almost non-existent and external sediment is not required to maintain its long-term stability. Whereas with a headland-bay beach under dynamic equilibrium, sediment supply from updrift and/or a source within the embayment is required to maintain its stability; otherwise shoreline would retreat as supply reduces. Should supply diminish, then a HBB in dynamic equilibrium could recede toward the limit defined by the static equilibrium under the same wave condition

3 headland bay formation

It is found that under the influence of a predominant swell, the headland-bay formation's curved periphery in natural environment may reach static equilibrium and remains stable without sediment supply from updrift and/or a riverain source within its own embayment. Which leads to the belief that this natural form could be the ideal solution to counteract coastal erosion. Scientist have researched this topic a lot and have attempted to describe the ideal bay shape by use of mathematical expressions, eventually a parabolic bay shape expression (PBSE) was found and is still used to determine the static equilibrium planform

(SEP) of different bays. This equilibrium can be reconstructed by use of manmade structures. (Hsu, Yu, Lee, & Benedet, 2010)

2.1.3 Sea stacks and arches

Rocky shorelines have a different resistance towards erosion which will lead to different landforms caused by erosion. As mentioned before sea cliffs are formed and regress slowly because of erosion at the toe of these cliffs leaving behind a wave-cut platform. This would go in a completely straight line if the coastline was made up of the same material and if it would undergo the same wave forces. This is nearly never the case, mostly rocks have some wake spots which will be exploited by the sea waves. Waves will form tiny cracks into the weak pieces of rock which will leave a tiny space into the rock, each time a wave crashes into the cliffs face the air in that tiny space will become compressed and will explode outwards, breaking of bits of rock. That process of the compressed air is called hydraulic action.

Everything is initiated by a sea cliff that can develop into a headland-bay area or a slowly regressing sea cliff. A headland-bay area still suffers from erosion which is concentrated onto the headland. Waves could force their way into the face of the headland using hydraulic action and break through the other side leaving an entire whole through the headland forming an arch (figure 4). Arch forms will eventually collapse leaving the headland on one side and a pillar (or stack) on the other, waves will continue to attack this stack at its base after which it will collapse as well leaving no trace of its existence behind.

4 sea arches and stacks

2.2 Depositional coasts

Contrary to erosional coasts, depositional coast does have an abundance of sediment supply which causes the coasts to widen despite the wave energy or ocean currents. New coastal landforms are developed due to deposition of sediment.

2.2.1 Deltas

Deltas develop through rivers that end in a sea or ocean, as an ocean or sea consist of a large slow moving water mass the river must slow down therefore losing its energy and competence. Resulting in the deposition of the different sand particles (sediment) to accumulate at the transition of the river and the ocean where the biggest and heaviest particles drop first and the finest further away creating a layer system. Coastal processes in the sea or ocean are unable to carry away the deposited sediment leading to a formation of a delta (figure 5).

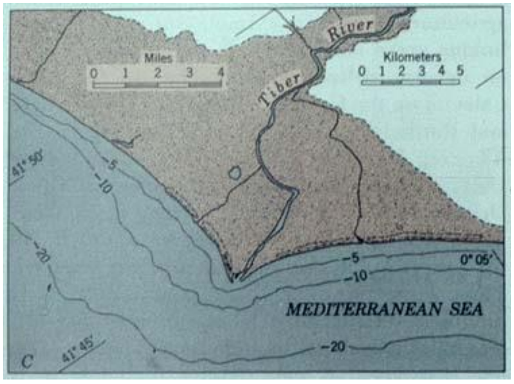
5 Delta formation

When a fresh water rivers mouths in a salt water sea or ocean an electric charge is produced, which causes clay particles to collect together forming bigger and heavier particles. The increased weight of the particles causes the particles to settle on the seabed. That process is called flocculation and leads eventually to the development of pieces of new land due to the accumulation of all the clay particles. These pieces of land are divided by multiple channels forming different distributaries. A maze of distributaries and active and inactive channels is formed.

The size of a delta is typically related to the size of the river, specifically to its discharge. The shape of a delta, on the other hand, is a result of the interaction of the river with tidal and wave processes along the coast. A classification utilizing each of these three factors as end members provides a good way of considering the variation in delta morphology. River-dominated deltas are those where both wave and tidal current energy on the coast is low and the discharge of water and sediment are little affected by them. The result is an irregularly shaped delta with numerous digitate distributaries. An example of a river-dominated delta is the Mississippi delta, also referred to as a bird's foot. (figure 6)

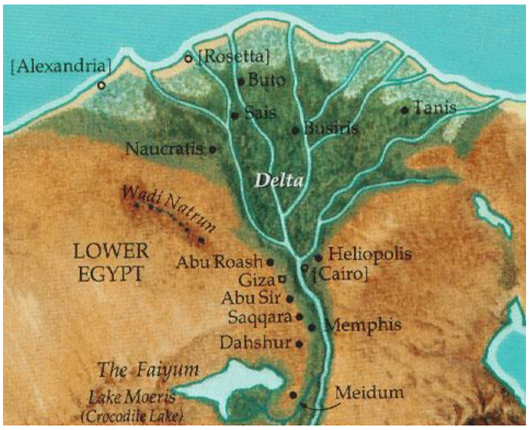
6 Mississippi Delta

Waves may remove much of the fine deltaic sediment and smooth the outer margin of the delta landform as well. This results in a smooth, cuspate delta that has few distributaries. Some wave-dominated deltas are strongly affected by longshore currents, and the river mouth is diverted markedly along the coast. An example is the Tiber delta (figure 7).



7 Tiber delta

Tide-dominated deltas tend to be developed in wide, funnel-shaped configurations with long sand bodies that fan out from the coast. These sand bodies are oriented with the strong tidal currents of the delta. Tidal flats and salt marshes also are common. They're also referred to as arcuate delta's and have rounded, convex outer margins. An example is the Nile delta (figure 8).



8 Nile delta

River delta's offer the surrounded land with very fertile ground and very good offshore fishing areas. But they're also associated with a higher flooding risk due to the delta's flatness. As they are known as very dynamic and unstable landforms, they're subject to channel migration as well as subsidence by the sea.

2.2.2 Barrier island/estuarine systems

Barrier islands are coastal landforms that are developed offshore parallel to the coastline due to tidal or wave action. They could consist of a singular island or could consists of an island chain made up out of multiple islands stretching over several kilometres. The area enclosed by these barrier islands are called estuarine systems which consist of embayments fed by streams. Estuarine systems receive a lot of sediments due to runoff from adjacent coastal plains.

Barrier islands (figure 9) are critically important in mitigating ocean swells and other storm events for the water systems on the mainland side of the barrier island, as well as protecting the coastline. This effectively creates a unique environment of relatively low energy, brackish water. Multiple wetland systems such as lagoons, estuaries, and/or marshes can result from such conditions depending on the surroundings. They are typically rich habitats for a variety of flora and fauna. Without barrier islands, these wetlands could not exist; they would be destroyed by daily ocean waves and tides as well as ocean storm events.

9 barrier islands in the Gulf of Mexico

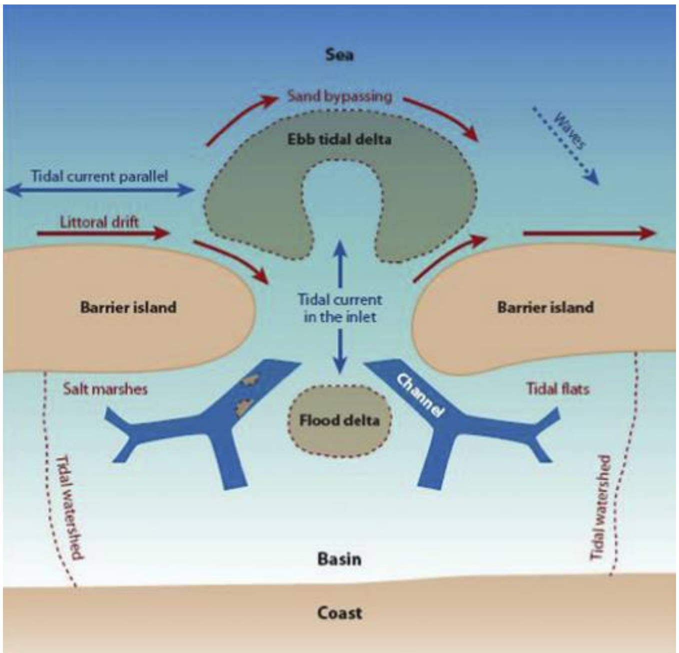
Barrier islands are thus a natural way of protecting the estuarines, lagoons and hinterland that's located on the landwards side of the barriers. They form a natural blockade which serve as a natural defence against wave action, swells, storm surges, and coastal storms and thereby protect thousands of people who live nearby as well as the fauna and flora in the embayments.

A good example is the Wadden Sea (figure 10) in the Netherlands, which consists of a lot of barrier islands and is subject to highly dynamic behaviour. The Wadden Sea is a coastal wetland of exceptional size, great beauty and richness in unique natural assets but is endangered by coastal process; of major concern is the movement of water and air and the transport, erosion and deposition of sand and mud. These processes result in an ever-changing morphology (topography/bathymetry) of the islands, tidal channels, inter-tidal shoals and tidal flats. This dynamic development of the shape and nature of the Wadden area forms together with the biotic systems, the present Wadden system. The morphodynamic development of the Wadden Sea is influenced by changing environmental conditions e.g. sea-level rise as well as by human interferences. For the management and protection of the Wadden system knowledge on the morphodynamic development is essential.

10 Wadden Sea

The mentioned morphodynamics consist of sediment erosion and stability of the tidal inlets. Concerning sediment erosion there has been noticed a trend over the years that this is becoming increasingly worse which leads to an increase of the beach nourishment. Use of soft engineering methods like beach nourishment is mandatory as in 1990 the concept of Basal Coastline ("Basis Kust Lijn" - BKL) was introduced in the Netherlands and the Dutch coast must be maintained by law. Local erosion and losses of sediment must be compensated by beach- and shore-face nourishment. As far as possible, "hard" engineering solutions such as seawalls and dikes must be avoided to combat coastal erosion.

The frequency and amount of beach nourishment depends on the rate of erosion. Which has increased over the past couple of years, during the period 1991-2000 6.5 million m^3 sand a year was added, this increased to 12 million m^3 for the period 2000-2010. If sea-level rise accelerates, and if the sediment loss from the coastal system is considered, even more sand must be nourished in future to keep the barrier islands intact.



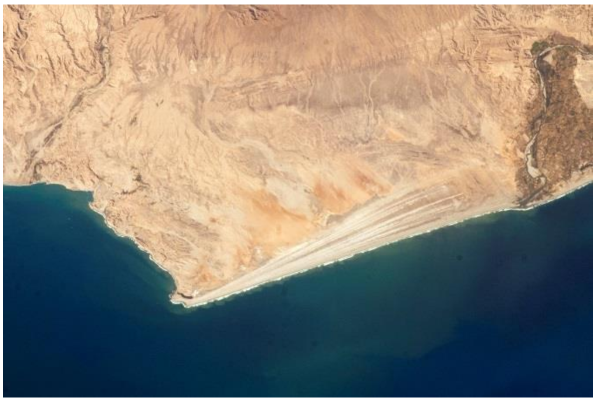
11 Tidal inlet scheme

Barrier islands are split by tidal inlets (figure 11). They're a consequence of water that goes over the barrier islands due to heavy storm, the water that has breached must be flooded back to the ocean/sea. This goes through the formation of a tidal inlet which is formed as follows: flood and ebb tidal deltas are deposited on the shoreward and oceanward side of an inlet (mesotidal/mixed energy). Channels occupied by the ebb and flood tidal flows are shown by light and dark blue arrows respectively. The flood ramp throat and main ebb channel form the inlet throat, the deepest portion of the channel. Sand moved into the inlet by waves and flood currents are either deposited on the flood tidal delta or circulated back out to the ebb tidal delta. Waves reorganize sediment in a series of swash bars which eventually migrate and attach to the ends of the barrier. Typically, the down drift end receives the most sediment.

The stability of a tidal inlet is determined by two competing processes: the tidal current which keeps it open, and the wind waves and associated littoral drift trying to close the inlet. Escoffier (1940) shows that a tidal inlet has a stable equilibrium if the tidal current (determined by the combination of tidal range and size of the basin) is sufficiently strong compared to the waves. The Wadden inlets belong to mixed energy tide-dominated inlets as per the classification of Hayes (1979, see Steijn, 1991), which apparently satisfy the stability condition. Extension of the analysis to system of several coupled inlets (Van de Kreeke, 1990; Tambroni and Seminara, 2006; Van de Kreeke et al., 2008) suggests that such a system tends to evolve into a single tidal inlet system, unless the interaction between the inlets themselves is weak. So, this suggests that a system like the Wadden Sea can only exist because the flow and transport across the tidal watersheds are limited. Salles et al. (2005) though claim that for the stability of a multiple tidal inlets system the nonlinear processes not included in the above stability analyses, are important. (Wang et al., 2012)

2.2.3 Strand-plain coasts

Strand-plain coasts (figure 12) are like barrier island formations without the presence of tidal inlets and thus tidal channels, lagoons and marshes. So, it is a barrier island system where the barrier island is directly attached to the shoreline, it defines itself by a broad belt of sand along the shoreline. Strand plains typically are created by the redistribution by waves and longshore currents of coarse sediment on either side of a river mouth so it's no completely flat and can contain beaches and dunes. Thus, they are part of one type of wave-dominated delta.

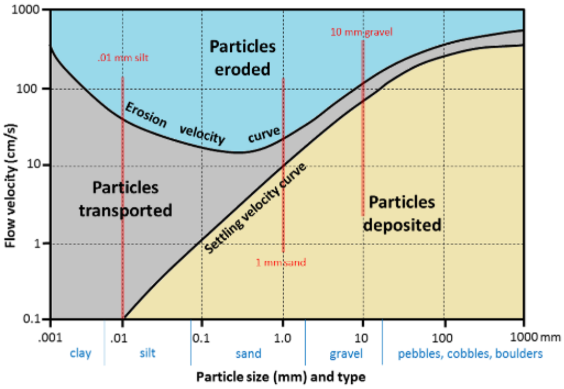


12 Strand plain, Peru

3 Sediment transport

Every coastal landform is managed by sediment transport, which means the transport solid particles through currents. The particles are only influenced by two forces being the gravity forces and current flows. Sediment that's resting somewhere can be picked up by strong currents and deposited somewhere downdrift forming new coastal landforms due to gravity forces. Sediment transport is the reason why erosion or accretion will happen to a certain landform. As these particles could be deposited somewhere and cause accretion of a certain landform, or the particles could be eroded away from an existing landform and be transported elsewhere through fluid currents.

Deposition and erosion all depends on the velocity of the current and the grain size. Hjulström investigated this matter and came up with a graph which describes the velocities and grain sizes at which to particles would erode, transport or deposit. Later this graph was refined by Ake Sundborg (figure 13)



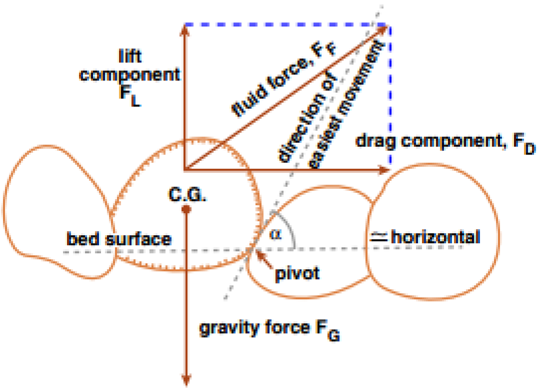
13 Hjulström-Sundborg diagram

In recent days, this curve has lost its academic value as it is physically incorrect, as an object (here the particles) can only be set in movement when an active force acts upon it. Whereas the graph states that the particles are set in movement by velocities which is incorrect as per the principles of Sir Isaac Newton. A force is the product of the mass of an object and its velocity.

A much better way to think about sediment transport is to think about the different stresses we know. These stresses are tensile, compressive and shear stress. It is shear stress that is rather different from the other two, as it deforms the object like a stack of papers that slide over each other do. The flow of the river acts upon the grain in a similar way causing the grain to move eventually. If the shear stress that acts upon the particles is greater than its critical shearing stress values, the particles will start to move.

Particles on the stream bed undergo multiple forces such as contact forces and gravity forces. And then there is the fluid force, coming from the fluid's velocity, which can be split up in its components being drag force and lift force. These forces depend a lot on the Reynolds number which determines if the fluid's current is laminar (Re is low) or if the current is turbulent (Re is high). If Re is low, the lift force is unimportant and the fluid force is almost entirely made up through the drag force. If Re is high, the lift force is the same as the drag component. Particles begin to move on the bed when the combined lift and drag forces produced by the fluid become large enough to counteract the gravity and frictional forces that hold the particle in place.

To describe this threshold of movement with a mathematical equitation, a single particle and its acting forces must be considered. Therefore, a lot of simplifications must be made such as only considering the drag forces, an average particle on an average slope subject to an average fluid force, friction prevents sliding of the particle past another and the particle can only pivot around an axis normal to the fluid's flow direction. The condition for the beginning of motion then is that the moments tending to rotate the particle downstream are just balanced by the moments (in the opposite sense) that tend to hold the grain in place. The forces that cause these moments are described in figure 14.



14 Particle on a stream bed with the resulting forces

A horizontal slope is considered which allows to only consider the drag force. The resulting fluid-force moments are:

$$a_1 F_G sin \propto = a_2 F_D cos \propto$$

It is an equitation where the moment caused by gravity, which holds the particle in place or wants to rotate the particle upstream, is in balance with the moment caused by the drag component, which want to rotate the particle downstream.

F_G can also be written as:

$$F_G = c_1 D^3 \gamma'$$

with c_1 being a factor that considers the shape of the particle. F_D is equal to the average boundary stress τ_0 times the surface.

$$F_D = c_2 D^2 \tau_0$$

where the coefficient c_2 considers not only the geometry and packing of the grains (which determines the "area of the grain") but also the variation of the drag coefficient. Substituting these two equations in the main one and writing τ_0 as τ_c to consider the critical conditions gives:

$$a_1 c_1 D^3 \gamma' sin \propto = a_2 c_2 D^2 \tau_C cos \propto$$

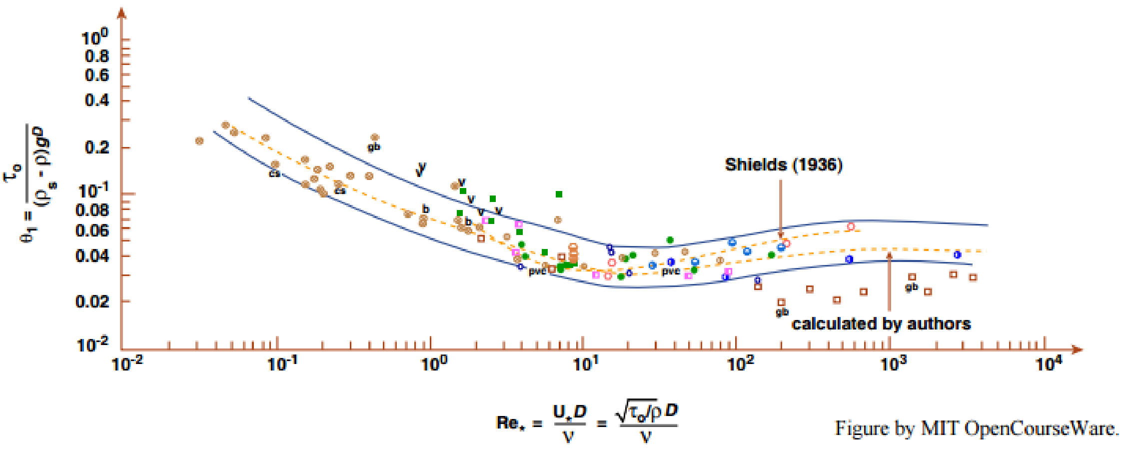
Solving for τ_C is:

$$\tau_C = \frac{a_1 c_1}{a_2 c_2} \gamma' D \tan \propto$$

Dividing both sides by γ’D:

$$\beta_C = \frac{\tau_C}{\gamma' D} = \frac{a_1 c_1}{a_2 c_2} \tan \propto$$

which is the Shield’s parameter, who put this parameter in a graph together with the Reynolds number. It shows the initiation of movement for different materials on a plane stream bed. Later his graphs were updated by Miller et al. (1977). (figure 15) (Southard, 2006)



15 Shields diagram modified by Millers et al. (1977)

From the moment when the particles are in motion they’re forming a sediment load inside the flow. This sediment load can be divided in bed load and suspended load, with the bed load keeping contact with the streaming bed and suspended load being in constant suspension. Suspended load is partially made up out of grains that are not found in appreciable quantities in the bed and is called wash load, which are usually finer grains that are derived from other sources than the bed. With respect to channel morphology, bed-material load must be considered as well. It consists of the bed load and suspended load without the wash load, and is very important to the morphology of the streaming bed as it is derived from erosion of the streaming bed, because bed-material load particles are constantly being exchanged with particles in the bed, and because it returns to the bed at the end of a transport event. These terms are explained below.

3.1 Transport mechanism

Sediment can be carried in different ways by the currents, in which manner the transport happens is function of the density and diameter of the particles, and the density and kinematic velocity of the fluid. These different parameters are all combined in the Rouse parameter. By use of the Rouse number the different manners of which the particles are moved can be classified. (table 1)

Transport mode	Rouse number
Movement	< 7.5
Bed load	2.5 – 7.5
Suspended load: 50% suspended	1.2 – 2.5
Suspended load: 100% suspended	0.8 – 1.2
Wash load	< 0.8

1 stream loads in relation to Rouse number

As mentioned in the table there are three different kinds of transport modes, being the bedload, suspended load and the wash load which are all explained more thoroughly in the following paragraphs.

3.1.1 Bedload

Bed load transport is the transport of sediment that stays in contact with the stream bed during the transport. Sediment will then move through rolling, sliding and hopping over the bed's surface, it travels with a velocity that's a fraction of the total fluid flow velocity. Bed load can have a big influence on the morphology of the streaming bed as it will carry away particles of the stream bed and deposit them somewhere downstream, this process can seriously alter the water currents in the ocean/sea. Therefore, it is important to know what the exact erosion rates are to know whether added nourishment is necessary. Current formulae in use for estimating the erosion rates are not yet accurately enough as they must deal with a lot of variables which are still not entirely clear for researchers. Such as bedform migration, sediment supply limitation and grain sorting. Although there are some big differences amongst researches concerning the formulae, there are some similarities such as the expression of the bed load transport rates. Bed load transport rates can be expressed as the excess of dimensionless shear stress (also known as Shield's parameter) raised to some power, to determine the excess of dimensionless shear stress it is deducted by the threshold of movement. Which gives the following dimensionless value:

$$(\boldsymbol{\tau_b} - \boldsymbol{\tau_c})$$

Another way to express bed load rates is giving a ratio of bed shear stress to critical shear stress and is indicated by T_S (or ϕ):

$$T_s = \varphi = \frac{\tau_b}{\tau_c}$$

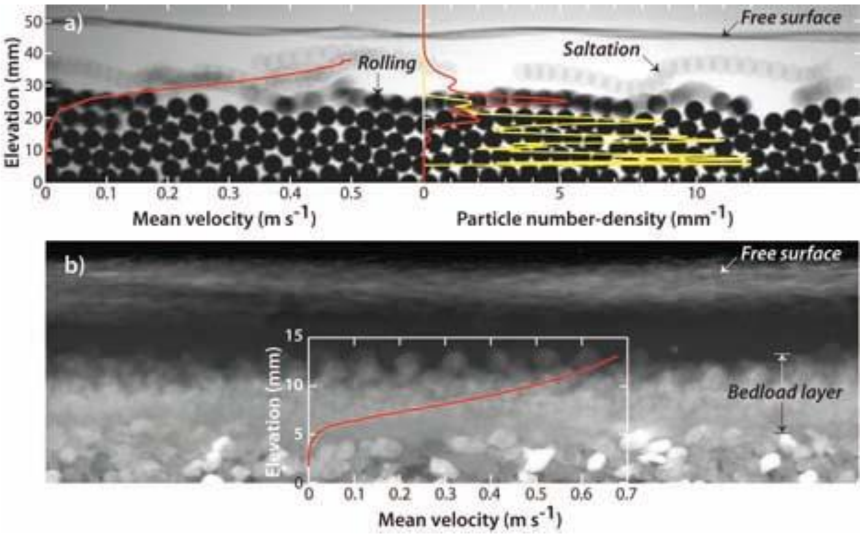
The formulae used nowadays to estimate the bed load flux (Q_S) are based on these transport rates. The bed load flux Q_S is mostly described as the bed load flux per unit channel width q_s. Specifically, q_s is a monotonically increasing nonlinear function of the excess Shields stress $\phi(\tau_b\text{-}\tau_c)$, typically expressed in the form of a power law. Notable authors of formulas are Meyer-Peter Müller, Wilcock and Crowe, Wilcock and Kenworthy and Kuhne.

These formulas consider sediment transport from the perspective of the correlation between fluid driving forces and the responding sediment flux. Yet it is a flow of different particles that are mostly supported by the streaming bed which means the grain-grain interaction should not be underestimated and that the bed load should also be treated as a granular phenomenon.

Treating bed load transport as a granular phenomenon leads to the division of the transport into three stages: (1) finer materials overpassing a locally static bed: the material resident on the bed does not participate in the transport – the mobile material originates upstream; (2) partial transport of local bed material: at any given time, part of the bed remains static, but any grain might eventually move; (3) general motion of the grains on the bed: all grains are equally apt to be moved.

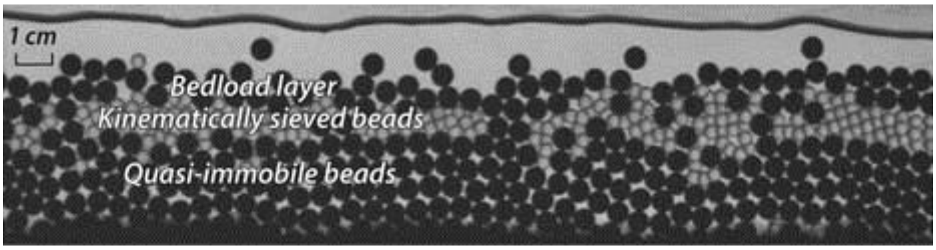
In general, the particles downstream will be smaller than the particles upstream, because the shear stress that must be overcome for small particles to move is much smaller than with bigger particles, since the shear stress required to entrain a grain is linearly proportional to the diameter of the grain. This process is known as the selective transport of sediment, this is however restricted by the hiding effect (Parker & Klingeman, 1982) which means that small particles can be swallowed by voids created by bigger particles where the small particles will be shielded and which will make it more difficult for the smaller particles to move. Division into different stages of the transport process allows for the hiding effect to be considered. With a view on coastal environments where most streaming beds consist of sand, the transport is mostly restricted to phase 3 where large pieces of sand will move as a whole and are called bedload sheets. Smaller sand particles upstream will fill up the voids created by the movement downstream, this is a process called 'kinetic sieving'.

Phase 3 consist of the full mobility bed load, researches have been done to determine the velocities of the different particle layers. The flow was turbulent and supercritical but shear stress was only about twice that necessary to move the grains over a depth of 2–3 diameters. This is more typical of gravel beds in a relatively active stage. The velocity profile exhibits three segments (figure 16a): an exponential tail at the transition between the stationary and the bedload layer; a linear domain; and a logarithmic region due mainly to saltating particles with velocities close to the fluid velocity. The lowest parts (exponential and linear) are like dry granular velocity profiles. The number-density profiles are broken down into moving and quasi-immobile particles (less than 5% of the maximal particle velocity). The moving particle profile shows three peaks, the uppermost corresponding only to saltation and the two lower peaks essentially to rolling beads. Figure 16b shows for comparison a velocity profile calculated with particle image velocimetry in a mono-sized bedload experiment with natural gravel. This velocity profile has again the same shape as in dry granular flows (Fraccarollo & Rosatti, 2009). These measurements further emphasize the interest to consider natural bedload as a granular phenomenon. Velocity and number-density (or concentration) profiles can eventually help to determine a bedload rheology by comparison with theoretically sound results obtained in dry granular flows.



16 Velocities & densities in full mobility bed load

Size segregation is another phenomenon which has been investigated concerning full mobility bed loads. It has been noticed that when smaller particles are introduced upstream of a certain bedload flow, these particles will eventually settle themselves under the top layer of the existing river bed. A very efficient vertical sorting process (figure 17) has been noticed and is the effect of kinetic sieving. Having a layer of finer particles under a layer of bigger particles could lead to bedload sheet mobilization and subsequent patch dynamics similar to that described by (Nelson et al., 2009). (Frey & Church, 2011)



17 Kinetic sieving

3.1.2 Suspended load

Suspended is that part of the total sediment load where the sediment has no contact with the stream bed for a considerable amount of time. As the flow velocity of the stream increases it will initiate the grains on the river bed to move, resulting in the grains to make jumps over the stream bed. These jumps will increase in time and frequency the more the flow velocity increases this until the full weight of the grain is supported by upwardly directed turbulent velocity fluctuations and the material moves within the water column forming the suspended load of the stream. As suspended load has big influence on erosion and deposition of sediment it's important to know how much sediment can be transported.

Similarly to bed load, suspended load gets into the waterbody if the shear stress that the fluids brings to the grains will surpass a certain threshold. This shearing stress can be described as:

$$\tau = (\upsilon + \xi_m)\rho \frac{d\overline{U}}{dz}$$

where τ is the shear stress, υ is the kinematic viscosity of the fluid, ξ_m is the kinematic eddy viscosity or kinematic eddy diffusivity coefficient, ρ is the density of the fluid, $\bar{U}$ is the mean velocity of the flow at a given height above the bed, and z is the distance above the bed. The eddy viscosity is the viscosity of a turbulent flow and is usually much greater than the viscosity of the fluid.

In a turbulent flow, the velocity is made of out of the following components:

$$U = \bar{U} + U'$$

$$V = \bar{V} + V'$$

$$W = \bar{W} + W'$$

with the components being the velocity components in the x, y and z direction respectively. A sediment particle will remain in suspension only if the strongest vertical velocity fluctuations (W') are greater than the particle fall velocity (ω_s).

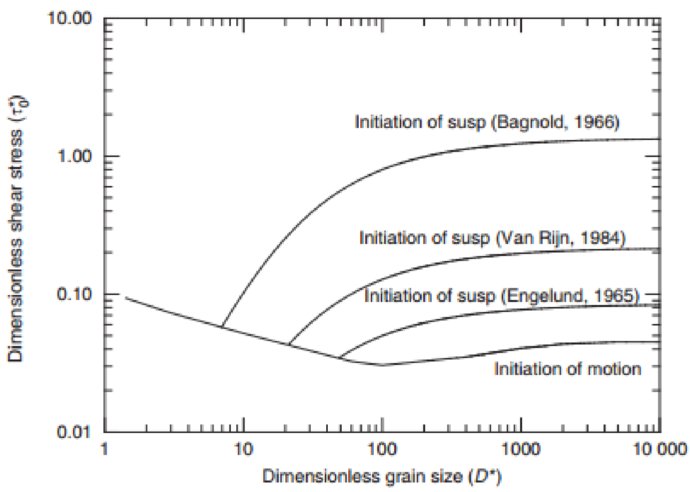
$$\sqrt{\overline{W'^2}} \cong u_* \cong \omega_s$$

where the following formulas are needed:

$$u_* = \sqrt{\frac{\tau_0}{\rho}}$$

$$\tau_0 = \rho g R S_0$$

with τ_0 is the bed shear stress, ρ is the density of the water, g is the acceleration of gravity, R is the hydraulic radius, and S_0 is the slope. This is the criterion of suspension as given by Bagnold, 1966. Other have also given their criterion of suspension with a slight difference (Engelund, 1965; Van Rijn, 1984). (figure 18)



18 Initiation of motion and initiation of suspension

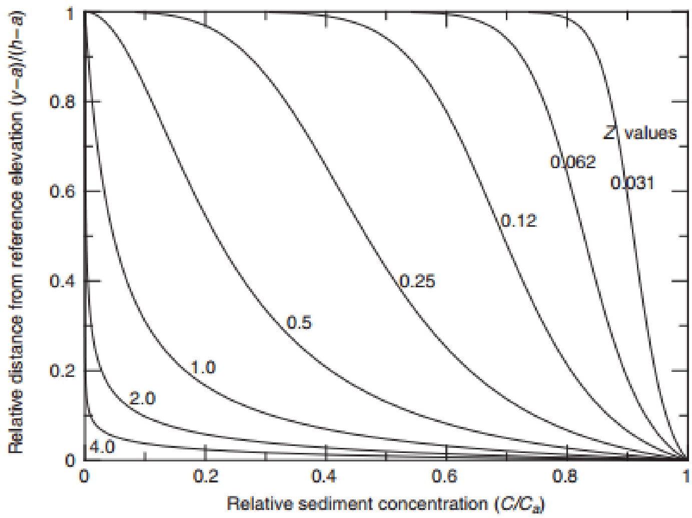
To know the concentration of sediment suspension at a certain point in time, the Rouse profile equation can be used. It uses the assumption that if the flow is assumed to be steady with the time-averaged sediment concentration at any level constant, the net vertical flow of sediment will be zero. In other words, the upward movement of sediment will be balanced by the settling of sediment through the water column. That gives the following equation:

$$\frac{C_s}{C_0} = \left[\frac{z(h - z_0)}{z_0(h - z)}\right]^Z$$

where C_0 is the sediment concentration at elevation z_0, C_s is the sediment concentration at elevation z, h is the flow depth, P is the Rouse number and Z is the following:

$$Z = \frac{\omega_s}{\kappa u_*}$$

where κ is the Karman constant. The term Z is also known as the ratio of the grain-fall velocity to the strength of the flow and is given in figure 19. (Kuhnle, 2013)



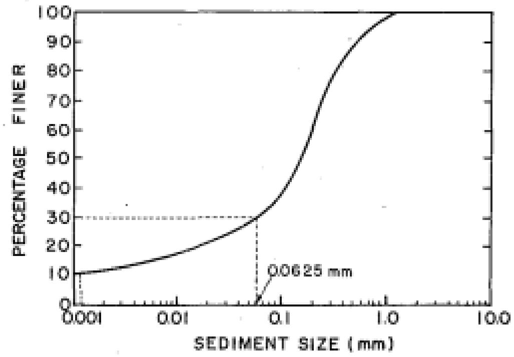
19 Values of Z-term

3.1.3 Wash load

There are different perceptions regarding the definition of wash load, but there are some researches with the same views regarding wash load resulting in three criteria which separate wash load from bed-material load.

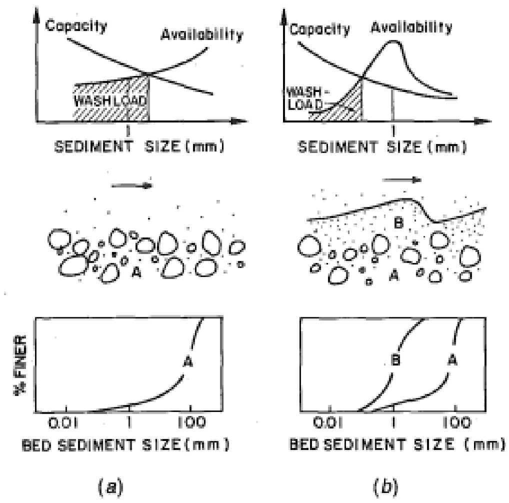
The first criteria that wash load consists of finer particles than the bed load and that these particles aren't derived from the streaming bed. Einstein states that the wash load sediment consists of particles that are within the 10% (D_{10}) finest particles of the bed, this is a relative size which means that the bigger the grains of the bed are, the bigger the grains of the wash load can be. The value of D_{10} is also an approximate value, the main condition that has to be fulfilled is that wash load is not found 'in appreciable quantities' in the bed of the stream.

Secondly wash load particles must be smaller than 0.063mm, although this criterion doesn't always interact with the D_{10} criteria nicely. As it is the case when there is a large concentration of fines among the stream bed, then it could be the case that D_{10} is much smaller than 0.063mm. (figure 20)



20 Large concentration of fines

The third criterion is dependent on the transport capacity of the stream and of the supply curve. Sediment can be in suspension for as long as the supply is smaller than the transport capacity, from the moment where the supply exceeds the transport capacity certain portions of the sediment will deposit on the bed and will act as bed sediment load. This can go on until the transport capacity increases again and the deposited sediment will go into suspension again. This is also visualised in figure 21.

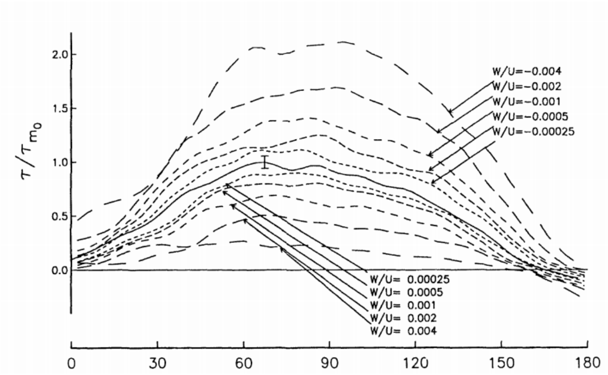


21 Wash load in function of supply and transport capacity

In figure 21a, wash load is in suspension for if the sediment supply is lower than the transport capacity of the stream. In 21b, a sudden increase of sediment availability is shown. At this peak in sediment availability the particles will deposit and the curve will (temporarily) shift from A to B. Temporarily because from the point when the capacity will increase again, these particles will come back into suspension leading for the curve to shift back to A. (Woo, Julien, & Richardson, 1986)

3.1.4 Bed ventilation

It is found that sediment transport can be seriously altered dependent on the level of the water table of the back-beach. This will have its effects on the swash zone, being the zone that consists of the zone between the point where the waves break and the point until where they wash up. The position of the back-beach's water table determines if there will be any groundwater flow because water will always flow to the lowest point. Having an elevated water along the back-beach allows water to flow from the elevated groundwater table to the swash zone, the groundwater will seep through the sea/ocean bed until the bed is fully saturated then the water will dissipate into the water mass of the sea/ocean. This result into two effects which are opposite to each other. (1) The exfiltrating water will pass through the particles from the riverbed to reach the water mass of the sea/ocean. This process creates an upward drag force which decreases the effective particle weight of the surficial sediment. This leads to destabilizing effect at the toe of the beach, the upper portion of the beach-face would accrete faster. (2) That process gets counteracted by another phenomenon caused by the groundwater flow. The groundwater flow also causes the streamlines of the sea/ocean flow to draw away from the bed which causes the bed shear stress to decrease (figure 22). (Conley & Inman, 1994)



22 Average bed shear stress for different values of $\overline{V}$

Conley et al. investigated the influence of infiltration and exfiltration on the average bed shear stress by conducting several experiments on a scale model. All the results are plotted in figure 22. The heavy solid line is the bed stress in the case of no ventilation and serves as the baseline for the other tests. The peak value from this curve has been used to normalize all the data. As can be seen in the figure, ventilation clearly affects the bed stress in oscillatory flow with infiltration ($\overline{V} < 0$) leading to increasing bed stress and exfiltration ($\overline{V} > 0$) causing reduced bed stress.

Parameter $\overline{V}$ which is defined by a ratio of fluid flowing with velocity u parallel to a solid permeable boundary through which a secondary fluid is flowing with velocity ω:

$$\overline{V} = \frac{\omega_m}{u_m}$$

The phenomenon that the bed shear stress would increase in the case of infiltrating can be attributed to the fact that the streamlines of the main flow (here the flow of the sea or ocean) are drawn closer to the stream bed leading to higher flow velocities near the bed. Which leads to higher bed shear stresses.

Nielsen (1997) suggested the use of the modified Shields parameter of the form:

$$\theta = \frac{u_{*0}^2 (1 - \propto \frac{w}{u_{*0}})}{g d_{50} (s - 1 - \beta \frac{w}{K})}$$

to account for the combined effects of increased shear stress and downward drag due to infiltration. K is the hydraulic conductivity of the bed material and by Darcy's law we have $\frac{w}{K} = \frac{-dh^*}{dz}$, α and β are dimensionless coefficients giving the strength of the shear stress increase and the downward drag, respectively. (Cartwright & Nielsen, 1999)

If values (α, β) get chosen as (16, 0.4), the data from experiments with and without steady infiltration gets brought together around the same curve. These values were determined by other authors. (Nielsen, Robert, Moller-Christiansen, & Oliva, 2001)

3.2 Longshore

Understanding of the longshore sediment transport (LST) is one of the most important matters in the field of coastal engineering. Because LST is responsible for an incredible amount of sediment transport along the coast and is therefore responsible for the morphodynamic changes of beaches around the world.

The process of LST start with incident waves that hit the shoreline on a certain angle. This diagonality comes through the wind direction or the direction of the swell. These incident waves transport a lot of sediment with them, which gets deposited on the beach. When the waves retreat due to gravitational forces, they return to the sea/ocean in a vertical way transporting big parts of the sediment back with them into the sea/ocean. This process keeps repeating itself and leads to a longshore drift of sediment, where the updrift side erodes and the downdrift side undergoes accretion.

The total longshore sediment transport (LST) rate is one of the most commonly required quantities in coastal engineering, as it used to estimate certain accretions and erosion along the beach and it also serves to calculate the morphological response to human interference (engineering works). There have been a lot of attempt to accurately estimate the sediment transport ratio. With the most notable being the CERC equation (USACE, 1984) and the equation by Kamphuis (Kamphuis, 1991).

The US Army Corps Of Engineers developed the CERC equation. It is based on the principle that the volume of sand in transport, Q_{lst} is proportional to the longshore wave power P per unit length of the beach; LST = K*P, with K the calibration coefficient. It has been calibrated using field data from sand beaches. It is given below:

$$Q_{lst} = \frac{\rho K \sqrt{g/\gamma_g}}{16(\rho_s - \rho) g (1-a)} H_{s,b}^{2.5} \sin(2\theta_b)$$

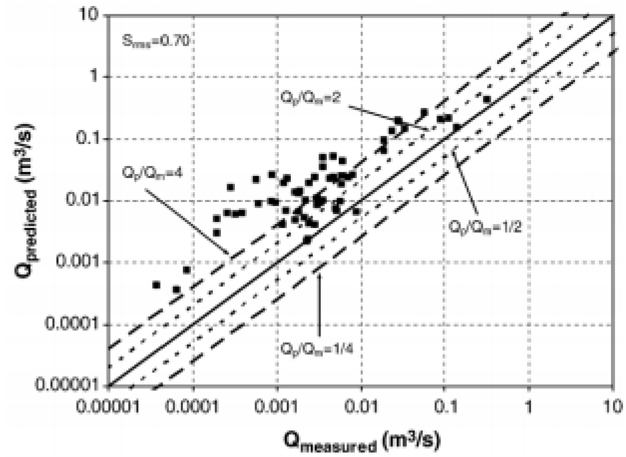
with Q_{lst} is the longshore transport rate in volume per unit time, K is an empirical coefficient, ρ is the density of water, ρ_s is the density of sand, g is acceleration due to gravity, a is the porosity index (≅0.4), $H_{s,b}$ is the significant wave height at breaking, γ_b is the breaker index (=H_b/h_b), and θ_b is the wave angle at breaking.

Kamphuis decided to study the effects of particle diameters and bed slopes, which resulted in a more accurate formula for longshore sediment transport.

$$Q_{lst,m} = 2.27 H_{s,b}^{2} T_{p}^{1.5} m_{b}^{0.75} D_{50}^{-0.25} sin^{0.6}(2\theta_{b})$$

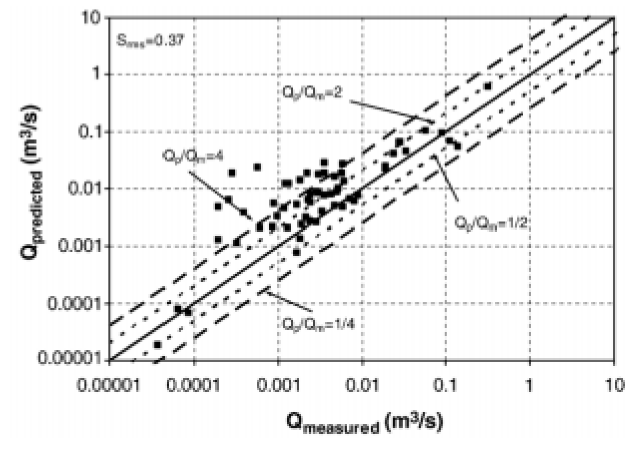
with $Q_{lst,m}$ as the transport rate of immersed mass per unit time, T_p is peak wave period, m_b is the beach slope near the breaking, i.e., the slope over one or two wavelengths seaward of the breaker line, and D_{50} is the median grain size. The immersed weight is related to the volumetric rate as $Q_{lst,m} = (\rho_s - \rho)(1 - a)Q_{lst}$. Contrary to the CERC equation, the Kamphuis formula considers grain size and wave period, which have their influences in getting the different particles to move and get into transport.

Although these formulae are still widely used they seem to come short on their accuracy levels. Bayram et al. decided to deduce a new formula that would get higher accuracy levels as opposed to the accuracy of the CERC equation and the Kamphuis equation. Their accuracy are plotted below, where they were tested using a verification data set to see how accurate they actually are. (Bayram, Larson, & Hanson, 2007)



23 Measured transport rates vs predicted by the CERC equation

Figure 23 depicts the measured transport rates of the verification data versus the values predicted by the CERC equation. The CERC (USACE, 1984) equation tends to overpredict the measured transport rates, and it produced the largest scatter and discrepancy ratio of 82%. Which means it only has an accuracy ratio of 28%.



24 Measured transport rates vs predicted by Kamphuis' formula

Figure 24 shows the values predicted by Kamphuis' formula versus the measured transport rates using the verification data set. The Kamphuis (1991) formula produced somewhat better predictions than the CERC equation having a discrepancy ratio of 58%. Like the CERC equation, the Kamphuis formula overpredicted low transport rates. These accuracy rates leave much room for improvement.

Bayram et al. found the accuracies reached by the CERC and Kamphuis equations insufficient. Because of the limitations in existing formulas, an effort was made in the present study to derive an alternative formula for the LST rate. This formula should (1) apply not only to wave-generated currents, but also to the case of sediment transport by wind and tidal currents; (2) include major physical factors governing LST; and (3) be validated with an extensive data set covering a wide range of conditions.

The main sediment that will be transported with the longshore currents is suspended sediment which is stirred up (brought into suspension) by breaking wave forces. Thus, the process will start with the suspension of sediment due to breaking waves and be followed by a transportation of this suspended sediment by any type of current. Therefore, the new formula needs to consist of parameters that describe breaking wave forces and any type of current. The wave breaking stirs up sediment and maintains an average concentration distribution c(x,z) in the surf zone (c in units of m^3 sediment/m^3 water). The total amount of work (W) needed to keep the sediment in suspension is a product of the concentration and the submerged weight of the particle with the fall speed w_s,

$$W = \int_0^{x_b} \int_{-h(x)}^{0} c(x,z)(\rho_s - \rho) g w_s dz dx$$

where x is a cross-shore coordinate originating at the shoreline and taken positive offshore (b denotes the break point), z a vertical coordinate originating at the still-water level, and h water depth. The work W can be attributed to a little part (ε) caused by wave flux F, being W = ε*F.

The LST rate is still known as the product of sediment concentration and the longshore current velocity (V).

$$Q_{lst} = \int_0^{x_b} \int_{-h(x)}^{0} c(x,z) V(x,z) dz dx$$

These formulas result in the following formula:

$$Q_{lst} = \frac{\varepsilon}{(\rho_s - \rho)(1 - a) g w_s} F \bar{V}$$

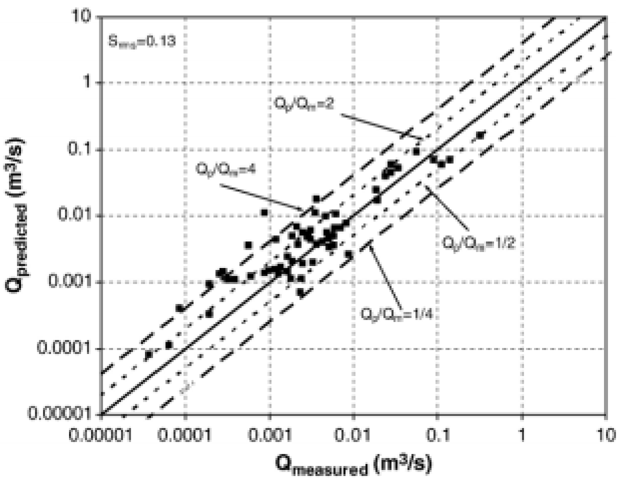
where the mean longshore current velocity ($\bar{V}$) and generated wave flux (F) have to be implemented. Following formula,

$$\bar{V} = \frac{5}{32} \frac{\pi \gamma_b \sqrt{g}}{c_f} A^{3/2} sin\theta_b$$

is used to estimate the mean flow velocity in longshore currents. And,

$$F = \frac{1}{8}\rho g H_b^2 \sqrt{g\frac{H_b}{\gamma_b}cos\theta_b}$$

is the mathematical description of the generated wave flux.



25 Measured transport rates vs predicted by the new formula

The new formula yields predictions that lie within a factor of 0.5 to 2 of the measured values for 62% of the data points. (figure 25)(Bayram et al., 2007)

Mil-Homens et al. tried to increase the accuracy of this formula using the most extensive LST data set presently available. This resulted in new calibration coefficients, which significantly increased the predictive ability of the formula. Although it is important to notice that despite the significant improvement in the prediction skills of the LST formulations, there is still considerable scatter. About 42% of the predictions (by all three improved formulas) deviate more than a factor 2 with respect to observations. This may be due to several reasons including: the non-consideration of parameters that may influence LST such as cross-shore profile features, 3-dimensional morphological features, tidal range and wind conditions in bulk LST formula; experimental errors that may have compromised data quality and insufficient data for high LST conditions. Another important shortcoming is the underestimation of LST in the higher energy region. This is most visible in the results of the CERC and Bayram formulas with the new coefficients. It would be desirable to have more data points in this region. (Mil-Homens, Ranasinghe, van Thiel de Vries, & Stive, 2013)

3.3 Cross-shore

Cross-shore sediment transport refers to the cumulative movement of sediment carries by waves, winds and currents perpendicular to the shore. The forces caused by waves, winds and currents lead to the movement of sand particles that are either in suspension in the water column or in flows at the surface of the seafloor. This occurs in a complex, three-dimensional pattern, varying rapidly with time. At any moment, some sand in the area of interest will have an onshore component while other sand is moving generally offshore.

Cross-shore can seriously alter the morphology of the beach and the surface zone in a short space of time. These morphologic changes brought to the coast can usually be attributed to changes in the elevation profile of the beach and adjacent ocean floor. Elevation profiles across the beach and through the surf zone are obtained with conventional surveying techniques, augmented as necessary with water depth measurements from a boat or another floating platform. A reduction in the amount of sand above mean sea level and a corresponding increase in the volume below this reference, for example, would be interpreted as offshore transport. If the beach volume increases at the expense of submerged sand levels, this implies shoreward transport. This is where the Bruun rule comes into play, this rule states that the volume of sediment that has been eroded on top of the profile is equal to the volume deposited in the bottom. (Bosboom & Stive, 2013) Although this is very difficult to measure due to the difficulty to separate longshore currents from cross-shore processes.

Cross-shore sediment transport is entirely dependent on the seasons, during the winter months there are mostly more storm and higher waves, which cause the beaches to erode more and a higher occurrence of sand bars. During the summer months, the waves are less steep leading beaches to accrete. Winter beaches may be characteristically lower and narrower with pronounced bars near the location of the largest breakers, while summer beaches are wider and with smaller, less distinct bars closer to shore.

The study of cross-shore transport has resulted in the concept of an equilibrium underwater profile (Dean, 1991). The shape of the offshore profile is approximated by

$$h(y) = Ay^{0.67}$$

where h(y) is the depth at distance (y) and A is a scale factor related to the grain size distribution of the sand forming the beach. This is an engineering simplification that assumes, under the constant application of a given incident wave, a particular beach will evolve toward a certain profile shape. The incident wave can be further generalized to include all the seasonal variations, resulting in a single equilibrium profile con- figuration for that beach. The actual profile may never achieve this contour because the incident waves are constantly changing. However, it is useful to coastal engineers and scientists concerned with understanding the natural variability of beaches. The simple exponential model for the equilibrium beach does not include any provision for bars or other contour complexities such as rock outcroppings.

4 Different defence mechanisms

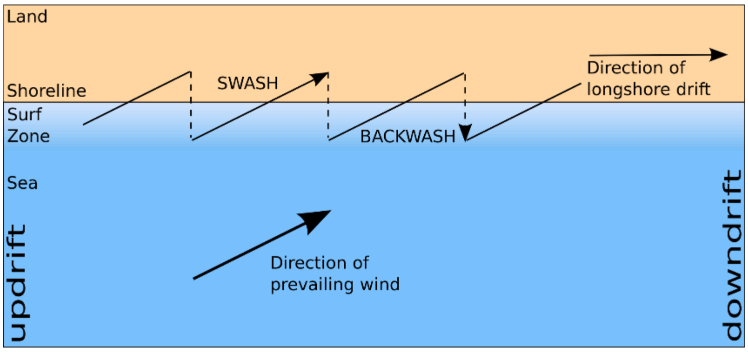
Coastal zone areas around the world are regions of great geographical importance as they inhabit the biggest part of the word's human population due to its opportunity for industry (fishing, harbours, transport, tourism). An area that's so precious to the present economy must be preserved from the devastating forces of nature. There are two different ways to protect these zones, being hard measures or soft measures. To select between these two can be very hard, that's why they are explained more thoroughly below.

4.1 Hard measures

Hard measures are typically used to protect coastal settlements by installing structures along the coastline that serve this purpose. They are used to deflect the power of waves. These are highly visible solutions which help reassure coastal communities. However, they are expensive to install and maintain. Also, installing these hard engineering solutions in one place can have a detrimental effect further along the coast.

4.1.1 Groynes

A rigid hydraulic structure that's constructed perpendicular to the shoreline, built to control the movement of the sediment material by interrupting the movements of the sea or river. In the matter of coastal protection, the main goal of such structures is to prevent the shore to erode or even disappear due to longshore drift. Longshore drift (figure 22) is the phenomenon caused by incident waves that hit the shore on a certain angle (incident angle), the diagonality of this phenomenon is triggered by the wind that controls the angle at which the waves hit the coast or by the direction of the dominant swell (surface gravity waves that cannot be affected by local winds). These angular waves carry little particles of sand and rock with them and drop them off at the point of the coast where the waves hit, eventually these particles will then be sucked back into the sea in a direction perpendicular on the shoreline because of the gravity. This process of diagonal drop off and perpendicular suction repeats itself time and time again, and causes a drift along the shore. Leading to erosion on the updrift beach and to accretion on the downdrift side. Figure 26 is a schematic presentation of this phenomenon.



26 Longshore drift

If this phenomenon wouldn't be stopped, the coast would erode and eventually disappear. To stop this from happening, groynes (figure 27) can be constructed. As mentioned before they're built perpendicular to the coastline and therefore form barriers for prevention of the longshore drift.

27 Groynes

When designing these groynes, the length and height of these structures must be determined. Because groynes that are too high or too long can lead to a strong wave flow in the zone between the groynes which also leads to a big entrapment of sediment. On the other side building too low or too short groynes could lead to little entrapment of sediment which would lead to a lot of erosion. So, a consideration must be made between the two extremes. Finally, it must be made sure that these structures stretch far enough land inward because otherwise the sea waves would just bypass the groynes by making a channel between the coast and the structure, this phenomenon is known as flanking.

There are lots of different kinds of groynes. Groynes can be constructed out of rock, wood or concrete. Next to this, groynes can differentiate from one another in the way that they can be fully submerged or not. Mostly groynes are constructed in groups that consist of several groynes, mostly in combination with seawalls

4.1.2 Seawalls

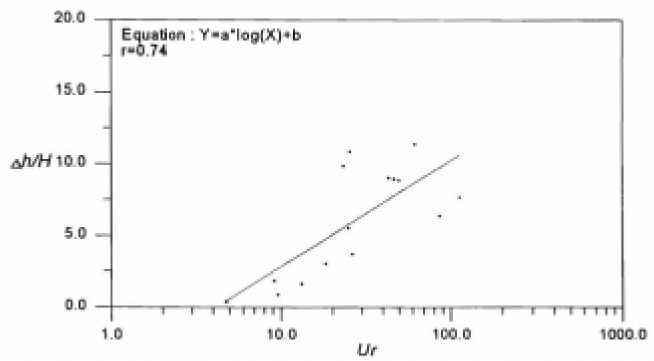
Sea walls (figure 28) are shore-parallel structures built from concrete, steel or stone along the coastline. And aim to prevent the shoreline, and therefore also the human areas of human habitation and leisure activities, from retreating (due to erosion) and to protect the upland from high waves and flooding by deflecting the energy of the waves. They're static structures that serve as a barrier between the dynamic nature of the coast and the inland. This barrier stops the exchange of sediment between the land and the sea.

28 Sea wall

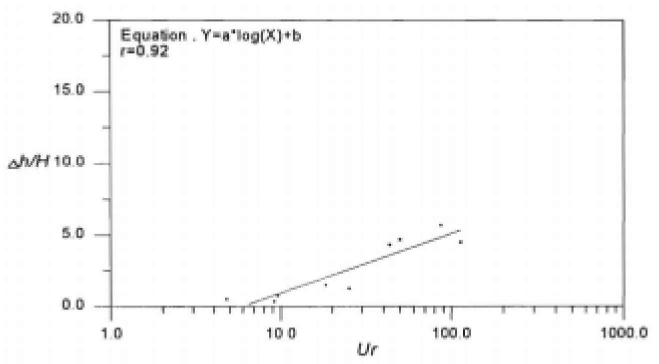
Seawalls are usually built on coasts experiencing chronic erosion or in danger of inundation and where further shoreline recession and flooding must be prevented. A seawall constructed on a beach with ample width and sediment supply, can introduce unnecessary problems by interrupting longshore sediment transport during times of high water and by preventing natural excursions of the beach in transformation between summer (swell) and winter (storm) wave conditions. On the other hand there are situations where seawalls have intermittently functioned during long-term cycles of erosion and become inactive and even buried during times of sediment abundance.(Kraus & Mcdougal, 2013)

Seawalls are very common structures constructed in coastal areas to protect land on the leeside. Especially, it is needed in reclamation areas. However, a seawall is a static rigid structure that must endure dynamic wave forces. Therefore, it must reflect these wave forces back into the sea/ocean, whereas the energy dissipates up and down. The energy that gets dissipated upwards doesn't form a problem as it will just result in splashing water, however the energy that dissipates downward will directly influence the toe of the sea wall. This toe usually suffers serious erosion and thus becomes troublesome problem for coastal engineers. This erosion, called scour, could lead to a total collapse of the seawall, if repairment has not been accomplished in time. There have been many evidences indicating that the failure of many coastal structures was attributed to the erosion in front of those structures. Therefore, the erosion in front of those seawalls has been a major concern for many coastal engineers.(Twu & Liao, 1999)

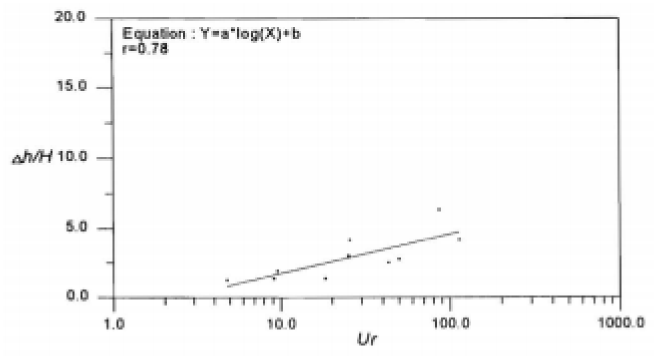
The scour depth in front of the seawalls with varied front slopes was studied. A three-dimensional wave basin was used to conduct a series of moving bed hydraulic tests to produce short-crested waves in front of the seawall which is believed to be significant in causing the scour. The relation of the scour to several parameters has been examined. Experimental results showed that the scour increases with increasing either the surf parameter or reflection coefficient, but they are both not well related. However, if the scour is expressed in terms of the Ursell parameter, which is an indicator responsible for describing the wave characteristics, as well as the front slope of the seawall, a better relation of the scour with the two parameters is found. Increasing either the Ursell parameter or the wall slope would make the scour worse. Nevertheless, it is realized that the scour depth in front of the seawall with $\tan \theta = 1/4$ (figure 30) is much less than that with $\tan \theta = 1/2$ or $1/3$ (figure 29), but is only slightly more than that with $\tan \theta = 1/7$ (figure 31). Figure 28 displays the relation of all the slopes to scour depth. This trend observed in three-dimensional tests is just the same as that obtained in two-dimensional tests conducted by TWU and CHIOU (1994). Since the seawall with $\tan \theta = 1/7$ possesses much larger section and occupies more base area than $\tan \theta = 1/4$, the latter may be suggested as a favourite option for the front slope of the seawall if both the scour depth and economical cross-section factor are taken into consideration.



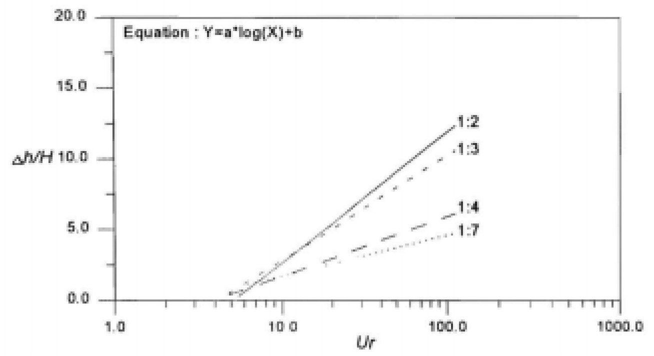
*29 Non-dimensional scour depth versus the Ursell parameter for tan **θ** = 1/3.*



*30 Non-dimensional scour depth versus the Ursell parameter for tan **θ** = ¼.*



*31 Non-dimensional scour depth versus the Ursell parameter for tan **θ** = 1/7.*



*32 Non-dimensional scour depth versus the Ursell parameter for tan **θ**.*

Apart from the problem of the scouring of the base, seawalls also disrupt natural shoreline processes and destroy shoreline habitats such as wetlands and intertidal beaches. As explained below.

Waves have the capacity to move tremendous amounts of sand in the surf zone. This sand movement on beaches can be conveniently considered as either longshore or cross-shore sand transport. This distinction, cross-shore vs. longshore transport, is somewhat artificial, in that the individual grains of sand may be moved both in the cross-shore and longshore directions at the same time. The movement of individual sand grains in response to wave motion and currents in the surf zone is extremely complex. Movement is related to instantaneous near-bottom water velocities under breaking irregular waves, the resulting shear stress on the bottom sand grains, and the subsequent transport of sand including the rich variations in transport mechanisms (bedload, suspended load, ripple and other bedform effects, bed ventilation effects). The complexities of surf zone dynamics and sediment transport processes preclude any meaningful analytic approaches. Thus, coastal engineers and scientists typically look for simplifications of the dynamics of the processes that can be modelled and compared with empirical results. One of the simplifications adopted is the separation of transport into the cross-shore and longshore directions.

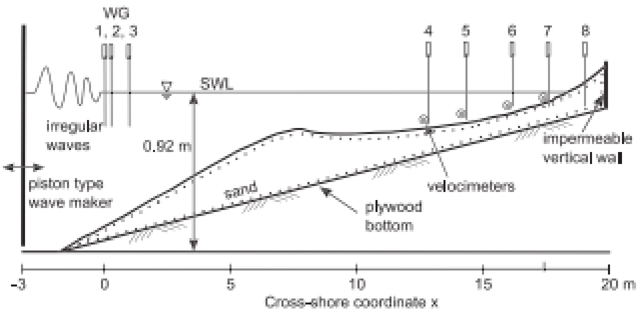
4.1.2.1 Effects of longshore transport on seawalls

Kamphuis et al. have conducted some experiments on the effects of seawalls on longshore sediment transport. Therefore, different hydraulic models were used to accurately describe the shoreline and its different characteristics. The following conclusions were found:

- The equilibrium profile developed in front of the seawall is a complex function of the initial profile, the storm surge, and the wave climate.
- The longshore sediment transport rate decreased as the beach eroded in front of the seawall.
- The location of the breaker peaks in the longshore suspended and bed load sediment transport rate distribution moved slightly offshore as the beach eroded.
- The bedload peak in the swash zone disappeared as the foreshore eroded. The local depth was found to be closely related to the local wave height; the ratio H/d approached a constant value as the beach approached an equilibrium condition. However, average scour depth in front of a seawall cannot be simply related to offshore wave height. (Rakha & Kamphuis, 1997)

4.1.2.2 Effects of cross-shore transport on seawalls

SAITOH and KABOYASHI investigated the effects the seawalls would have on the natural cross-shore sediment transport. An experiment was conducted in a wave flume that was 30 m long, 1.15 m wide, and 1.5 m high, as shown in figure 33 below.



33 Experimental setup for a sand beach in front of a vertical wall

This wave flume is equipped with 8 wave gauges, responsible for measuring the height of the several waves above the still water level (SWL). Four Doppler velocimeters were installed to measure the different velocities of the waves on different locations. The sand bottom elevation in the wave flume was measured using two profiling systems (Figlus et al., 2011). A laser line scanner mounted on a motorized cart was used to measure the 3D bathymetry of the subaerial portion of the beach profile.

An experiment was conducted in a wave flume to investigate the cross-shore irregular wave transformation and sediment transport on a sand beach in front of a vertical wall situated above the SWL. Seven 400-s runs of irregular waves were generated on a semi-equilibrium beach. The cross-shore numerical model CSHORE (Kobayashi et al., 2010) is shown to reproduce the cross-shore variations of the mean and standard deviation of the free surface elevation η measured at eight cross-shore locations. The comparison for the horizontal velocity U measured at four cross-shore locations is approximate because of the difference between the computed depth-averaged velocity and the measured velocity at the elevation of (2d/3) below SWL where d is the local still water depth. The measured extreme values of η and U were larger than the values corresponding to the Gaussian probability distribution. The measured maximum values of η and U are expressed empirically in terms of the mean and standard deviation of η and U to predict the cross-shore variations of the maximum free surface elevation and onshore velocity that are needed to estimate the extent of wave action. The computed cross-shore variations are discussed considering the simple approach suggested by Federal Emergency Management Agency (1996). In addition, CSHORE is shown to predict the semi-equilibrium beach with the elevation difference of about 1 cm in front of the wall. The computed cross-shore variations of the bedload and suspended sand transport rates are examined to identify the cause of the observed slight accretion in front of the wall. CSHORE predicts the onshore sediment transport but cannot reproduce this slight accretion using the empirical bedload and suspended load parameters for the ranges calibrated previously because the computed profile evolution is not very sensitive to these parameters. (Saitoh & Kobayashi, 2012)

4.1.3 Revetments

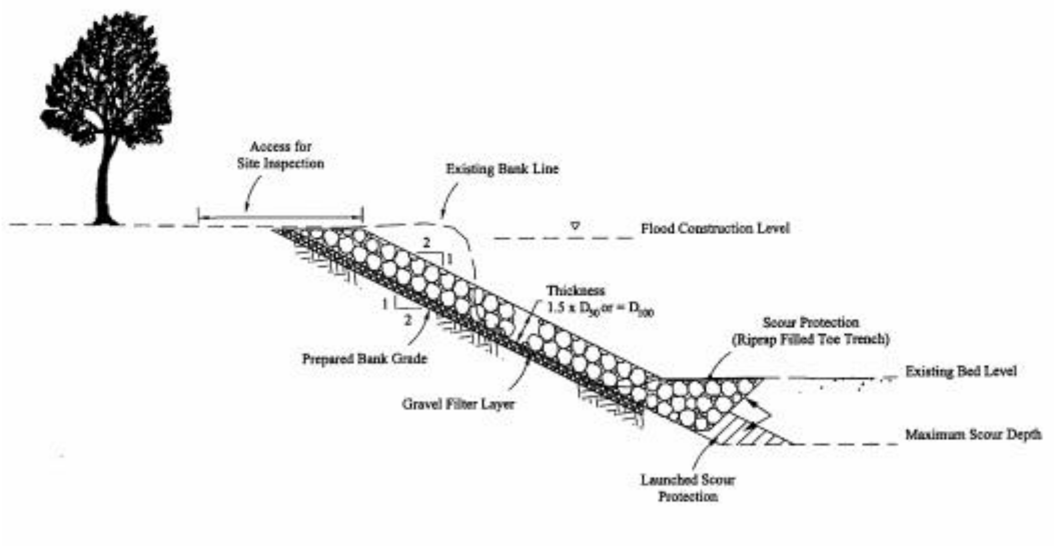
Revetments are sloping structures placed on banks or cliffs in such a way as to absorb the energy of incoming water. In coastal engineering, they're mainly used to protect a certain shoreline from erosion, by absorbing the energy of the incident waves. These structures can be built out of stone, concrete or other material, there are a couple of different forms in which revetments can be built, where riprap and tetrapod are the most common.

4.1.3.1 Riprap

34 Riprap revetment

Rock riprap (figure 34), also known as rock armour, resists erosion through a combination of stone size and weight, stone durability, and the gradation and thickness of the riprap blanket. The interlocking of angular rocks provides resistance to movement for the individual blocks in the revetment. Stream characteristics also strongly affect the stability of riprap revetments. Local scour, as affected by stream characteristics and bed materials, determines the protection required against undermining of the toe of the revetment; channel slope and alignment affect the impingement of flows on the bank and the hydraulic conditions that the rock must resist.

The four most common types of riprap failure are particle erosion, translational sliding, modified slumping and slumping (Blodgett & Mcconaughy, 1986). Particle erosion results from the displacement of individual rocks and is often a result of undersized rock, debris impact or direct impingement of flow. Translational sliding, where the revetment fails parallel to the side slope, generally results from toe scour and loss of support along the base of the revetment. Modified slumping, where the blanket moves without toe failure, usually results from overly steep slopes. Slumping is a rotational failure of the bank beneath the revetment, that generally occurs on high, unstable banks.



35 Cross-section of the typical riprap revetment

As can be seen from Figure 35 above, there are some things to which must be paid much attention during the design of the riprap revetment. Very important for the working of the whole structure are the scour protection, the thickness and the filter.

Protection against scour is provided by reinforcements at the toe of the structure, which should reach at least as deep as the predefined maximum scour depth. There are some methods who try to give approximations of this maximum scour depth, such as the methods provides by the *scour manual* by (Hoffmans & Pilarczyk, 1995) and *scouring* by (Breusers & Raudkivi, 1991).

The revetment should be at least thick enough to store all the rocks which would provide to actual reinforcement. It must include all the rocks in the specified gradation within the layer. Oversize stones that project through the layer may contribute to failure by creating turbulence. Based on Brown and Clyde (1989), the riprap thickness normal to the slope should meet the following criteria; not less than 350 mm, not less than 1.5 x D_{50} and not less than a D_{100}.

Filters are necessary to limit the loss of bank material through the riprap. The traditional filter material is gravel or crushed rock. Geotextiles are also an alternative because they may be cheaper and easier to install in certain circumstances. For gravel or rock filters, Brown and Clyde (1989) recommend the following sizing criterion: $D_{15c}/D_{85f} < 5 < D_{15c}/D_{15f} < 40$ where D_{15} and D_{85} refer to the 15% and 85% sieve passing sizes, and subscripts “c” and “f” refer to the coarse and finer layers respectively. The criterion should be imposed at the interfaces between the underlying material and the filter, and between the filter and the overlying riprap. If a single filter layer cannot meet the criterion at both interfaces, two or more layers may be required.

4.1.3.2 Tetrapod

36 *Tetrapod revetment*

The tetrapods are designed in such a way that they dissipate the force of incoming waves by making the water flow around rather than against them (figure 36). They also reduce displacement by allowing the random distribution of tetrapods to mutually interlock. Due to their weight and design, tetrapods can remain stable even under the most extreme weather conditions. Several tetrapods arranged together form an interlocking, porous barrier that dissipates the power of waves and currents.

No structural design or concrete structures used in breakwaters can last forever. Even tetrapods or any other form of concrete blocks, tend to become dislodged over a period of time due to the forces of nature constantly crashing against them. Thus, all the concrete structures are replaced after some point of time. Tetrapods are generally monitored through satellite photography for any kind of displacement or change in structural form.

Though tetrapods are helpful structures, they have also faced a lot of criticism mainly because of their shape. Many people argue that they pose a danger to swimmers, surfers, and boaters, while others say that tetrapods in fact accelerate beach erosion by disturbing the natural processes that shape the coastal environment. Tetrapods have also been criticized for spoiling the natural coastal scenery. It is also being said that the wave action on tetrapods pulls the sand away from the shore faster than what happens in the natural process.

Thus, even though tetrapods are widely criticized, the fact remains that they cannot be neglected. The main feature of a tetrapod lies in its design, which is not found in nature. Tetrapods are a symbol of artificiality and not aesthetics, and despite all the criticism have been extremely helpful in numerous ways.

4.1.4 Gabions

37 gabion revetment (foreground) gabion walls (background)

Gabions (figure 37) are big cubes made of steel wire filled with pebbles, stones and rocks. Mostly they are filled in situ, often with locally available material and therefore have a relatively low capital cost. Because they are flexible and porous they can absorb some wave and wind energy, thereby reducing the scour problems associated with impermeable sea defences such as concrete seawalls. Gabions can be placed as sloping “mattresses” or as near vertical cubic baskets. The latter are intended for bank or cliff stabilisation and are not normally suitable for use in shoreline situations.

The low-cost and ease of assembly on the construction site make gabions an attractive alternative to other shore hardening structures. But mostly it is not the most effective measure that can be taken to protect the shoreline. The following problems with gabions on open-ocean shorelines have been observed: (1) gradual failure by degradation, (2) instantaneous failure during a storm, (3) rock leakage onto the beach, (4) protruding wire on the beach, and (5) loss of the recreational value of the beach. This leads to repairment or even replacement. Four stages of gabion life have been identified in Puerto Rico: (1) emplacement - newly emplaced gabions can cause active, passive, or placement beach loss, similar to seawalls and revetments (2) initial weakening - piping, wire deterioration and rupture, and gabion slumping or toppling signal the eventual failure of the gabion structure, (3) failure - the gabions leak rock content and/or collapse onto the beach, and (4) replacement - failed gabions are replaced by more substantial hard structures or covered in place with concrete. The alleged low-cost of gabions is misleading and more than offset by their high failure rate, negative environmental impact, and threat to the safety of beach users. The Puerto Rico experience indicates that even with regular maintenance, gabions are a poor choice for open-ocean shoreline protection. (Jackson, Bush, & Neal, 2006)

4.1.5 Offshore breakwater

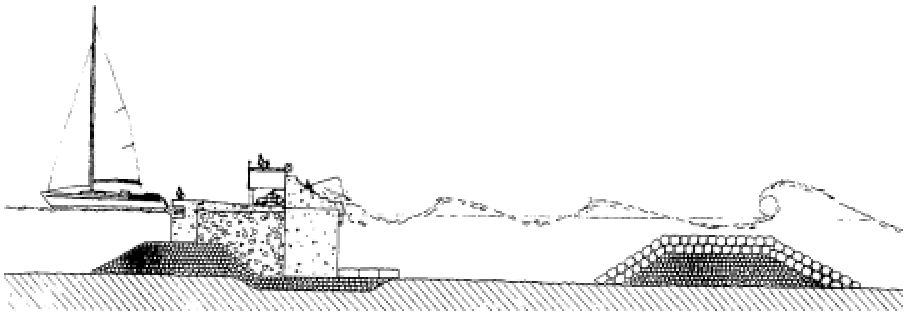
38 Offshore breakwaters

Breakwaters are structures constructed on coasts as part of coastal defense or to protect an anchorage from the effects of both weather and longshore drift. From all hard-engineering measures they tend to have the least impact on the environment as they are completely detached from the shoreline (figure 38). These structures could be, according to their position relatively to the mean water level, emerged or submerged. Both constitute an obstacle to the normal wave propagation, permitting the dissipation of the incident wave energy and providing a “filter” shelter for the coast at their lee-side. The submerged breakwater (figure 39) is also a particularly attractive solution for the creation and preservation of beaches, due to its low environmental and visual impact.

Submerged breakwaters could be constructed for several reasons, the most common purposes being the following:

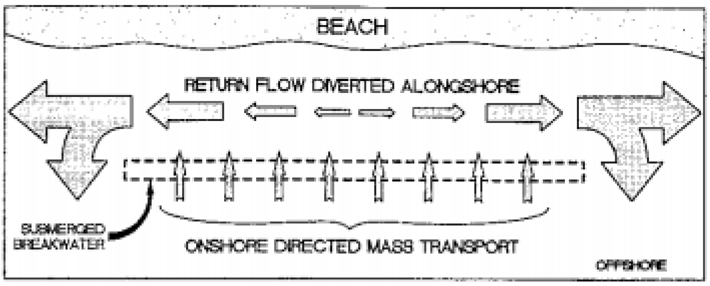
- Beach protection caused by the wave dissipation/attenuation “shelter” effect

- Creation of a calmer zone in a harbour, protecting them or preventing siltation in port access ways
- Protection of a main structure by reducing the intensity of wave action on the principal coastal defence structure
- Redistribution of sediment transport patterns, to create desirable beach features or alteration of the sediment deposition area in a navigation channel entrance.



39 effect of a submerged breakwater on wave propagation

The breakwaters capability for retaining or permitting sediment accumulation (if there is shore sediment transport) at its backward side is responsible for its important role in beach protection. This is due to the attenuation of the wave height, caused by the energy dissipation and the formation of diffraction currents at the ends of the structure. Figure 40 illustrates the diffraction currents formed in the extremities of the breakwater. Their role is very important, even if the long shore transport is not significant, which justifies their use when this transport is reduced. This importance is not only related with the sediment accumulation but also with the bathing water quality in the area and its re-circulation.

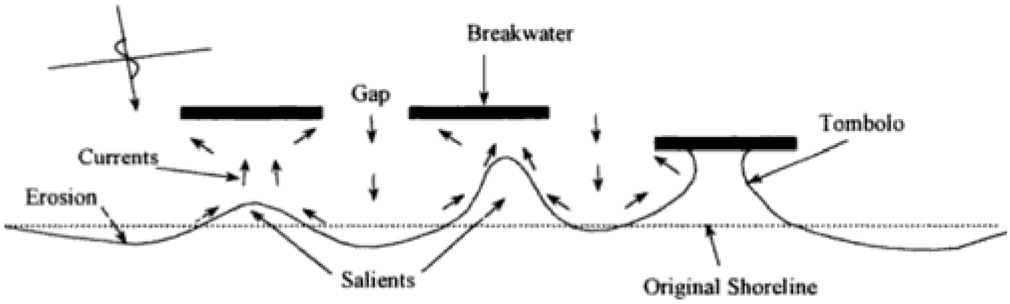


40 diffraction current lee-side of submerged breakwaters

Much research has been conducted on the effects of submerged breakwaters on wave dissipation, but there are still some unresolved questions due to the dependence on a wide scale of variables. These variables include: bathymetry, wave climate, sedimentation, implantation depth, length, distance to the coast, gaps between structures, crest height and length and submerged breakwaters structural configuration. With bathymetry being the underwater equivalent to topography.

Detached breakwaters (emerged type) are designed to attenuate the whole wave action and are submitted to the direct impact of wave breaking, resulting in larger structures that often eliminate water circulation at the lee side (in the protected area). Consequently, degradation of water quality and of natural habitats in the lee-side is a frequent phenomenon. The environmental result is obviously not highly appreciated, due to its big visual impact, together with the strong erosion phenomena noticed at the gaps between barriers.

Offshore emerged breakwater intercept much of the incident wave energy, resulting in reduced wave action behind the structures. The waves enter through the breakwater gaps and then diffract as they travel towards the shore. The diffracted waves change the beach shape from a relatively to an attractively curved shoreline with salients and tombolos. A salient is an accretion formation that doesn't reach the breakwaters itself; a tombolo is an accretion that does reach the breakwater. In general, breakwaters that are longer or closer to the coast form tombolos. Salients form when the breakwaters are further from shore and they are substantial gaps between the breakwaters. (figure 42)



41 tomboloformation

Salients are mostly preferred over tombolos, because they do not block the currents behind the breakwaters, thus enhancing water quality in the swimming areas. However, they are essentially an unstable beach form between a straight beach and a tombolo. Small changes in conditions can convert a salient into a tombolo, which means that incident wave and water level conditions must be more or less constant in order to produce salients. (Kamphuis, 2000)

On the other hand, submerged breakwaters allow circulation along the shoreline zone as they are constructed below the mean water level and thus permit some overtopping. It is assumed that submerged breakwaters can have a crest height of 40-50% of the water depth. This leads to sufficient water exchange between the shore side and the offshore side of the structure, preserving healthiness and the bathing use of the water in the protected area. Some advantages are connected to the higher water quality; maintenance of fish habitats due to its lower impact of coastal development on aquatic habitat and a better integration of the coastal defence structure in the shore zone, are examples of the advantages of submerged breakwaters over the conventional structures.

Where emerged breakwater break waves in a direct manner, submerged are less subjected to wave action as their height is much lower and the required volume of material is much less than similar emerged counterparts. They're considered as a good solution for coastal protection because of their low visual (they do not spoil the aesthetic value of the beach) and minimal environmental impact.

Though it is not all advantages for these submerged structures as their design is generally more sophisticated and an adequate marker is needed to avoid navigation hazards due to their invisible (below sea level) nature. From the construction point of view, this kind of structures needs the employment of floating equipment and more delicate constructive techniques, being not convenient in high energetic sea states.

It must be stated that generally emerged breakwaters offer a better protection of the coastline. Although there are some cases where the submerged structures are believed to have more effect on wave dissipation than the emerged counterpart. They could be used in combination with a main defence structure that will attenuate the biggest waves before they reach the main structure.

Another disadvantage of emerged breakwaters, in terms of environment, is the necessity of gaps between the barriers that often give rise to rip currents, bed irregularities and tombolos, Pilarczyk (1996). Submerged breakwaters do not offer this inconveniency, as they can be constructed in the form of long continuous structures without gaps. As for lower waves, these submerged structures are much more permeable, they do not need gaps for the necessary continuous water exchange to and from the internal area, like in their emerging counterparts.

In a time where the emphasis lies on the visual aspect of the beaches, a submerged breakwater could offer a great solution. Although these structures struggle to cope with high incident waves. (Pinto & Neves, 2004)

4.1.6 Cliff stabilization

4.1.6.1 Causes of instability

One of the erosional coastal landforms are the seacliffs, as we know these landforms can be quite unstable. Causes of this instability can be due to the combined effect of several factors, such as:

- Erosion of the foot of the cliff caused by sea action that includes waves, wind, and tide action. This is mainly the starting point for the further destabilization of the cliff. It undercuts the cliff and initiates other sliding collapses, which are determined by the nature of the cliff materials and their geotechnical properties.
- Sliding or weathering of the slope due to geo-technical instability. This phenomenon is mostly preceded by previous mention, but the sliding or eventual collapsing of the cliff can be of different nature depending on the geo-technical conditions (materials and geotechnical properties) of the slope. There are basically three different situations:
    1. If the cliff is built up out of non-cohesive materials and rock, the collapse of the upper layers could cause the formation of talus or a collection of materials at the foot of the cliff caused by the material falling. This may act as a protection for the eroded base and reduce the effect of wave action or storm surge and the cliff may get stabilized without any further action being needed.
    2. If the material is a mixture of clay, silt, sand and boulders, such as in the case of moraine till, the resulting slope can be become very steep for a period due to the significant cohesive forces. Although the slope will eventually further deteriorate with the increase in water pressure due to ground water from the land area above the cliff. This phenomenon will only increase during periods of frost, where the groundwater will freeze which will increase the groundwater pressure even more. If the cliff will collapse due to the water pressure it will probably happen because of sliding.

3. If the material consists of clay and silt, which are more plastic, the collapse of the cliff will be in the form of slides, which can go far behind the top of the cliff.

- Weathering of the cliff by wind transport of sand. This will be most pronounced if the cliff material is sand; however, also exposed cliffs consisting of other types of material can be eroded by sand blown over the cliff from the beach.

Next to before mentioned causes of cliff erosion, it has been researched by *Earlie et al. (2015)* that cliffs also suffer from seismic motions during heavy storm surges. Wave pressure fluctuations on the ocean floor generate micro seismic ground motions both at the coast and hundreds of kilometres inland. Combined observations of coastal ground motions and in situ nearshore hydrodynamic data have advanced our understanding of ground motion on different coastal morphologies and shelf bathymetries under varying tidal and wave conditions. In most instances, considered cliff-top ground motions increase with increasing wave height and tidal elevations.

The cliff-top ground motions generated from local ocean waves can be categorized into three major frequency bands: (1) high frequency (HF) 1–50 Hz (1–0.02 s), reflecting the natural frequency of the ground as it "rings" in direct response to wave impact and breaking waves; (2) low-frequency cliff motion or "flexing" generated by individual sea swell or single-frequency waves (SF) 0.1–0.05 Hz (10–20 s); and (3) infragravity waves (IG)< 0.05 Hz (>20 s) which load the foreshore, causing pressure fluctuations. Microseisms are also detected, and motions at double frequencies (DF, twice the primary sea swell frequency) (0.1–0.2 Hz, 1–5 s) exhibit similar amplitude at the coast and tens of kilometres inland.

Cliff-top ground motions measured in wave conditions with significant wave height H_s less than 3 m show vertical ground displacements in the region of 0.5–10 μm during each wave loading cycle. It has been suggested that this repetitive flexure of the cliffs ultimately fatigues rock strength and leads to cliff failure. Experiments using cross-shore seismometer arrays show an exponential decay in the ground motion signal (in the IG and SF bands) with distance inland. The stresses created by the decrease of displacement inland are thought to be responsible for potentially weakening the integrity of the rock structure. This hypothesis was examined in sedimentary cliffs capped with glacial till deposits under a range of wave conditions (H_s< 5 m) and argued that "background" micro seismic cliff-top motion caused by cyclical loading is usually not of sufficient amplitude to drive the growth of microcracks. However, it was also suggested in this experiment that larger displacements associated with episodic wave events (H_s> 5 m) can be responsible for less frequent, cliff-normal displacements, leading to an interaction between groups of microcracks that could ultimately damage the integrity of the rock structure.

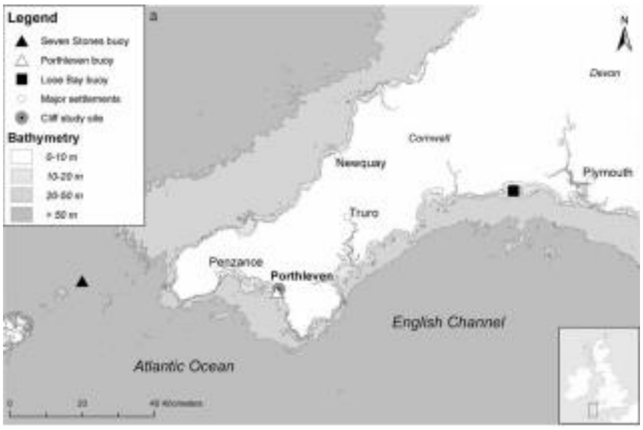
As it is not easy to conduct experiments on sea cliffs during severe storms, the experiment was conducted in southwest UK from the 31st of January to the 6th of February 2014, it is important to have an explanation of the used method which *Earlie et al. (2015)* used to investigate the displacements along the cliff.

The deepwater wave conditions were obtained from a vessel anchored 55km to the west of the investigated site, on the location where the ship was anchored it was floating 60m above the sea-floor. Hourly statistics of offshore wave heights were recorded during the 7-day testing period. Next to that an inshore buoy was used, located 1km offshore (Porthleven buoy). This buoy began malfunctioning on the night of the 4th of February, another buoy (Looe Bay buoy) was then used after it was firstly verified that the data of both buoys were the same. The deepwater significant wave height (H_s) and peak wave period (T_p) were subsequently used to compute the deepwater wave energy flux (P) using (Arnott, 2009; Howd, 1998):

$$P = \frac{1}{16} \rho g H_s^2 C_g$$

where ρ is the density of seawater (1025 kg/m^3), g is the gravitational acceleration (9.81 m/s^2), and Cg is the deepwater group wave speed:

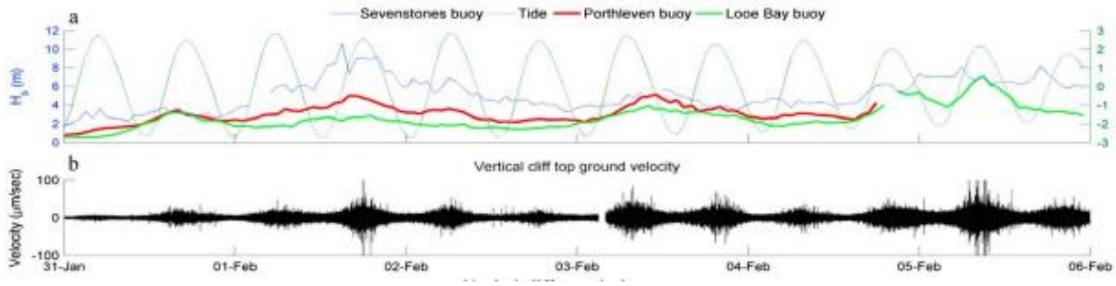
$$C_g = \frac{1}{2} (\frac{gT_p}{2\pi})$$



42 plan of investigation site

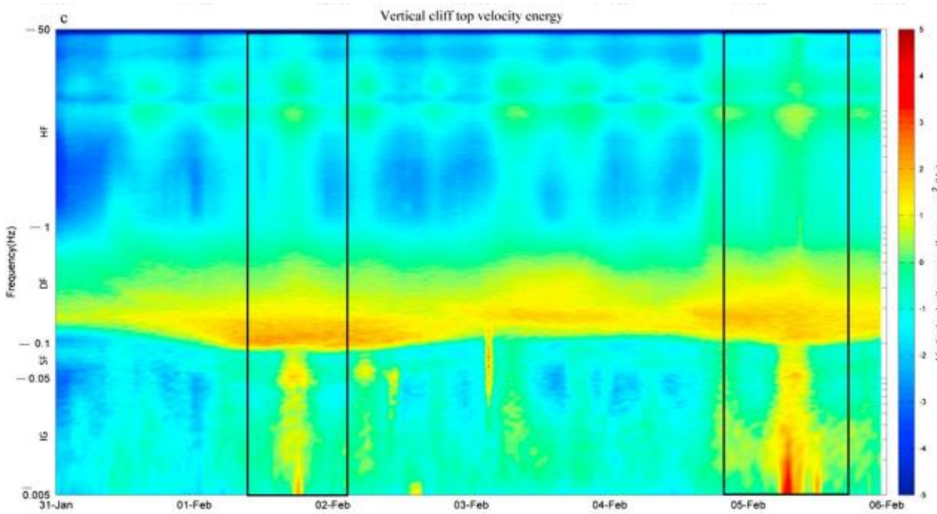
Not only the data about the height of the waves was assembled but also visual data was assembled using a waterproof camera, that would capture every cliff collapse, large wave impacts and wave overtopping events. As the camera was GPS time synced, the exact time of certain changes in the original structure could be determined. This video-monitoring was used in combination with a terrestrial laser scanner, a Leica P20, to more accurately compute the volumetric changes on the cliff using a direct point-to-point cloud comparison method.

The cliff-top motion was measured using a Nanometrics Compact Trillium broadband seismometer sampling at 100 Hz and was buried on the investigated cliff, about 5m inland. The found data would be compared with data acquired from the British Geological Survey inland broadband seismometer located at Carmellis, Cornwall, 17 km inland from the site, sampling at 50 Hz. All this acquired data was modified to the three known frequency domains, being: high frequency (HF) 1–50 Hz, single frequency (SF) 0.1–0.05 Hz, and infragravity frequency (IG) 0.005–0.05 Hz. Apart from these double frequency (DF) was also considered: 0.1–0.2 Hz. This was only the case for vertical velocity components, horizontal components were neglected in this case because they contain tilt effects on low frequencies.

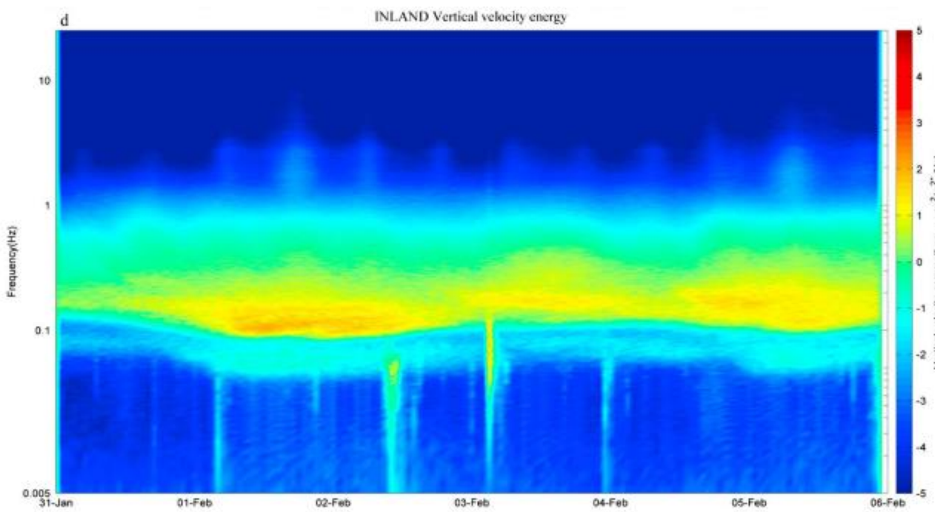


43 Results from the different buoys

In the graph above (figure 44), there were two heavy storms. Namely during the 1st of February and the 5th of February. The different lines don't coincide that much during the storm of the 1st of February, that is because the direction wasn't quite towards the Porthleven coast which explains why the vessel gives higher ratings than the more inlands buoys. During the 5th of February storm both lines coincide during high tide because this time the direction was exactly right.



44 vertical cliff-top velocities

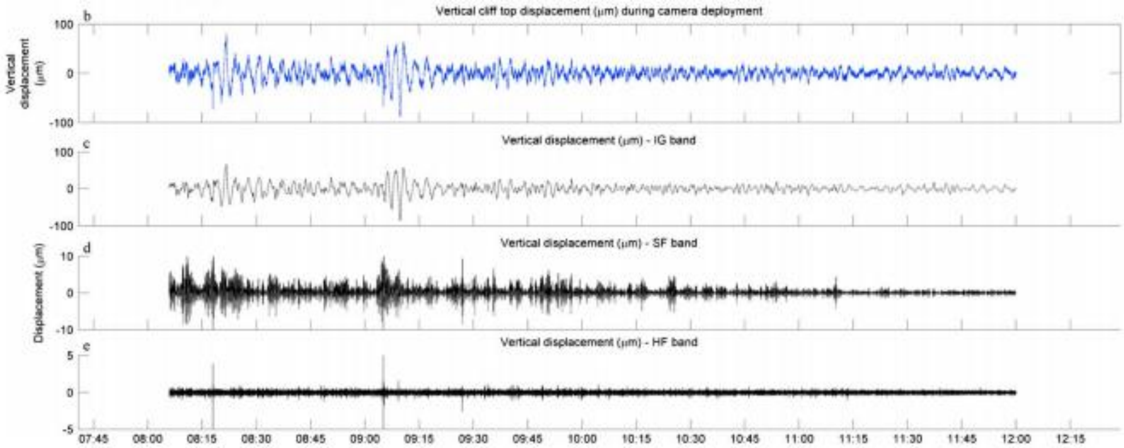


45 vertical cliff-top velocities (British Geological Survey)

Comparison of the implanted seismometer (figure 45) and the data from the British Geological Survey (figure 46) could help identify the local and nonlocal sources of energy. For each frequency spectra, the comparison is done:

HF – detected on the coast, not detected inland. So, this must be a locally generated signal.
DF – inland and coastal signals are similar suggesting a nonlocal dominance at the coast
SF – Inland there were 3 situations (1, 2 and 4 February) where a heightened signal was detected, while this is not noticeable in the coastal image, suggesting it concerns a local source. The spectral peak located around 0.1 Hz on 3 February was present in both the inland and the coastal spectra and coincided with a magnitude 5.7 earthquake located at Lixourion, Greece
IG - A clear IG energy peak occurred during the storm periods only in the coastal spectra.

The conclusion is that during storm peaks, the cliff suffers vertical velocity so a vertical displacement will probably be also the case. As can be seen in the figure 47:



46 vertical displacements from 08:00 to 12:30 h on 5 February 2014

In previous cliff-top ground motion studies with significant wave heights up to 5 m, vertical displacements rarely exceeded 10 µm [Adams et al., 2005; Young et al., 2011, 2013]. At our site, ground displacements during both the extreme storm wave events increased by an order of magnitude, where the vertical displacements increased from 5–10 µm under calmer periods to >50 µm under energetic conditions. These greatest vertical displacements occurred during the second storm event at high tide.

Under energetic conditions, the largest vertical displacements were coincident with periods of successive cliff overtopping followed by water cascading down the cliff face. This suggests that wave loading and unloading on the cliff-top might significantly increase cliff motion and the associated strains and flexure mechanisms during times of wave overtopping at higher tidal elevations.

Previous research has suggested that although displacements under normal conditions are not likely to contribute toward the weakening of rock structures, episodic displacements caused by extreme wave conditions may be responsible for failure in metasedimentary cliffs. A reason for these thoughts is a simple LIDAR scanning of the area which would could picture the geomorphic perspective of the area; the long-term annual retreat rate for Porthleven, obtained from aerial photography and averaged over 50 years, is 0.1 m yr^{-1}. This value was corroborated by *Earlie et al. [2014]* using airborne lidar over a 3.5-year period. Assuming a cliff height of 10 m, a long-term cliff recession rate of 0.1 m/yr^{-1} equates to an annual cliff volumetric loss of 1 m^3 per meter length of cliff. Terrestrial laser scans over the 2-week storm period show that the 300-m long cliff section eroded 1350 m^3, which represents 4.5 m^3 average erosion volume per meter length of cliff over the 2-week period, or an annual cliff volumetric loss of 113 m^3 per meter length of cliff.

During this study, it was found that the vertical cliff-top ground motions measured during an exceptionally stormy winter period in the UK were found to increase with increasing Hs and tidal elevation. Capturing these events during one of the stormiest periods the region has seen in 60 years highlights the role that extreme events play in contributing toward coastal cliff erosion. (Earlie, Young, Masselink, & Russell, 2015)

4.1.6.2 Measures

If erosion is already in place, it is very likely that cliffs have been eroded at the toe. In that case the only suitable solution is the construction of revetments (as discussed above) at the bottom of the cliffs, although at this point the slope of the cliff could have been se steep that eventual sliding or collapsing could happen nevertheless. So, other measures need to be looked at as well:

- Drilling long anchors into the cliff's face and grouting the holes with cement
- If the rock is fissured, a steel mesh can be used to cover the face of the rock, with the mesh being held again by anchors into the rock. Such mesh may however require constant maintenance in areas exposed to sea air.
- Artificial smoothing of slopes is the best method of stabilizing cliffs made of other material than rock. These slopes must have a gradient lesser than the angle of repose of the soil for the best effect.
- Cliff dewatering by creating horizontal or vertical drains that reduce the effect of water runoff.
- Cliff stabilization on such slopes can be helped by the growth of vegetation cover in the form of shrubbery that can hold the soil together.
- Addition of granular material at the base of the cliff can help stabilization by preventing further erosion that had started the destabilization process in the first place.

Previous measures are only possible when there is enough space at the bottom and top of the cliff, if otherwise more expensive measures are the only solution like retaining walls.

4.1.7 Entrance training walls

Entrance training walls (figure 48) are breakwaters built where rivers discharge into the adjacent sea or ocean. It allows for the flow of the river to be discharged in a controlled manner. The zone where the river discharges into the sea/ocean is often one of a very wild nature with unpredictable current, by building the training walls this problem gets avoided. And the beaches nearby gain their recreational factor.

However, there is a big negative effect when building those training walls, and that is causing an interruption of the longshore drift. Which causes the adjacent beaches to undergo profound morphological changes.

There were four beach behaviour patterns in response to the impact of the training wall. The first was the progressive accretion in the downdrift. Lighthouse Beach is the example. Under the protection of the training wall, the average rate of the accumulation of total sand dune was about 2% per year from 1965 - 1991. The rate of shoreline accretion was 3.8 -5.5 m/a from the beginning (1889) of the building of the training wall to 1973. The 1974 storm apparently did not disturb the special beach behaviour just described above. The protection of the training wall, the small, semi-closed bay, and rich supply of sediments were the special environment factors favouring this type. The second was beach accretion in the updrift of the training wall, generally characterized by accumulation of sand dune and shoreline accretion. Although there were impacts of big storm erosion, the total amount of sand exchange and range of contour change were minor. The accreting area dunes provided a reservoir of beach sand during severe storms and thus helped prevent wave erosion. In areas where substantial dunes existed, the post storm beach width being greater than the pre-storm width is proof. The third type was observable at the downdrift of the training wall too, where natural storm erosion and training wall induced serious erosion occurred. The fourth type is a special one when there was no net littoral drift along the beach. The wall functioned as an accumulator of sediment. There was deposition near the training wall and erosion away from the wall at both sides of the walls.(Huang, Short, Zeng, & Hanslow, 2004)

47 Entrance training walls at the Gold Coast, Australia

4.2 Soft measures

Soft measures are applied to prevent coastal erosion form happening, with as little environmental impact as possible. This approach potentially provides environmentally-friendly protection, is aesthetically pleasing, and can usually be implemented within a reasonable budget.

4.2.1 Beach nourishment

Beach nourishment is the process of mechanically or hydraulically placing sand directly on an eroding shore to restore or form, and subsequently maintain, an adequate protective or desired recreational beach. This process implies that by artificially widening of the mainland in seaward direction by adding sand, the risks may be relieved.

Although all is not well concerning beach nourishment as this form of beach restoration also brings about sizable changes in the sandy beach ecosystem. In the short term, a large proportion of the resident flora and fauna is destroyed by the addition of a thick layer of nourishment sand. Changes in the beach habitat after nourishment, such as altered beach profile and sedimentology, will influence the rate of recovery of the ecosystem's natural equilibrium.

The negative effects of beach nourishment can be divided in three main groups, being (1) during construction phase, (2) quality characteristics of the used sediment and (3) quantity characteristics of the used sediment supply.

(1) Construction phase
The environment gets influenced by the machines needed to put the extra sand in place. They have a negative impact on the environment through the excess noise they produce, which can possibly scare away fauna living in the area. Also, these machines create pollution due to the exhaust gasses, fuel leaks, etc.

(2) Quality characteristics
Adding sediment with a different grain size than the existing grain size can cause unnecessary compaction, which leads to changes in the interstitial space, the capillarity, the water retention, the permeability and the exchange of gases and nutrients. For example, female sea turtles dig holes in the beaches sand to make their nest. This could be hampered due to the compaction.
Changes in the beach profile (linked to the grain size distribution of the nourishment sands) can lead to changes in the hydrodynamics of the intertidal zone; an increase of the slope angle will increase wave energy on the beach, creating a hydrodynamically more stressful environment, leading to a reduction in diversity and abundance of the infauna. Significant changes of the profile can give rise to a change in the morphodynamic state, causing a slow recovery and maybe even a permanent shift in the ecological community structure.
The added sediment could contain toxic substances such as heavy metals, PACs, PCBs, …

(3) Quantity characteristics
The quantity of added sediments has effects on the possible burial of different species. A rule of thumb that goes for most organic species is that they can survive while buried under a maximum of 90cm of sediment. In most cases of traditional nourishment, the deposited sand layer is about 1 to 2.5 m and remains for a long period of time, thus resulting in total mortality of benthic macrofauna. (Speybroeck et al., 2006)

So, beach nourishment doesn't have a solely positive effect on the environment. But is still preferred over the use of hard measures, as it has less negative effects.

4.2.2 Dune stabilization

Dunes are large heaps of sand located at the end of the beach, and serve as a natural protection mechanism that prevents coastal waters to intrude the hinterland. The formation of dunes is initiated by beach grass (figure 48), which can nestle itself in the sand and survive there. This vegetation stops sand that's is saltating over the beach, and after a while it accumulates a whole mass of sand forming a dune.

It is known that dunes have suffered from human activities the last couple of decades. The number of dunes have profoundly decreased. These reasons can be due to activities directly on the beach such as the construction of foot paths, off road vehicle tracks, roads and use of sand dunes for water extraction. Or these reasons can be less direct by stopping the sediment supply to dunes, leaving them prone to erosion. It is known that stable dunes have an equilibrium between accretion due to sediment supply and erosion. The sediment supply come from longshore drift leaving sand on the beach during low tide, strong enough winds will blow this sediment to the back of the beach where it gets stopped by the beach grass. Humans affect sediment supply in several ways including the damming of rivers, dredging in coastal water and the construction of harbors, all of which depletes sediment supply. They also disrupt the long shore transport of sediment through the construction of piers, marinas or groynes, breakwaters and other hard structures.

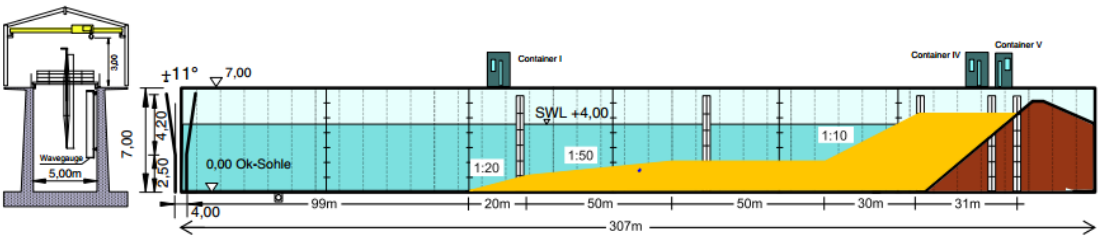
Measurements taken to preserve dunes from erosion should consider the natural processes that caused the dune in the first place and where needed restore and conserve these natural processes. This means that for the actual protection measurements, the cause of the sediment erosion must be investigated. That cause will mostly be a shortage of beach vegetation which are used to accumulate the sand. In zones where this is the case, a plantation of extra beach vegetation must be considered. It is important to note that only indigenous species can be used, as imported species could compete with the existing species and could adjust the dunes in a bad way (Van der Meulen & Salman, 1996). Following the plantation of extra beach vegetation, dune zones must be closed off from the public. As public access to dunes is one of the main reasons why dunes disappear.

48 Beach grass

4.2.3 Beach drainage

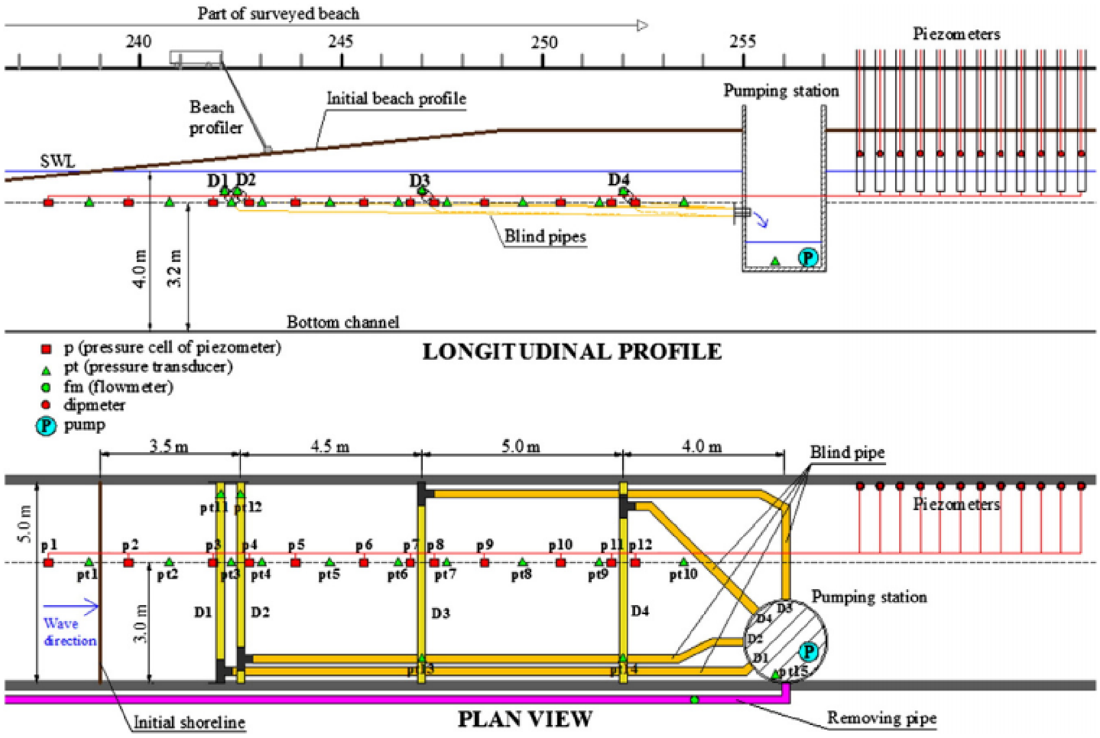
The main principle of beach drainage is keeping the beach dry by keeping the ground water table low, this leads to less back-wash. Because the grain-grain reaction is much stronger when the sand is dry, the back-washing wave cannot generate enough shear stress to bring the grains to motion. This whole process will lead to less erosion of the beach.

P. Contestabile et al. investigated the positive effects of the beach drainage system (BDS), as very little research was available. The previous experiments concerning BDS all suffered from bad scaling factors. That's why P. Contestabile et al. decided to conduct a series of new experiments utilising the large wave flume of the "Grosser Wellerkanal", it's a wave flume of 307 m long, 7 m deep and 5 m wide flume. It represents a 1:1 scale. (Figure 50)



49 Longitudinal profile of the large wave flume

In this wave flume, the beach was modelled with an initial slope of 1:20 (2.86°) for 20 m, changing to 1:50 (1.15°) in the next 50 m and a horizontal part for the following 50 m. The last 30 m, corresponding to the surf and the swash zone, were reached with a slope 1:10 (5.71°). The sand used was quartz sand with a characteristic diameter (D_{50}) of 0.33mm. This beach model was equipped with a drainage system of PVC drain pipes (named D1, D2, D3 and D4), with diameters of 0.2m, which are buried 0.4m under the still water level. They are all connected to a pumping station in a sloping way so that the water transport could happen through gravity alone, the pumping station would transfer the collected water to the initial part of the flume. The measurements of hydraulic head were measured with piezometers and morphological changes of the beaches profile were measured by a beach profiler mounted on a carriage. (Figure 51)



50 longitudinal and plan view of flume model and instrument location

The experiments were conducted for three different wave conditions; high energy (HE), medium energy (ME) and low energy (LE). It's important to mention that the beach profile wasn't remodelled after each cycle of experiments due to the size of the flume it would be cost-ineffective. The experiments went as follows:

- HE: 3h without drainage; 3h with D1; 3h undrained; 4h with D1 & D2; 3h with D3.
- Remoulding of the beach profile
- LE: 4h undrained; 5h with D1; 4h undrained; 6h with D1+D2.
- ME: 3h undrained; 3h with D1; 3h with D1&D2; 3h with D1&D2&D3; 3h with D3.

The D4 drain pipe was never considered since the wave run-up wasn't strong enough for the pipe to have any influence.

The results of the present experimental investigation reveal that for HE (High Energy) wave conditions the operation of the drains leads to a local stabilization effect near the cone of depression. When two drains (D1+D2) were operative, the stabilized area was double in size with respect to that induced by drain D1 only. In any case, the drainage system did not have an overall beneficial effect on the beach stabilization. The comparisons of relative vertical variation of bed level (Δz) in tests with drains D1 and D3 operating highlights that drainage acting in the saturated zone (under exit point), D1, and in the unsaturated zone (above the exit point), D3, have similar effects. For ME (Medium Energy) wave conditions drainage generated by a single drain in any position (D1, D2 or D3) seemed to be inadequate to produce a global stabilization effect. The simultaneous operation of drains D1 and D2 after 3 h of tests resulted in a good stabilization of the beach. The operation of drains D1, D2 and D3 simultaneously surprisingly triggered again the original erosive trend. The asymmetric increases in normal shear stresses due to the increase in infiltration and decrease in exfiltration could provide a heuristic explanation about the inefficacy of 3 drains working simultaneously. For LE (Low Energy) wave conditions, the drainage system with one (D1) or two (D1 and D2) drains generated an increase in the natural accretive trend of the beach. Under ME and LE tests, it was found that the local morphodynamic effect above D1 and D2 in the initial part of the swash zone was related to the lowering of the hydraulic head.

In conclusion, under HE wave conditions the drainage system seemed to be inadequate in giving any stabilization effect; the largest benefits were visible with the simultaneous operation of two adjacent drains, simulating a single drain of double diameter, for ME and LE tests, where a global beach stabilization and an increased accretion were, respectively, observed. The results obtained under ME conditions with 3 drains operative were uncertain.(Contestabile, Aristodemo, Vicinanza, & Ciavola, 2012)

4.3 Hard vs soft measures

Hard engineering methods provide a more effective way in protecting the coast against erosion and do this in a very quick way compared to soft engineering methods. Because most soft measures take several months to be used to the fullest of its possibilities, for example dune stabilization will only work effectively from the point where the vegetation is fully grown and a dune has formed. Although hard engineering methods can seriously alter the environment and natural coastal processes such as longshore drift, for example installing of a breakwater will interrupt the normal longshore drift leading to serious erosion or accretion on adjacent beaches. Soft engineering methods offer much less of a treat to the environment or coastal processes, although they are not entirely innocent either.

To select to best measurement between hard and soft solutions, the timeframe must be considered as well as the touristic appeal of the particular beach. If a solution must be reached on a short period of time, a hard solution would seem to be the best outcome. This depends on the touristic appeal on the beach, if it’s a beach which has a big touristic appeal, a soft solution would be the better option. For longer periods of time, it’s preferred to choose a soft solution over a hard one.

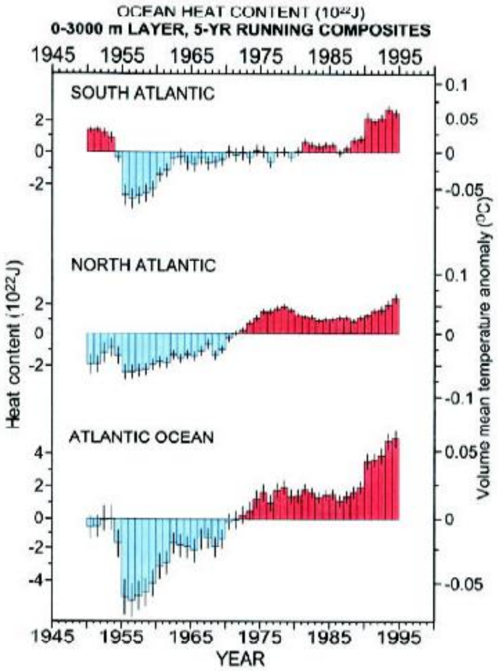
5 Climate change

5.1 Consequences of climate change

Observational records and climate projections provide abundant evidence that different shorelines are vulnerable and have the potential to be strongly impacted by climate change, with wide-ranging consequences for human societies and ecosystems.

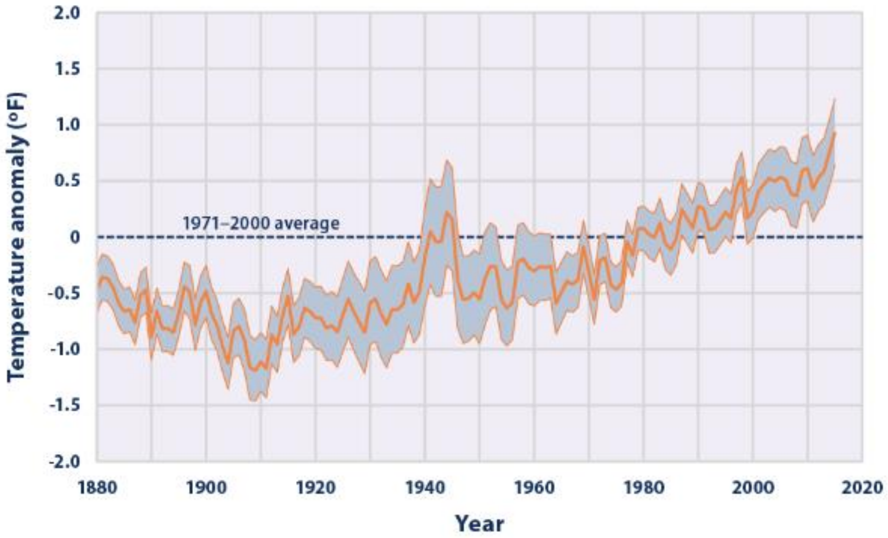
5.1.1 Rise of the ocean's surface temperature

Perhaps the most important and most frequently measured ocean parameter, in no small part due to its effect on the temperature of the overlying atmosphere, is sea surface temperature (SST). There are very few historical records known of sea surface temperature, the first real contribution came through *Levitus et al. (2000)*, who evaluated some five million profiles of ocean temperature taken over the period from 1948 to 1998 (figure 52). Over that period, their results indicate that the mean temperature of the oceans between 0 and 300 meters has increased by 0.31°C, in energy terms this translate to a mere 1 x 10^{23} joules of energy.



51 Surface temperature (°C) 1945-1998

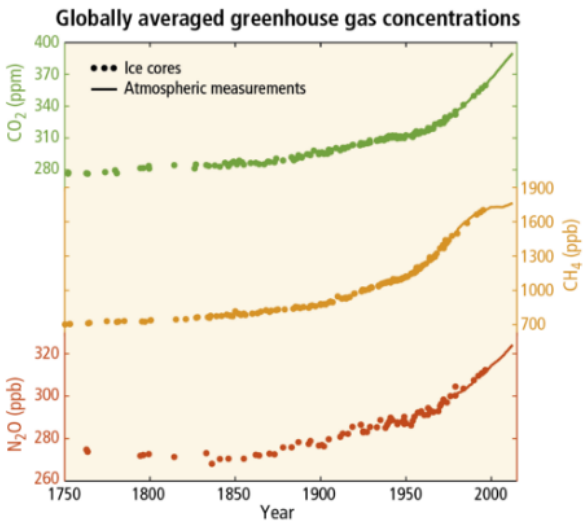
Due to recent evolutions in technology, the measurements can happen more accurately by use of remote sensing techniques, special equipped buoys and vessels across the oceans. Like the measurements done by NOAA (figure 53). Although the technological evolution makes the measurements more accurate, it doesn't mean that the sea surface temperature is the most accurate indicator for climate change. Because the heating or cooling of the oceans is still function of a couple of variables, being the exchange of heat between the ocean and atmosphere, the vertical stratification of the water column and the horizontal and vertical advection of heat. Next to this precipitation and wind play their role in modifying the ocean's temperature. So not all the gained or lost heat in the oceans and seas is due to climate change.



52 Average sea surface temperature (NOAA)

Both graphs, the historical one by *Levitus et al.* and the more recent one by NOAA, show a certain increase in the mean temperature. Temperature increased over the 20th century and continues to rise. From 1901 through 2012, temperatures rose at an average rate of 0.13°F (0.07°C) per decade. Sea surface temperatures have been higher during the past three decades than at any other time since NOAA's observations began in 1880. The rise of sea temperature is function of a wide range of physical processes, such as sensible heat flux, latent heat flux and long and short wave radiation and heat of fusion from ice formation. The differences in these heat fluxes, advection and mixing processes all help to determine the sea surface temperature.

A large part in the heating of the atmosphere is due to natural causes, but also human-induced effects come into play here. From these effects, the biggest influence comes from the emission of greenhouse gases such as carbon dioxide, methane and nitrous oxide. These heat-trapping gases assemble under the Earth's atmosphere and act as a shield which reflects all the heat the Earth wants to bounce back into space, and in the meantime, these gasses let through all the visible light and warmth coming from the sun. Figure 54 shows the concentrations.



53 Greenhouse gases (5th rapport IPCC)

This leads to a global increase of the surface temperature. Climate models have predicted that during the 21st century the global surface temperature is likely to rise a further 0.3 to 1.7 °C for their lowest emissions scenario and 2.6 to 4.8 °C for the highest emissions scenario.(Stocker et al., 2013) These findings have been recognized by the national science academies of the major industrialized nations and are not disputed by any scientific body of national or international standing. The mean sea temperature follows the temperature of the atmosphere, but with a certain delay.

Although the sea's temperature rises much slower than the atmosphere's temperature, marine ecosystems can be far more sensitive to even the most modest temperature change. Global warming caused by the greenhouse effect had a mere increase of 0.7 °C (NOAA) in the 20^{th} century, as opposed to a 0.1°C increase in the ocean's temperature. This warming can be felt from to surface till a depth of about 700 meters, it's in this area that most of the marine life thrives.

The rise of temperature and the increase of the atmosphere's carbon dioxide concentration, will lead to a down-grade of the existing coral reefs. This will have its repercussions on the reef-associated fisheries, tourism, coastal protection (Wells, Ravilious, & Corcoran, 2006), and people. Atmospheric carbon dioxide concentration is expected to exceed 500 parts per million and global temperatures to rise by at least 2°C by 2050 to 2100. These events could cause coral to bleach, which means the corals will expel the algae which gives colour to the coral reefs. When coral bleaches, it is not dead. Corals can survive a bleaching event, but they are under more stress and are subject to mortality. (Hoegh-Guldberg et al., 2007)

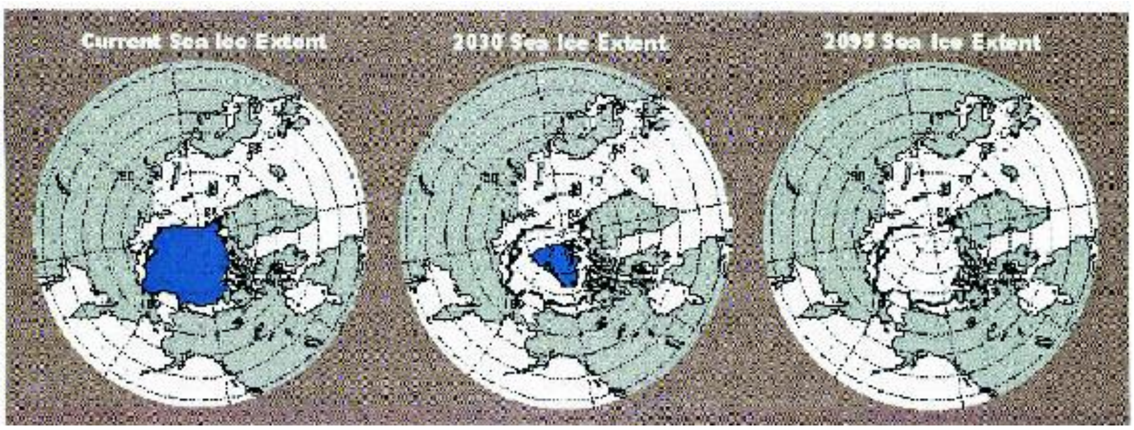
Another important phenomenon that can be linked to SST, is stratification of ocean layers. Because of global climate change, it is projected that the hydrologic cycle will be intensified, with increased precipitation and evaporation, and varying impacts to coastal runoff. Solar radiation and freshwater inputs result in density differences between surface and deeper waters that have important effects on the stability of the water column and on nutrient regeneration. The result is a layering or stratification with lower density water on the surface and higher density water below. Increases in precipitation and runoff combined with warmer surface temperatures increase the intensity of stratification. This has the potential for both positive and negative effects.

The development of well-defined stratified areas has been linked to the population structure of marine organisms. The development of retentive zones defined by stratified waters and associated fronts can be important in maintaining planktonic organisms within regions where the probability of survival is enhanced. However, strong stratification can impede mixing and nutrient regeneration, potentially resulting in a decrease in primary production in some areas. Increased temperatures and enhanced stratification have been implicated in a decline in production. In general, we can expect increased stratification in coastal locations that will be subject to increased runoff and river discharge. In contrast, in open ocean waters, it is likely that higher levels of evaporation could lead to increased salinity, reduced stratification and an increase in the mixed layer depth.

Next to its part in maintaining a planktonic sea population, SST also has its part to play in the world's sea ice extent. Which will decrease because of the heightened temperature of the sea water. Not only would a melting of these icecaps mean a further rising of the sea-level it would also mean that there would be ocean in places where there normally would have been ice. The reduction and potential loss of sea ice has enormous feedback implications for the climate system; ice and snow are highly reflective surfaces, returning 60 to 90% of the sun's incoming radiative heat back to outer space. By contrast, open oceans reflect only 10 to 20% of the sun's energy. Thus, the conversion of the Arctic ice cap to open ocean could greatly increase solar energy absorption, and act as a positive feedback to global warming.

Observations in the Arctic confirm the whole global warming theory, with ice extent shrinking by as much as 7% per decade over the last 20 years (Johannessen, 1999). By some other estimates, Arctic sea ice has been thinning (and subsequently decreasing in volume) by as much as 15% per decade (Rothrock, Yu, & Maykut, 1999). This does not only happen in the Arctic, but also in the Bering and Chuckchi seas. (Field et al., 2001)

Based on these observations, people have been working on certain numerical models to predict the annual decrease of the icecaps (figure 55). Such as the Canadian Climate Centre Model. (Field et al., 2001)

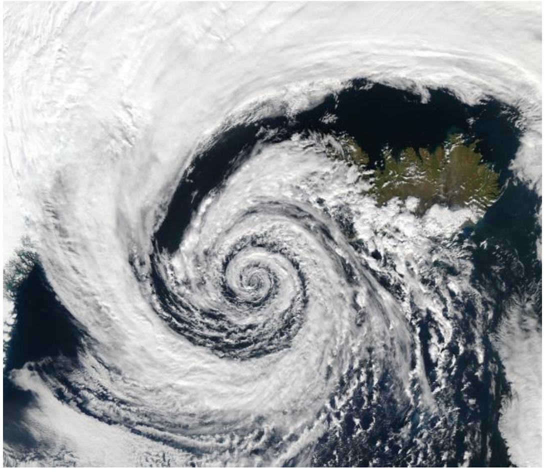


54 melting of the icecaps as predicted by the Canadian Climate Centre Model

5.1.2 Storm surges

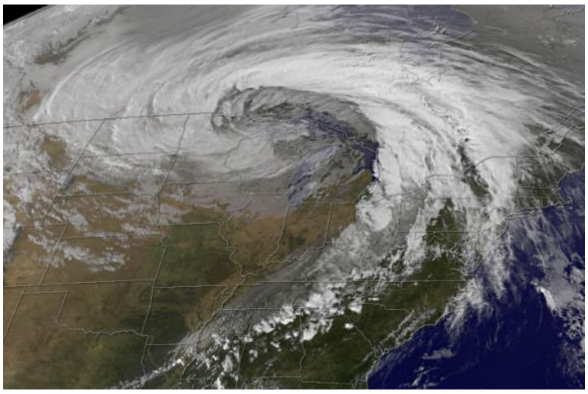
Another not to be underestimated risk that climate change carries with, is the undeniable rise of heavy storms the further climate change goes its way. Even more than earthquakes, storms have the capability to cause great damages which will result into costs. Coastal areas are especially at risk, as onshore winds accentuate tides and enhance storm surge, battering shorelines and damaging structures. Increasing sea levels extend the impact zone inland. In general, there are two different kinds of those heavy storms, known as hurricanes and extratropical storms.

A hurricane (figure 56), also known as tropical storm or cyclone, is a rapidly rotating storm system characterized by a low-pressure centre, a closed low-level atmospheric circulation, strong winds, and a spiral arrangement of thunderstorms that produce heavy rain. Tropical cyclones typically form over large bodies of relatively warm water. They derive their energy through the evaporation of water from the ocean surface, which ultimately recondenses into clouds and rain when moist air rises and cools to saturation. The strong rotating winds of a tropical cyclone are a result of the conservation of angular momentum imparted by the Earth's rotation as air flows inwards toward the axis of rotation. Thus, they rarely form within 5° of the equator. As they have warm cores, they will cause the most damage during the hot months. (Henderson-Sellers et al., 1998)

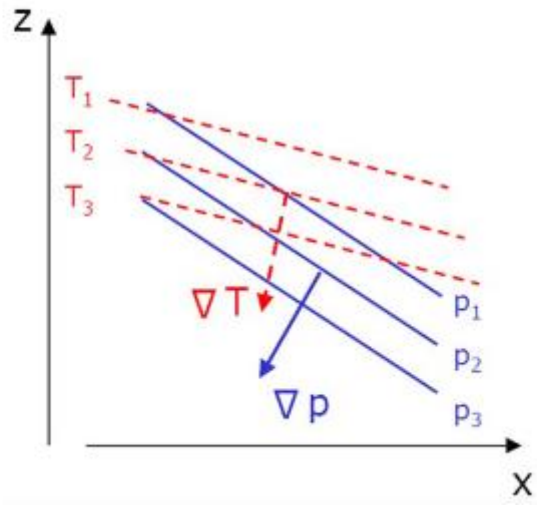


55 Hurricane

Extratropical cyclones (figure 57) are low-pressure areas which drive the weather over much of the Earth. They can produce anything from cloudiness and mild showers to heavy gales, thunderstorms, blizzards, and tornadoes. These types of cyclones are defined as large scale low pressure weather systems that occur in the middle latitudes of the Earth. In contrast with tropical cyclones, extratropical cyclones produce rapid changes in temperature and dew point along broad lines, called weather fronts, about the centre of the cyclone. Where hurricanes get their energy from the evaporation of water, extratropical storms get their energy from horizontal temperature contrasts, these horizontal temperature contrasts are often referred to as a baroclinic atmosphere (figure 58). (Vose et al., 2014) That is an atmosphere in which the density depends on both the temperature and the pressure, which means as much as that the lines of the same mean temperature, isotherms, cross with the lines of the same mean pressure, isobars. (Marshall & Plumb, 2008) In contrast to hurricanes they have cold cores, which means they will cause most of the damage during winter months.

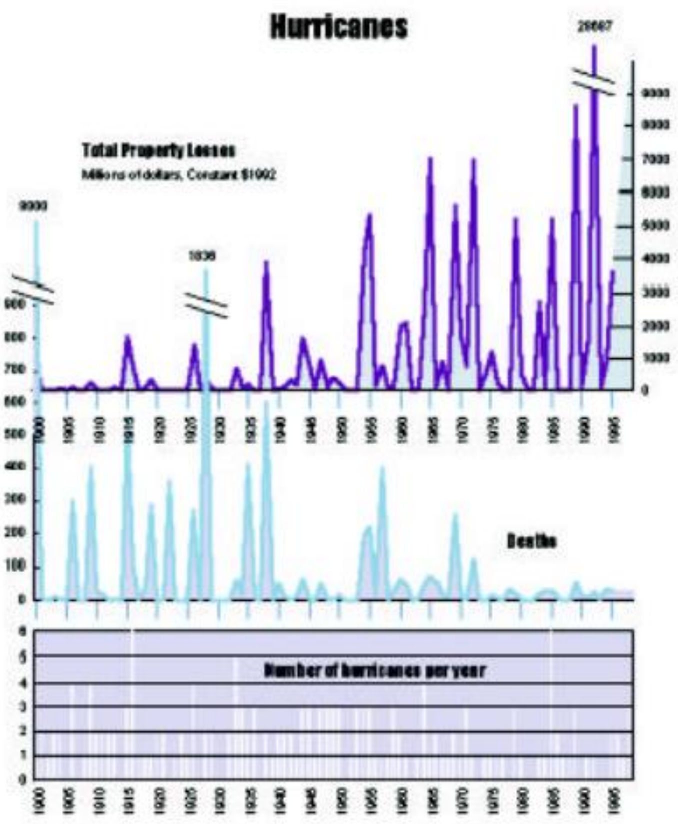


56 Extratropical storm



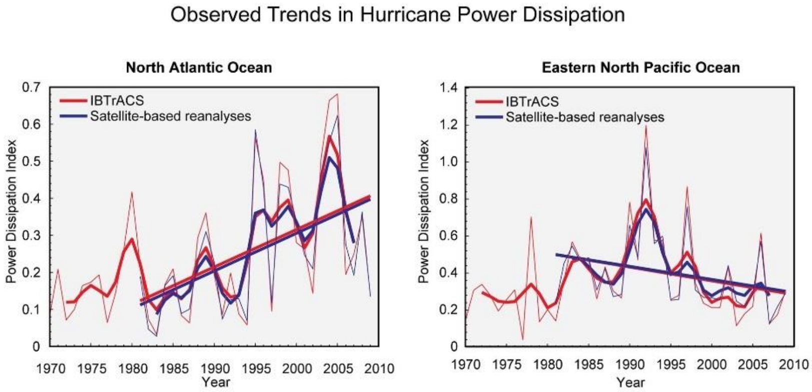
57 Baroclinic atmosphere

Coastal storms pose great risk for the coastal areas because a lot of the human population and activities are still situated at the coast or nearby the coastline, which is subject to storm flooding, wave forces, coastal erosion and hurricane force winds. Although there is a trend noticeable that the losses of life are decreasing throughout the years, this could be due to the higher level of protection and to the warning systems installed over most of the modern coastlines. Damages brought upon property are increasing, as the concentration of people and infrastructure along our coasts continues to increase. This is illustrated in Figure 59.



58 Damages to property and death toll caused by hurricanes in the US (NOAA)

Observed trends considering the number of hurricanes per year are most of the time very contradicting, as the accurate measure equipment became only available since 1980. Measurements taken before that period are mostly inaccurate. Still, with the advanced technology from this last period it is very difficult to notice conclusive trends of increase in the hurricane occurrence. As hurricane occurrence is function of location, that means that in a certain area hurricane occurrence can increase while in another area it can decrease. This happens in the Atlantic Ocean for example, as the north Atlantic Ocean shows a certain increase in occurrence and severity of the hurricanes while the measurements in the eastern north Atlantic Ocean show the exact opposite of this conclusion. (figure 60)



59 hurricane occurrence in the Atlantic Ocean

Although globally can be said that there is a substantial increase of hurricane activity since the early 1980s (Landsea & Franklin, 2013). This can be related to the increase of the sea surface temperature, as discussed in the paragraph above. Is it, though, much more complex than "a warmer surface leads to an increase in hurricane activity". How hurricanes develop also depends on how the local atmosphere responds to changes in local sea surface temperatures, and this atmospheric response depends critically on the cause of the change.(Emanuel & Sobel, 2013) For example, the atmosphere reacts differently when increases of the sea temperature happen locally than when they happen on a more global scale.

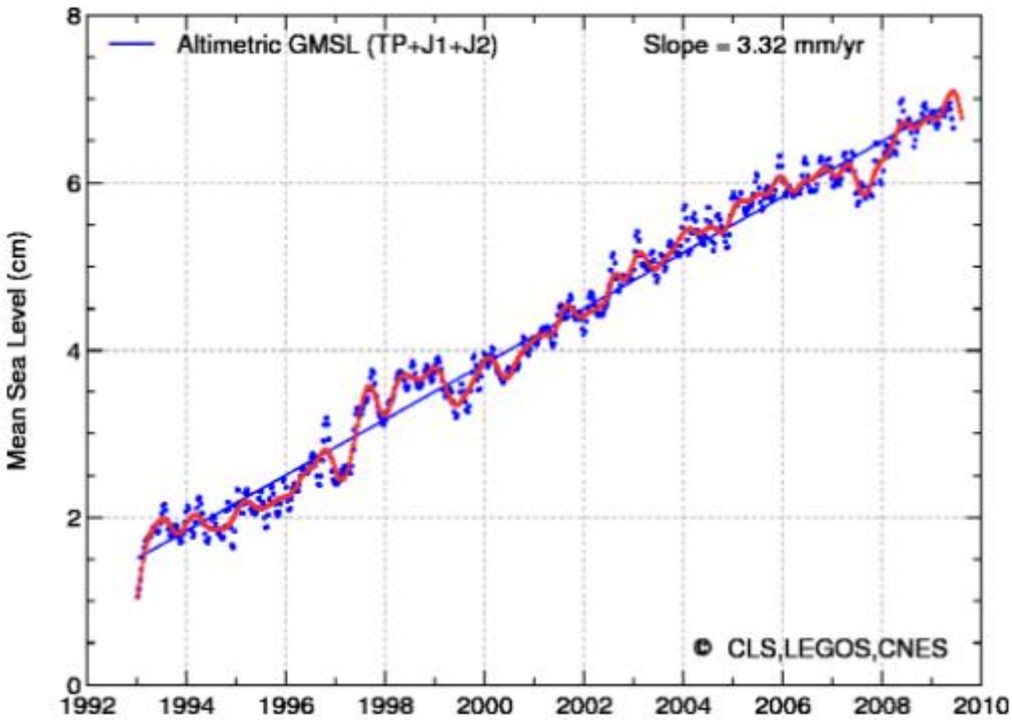
Concerning future trends different climate models have conclude that or a sea-surface warming of 2.2 °C, the simulation yielded storms that were more intense, showing a 5 to 12% increase in wind speed. For a moderate typhoon, these increases in wind speed translate to increases of 11 to 25% in the destructive power of winds. Similar percentage increases would be expected for other factors determining storm impact such as wave height and storm surge. Further, the simulation predicts large increases in rainfall, 28% greater than present. (Field et al., 2001)

Even if these storm magnitudes and increases of occurrence would stay the same, they would still cause more damage in the future due to the sea level rise. Because of the higher reach of waves on the beaches and barrier islands of the nation's coast, flooding and erosion damage will be expected to increase.

5.1.3 Sea level rise

Sea level rise is caused by two primary factors, the first is that due to the increase of the sea/ocean temperature the water expands which leads to the fact that the volume of water will increase the second is also due to the increase of the sea temperature which will melt the glaciers and ice sheets on the Earth's poles what will lead to an increase of the total water mass present around the globe.

Global mean sea level has been rising and there is high confidence that the rate of rise has increased between the mid-19th and the mid-20th centuries. The average rate was 1.7 ± 0.5 mm/ yr for the 20th century, 1.8 ± 0.5 mm/yr for 1961–2003, and 3.1 ± 0.7 mm/yr for 1993–2003. It is not known whether the higher rate in 1993–2003 is due to decadal variability or to an increase in the longer-term trend. Higher accuracy of measurements was reached, when the use of satellites became more common. One of these satellites is the Jason-1 which measured the rise of the sea level between 2001 and 2012, it replaced the TOPEX satellite and was later succeeded by Jason-2. (Figure 61)

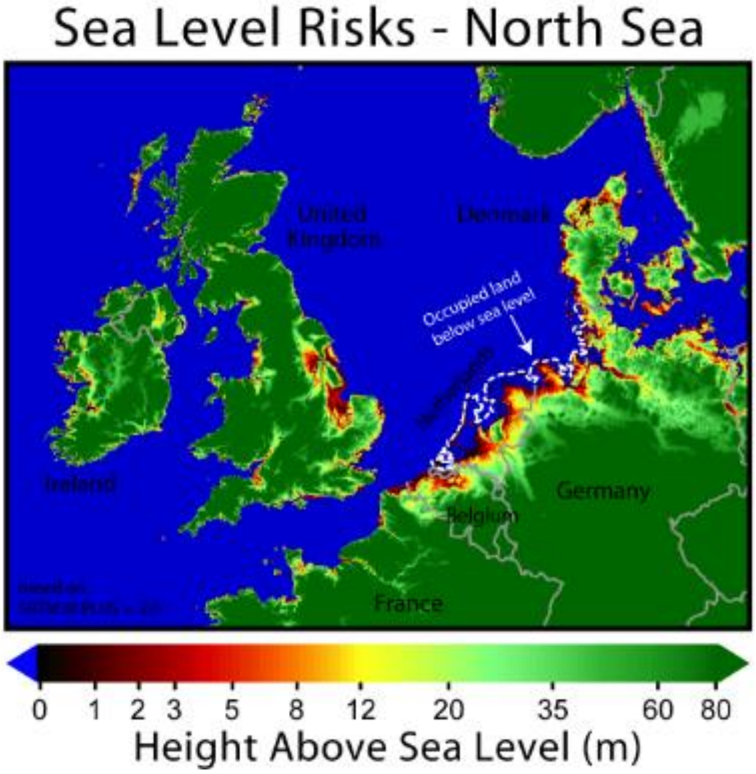


60 Sea level rise measured by satellites (Jason-1)

There are uncertainties in the estimates of the contributions to the long-term sea-level change. For the period 1993–2003, the contributions of thermal expansion (1.6 ± 0.5 mm/yr), mass loss from glaciers and ice caps (0.77 ± 0.22 mm/yr) and mass loss from the Greenland (0.21 ± 0.07 mm/yr) and Antarctic (0.21 ± 0.35 mm/yr) ice sheets totalled 2.8 ± 0.7 mm/yr. For this period, the sum of these climate contributions is consistent with the directly observed sea-level rise given above, within the observational uncertainties. For the longer period 1961–2003, the sum of the climate contributions is estimated to be smaller than the observed total sea-level rise; however, the observing system was less reliable prior to 1993. For both periods, the estimated contributions from thermal expansion and from glaciers/ice caps are larger than the contributions from the Greenland and Antarctic ice sheets. The large error bars for Antarctica mean that it is uncertain whether Antarctica has contributed positively or negatively to sea level. Increases in sea level are consistent with warming, and modelling studies suggest that overall it is very likely that the response to anthropogenic forcing contributed to sea-level rise during the latter half of the 20th century; however, the observational uncertainties, combined with a lack of suitable studies, mean that it is difficult to quantify the anthropogenic contribution.

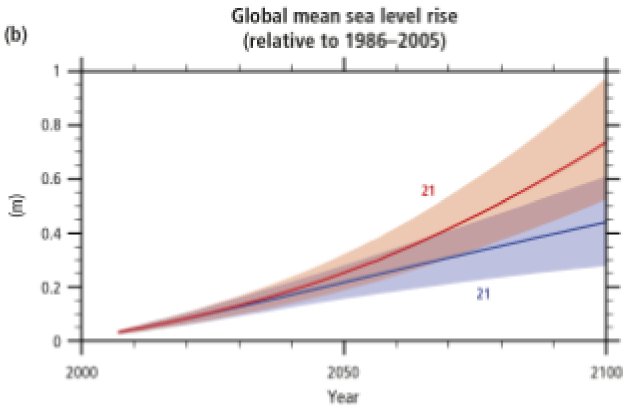
Rising sea level potentially affects coastal regions, but attribution is not always clear. Global increases in extreme high water levels since 1975 are related to both mean sea-level rise and large-scale inter-decadal climate variability.(Bates, Kundzewicz, Wu, & Palutikof, 2008)

Rising of the mean sea level will eventually cause the amount of beach to decrease. A slight increase in the sea level can have a big effect on the decrease of certain beaches, all this is dependable on the slopes of these beaches which will decide how much loss of beach there will be. Flatter beaches will disappear quicker than beaches that are more steep. The beaches are only the beginning here, as the sea level is predicted to rise even more the zone behind these beaches must be considered too. Following map (figure 62) shows the height above the North Sea.



61 Height above the mean sea level

As can be seen from the figure, the Netherlands are in great danger from even a slight increase of the mean sea level as part of the country is situated below the sea water level. This is possible due to the many dikes and dunes constructed to protect these zones. It’s exactly for these zones that it is very important to know how much the sea level will rise, that’s why the Intergovernmental Panel on Climate Change (IPCC) publishes several reports on how the climate evolves and how it will continue to do so in the future by use of different prediction models. In their fifth and last assessment report they've once again stated how much the sea level will rise until 2100 (figure 63). (IPCC, 2013)



62 mean sea level rise (IPCC)

As mentioned, climate change causes a whole lot of consequences of which certain have a direct impact on the coastal zones such as an increase of the sea level temperature, an increase of the sea level and an increase in severity and occurrence of storms. Coastal structures must be adapted to these consequences therefore the consequences must be 'translated' into risks by use of numerical models. Numerical models offer the possibility to design coastal structures within the limits of risk.

5.2 Rise of risk

As mentioned before, the whole world undergoes a change of climate which leads to a surge in severe storm, rising of the sea level and temperature. Many of the word's coastal zones have long been subject to the risk of severe storms and subsidence. There is a long history of investment in large-scale shore protection by public agencies and by private entities guarding facilities. Traditionally these defence structures have been based on 100 and 500-year flood maps, based on current climatology prepared by the Federal Emergency Management Agency (FEMA, 2014). These maps are used for the calculations for the damage caused by floods, and are guidelines for the design of the protection investments. There must be noted that protection from a certain event, like the 100-year returning flood, may be augmented by judgmental safety factors that consider economic damages and human lives at risk.

When climate change must be considered, the protection decisions become more complex due to the risks of rising sea level and hurricane destructive potential. The flood maps based on current climatology are no longer adequate to convey information for decisions.

Many coastal cities are proceeding to plan for increasing flood risks, as do public and private agencies but there is limited information available on different parameters changed by climate change. There have been made previous efforts to determine the effect of climate change in analysis of sea level rise and storm surge. The EU-funded DIVA model (Hinkel & Klein, 2009) has been developed for analysis of vulnerability and adaptation from regional to global levels, assuming scenarios of sea level rise. Also there have been conducted other experiments on particular protection investments such as those by (Condon & Sheng, 2012), (Yohe, Knee, &

Kirshen, 2011) and (Tsvetanov & Shah, 2013), although they're studies on particular defence structures they do consider the scenarios of sea level rise and/or fixed return periods of severe events. While these approaches produce useful pictures of expected increases in risk and of the adaptation challenge, they do not represent the risk over coming decades in a way that can support consideration of potential future adjustments when making today's investment choice. Determining when the needed reinforcement on the protection investment must be build is still a very complex question; building it too early may be wasteful, building it too late can lead to costly damages. In some cases, the cost for protection is so high, that abandonment may prove to be the better option.

In the following, a method is explained to estimate the rise of the risk. The method is an application of dynamic programming that can be used to analyse investments in adaptation today when the coastal risk is rising over coming decades in an uncertain way. When a certain investment is made today, possible additional investments must be considered as well, this will lead to the most cost-effective solution. With view to climate change the sequential nature of the decision must be considered.

The use of current flood maps leads to a cost ineffective solution for adaption of a protection investment, because the rise of risk due to climate change hasn't been considered. It's an assumption that the probability of different water levels would constant over time, which is wrong. That is why a new system of dynamic programming must be considered, where risk is believed to increase from decade to decade, which will lead to a more accurate and cost effective solution for the consideration of certain adaption to protection investments.

The decision to protect a certain area with an uncertain climate future, the area is being considered for a time span with finite horizon (for example ending in 2100), where choices are made sequentially in discrete time periods. The main goal of this system is to get a cost-effective solution, to minimize the present value of sum of the expected future costs on protection and flood damage. To reach this objective, Dynamic Programming starts in the last time period proceeding to the first, while taking into account Markov decision processes (Puterman, 1994), and is solved through backwards induction. The state of each time is defined as the level of protection in place, the decisions that could be taken from then on are either addition to the current state of protection or the leave the current situation like that. The cost at any time is a function of current and future states and actions, as well as flood damage.

Dynamic programming begins by calculating the cost for every possible state for the last time-period then iteratively moves his way till the first time-period. In each time-period the current state is assessed and decisions are computed considering the previously calculated cost. The process determines the least costs of each time-period whereupon the algorithm decides what the decision in each state that results in the cumulative least costs, discounted over time is. From these conclusions, some formulas can be deducted:

We assume a risk-neutral decision maker and define S_t to be the state, which is the height of protection at decade t because the grade of protection is equal to the height of the structure. Action $A_t(S_t)$ is defined to be the best action to take in decade t, yielding the lowest expected costs for decade t in state S_t, where the action in this context is the height of additional sea wall built in decade, t. Both S_t and A_t are real positive numbers. The cost $C_t(S_t, A_t)$ is defined to be the expected costs during decade t given state S_t and action A_t. It is a function of the expected costs of damage due to flooding, the costs of building new protection and maintaining existing protection. The function $V_t(S_t)$ returns the action A_t that produces the lowest cost, V_t in state St. For each time period, we calculate the best action and lowest value such that

$$A_t(S_t) = \; min_{A_t}[C_t(S_t, A_t) + \frac{1}{(1+r)^{10}} V_{t+1}(S_t + A_t)]$$

and

$$V_t(S_t) = \; min_{A_t}[C_t(S_t, A_t) + \frac{1}{(1+r)^{10}} V_{t+1}(S_t + A_t)]$$

Where r is the discount rate. $C_t(S_t, A_t)$ is defined as

$$C_t(S_t, A_t) = E(Damage|S_t, A_t) + c_m * m * S_t + c_b * m * A_t$$

Where $E(Damage|S_t, A_t)$ is the expected damage during the time-periods, in which the designperiod is divided, given the state and decisions made at the beginning of that period, c_m is the cost of maintaining one meter-km of sea wall, c_b is the cost of building an additional meter-km of sea wall, and m is the length of the sea wall in km. Since we choose to look at decisions made every decade, each of these costs are accumulated over the decade, at discount rate r. Then, by iterating backwards over time, we can derive the optimal levee height for each state.

The estimation of expected damage, $E(Damage|S_t, A_t)$, involves a calculation of the probability that the annual maximum water level above today's mean sea level, X, during decade t will exceed the height of provided protection, $S_t + A_t$, in a facility located n meters above today's mean sea level, $P_t(X > \xi + S_t + A_t)$. If we assume damage to be a function of flood height, then to derive the expected damages over the t^{th} decade we calculate the sum of expected damages for each year (discounted to the beginning of the decade):

$$E(Damage|S_t, A_t) = \sum_{i=1}^{10} \frac{1}{(1+r)^{i-1}} \int_{x=\xi+S_t+A_t}^{\infty} D(x-\xi) * P_t(X > x) dx$$

To estimate the flood risk, $P_t(X > x)$, we require information on uncertain sea level rise, storm surge and subsidence at each time period.

(Lickley, Lin, & Jacoby, 2014)

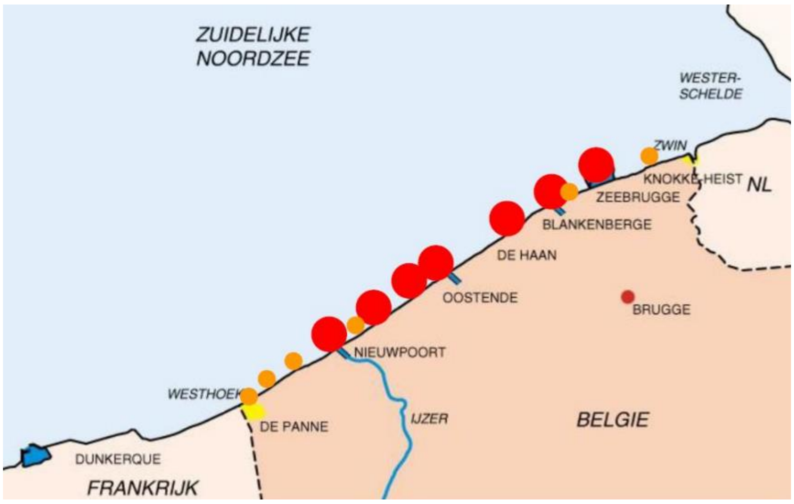
6 Practical application

6.1 Introducing the problem

Although the Belgian coast has a mere distance of 67 km, it's still a very frequently visited place with lots of inhabitants, with about 450000 inhabitants, and furthermore it boasts a lot of important economic, recreational, industrial and nature zones. Knowing this, the Belgian coast is still very susceptible for flooding, this will only become worse in the future because of climate change which will cause the sea level to rise and the occurrence and severity of storm to increase.

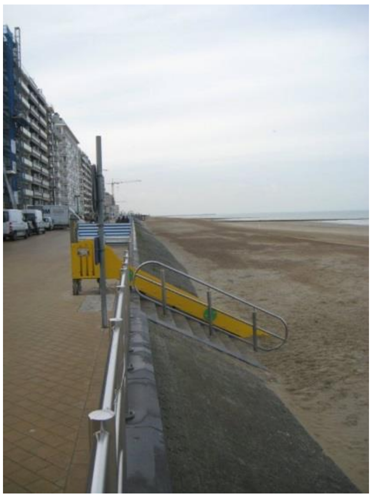
Before real measures were taken to protect the Belgian coast against the climate change and it consequences the Belgian coast itself consisted of erosive, sandy beaches that were subject to longshore drift going from France to the Netherlands. Besides this there was also a tidal variation of about 4 meters which will led to cross-shore drift. These two phenomena would have caused for the whole coast to erode; they were accounted for by the construction of 171 groynes and 38 kilometres of sea dikes which serve as hard measures to counteract the erosion.

In 2010 the Belgian government decided to assign a study concerning the enforcements of the weak points along the Belgian coastline (figure 64), this resulted in the integrated coastal safety plan (Dutch: "geïntegreerd kustveiligheidsplan"). It is to protect the North Sea coast in the long term against flooding, considering the rising of the sea level until 2050, to make the coastal area an attractive and natural region.(Mertens, Wolf, & Verwaest, 2008)



63 weak points along the Belgian coast

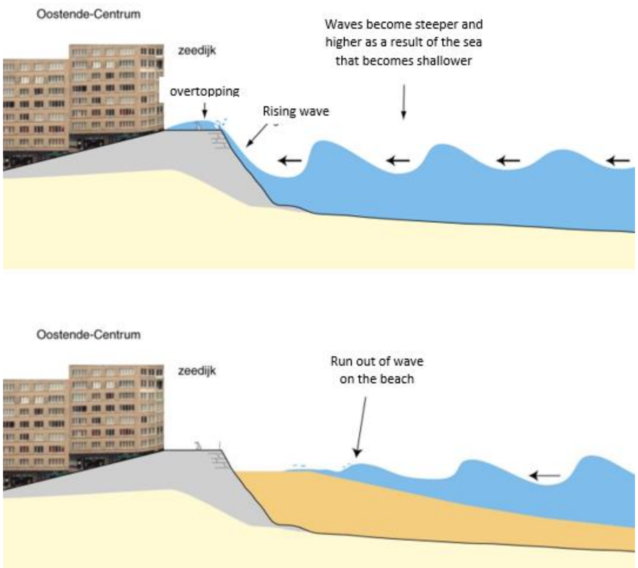
The policy of the safety plan is based on the principles of integrated coastal zone management: all the seawalls along the Flemish coast must withstand to a 1000-year storm and this must be accomplished while in equilibrium with environmental, social, economic and recreational goals. To consider economical and practical limitations, a certain quantity of skipping water per unit of time is allowed. The permissible overtopping flow rate must be as such that the global and local stability of the protection construction is assured and that the inland is protected from flooding.



64 Typical example of a Belgian sea dike

From preliminary results of previous researches at the Belgian coast, it has been found that about 30 percent of the Flemish coast is insufficiently protected against the 1000-year storm. Every vulnerable area must be equipped with the necessary measures. In high risk bathing zones, mostly the beach will be made higher and wider and extra sand dunes will be constructed. In the ports, seawalls can be constructed along the port channels, quay areas can be raised and barriers for floods are considered. Wherever needed sea dikes are being strengthened with permanent or mobile storm walls.

The maritime policy that is being used by the government in recent times differs from what was being used in the past. One will be less tempted to go back to hard measurements as a coastal protection solution as they destroy the natural appeal of the beaches, the new solutions consider not only the protection aspect but also consider how it will all look. That's why measures are built from the following thought: if the sea rises, the beach must rise as well. This means a widening and an increase of the beaches, as well as making the dunes more robust so they well be less susceptible to erosion. Mostly this is done by the suppletion of sand on beaches. Resulting in a recovery of the natural coastal dynamics. (Figure 66)



65 hard vs soft measures

One of the points highlighted by the coastal safety masterplan is Wenduine, a small coastal town along the shoreline. It’s one of the weakest points along that shoreline due to its low-lying foreshore, proximity to an urban area, low lying hinterland behind the sea dike, sea dike that offers too little protection and the promenade.

6.2 Location

66 location relative to Belgium

67 location relative to the Belgian coastline

68 satellite image of Wenduine

Wenduine is a small town, in the province of West-Flanders in Belgium, it's located on the shoreline between De Haan and Blankenberge. It is one of the towns that make up the entire Belgian coastline, the Belgian coastline has an orientation of 320° which makes it a very favourable position towards the mostly dangerous north and northwest winds. Another favourable property of the Belgian coast is that it is in 'the shadow' of the English island, which will act as a buffer for the heavy storms.

6.3 Measures

Wenduine was specified by the integrated coastal safety plan as one of the weak points which needed strengthening. Because of the following factors:

- Low level of existing crest
  With the existing crest height, an overtopping flow of 342 l/s/m was calculated, which meant that the security level was less than 50 years.
- Shallow foreshore
- Close proximity to urban area
- Low lying hinterland
- High recreational and touristic value of beach and promenade

Suiting measures needed to be taken to make Wenduine and its beach more resistible to future storms with respect to the 1000-year storm, while also the effects of the "+8m superstorm" were examined. The storm has the following overtopping rates in because they should happen (table 2). The overtopping rates have been estimated or extrapolated from tests conducted on physical scale models.

Table 1. Existing and tolerable mean wave overtopping discharge at Wenduine

Storm Event	Existing Overtopping Discharge	Tolerable Discharge Criteria
1000 Year ARI	q ~ 70 l/s/m	q < 1 l/s/m
+8m Superstorm	q ~ 500 l/s/m *	q < 100 l/s/m

* extrapolated value

2 overtopping rates

As can be seen in the table, the limits are exceeded for both storms this means that the existing seawall is not sufficient to control the area during these types of storms.

The previous situation consisted of a 25m wide sea dike with a sand beach of about 500m as shown in Figure 35 and would have been flooded considering the 1000-year storm period. That beach is erosive, as more the amount of sand that gets washed away is greater than the amount of sand that gets deposited by sea currents or wind transport.

69 previous sea dike, Wenduine

6.3.1 Beach nourishment

After Wenduine was considered a weak point by the integrated coastal safety plan it was decided to strengthen the coast by means of sand suppletion. It was decided to make the beach higher and wider by spraying it with sand. Sand suppletion is a soft measure method by which the beach is heightened and widened by adding sand to it, this new sand comes from the dredging of harbours, fairways and sandbanks. In more recent times, the sand used for suppletion purposes along the Belgian coastline comes from Sierra Ventana, which is a shallow region in front of Zeebrugge with a plethora of dredging material.

In 2012 the beach was supplied with extra sand, which increased the normal beach level. At some point the dry beach was widened by 35 to 55 meters. During the suppletion about 717.000 m^3 of sand was used. Inherent to the sand suppletion method is that the added amount of sand will eventually wash away or erode way due to winds, the result is that the whole process must be redone. This is exactly what happened with the sand suppletion in front of the beaches of Wenduine after the sand suppletion of 2012. The whole process of sand suppletion was redone in 2016 and two months later the Belgian coast was hit by the storm Dieter, which washed away about 1 million m^3 of sand. (Figure 71)

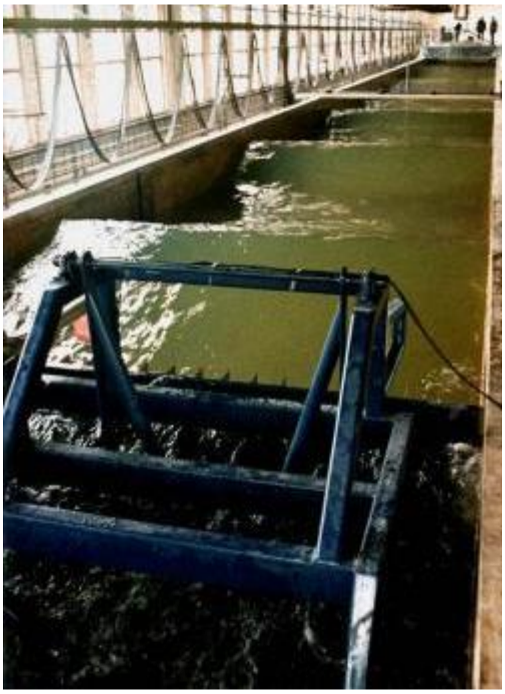
70 Damages caused by storm Dieter

Sand suppletion is a process which needs a lot of maintenance as can be noticed from the above-mentioned situation and therefore it costs a lot of money. All negative things aside, it is an effective method with a lot of respect for the ecosystem and the overall appearance of the beach. Use of the sand suppletion defence method doesn't negatively influence the appearance in any way.

6.3.2 Seawall

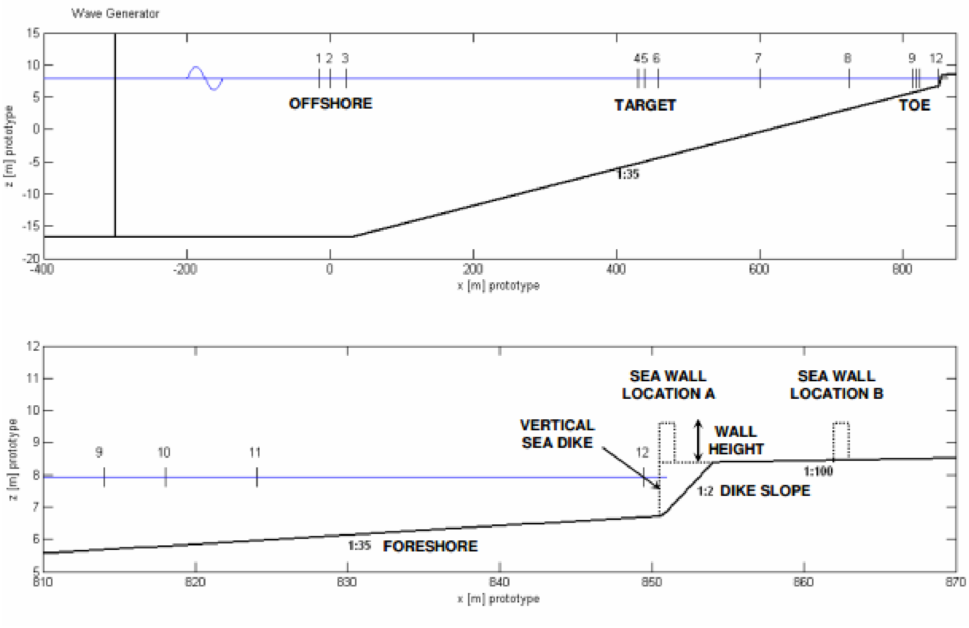
Beach nourishment alone wouldn't be sufficient to protect the beach against the two design storms (1000-year storm and the +8m superstorm), in combination with sand suppletion a new wave return wall would have to be constructed to reduce the risk. These measures must be sustainable as they must offer sufficient protection until 2050. While there are also some limitations that must be considered, concerning the social, environmental and economic properties have the new construction, which means that the construction can't exceed a certain height for community acceptance while it must withstand the overtopping hazards by limiting the discharge rates as prescribed by the coastal safety masterplan.

Design of the new sea wall was led by the information coming from tests on a 1:25 scale model in the Flanders Hydraulics Research laboratory, where experiments to determine the average overtopping flow rate, individual overtopping flow rates and to estimate forces resulting from the waves have been conducted on a smooth dike that could be expanded with a seawall, parapet or a bank. Test were conducted in the large wave flume of the laboratory with the following dimensions: 70 m long, 1.4 m high and 4 m wide (Figure 72). The wave flume is equipped with a piston type wave generator. The stroke length of the baffle is 60 cm allowing for a wave height of 65 cm to be generated at a depth of 90 cm.



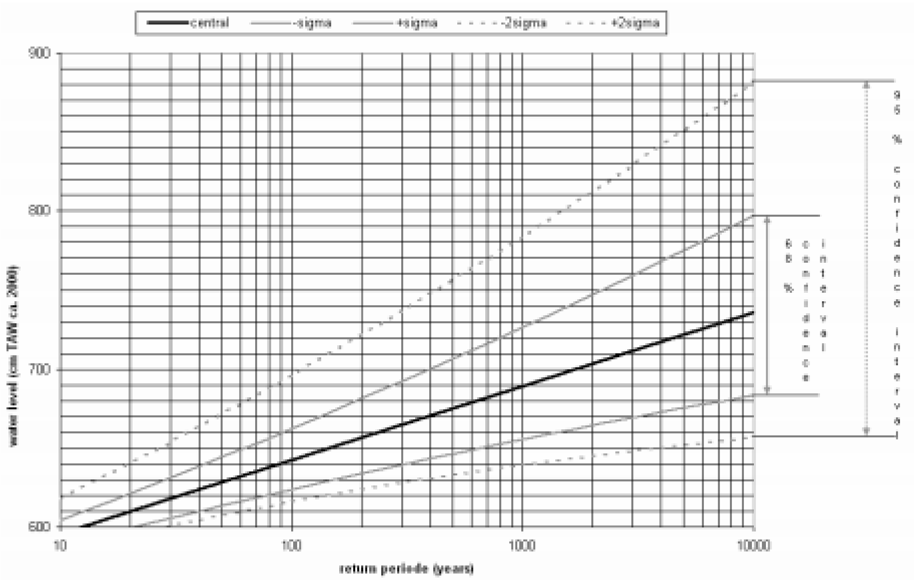
71 large wave flume

The scale model is then constructed inside the wave flume. A model version of the foreshore is constructed with a slope of 1:35 from smooth concrete which is a simplified presentation from what the foreshore would look like after a 1000-year storm event based on the PhD thesis of Steetzel (Steetzel, 1990). At the end of this foreshore, the actual dike is modelled from laminated timber. It's the sea dike that will do most of the flooding prevention so this is the part were different models will be tested. This is also presented in the scheme below (figure 73).



72 model in large wave flume

For the scaling down of the waves, a dataset is used which describes a time series of 76 years (1925-2000) of high water levels along the Belgian coast measured at Ostend. Statistical analysis is conducted on these values to estimate the confidence intervals (figure 74). (Verwaest et al., 2009)



73 surge levels with confidence intervals

From this graph, water levels during storm surges can be determined. They are measured in TAW which is a standard method in Belgium to measure heights (TAW = Tweede Algemene Waterpassing). Furthermore, these water levels can be scaled down using the scaling factor of 1:25 which gives the following values (table 3).

Table 2. Incident wave boundary conditions at "target" wave gauge array								
Storm Event	Prototype Scale				Model Scale (1:25)			
	SWL (m TAW)	H_{m0} (m)	$T_{m-1,0}$ (s)	T_p (s)	Model depth (m)	H_{m0} (m)	$T_{m-1,0}$ (s)	T_p (s)
"+8.0 m Superstorm"	7.94	4.97	9.00	12.75	0.99	0.20	1.80	2.55
1000 Year Storm	6.84	4.75	8.60	11.70	0.94	0.19	1.72	2.34

3 wave boundary conditions

These wave boundary conditions are reproduced at the 5m TAW boundary located at the "target" gauges.

Different wave gauges are installed on the points indicated by the numbers on the scheme (figure 39) and are calibrated on 20Hz which allows them to determine the wave energy for incoming as well as reflecting waves. Wave overtopping discharge is measured in the overtopping collection box where a micro pulse meter is installed which could measure the mean and the instantaneous overtopping rates.

As mentioned before different forms of seawalls are tested, they're shown in table 4.

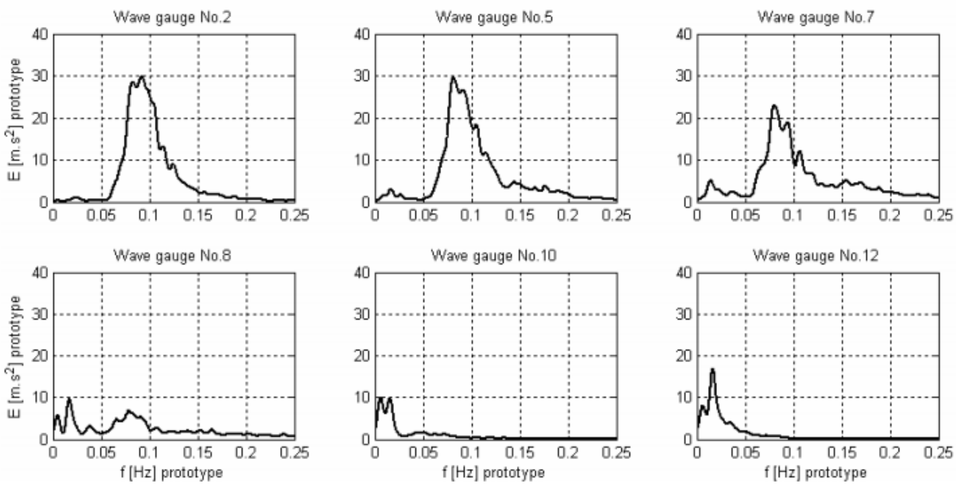
Table 3. Matrix of test parameters				
Variable	**Parameters Tested**			
Dike slope	1:2 slope	Vertical	-	-
Wall height	0 m	0.6 m	1.2 m	1.8 m
Wall location	No walls	A	B	A & B (SWB)
Wall geometry	Vertical wall	Parapet	-	-

4 different forms of defence structures

The first form that's tested is the current situation, being a normal sea dike with a 1:2 slope. Following this a vertical seawall is investigated with a parapetwall on top which has a nose angle of 50 degrees. Furthermore, the concept of a stilling wave basin is tested which consists of two separate seawalls with an offset of about 10metres. A basin is created between the two walls with holes in the seaward wall to allow drainage of the basin.

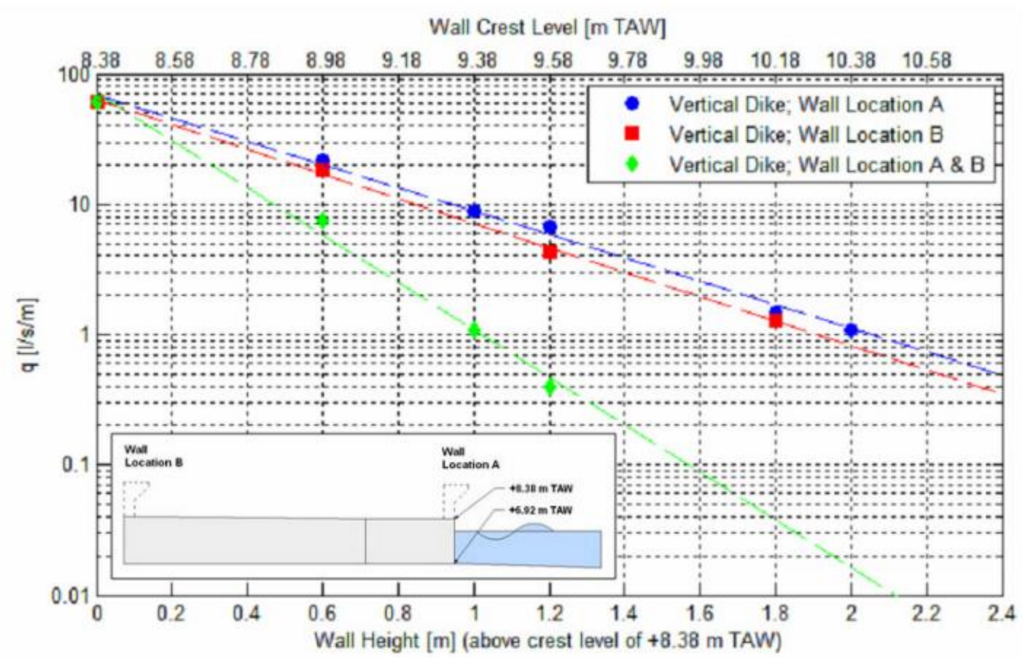
Results

The results show a resemblance while testing all the different defence mechanisms, that is the influence of the shallow foreshore which visibly weakens the force of the incident waves. Looking at graphs who measure energy in function of frequencies the frequency peak flattens out the further the wave finds itself on the foreshore where the energy shifts to form infragravity waves. This is clearly shown in the graphs on figure 41. The gauge locations are shown in figure 75.

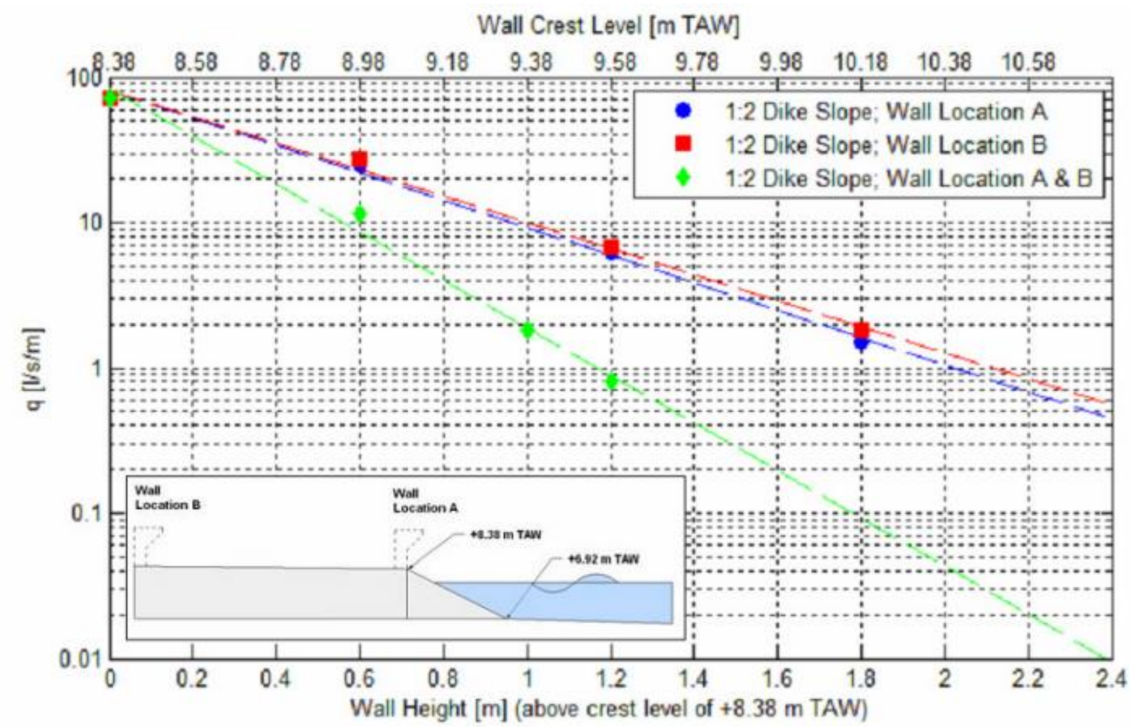


74 shift of the frequency spectra

The waves get shifted to infragravity waves but they still reach the main defence structure, that's why it must be determined which of the possible defence structures must be used. Physical model test is conducted on the vertical wall and on the sloped (1:2) wall, both equipped with the parapet wall. Figure 76 & 77 show the mean overtopping rate in function of the wall height. Different lines are plotted in the graphs because of the different locations of these parapetwall on the sea dike, different locations have been predefined with location A on the edge of the sea dike and location B 8 meters landinward of that edge. The edge of the sea dike finds itself at +6.92m TAW, which is the constant value for the eroded beach, while the top of the dike reaches +8.38m TAW without the parapetwalls.



75 wave overtopping in function of wall height & location – vertical dike



76 wave overtopping in function of wall height & location - sloped dike

The parapetwall on location B has a better result, that means a smaller overtopping rate, than the parapetwall on location A with the vertical wall. This is different with the sloped dike where the parapetwall on location A has the better results. What's the same with both models is the stilling wave basin which has by far the best results, it is more efficient than the single wall designs. Considering the social aspects of the dike design, it is obligated to choose the stilling wave basin model as for a given overtopping rate limit it requires lower walls. For example, given the 1000-year storm limit of 1 l/s/m it requires wall of 1m and 1.2m, for a vertical dike and a sloped dike respectively. While the single wall design requires at least 1.8m high walls.

Vertical sea dike with parapet walls have the bests results and are tested for the 1000-year storm with a limit of 1 l/s/m and for the +8m superstorm with a limit of 100 l/s/m. Table 5 gives the crest levels considering these boundary conditions.

Table 4. Wall crest levels required to meet tolerable mean overtopping discharge standards

Standard of Protection	Storm Event	Wall Crest Level [m TAW]					
		Dike with vertical seaward slope and wall(s) at location:			Dike with 1:2 seaward slope and wall(s) at location:		
		A	B	A & B	A	B	A & B
q < 1 l/s/m	1000 Year ARI	+10.42 (2.05)	+10.29 (1.91)	+9.40 (1.02)	+10.40 (2.02)	+10.49 (2.11)	+9.55 (1.17)
q < 100 l/s/m	+8.0 m Superstorm	+9.79 (1.41)	+9.53 (1.15)	+9.29 (0.91)	NA	+9.59 (1.21)	NA

Note: Wall heights, relative to minimum dike crest level of +8.38 m TAW, are stated in brackets below crest levels

5 required crest levels

As mentioned before the design of the dike is not only influenced by engineering safety but is also subject to architecture/urban-planning, and community considerations. Which is why the height of the parapetwalls can't be chosen freely, table 4 shows that the lowest walls possible while still meeting the safety requirements is about 1 meter high. Even this is too high as meetings with stakeholders have shown. They want a wall that's not higher than 0.7m.

To solve this contradiction the height of the parapetwall, 0.7m, is used as a boundary condition itself with the only variable being the width of the sea dike. This is widened until the expected values are reached for a parapetwall height of 0.7m.

6.4 Cost-benefit analysis

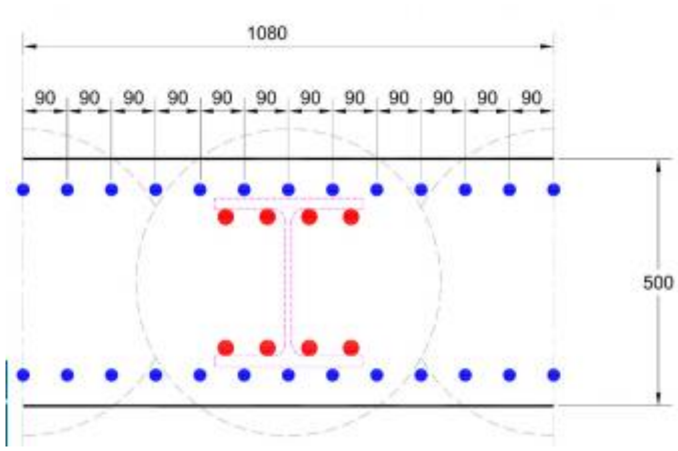
An analysis of the costs and benefits of the proposed measures is interesting to provide a conclusive answer as to which measure would be the best solution. The analysis conducted herein considered social, economic, and environmental implications of the options under consideration. All of the measures have some kind of influence on these different matters. All of the options represent loss to some party – loss of the greater part of the beach, in some scenarios; loss of private property in others; loss of sensitive habitat; or lifestyle for others.

6.4.1 Costs

Costs caused by sea wall construction can be divided into two different categories: direct and indirect costs. The direct costs include all the costs that are needed for the construction and preservation of the structure. As the structure is designed for a lifetime of 50 years, a proper working has to be guaranteed. A close monitoring of the structure is necessary, for a sea wall this will mostly consist of regularly checking the level of the adjacent beach. Because one of the main reasons of sea wall failure is the scouring at the toe of the structure. It should be made sure that the toe is covered at all times. This procedure leads to yearly maintenance and monitoring costs. Indirect costs of a sea wall include a reduction of the area's touristic appeal during the construction phase.

The construction costs of the new construction is composed of the following elements:

- Demolition of the existing sea dike
- Sand suppletion for the widening of the dike
- Excavation works
- Fencing of the temporary pile wall
- Secant piles
- Steel HEB340 profile in the seaward sea wall (figure 77)
- Reinforcement cages in the landward sea wall
- Placement of the sea walls
- Payment of the workers
- Rent of the necessary machines



77 Steel HEB340 profiles in the seaward sea wall

The whole construction of the seawall and the widening of the sea dike takes about 8 months, with the planning of the different tasks as follows, with the periods divided in weeks:

ACTIVITY	PLAN BEGIN	PLAN DUUR	PERIODS
Demolition	1	4	32
Sand suppletion	5	8	32
Excavation + fencing of the temporary pile wall	13	5	40
secant piles + HEB340 profile + reinforcement cages	18	10	32
Sea wall elements	21	10	32
Finishing	31	4	32
Total			168

78 Planning of the project

The working hours of the workers are given in the figure (figure 78) above, with use of 8 working hours a day per worker, which equals in a total of 5376 working hours for all the workers together. At all times there was a supervisor present at the working site, who will be paid a bit more than the workers so they are treated as two different expenses.

The construction of the secant piles requires a lot of machinery, being the piling rig, silos to store the cement and a pump system to transport to cement to the piling rig. This machinery has to reach the construction site and that costs money, it will cost around 4037.77 €. The cost for the construction of the piles themselves is calculated with use of a diameter of 0.62m a pile and there are around 750 piles with a length of 7.5m and 750 piles with a length of 2.5m, which results in a cost of € 269865 by use of €39.98 per meter of secant pile.

Demolition of the existing sea dike consists of the destroying of the existing concrete and stone elements and transporting them away. A crane, a dozer and a couple of pneumatic hammers will be used for this, with respective cost of €41.91/h, €59/h and €6/h if you use two hammers. The actual demolition costs 11820 € a square meter. All the debris from the demolition works has to be transported to places to will do the recycling, which is rated at € 8.99/m³.

The temporary fencing consists of metal sheets that are fenced into to ground to offer a better condition to make the secant piles. Transport of the fencing machine will cost 4037.77 € and the actual fencing is charged per square meters and is equal to 142.7 €/m².

The sea walls (landward and seaward) are both prefabrications, and will cost around 100 € per meter. In this price the transport and placement is already included.

The following table gives all the prices of the construction and widening of the sea dike.

Task	Unit	Quantity	Price	Product
Demolition	m^2	6000	1.97 €/m^2	11820 €
Transport debris	m^3	600	8.99 €/m^3	5394 €
Sand suppletion	m^3	4800	16.80 €/m^3	80640 €
Transport sand	m^3	4800	8.99 €/m^3	43152 €
Excavation works	m^3	1200	21.74 €/m^3	26088 €
Transport sand	m^3	1200	8.99 €/m^3	10788 €
Fencing	m^2	4800	142.7 €/m^2	684960 €
Transport machine	Unit	1	4037.77 €/unit	4037.77 €
Concrete secant piles	m	6750	39.98 €/m	269865 €
Transport machine	Unit	1	4037.77 €/unit	4037.77 €
HEB340 profiles	m	2250	141 €/m	317250 €
Reinforcement cages	Per length of 3m	300	33 €/piece	9900 €
Sea walls	m	1200	176.5 €/m	211800 €
Workers	h	5376	18.19 €/h	97789.44 €
Supervisor	h	1344	20.9 €/h	28089.6 €
Machines	-	-	-	90860.4 €
Total	-	-	-	**1.671.657 €**

6 Costs of the new sea dike

The entire construction cost lies around 1.7 million euros, this is quite low compared to values estimated by Arcadis and Fugro (2006) who found an estimated value of 5.82 m€/km of North Sea dike . This can be explained due to the fact that this is very roughly estimated value and doesn't consider maintenance costs to the machines, doesn't consider security measurements, doesn't consider influences on the public traffic, doesn't consider the setting up of the construction site and there should also be noted that this only contains the cost for the construction of the sea wall only, the construction of the new boulevard wasn't considered as well as the investigation of the ground layers.

The maintenance and costs are estimated to be around €50000 a year by Arcadis and Fugro (2006). Which will come down to a mere € 2500000 after its entire lifespan. So the entire costs for the entire lifespan of the sea wall will be around €4171657.

Apart from these normal construction costs to build a sea wall and also apart from the physical size and shape of the construction there are some other issues that have a profound influence on the cost of the whole structure:

- Accessibility of the site, which is here pretty good with 4 big streets towards the dike allowing a good and fast transport of equipment and materials.
- Weather. Cold weather has a tendency to slow the works down leading to higher costs.
- Quality of the used materials. Preferably they are of a high quality, as they will positively influence the quality of the whole structure and possibly prolong the lifetime of the construction.

- Size. The construction is pretty big, which will allow for materials to be bought in bulk what will decrease most of the prices.
- Precast. For the construction of the parapetwalls there's chosen for a precast version, which will positively influence the speed and quality of the works.

6.4.2 Benefits

To analyze the benefits associated with the construction of a sea wall, a comparison with a Business as Usual scenario has to be made, where no measures are taken and thus will suffer from damage caused by the overtopping of the sea wall due to high energy waves leading to costs. The caused damage will translate itself into two different costs: direct costs and indirect costs. Direct costs are costs caused by demolition of properties, the government would show a reactive approach to properties that will have become inhabitable. For indirect costs, it is assumed that owners of properties that have become inhabitable will move out and therefore the household income of these properties would stop. Further indirect costs include lost value of the affected town and properties, which would also have a negative impact on the revenues generated by tourism, which has a yearly revenue of 2667.7 million euros (Westtoer) along the entire Belgian coast.

Most of the previous costs can be limited or prevented by installing a decent coastal defense structure. Investment in an adequate sea wall will lead to a decrease of most of the risks compared to the business as usual strategy. The big investment is justified by all the benefits caused by the structure. It serves as a last measure barrier between the beach and the hinterland and therefore protects the town of Wenduine against severe waves caused by storm surges. Wenduine has a population density of 825 inhabitants per square kilometer, which is a very dense populated area with the average population density of Belgium as much as 368 inhabitants per km². This population is mostly concentrated directly to the beach in big apartment buildings. These buildings are located on the exact place where the dunes would naturally form, so there is no real protection in the case of high energy waves during storm surges. A well-constructed sea wall is a necessity to guarantee the safety of the people living there and to help preserve the value of the hinterland by preventing demolition or flooding. By doing this the important aspect of coastal tourism is kept intact.

To further amplify the benefits of the sea wall, the social aspect has been firmly considered as well. With the new design providing a wider boulevard, which offers more space for coastal commerce such as restaurants, bars and the typical rental of go-carts. Also the seaward parapetwall has been kept quite low to keep the view, and the landward wall has been designed into the shape of a bench to improve the touristic appeal of the dike.

The damages at Wenduine in case of flooding during a severe storm would amount to a mere € 100 million as calculated by Van der Biest et al. (2009) with CLIMAR software. These costs make up a big part of the costs of the Business as usual scenario, although business as usual also consists of reoccurring beach nourishments. The price of such nourishments is about 0,39 euro/m³ as of 2015 (federal government, department of finances) if it's dredged of Sierra Ventana, a three-time supply of 700000 m³ during the 50-year lifespan is considered, which amounts to €819000.

6.4.3 Comparison

The following table (table 7) tries to give an image of the costs and benefits of the proposed measures. This table has to be viewed with the necessary skepticism as the values are very rough estimations, as the effects caused by the measures are difficult to measure.

The costs of the business as usual scenario is equal to all the predicted damage plus the costs of the beach nourishments. The benefits are the ones that are caused because of the aesthetic benefits and is usually equal to a small factor (25 %) of the recreational value of the beach, this is calculated using the touristic value of the entire Belgian coastline (2667.7 million euros) and divide it by the entire length of the shoreline (67km), which results in 39.8 million euros per km of Belgian shoreline, which is about the length of the coast of Wenduine. Divided by 4 this gives about 10 million euros.

The costs of the sea wall were calculated and have a value of about € 4171657. The benefits amount for a fraction of the total predicted damages (100 million euros), because it consists of an occurrence of severe storm and the sea wall was developed to allow certain flooding during these storms, which will of course lead to some damage and decreasing of property value. A very rough estimation would be € 80 million.

Measure	Costs	Benefits	Costs-Benefits
Business as usual	€ 100.819.000	€ 10.000.000	€ 90.819.000
Sea wall	€ 4.171.657	€ 80.000.000	€ 75.828.343

7 Costs and benefits for the proposed measures

The absolute value of costs-benefits is considered, and it shows that the construction of a sea wall offers an advantage of € 166.647.343 over the business as usual scenario.

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