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Coastal erosion and control

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ABSTRACT

Coastal erosion is a problem at many coastal sites caused by natural effects as well as human activities. This paper explores the coastal cell concept to deal with coastal erosion by identifying and analysing the sediment volumes accumulated in large-scale and small-scale coastal cells at various sites. Mechanisms causing chronic erosion and episodic erosion related to coastal variability are identified and discussed. The effectiveness of soft and hard remedial measures for sandy beaches are assessed based on laboratory, field and modelling experiences.

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1. Introduction

Nearly all coastal states have to deal with the problem of coastal erosion. Coastal erosion and accretion has always existed and contributed to the shaping of the present coastlines. However, coastal erosion now is largely intensified due to human activities. Presently, the total coastal area (including houses and buildings) lost in Europe due to marine erosion is estimated to be about 15 km² per year. The annual cost of mitigation measures is estimated to be about 3 billion euros per year (EUROSION Study, European Commission, 2004), which is not acceptable.

Although engineering projects are aimed at solving the erosion problems, it has long been known that these projects can also contribute to creating problems at other nearby locations (side effects). Dramatic examples of side effects are presented by Douglas et al. (2003), who state that about 1 billion m³ (10⁹ m³) of sand are removed from the beaches of America by engineering works during the past century.

The EUROSION study (2004) recommends to deal with coastal erosion by restoring the overall sediment balance on the scale of coastal cells, which are defined as coastal compartments containing the complete cycle of erosion, deposition, sediment sources and sinks and the transport paths involved. Each cell should have sufficient sediment reservoirs (sources of sediment) in the form of buffer zones between the land and the sea and sediment stocks in the nearshore and offshore coastal zones to compensate by natural or artificial processes (nourishment) for sea level rise effects and human-induced erosional effects leading to an overall favourable sediment status.

In the CONSCIENCE Project (2010) the coastal cell concept to deal with coastal erosion is further explored by identifying and analyzing the sediment volumes accumulated in large-scale and small-scale coastal cells at various pilot sites. Mechanisms causing chronic erosion and fluctuation erosion related to coastal variability are identified and discussed. The effectiveness of soft and hard remedial measures for sandy beaches are assessed based on laboratory, field and modelling experiences (see also, Van Rijn, 2010a,b).

2. Coastal cells

Many coasts consist of relatively straight and flat (low-gradient) beaches. These simple, flat beach coasts may differ greatly from the originally submerged coasts. The most basic coastal form is an indented coast (bay-headland coast or embayed coast) resulting from subsidence or from submergence due to sea level rise. Wave attack on an indented bay-headland type of coast will result in concentration of wave energy on the headlands (due to refraction) and reduction of wave energy in the bays, which may lead to headland erosion and bay deposition, if these coastal forms consist of erodible material. Longshore currents accelerating along headlands and decelerating in the bay area will enhance headland erosion and bay deposition. Thus, headlands are cut back and bays are filled up. In case of uneven resistance against erosion, the 'softer' headlands will erode more rapidly and the more erosion-resistant headlands remain present as promontories along the coast. Rock-type and cliff-type coasts consisting of variable erodibility retain as irregular crenulate coasts. If the headlands are equally erosive, the coastline will be straightened. This can be demonstrated by considering an undulating sandy shoreline under wave attack from a constant direction. The longshore transport depends on the angle between the nearshore wave crest (based on

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refraction) and the coastline. The longshore transport rate is maximum on the downdrift flank (maximum wave angle) of the protruding headland resulting in erosion on the updrift flank and accretion on the down drift flank and coastal straightening on the long term.

On larger spatial scales this process, dominated by littoral drift, will continue until the coastline consists of a series of smooth beach curves (arcs with curvatures between 1 and 100 km, depending on wave climate and erodibility). The end points of the arcs may be associated with old, more erosion-resistant headlands, with outlets and deltas of rivers, with ebb deltas of tidal inlets or with man-made structures. The dominant waves will turn the beaches to face the direction of the dominant wave approach by moving sediment to the downwave end of the arc resulting in a (hollow) arc-type coast. The formation of smooth arc-type barrier beaches is the most basic element of coastal straightening and is the ultimate stage of wave-dominated coastal evolution.

Hard headlands present along a sandy shoreline act as natural groyne and compartmentalise the shoreline into sediment cells. One large isolated headland usually causes an embayment to form on its downdrift shoreline. A series of two or more headlands spaced closely generally causes the formation of embayments that are semi-circular in shape. Headlands with broad faces block significant amounts of wave energy sheltering the beaches in the lee zone.

Possible sources of sediment within a cell are: sediment input by rivers and estuaries, cliff and dune erosion, onshore transport due to wave asymmetry from the shelf, artificial nourishment, biogenic deposition (shell and coral fragments). The most important sinks are: offshore transport due to undertows and rip currents during storms, trapping in local depressions (canyons) and mining. Sources and sinks are herein identified as phenomena of an irreversible nature; a sediment particle eroded from a cliff system cannot return to this system and a particle deposited in a canyon is a permanent loss for the coastal zone.

Besides sources and sinks, stores or accumulations can be distinguished. Stores can be sand/gravel bars and banks migrating or resting in the coastal system. Sediment particles may be stored for a certain period in these features, but later the sediments may be mobilised again to take part in the transport process.

Coastal evolution and hence coastal sediment budgets in cells are strongly related to long term sea level rise (relative to the land). Shoreline response to relative sea level rise can be broadly divided into two main categories: erosional transgression and depositional regression (Van Rijn, 1998).

Erosional transgression refers to a net landward movement of the shoreline in the case of rising relative sea level. The well-known concept relating shoreline recession to water level rise is the geometric shift concept of Bruun (1962, 1988), which is based on the idea that the (dynamic) equilibrium profile of the beach and surf zone moves upward and landward in response to sea level rise (Bruun-rule). Using this concept, the required annual input of sediment (accommodation space) to the nearshore zone is equal to the area (m^2) of the nearshore zone times the annual rate ($m/year$) of relative sea level rise. Assuming that relative sea level rise is 2 mm/year and that the width of the nearshore zone is in the range of 1–10 km, the required sediment supply to the nearshore zone per unit length of shoreline is about 2–20 $m^3/m/year$ to keep up with sea level rise. This volume of sediment will be eroded from the coast, if nothing is being done. This type of coastal erosion can be prevented (compensated) by coastal nourishment of the same amount (2–20 $m^3/m/year$). Examples of eroding coasts due to sea level rise are: Mississippi delta coast, USA; Egypt coast.

3. Mechanisms of coastal erosion and coastal variability

The erosion of sandy beach-dune systems and soft cliff systems due to storm waves has been studied by many researchers. Reviews are given by Komar (1976), Vellinga (1986) and by Van Rijn (1998). Most of the studies involve the analysis of experimental results in small-scale and large-scale flumes. Detailed and complete field data sets are scarce, because usually the pre-storm bed profiles are missing.

Coastal erosion is the permanent loss of sand from the beach-dune system and strongly depends on the type of coast (exposure, wave climate, surge levels, sediment composition, beach slope). Coastal erosion has both cross-shore and long shore components. Dune and soft cliff erosion during extreme events mainly is a cross-shore process bringing the sediments from the immobile dune front into the mobile littoral system. Dune and beach erosion also is an alongshore process due to the presence of eroding longshore currents including tidal currents.

Various empirical models are available to estimate dune erosion. A semi-empirical model (S-beach) has been proposed by Larson and Kraus (1989). This model is based on equilibrium theory with limited description of the physical processes. A beach profile is assumed to attain an equilibrium shape if exposed to constant wave conditions for a sufficiently long time. An equilibrium profile ($h = Ax^{2/3}$ with x = cross-shore coordinate and A = shape parameter depending on bed material diameter) dissipates incident wave energy without significant net change in shape. The transport rate is related to the difference between the actual wave energy dissipation and the equilibrium wave energy dissipation along the equilibrium profile. The transport direction is determined from an empirical criterion. Steetzel (1993), Van Thiel de Vries et al. (2006) and Van Rijn (2009) have used process-based mathematical models based on cross-shore wave propagation, wave shoaling, wave refraction and wave breaking. The output of the wave model is used to compute the local cross-shore sand transport rate. Bed level changes are determined from cross-shore gradients of the transport rate in a numerical loop system.

Fig. 1 shows plots of the dune erosion area (above the storm surge level) after 5 h as a function of the sediment size and the storm surge level based on the simplified cross-shore model of Van Rijn (2009) for the case of waves normal to the coast. Vellinga (1986) has found that the most effective duration of a storm along the North Sea coast is about 5 h. The significant offshore wave height in the North Sea is assumed to vary between 4 and 8 m for surge levels between 1 and 5 m above mean sea level (MSL). Dune erosion after 5 h is largest for relatively fine sediments (0.15 mm) and reduces rapidly for coarser sediments. Dune erosion of gravel (1 mm) is only 15% of that of fine sand (0.15 mm). The shoreline recession (E) due to dune erosion can be estimated from $E = A/h$ with A = dune erosion area above storm surge level SSL and h = dune height above the storm surge level. Fig. 1 shows dune recession values (axis on right side of plot) based on a dune height of 10 m above SSL. Dune recession values are twice as large for dune height of 5 m.

The simplified model of Van Rijn (2009), applied to compose Fig. 1, produced fairly good results using measured dune erosion data of Inch Beach (sand of 0.24 mm) in Ireland (pilot site of CONSCIENCE project). The data represent accumulated dune recession values in the range of 14–28 m over the period December 2007 to May 2008 with offshore wave heights in the range of 2.5–5.5 m (periods of 12–16 s). The computed total dune recession value for this period is about 20 m (accumulation of various storm events, each with duration of 5–6 h). This confirms that Fig. 1 yields realistic results.

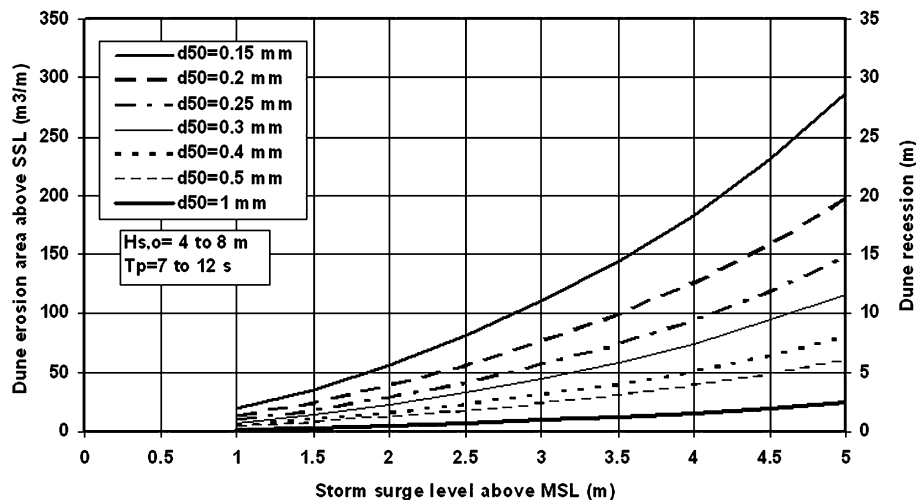


Fig. 1. Dune erosion after 5 h during a storm event as function of sediment size and storm surge level.

Dune erosion is very much related to extreme events with high surge levels including tidal effects. The maximum dune erosion during a super storm on a sandy coast (0.2 mm) is of the order of 200 m³/m. In 'normal' conditions with two or three storm events per year with surge levels between 1 and 2 m per year, the total annual dune erosion is much smaller and estimated to be about 50 m³/m/year locally along the sandy North Sea coasts. Most of the eroded dune sand will be deposited on the beach from where it can be returned to the dune front by wind-induced forces or carried away by cross-shore and longshore currents. Dune accretion at the dune front due to wind effects is of the order of 10–20 m³/m/year (Van der Wal, 2004) and is generally not sufficient to compensate for dune erosion on the annual time scale by natural processes. Thus, dune erosion generally leads to a permanent loss of sand which can only be compensated by artificial nourishment (dune restoration).

Coastal variability (also known as shoreline variations) is not the same as coastal erosion, the latter being the permanent loss of sand from the system. Coastal variability is herein defined as the temporary (fluctuation) loss of sand from the system. Coastal variability generally is visible through variation of the shoreline (LW-line, HW-line, dune foot-line) around a systematic trend line (chronic erosion or deposition); the trend line erosion may be caused by natural (autonomous) processes or related to man-made structures.

Spectral analysis (Stive et al., 2002) of time series of the shoreline over a period of about 10 years for three typical ocean-fronted beaches (Duck, USA; Ogata, Japan and Ajigaura, Japan) shows pronounced peaks corresponding to a 1-year cycle indicating the effects of seasonal (summer-winter) changes. Higher frequencies are also present in the data sets associated with the typical return period of storm events. Peaks at lower frequencies (2–4 years) are also present, most probably associated with migrating sand waves.

At many natural beaches the cyclic beach behaviour is strongly related to the cyclic breaker bar behaviour. The typical beach-bar behaviour on the time scale of the seasons is the offshore-onshore migrational cycle with offshore migration of the bar system during the winter season and onshore migration and beach recovery during the summer season (low waves). Seasonal variation resulting in so-called winter and summer profiles is a general characteristic of nearshore morphological behaviour, but the degree of seasonality varies widely.

4. Controlling coastal erosion by soft nourishments

4.1. Available methods

The available options of shoreline management to deal with erosion problems, are:

- to accept retreat in areas where beaches and dunes are wide and high;
- to maintain the coastline at a fixed position by of hard structures and/or by soft nourishments;

To distinguish between long term chronic erosion and short-term fluctuation erosion (natural coastal variability), cross-shore profile data should be available covering at least 10–20 years in the area of interest. Based on the profile data, the total volume of sediment within the active zone (say landward of the –8 m depth contour) at the problem area (length scale of 5–10 km) can be determined and plotted as a function of time to reveal erosional or depositional trends. If there is a substantial loss of sediment over a period of 5–10 years, it may considered to nourish the area with a sediment volume equal to the observed volume loss, either as shoreface nourishment or as beach nourishment or both.

Shoreface nourishments (also known as feeder berms) are used in regions of relatively wide and high dunes (relatively safe coastal regions) to maintain or increase the sand volume in the nearshore zone with the aim to nourish the nearshore zone on the long term by natural processes (net onshore transport). The nourishment volume is of the order of the volume of the outer breaker bar (300–500 m³/m). The length scale (alongshore 2–5 km) of a shoreface nourishment is of the order of several times the width of the surf zone. Shoreface nourishment is relatively cheap as the sand can be dumped during sailing in shallow water (5–10 m). Relatively large nourishment volumes are required as only part of the nourishment volume (approximately 20%–30%) will reach the beach zone after 5 years.

Shoreface nourishments have both longshore and cross-shore effects. The shoreface nourishment acts as a wave filter (larger waves are reduced by breaking), resulting in a decrease of the longshore transport landward of the nourishment location; updrift sedimentation and downdrift erosion. The cross-shore effect is that the large waves break at the seaward side of the shoreface nourishment and the remaining shoaling waves generate onshore

transport due to wave asymmetry over the nourishment resulting in an increase of the onshore sediment transport. Both effects result in the trapping of more sand behind the shoreface nourishment area. Basically, a shoreface nourishment behaves in the same way as a low-crested, submerged breakwater as discussed by Sánchez-Arcilla et al. (2006). However, the wave filtering effects will reduce in time as sand will be eroded from the shoreface and carried away in both cross-shore and longshore directions.

Practical experiences along the Dutch coast (Witteveen and Bos, 2006) show that shoreface nourishments at depths -6 to -5 m (below mean sea level) have an efficiency (defined as the ratio of volume increase of the nearshore zone and the initial nourishment volume) of 20%–30% after about 3–5 years; the nearshore zone is defined as the zone between -1 and -5 m NAP (landward of the nourishment area). The efficiency with respect to the beach zone between -1 m and $+3$ m is extremely low (about 2%–5% after 3–5 years). Given a typical shoreface nourishment volume of $400 \text{ m}^3/\text{m}$, the potential increase of the beach volume after 3–5 years is not more than about 10 – $15 \text{ m}^3/\text{m}$ or a layer of sand with thickness of about 0.1 m– 0.15 m over a beach width of 100 m.

Beachfills are mainly used to compensate local erosion in regions with relatively narrow and low dunes (in regions of critical coastal safety) or when the local beach is too small for recreational purposes. Practical beachfill volumes per unit length of coast are: 10 – $30 \text{ m}^3/\text{m}/\text{yr}$ for low-energy coasts (Mediterranean); 30 – $75 \text{ m}^3/\text{m}/\text{yr}$ for moderate-energy coasts (North Sea); 75 – $150 \text{ m}^3/\text{m}/\text{yr}$ for high-energy coasts (Atlantic/Pacific Ocean). Practical lifetimes are of the order of 1–5 years. An elongated beachfill should be placed as much as possible landward of the high tide line in a layer of 2 – 3 m thick (volumes of 50 – $100 \text{ m}^3/\text{m}$) with a berm of about 20 – 30 m wide (if required) at the dune foot level; the length of the fill should be larger than about 3 km to minimise the sand losses at both alongshore ends due to dispersion effects under normal wave attack. The initial lower slope of the beachfill should not be too steep (not steeper than 1:20).

Ideally, the beach fill material should be slightly coarser than the native beach material in the beach/swash zone. Fine fill materials will require a relatively large overfill volume to compensate the losses during construction. The sand size largely depends on economically available sand in the borrow area. The effectiveness of beachfills increases considerably for sand larger than 0.3 mm. Beachfills are relatively expensive as a pumping line to the beach generally is required. Beach nourishments of fine sand (0.2 mm) have extremely low lifetimes of 1–2 years along the Holland coast.

Although, sand nourishment may offer significant benefits, it may also be a costly method if life spans are fairly short at very exposed beaches or if the long term availability of adequate volumes of compatible sand at nearby (economic) locations is problematic. For example, sand material suitable for beach nourishment cannot easily be found at most Italian and Spanish sites along the Mediterranean.

4.2. Experimental results of laboratory tests (cross-shore)

Beach nourishment generally results in a largely disturbed beach profile. The natural beach profile is covered by a thick layer of sand (1 – 2 m) with relatively straight slopes. The beach slope may be in the range between 1:50 and 1:100, but the slope of the seaward flank of the fill usually is quite steep (1:10). Relatively steep beach profiles are very vulnerable to erosion. The initial losses of beach nourishments are largely determined by the initial slope of the seaward flank of the fill. Practical experience at Sylt beach Germany (Raudkivi and Dette, 2002) shows an initial beach loss of about $120 \text{ m}^3/\text{m}$ in about 4.5 months (winter period) or about $1 \text{ m}^3/\text{m}/\text{day}$ for a beach nourishment at 28 September 1992 with an initial

toe slope of 1:5. Experience at Egmond beach (The Netherlands) also shows relatively large initial losses and relatively short beachfill lifetimes of about 1–2 years. Preferably, beach nourishments should have a relatively flat initial slope (1:20 or flatter). If possible, an underwater berm at about -1 m should be included to minimise the initial sand losses.

To better understand the erosional behaviour of beachfills, it is of crucial importance to understand the erosion/accretion processes at natural beaches (without nourishment). Data from two beaches along the central Holland coast are available: Egmond beach and Noordwijk beach. The tidal range at both beaches is of the order of 2 m; the beach sediment is sand with a median particle diameter of about 0.25 mm. Field experience at Egmond beach along the Holland coast (Van Rijn et al., 2002) clearly shows that high and low areas on the beach co-vary with high and low levels of the crest of the inner surf zone breaker bar. The beach volume per unit width increases/decreases with increasing/decreasing crest level of the inner bar. The beach volume per unit width increases by about $30 \text{ m}^3/\text{m}$ if the crest level of the inner bar increases from -1.5 m to -0.5 m NAP (NAP is approximately MSL); and decreases by about $30 \text{ m}^3/\text{m}$ if the crest level decreases from -1.5 m to -2.5 m NAP. Given a beach width of about 100 m, this means a maximum vertical change (increase/decrease) of the beach level near the water line of about 0.6 m assuming a triangular accretion/erosion pattern. The daily beach volume changes (erosion/accretion) vary between 1 and $3 \text{ m}^3/\text{m}/\text{day}$ in a storm month with wave heights up to about 5 m. The daily accretion is maximum if the crest level of the inner bar is at -0.5 m NAP; the daily erosion is maximum if the crest level of the inner bar is at -2.5 m NAP. The beach volume changes of about $30 \text{ m}^3/\text{m}$ can occur over a period of about 10 – 15 days in a storm month (maximum storm surge level SSL of about $+2$ m above NAP); the beach volume is almost continuously adjusting to a new equilibrium, if the inner bar crest level is continuously changing. Assuming a maximum beach volume variation (erosion) of about 20 – $30 \text{ m}^3/\text{m}$ due to a storm event and a net daily onshore transport rate of about 1 – $3 \text{ m}^3/\text{m}/\text{day}$ due to fair-weather processes, the restoration time of the beach morphology to the changing inner bar morphology is of the order of 10 – 30 days (a few weeks) after a storm period.

Similar volume variations have been observed over a period of three years at the beach of Noordwijk along the Holland coast (Quartel et al., 2008). The mean beach width is about 120 ± 15 m; the mean beach volume (above MLW) is about $190 \pm 25 \text{ m}^3/\text{m}$. Thus, the maximum beach volume variation over a period of 3 years is about $25 \text{ m}^3/\text{m}$ or about 15% of the total beach volume above the mean low water line (MLW; about -0.7 m below mean sea level MSL). The volume variations are largest ($\pm 15 \text{ m}^3/\text{m}$) in the lower beach zone with the inner bar between the MLW (at 1.3 m above MSL) and MSL and smallest ($\pm 5 \text{ m}^3/\text{m}$) in the zone between MHW and MSL and in the upper beach zone above MHW ($\pm 5 \text{ m}^3/\text{m}$). The beach volume is found to be largest at the beginning of the winter season and smallest at the end of the winter season (storm waves). The maximum volume variation of about $25 \text{ m}^3/\text{m}$ (above MLW $\cong -0.7$ m to MSL) at Noordwijk beach is somewhat smaller than that at Egmond beach, which is about 30 – $50 \text{ m}^3/\text{m}$ (above -2.5 m to MSL) including the inner bar volume variation at that location.

The erosion of nourished beaches with straight slopes has been studied extensively by performing small-scale and large-scale tests in wave tanks/flumes. Fig. 2 shows beach profiles for an initial slope of 1:10 (V:H), 1:20 and 1:40 based on experimental results in a small-scale laboratory flume at Deltares/Delft Hydraulics with sand of 0.13 mm and approaching (irregular) waves of about $H_{s,0} = 0.17$ m (Deltares, 2008). Similar tests have been done in the large-scale Hannover wave flume with $H_{s,0} = 1$ m and $d_{50} = 0.27$ mm (EU SANDS Project, see Sánchez-Arcilla, 2011). The

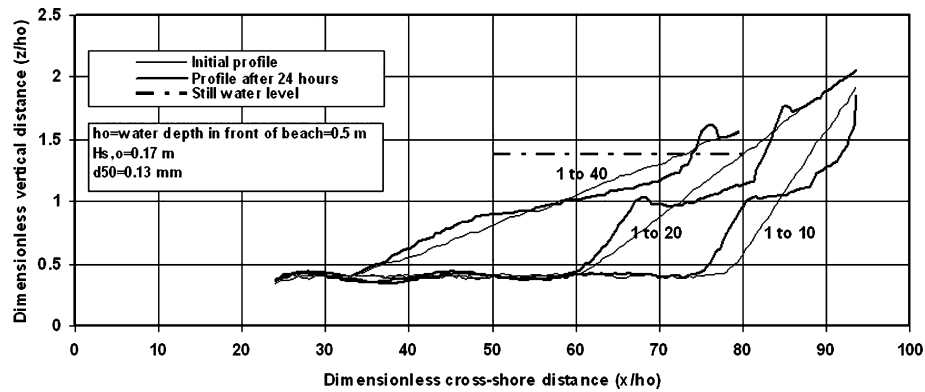


Fig. 2. Beach profile development for initial slopes between 1:10 and 1:40 (laboratory tests).

small-scale test results have been upscaled to the Hannover flume scale representing field scale (length scale = depth scale = 5.5, sediment size scale = 2, morphological time scale = 2, Van Rijn et al., Submitted for publication). Based on this upscaling approach, Fig. 3 shows the beach erosion volumes as a function of time. The upscaled results represents prototype erosion by waves of about 1 m at the toe of the beach (minor storm events; offshore waves of 3–4 m). The steepest initial slope of 1:10 yields an erosion volume after 1 day of about 12 m³/m. The beach erosion is of the order of 6–9 m³/m/day for milder slopes between 1:20 and 1:40.

4.3. Numerical modelling (cross-shore)

To determine the overall efficiency of beach nourishments, the process-based CROSMOR-model (Van Rijn, 2009) has been used to compute beach erosion volumes for various schematised cases (see Fig. 4) along the Dutch coast (North Sea wave climate). The initial beach nourishment volume is about 220 m³/m. The slope of the upper beach is set to 1:150; the initial slope of the lower beach is 1:20. The North Sea wave climate along the Dutch coast can be characterised as: $H_{s,o} < 1$ m during 50% of the time, $H_{s,o} = 1$ –3 m during 45% of the time and $H_{s,o} > 3$ m during 5% of the time. Three wave conditions have been used: $H_{s,o} = 0.6$ m and wave period of $T_p = 5$ s over 100 days normal to the beach, $H_{s,o} = 1.5$ m and wave period of $T_p = 7$ s over 30 days normal to beach and $H_{s,o} = 3$ m and wave period of $T_p = 8$ s over 10 days normal to beach representing a standard winter season. The tidal range is set to 1 m. The peak

flood tidal current to the north is set to 0.6 m/s and the peak tidal ebb current to the south is set to -0.5 m/s. These values apply to the offshore boundary. Nearshore tidal currents are computed by the model and are much smaller due bottom friction (decreasing depth). A storm set-up of 0.5 m has been used during offshore waves of $H_{s,o} = 3$ m. Three types of beach material have been used: $d_{50} = 0.2, 0.3$ and 0.4 mm. The offshore boundary conditions were applied at a depth of 15 m to MSL.

Fig. 5 shows computed results for the wave height of $H_s = 3$ m over 10 days and three types of beach materials (0.2, 0.3 and 0.4 mm). Erosion mainly occurs in the beach nourishment section above the water line (0 to +1.5 m). The eroded sand is deposited at the toe of the beach nourishment between the 0 and -1.5 m depth contours. The deposition layer in front of the beach nourishment slows down the erosion in time by reducing the wave height. The cumulative erosion volumes (in m³/m/day) are shown in Figs. 6, 7 and 8. The large-scale Hannover flume data are also presented in Figs. 6 and 7, showing reasonably good agreement with the computational results for waves of about 1.5 m. Fig. 6 shows that the cumulative beach erosion volume stabilizes after about 30 days due to the generation of an equilibrium beach profile. Erosion values after 100 days are only 10% larger. Runs without tide show similar values, as the nearshore tidal currents are not very strong and are smaller than the wave-induced longshore currents.

The initial erosion volumes (after 1 day) for waves < 1.5 m are about 6 m³/m for sand of 0.4 mm–10 m³/m for sand of 0.2 mm. These values are in line with the initial erosion volumes of the

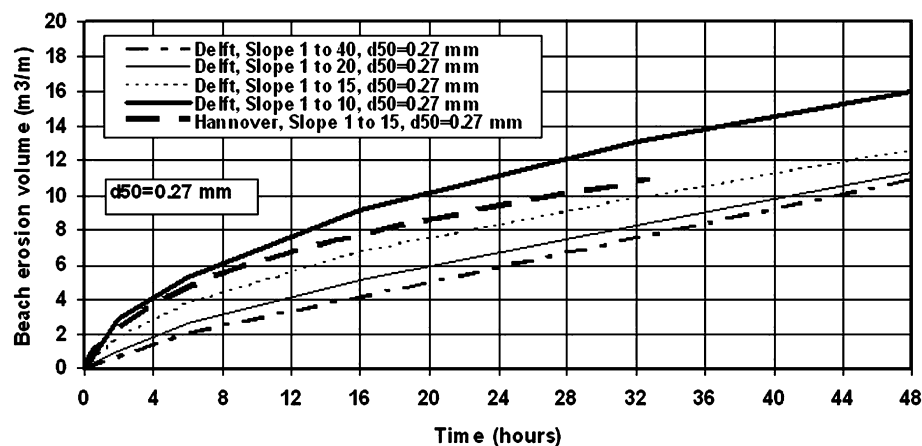


Fig. 3. Beach erosion volumes for plane sloping beaches in nature with daily waves of about 1 m at toe of beach; slopes between 1:10 and 1:40, $d_{50} = 0.27$ mm (Delft tests upscaled to Hannover flume scale using length scale = depth scale = 5.5, sediment size scale = 2, morphological time scale = 2).

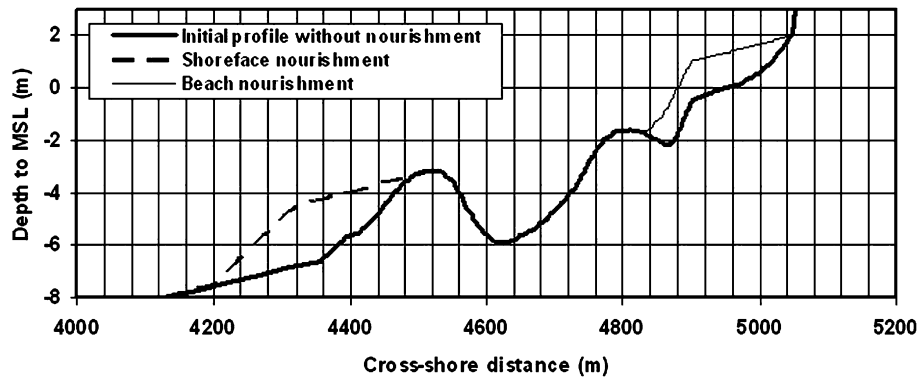


Fig. 4. Schematized beach and shoreface nourishment profiles along Dutch coast (Egmond profile).

upscaled laboratory data (Fig. 3) yielding values of about 6–8 m³/m after 1 day for sand of 0.27 mm (slopes between 1:15 and 1:20).

Over a month these values are significantly smaller. Monthly-average erosion volumes (cumulative erosion divided by total duration) are in the range of 1 m³/m/day for sand of 0.4 mm–2.5 m³/m/day for sand of 0.2 mm and waves with $H_s < 1.5$ m (based on Fig. 7). For waves of $H_s = 3$ m these values increase to 4 m³/m/day to 10 m³/m/day.

The computed bed profile of a run with $H_{s,0} = 3$ m and an offshore wave angle of 30° has also been plotted in Fig. 5, showing a slight increase of the total erosion volume by about 20%. The cumulative erosion is plotted in Fig. 6. The wave-induced longshore current at initial time is also shown in Fig. 5. The maximum longshore current is of the order of 1 m/s just in front to the beach nourishment, which enhances the transport capacity and hence the erosion power of the system. The eroded sediments are deposited at the seaward edge of the inner breaker bar. This wave angle effect is a typical storm feature, as it is hardly noticeable for an offshore wave height of 1.5 m (see Fig. 7).

The cumulative erosion is slightly reduced, if a shoreface nourishment is present due to additional wave breaking at the shoreface nourishment location, see Fig. 8.

These computational results with daily-average erosion values of the order of 1–10 m³/m/day show that a beach nourishment volume of the order of 100–200 m³/m can be easily eroded away in one to two winter seasons in line with observations at the Dutch coast where beach fills have, on average, to be repeated at two year intervals. The trough (depression) beyond the inner breaker bar acts as a sink to the erosion of beach sediments. Therefore, the presence of a trough in front of the beach nourishment should be

avoided (trough between inner bar and beach should be filled with sand). Fig. 9 shows the computed bed level after 1 winter period with a sequence of waves, as follows: 100 days with $H_{s,0} = 1$ m, 30 days with $H_{s,0} = 1.5$ m and 10 days with $H_{s,0} = 3$ m for three sediment diameters ($d_{50} = 0.2, 0.3$ and 0.4 mm). The beach erosion is approximately 150 m³/m for $d_{50} = 0.2$ mm; 100 m³/m for $d_{50} = 0.3$ mm and 90 m³/m for $d_{50} = 0.4$ mm. As can be seen, the beach nourishment volume of 0.2 mm sand is almost completely removed after 1 winter season. At the landward end of the beach a typical scarp-type erosion front is present, which is often observed in nature. Beach nourishment of more coarse material of 0.3 mm has a lifetime which is 50% larger than that of 0.2 mm material. The eroded beach sediment is deposited as a new breaker bar beyond the –4 m depth line.

Using data of Figs. 6, 7 and 8 and adding the results of each wave class ($H_s = 0.6, 1.5$ and 3 m) linearly, yields beach erosion volumes of 210, 130 and 120 m³/m for $d_{50} = 0.2, 0.3$ and 0.4 mm. This approach leads to an overestimation of about 30%, as the time history effect of the beach profile is not taken into account.

The CROSMOR-model has also been used to evaluate the efficiency of shoreface nourishments beyond the –6 m depth line. Fig. 10 shows the morphological changes of a shoreface nourishment at the seaward flank of the outer breaker bar for a wave height of $H_{s,0} = 1.5$ m (post-storm waves and fairweather waves) for $d_{50} = 0.2$ mm and 0.4 mm. Onshore sand transport in the range of 20–100 m³/m over 100 days can be observed for waves of 1.5 m depending on the bed material diameter and the modelling of the suspended transport due to wave asymmetry. The largest values occur for relative coarse sediment and inclusion of the suspended transport due to wave asymmetry. The migration distance varies

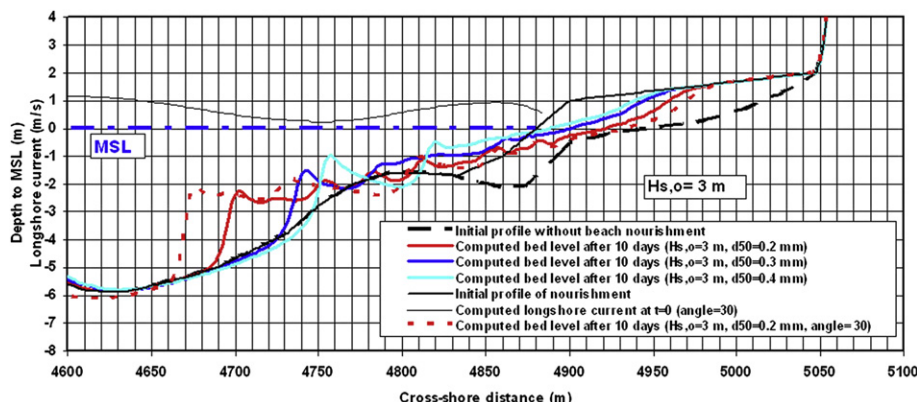


Fig. 5. Erosion of beach nourishment; $H_{s,0} = 3$ m; sediment $d_{50} = 0.2, 0.3$ and 0.4 mm.

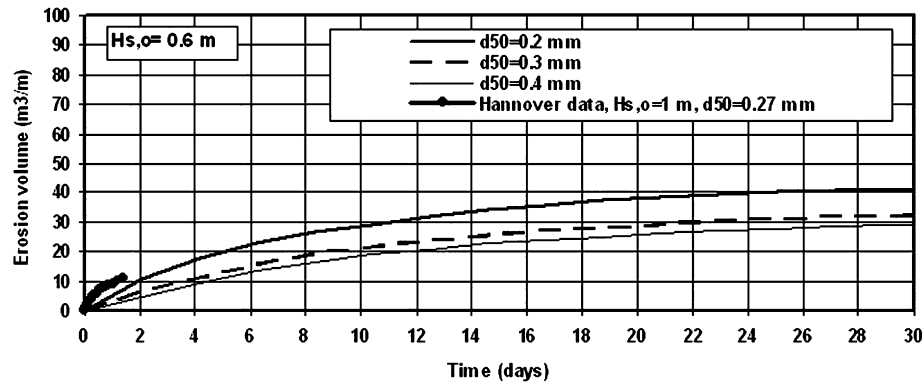


Fig. 6. Cumulative erosion of beach nourishments; $H_{s,0} = 0.6$ m; sediment $d_{50} = 0.2, 0.3$ and 0.4 mm.

between 10 and 40 m over 100 days. The nourishment profile shows a slight tendency to grow due to the shoaling wave of 1.5 m as has been observed in nature (see Fig. 9). As the beach zone ($-3/+3$ m) is situated at about 200 m shorewards from the shoreface nourishment, it will take at least 5 years of low wave conditions (which occur during about 75% of the time; $H_{s,0} < 1.5$ m) before the nourishment can migrate to the beach zone (-3 to $+3$ m). Hence, it is rather difficult for the sediments to pass the deep trough landward of the outer bar.

Fig. 11 shows the morphological changes (offshore migration) of the shoreface nourishment at the seaward flank of the outer breaker bar for storm events with $H_{s,0}$ in the range of 2.25–5 m (which occur during about 25% of the time) and $d_{50} = 0.2$ mm. As can be observed, these conditions result in offshore-directed migration of the nourishment. The sediment (in the range of 50–100 m³/m) is eroded from the crest region and deposited at the seaward flank over a period of 5–50 days.

On the seasonal time scale with low and high waves, the shoreface nourishment will be gradually spread out in both onshore and offshore direction. The annual transport from the crest region to both flanks (seaward and landward) of the bar is of the order of 50–100 m³/m/year yielding a lifetime of the order of 5 years (as observed along the Dutch beaches in North Sea conditions) given an initial volume of about 400 m³/m. The computed net onshore transport over one year is of the order of 25–50 m³/m/year. Practical experience shows that about 25% of the initial shoreface nourishment volume will eventually (after 5 years) be transported to the nearshore zone. Assuming an initial shoreface nourishment volume of about 400 m³/m, the net onshore transport involved will be about $0.25 \times 400 / 5 = 20$ m³/m/year, which is somewhat smaller than the computed value of 25–50 m³/m/year.

These values refer to North Sea wave conditions. The onshore feeding potential of a shoreface nourishment will be much smaller in milder wave climates (Mediterranean).

4.4. Numerical modelling (longshore)

In plan form two types of beach nourishments generally are designed: rectangular, elongated beachfills or a triangular, headland-type beachfills (stockpiles). The latter are more attractive from economical point of view (lower construction costs). Using headland-type fills, the nourished beach is divided into a series of sediment stocks and cells (compartmentalisation) in which the sediments are supposed to be spread out by natural processes. This idea will hereafter be explored by example computations for an eroding coastal section in a severe wave climate (North Sea) with a length of 15 km using the LONGMOR-model (see Equation (1)).

The local wave climate (offshore waves of 0.5–4 m and incidence angles of 30° and -15° with respect to the coast normal) is assumed to generate a net longshore transport of about 375,000 m³/year at $x = 0$ and about 500,000 m³/year at $x = 15$ km. Hence, a significant longshore transport gradient of 125,000 m³/year is assumed to be present to impose a chronic coastal erosion of about 7 m in 5 years along this coastal section (see Fig. 12). The LONGMOR-model has been used to determine the consequences of creating coastal cells by means of headland-type beach fills with a cross-shore length of 50 m and a spacing of 5 km. The active layer thickness of the coastal profile is assumed to be 6 m. The beach sediment is sand with $d_{50} = 0.2$ mm and $d_{90} = 0.3$ mm. The local beach slope is assumed to be $\tan \beta = 0.05$ (slope of 1:20 from water line to 6 m depth contour). The local wave breaking coefficient is assumed to be 0.6. The longshore grid size is 50 m and the time step

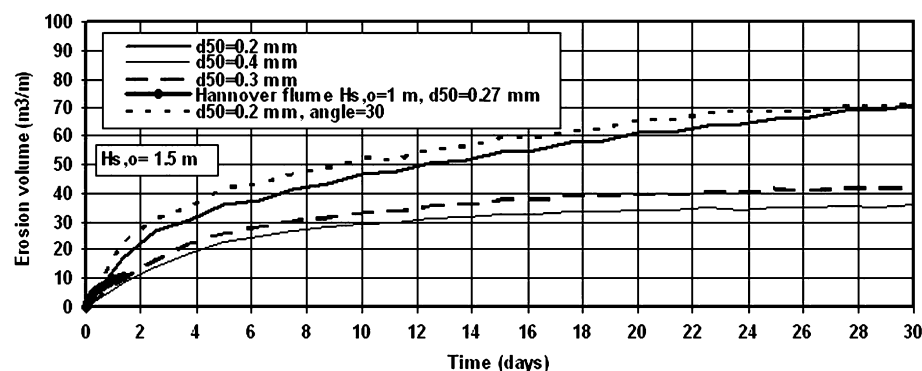


Fig. 7. Cumulative erosion of beach nourishments; $H_{s,0} = 1.5$ m; sediment $d_{50} = 0.2, 0.3$ and 0.4 mm.

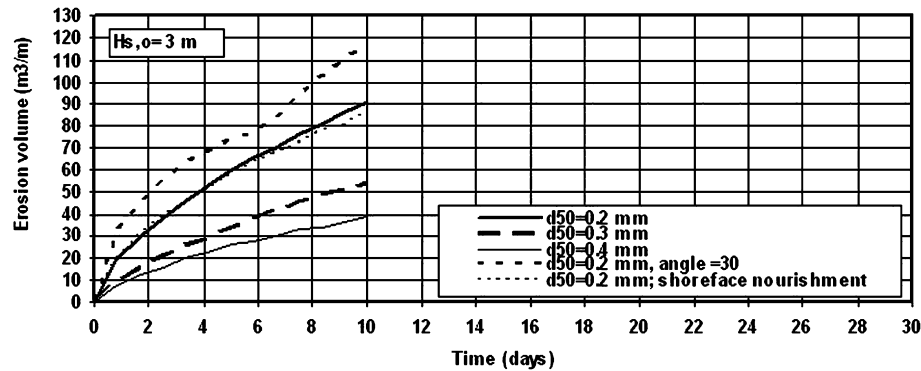


Fig. 8. Cumulative erosion of beach nourishments; $H_{s,0} = 3$ m; sediment $d_{50} = 0.2, 0.3$ and 0.4 mm.

is 0.01 days. The shoreline changes over a period of 5 and 10 years have been determined using the schematised wave climate yielding a net longshore transport gradient of $125,000 \text{ m}^3/\text{year}$ based on the method of Van Rijn (2002, 2005).

Fig. 12 Top shows the typical shoreline behaviour with gradual decay of the beach fills and a shoreline erosion of 7 m in 5 years (and 14 m in 10 years) on both ends of the beach due to the imposed longshore transport gradient. These results are based on the assumption that the longshore transport is symmetric with respect to the shoreline angle; a positive angle of $+15^\circ$ yields the same results as a negative angle of -15° . When the longshore transport is modified by assuming that the transport rate on the updrift flank of the headland is about 20% larger than that on the downdrift flank, the shoreline shows a migrational behaviour of the sandy headlands, see Fig. 12Bottom. The beach in the middle between the two headlands even shows significant erosion of about 20 m after 10 years. Thus, a side effect of headland-type beach fills may be the generation of local erosion spots due to (minor) alongshore variations of the net longshore transport, which is not attractive from a management perspective as it will require remedial measures. This example for an exposed coast in a severe wave climate (see Fig. 12) shows that the introduction of artificial alongshore variations by headland-type beach fills is tricky and should be avoided as much as possible.

4.5. Practical nourishment experiences

A massive programme of large-scale and long term beach and shoreface nourishments is being executed along the Holland coast to mitigate the chronic long term erosion. The Holland coast is the

central coastal section of the Netherlands bordering the North Sea (Van Rijn, 1995, 1997). It consists of two large-scale cells (compartments), each with a length of about 55 km, see Fig. 13. The southern cell is situated between two long harbour jetties; the Hoek van Holland jetty (south) and the IJmuiden jetty (north). The northern cell is situated between Den Helder bordering the Texel inlet (north) and the IJmuiden jetty (south). Each cell is divided in two subcells (each with a width of about 0.5–1 km) covering the shoreface zone between the -8 m and -3 m depth contours (depth to MSL) and the nearshore zone or beach zone between the -3 m and $+3$ m depth contours. The landward boundary is situated at the dune toe line ($+3$ m to MSL).

During the period between 1600 and 1800 the retreat of the coastline in the eroding sections was of the order of 3–5 m/year caused by the eroding capacity of the flood and ebb currents near the tidal inlets in the south and in the north. From 1800 onwards coastal defence was improved by building stone groynes in the sections 0–30 km and 100–118 km. As a result of these man-made structures, the retreat of the coastline in the eroding sections was considerably reduced to about 0.5–1.5 m/year. Long rubble-mound barriers (breakwaters) normal to the shore were built around 1870 near Hoek van Holland and IJmuiden to ensure a safe approach of vessels to the harbour of Rotterdam and Amsterdam. These man-made structures have compartmentalized the Holland coast into two large-scale sediment cells, each with a length of about 55 km.

In 1990 a new coastal management policy was initiated aimed at compensating all coastal erosion by regular beach and shoreface nourishments using sand dredged from offshore borrow regions beyond the -20 m depth contour where large quantities of sand are available in the North Sea. This new policy is now in operation for

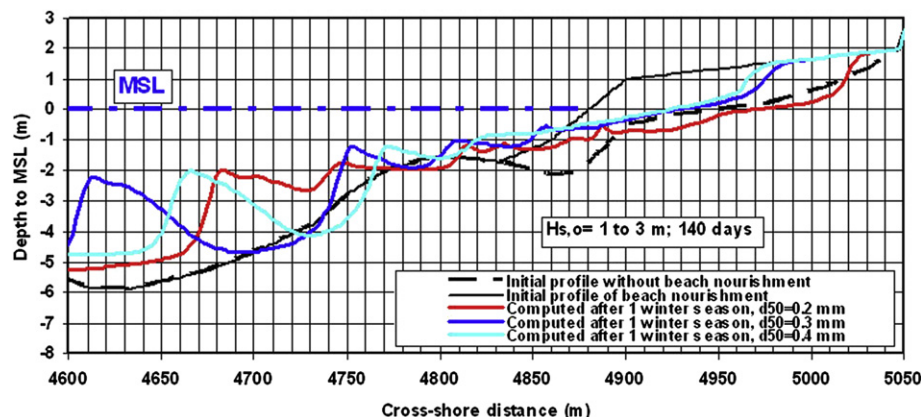


Fig. 9. Erosion of beach nourishment after 1 winter season ($d_{50} = 0.2, 0.3$ and 0.4 mm).

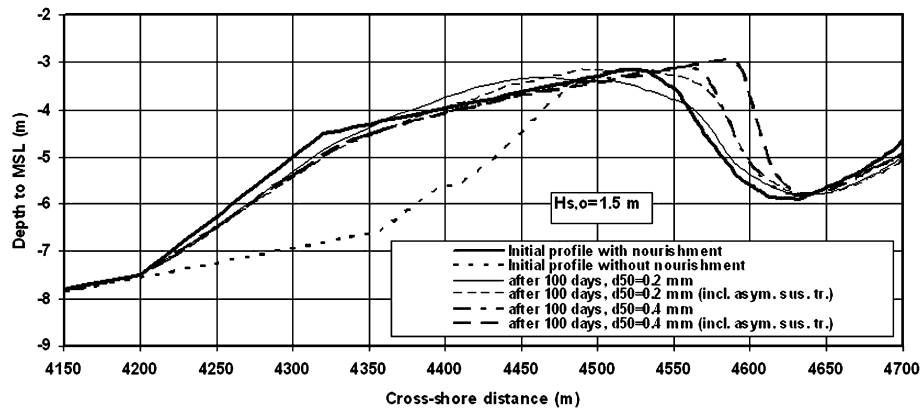


Fig. 10. Onshore migration of shoreface nourishment; $H_{s,o} = 1.5$ m, sediment $d_{50} = 0.2$ mm and 0.4 mm.

more than 15 years and the effects on the overall sediment budget of the Holland coast can be evaluated based on analysis of measured long term cross-shore profile data.

Figs. 13 and 14 show the annual erosion and deposition volumes (in m^3/year) for various alongshore sections in both cells of the Holland coast focussing on the period 1964 to 1990 (with minor nourishments) and 1990 to 2006 (with major nourishments) to study the effect of the nourishment scheme set into operation since 1990.

Analysis of the 1964–1990 data shows (Fig. 13left and 14left) substantial erosion of the order of $100,000 \text{ m}^3/\text{year}$ (or $20 \text{ m}^3/\text{m}/\text{year}$) and coastal recession of the order of $1 \text{ m}/\text{year}$ along the northern sections (km 0 to 30) of the north cell. Large deposition values can be observed on both sides of the long harbour jetties in the middle of the Holland coast (IJmuiden sections, km 50 to 60) and erosion in the neighbouring sections in both cells (in line with the data presented by Van Rijn, 1997).

Analysis of the 1990–2006 data (Fig. 13right and 14right) shows substantial deposition at nearly all sections due to the massive nourishment programme. The total annual volume increase (based on bathymetric data) is about $2.4 \text{ million m}^3/\text{year}$. The total annual nourishment is approximately $3.4 \text{ million m}^3/\text{year}$ (between 1990 and 2006).

Comparison of the volume data of both periods (1964–1990 and 1990–2006) shows that the erosive trend along the northern sections of the north cell has changed into an accretive trend due to the massive nourishment programme.

The net longshore transport across the southern boundary is about zero due to the presence of the deep navigation channel to the Port of Rotterdam. The net longshore transport across the

northern boundary into the Texel inlet was estimated to be about $0.3 \text{ million m}^3/\text{year}$ (Elias, 2006). The net cross-shore transport across the seaward boundary at the -8 m depth line is assumed to be about zero. The net cross-shore transport by wind across the dune toe line ($+3 \text{ m}$) is estimated to be $0.7 \text{ million m}^3/\text{year}$. Thus, the total annual loss of sediment from the Holland cells after 1990 is about $1 \text{ million m}^3/\text{year}$. As the total annual nourishment volume is about $3.4 \text{ million m}^3/\text{year}$, the volume increase of the Holland cells now is $2.4 \text{ million m}^3/\text{year}$ in line with observations. About $0.4 \text{ million m}^3/\text{year}$ is required to compensate for sea level rise of $2 \text{ mm}/\text{year}$. So the over-nourishment is about $2 \text{ million m}^3/\text{year}$ at present.

Using this approach of massive nourishment, the Dutch shoreline can be substantially extended and reinforced in seaward direction over a period of the order of 100 years as an extra coastal buffer to better deal with sea level rise effects and intensified wave climates.

This example of the Dutch coast shows that long term and large-scale erosion can be stopped by massive beach and shoreface nourishment over long periods of time. This approach is only feasible if sufficient quantities of sand are available and the dredging and dumping costs are acceptable (about 10–15 million Euro per year or 100 to 150 Euro per m coastline for the Holland coast with a total length of about 100 km).

Although, sand nourishment may offer significant benefits, it is a costly method if life spans are fairly short at very exposed beaches. A recent study of nourishment projects along Californian beaches (USA) has shown that about 20% of the projects survived less than 1-year, 55% lasted only 1–5 years and about 20% survived over 5 years (Leonard et al., 1990). Another constraint is the long term

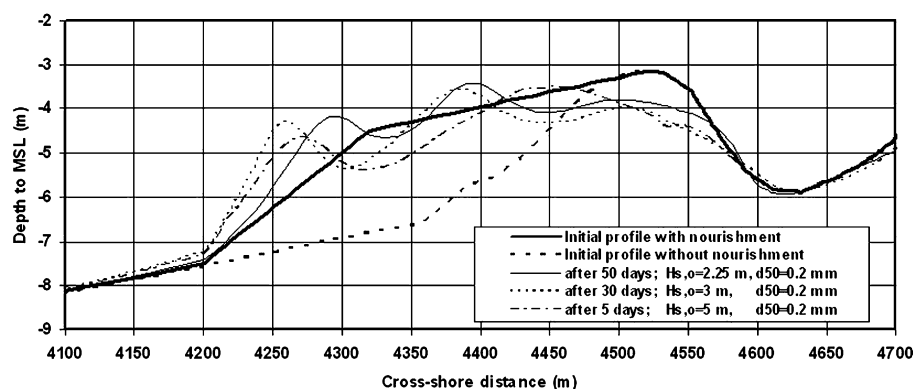


Fig. 11. Offshore migration of shoreface nourishment; $H_{s,o} = 2.25$ – 5 m, sediment $d_{50} = 0.2$ mm.

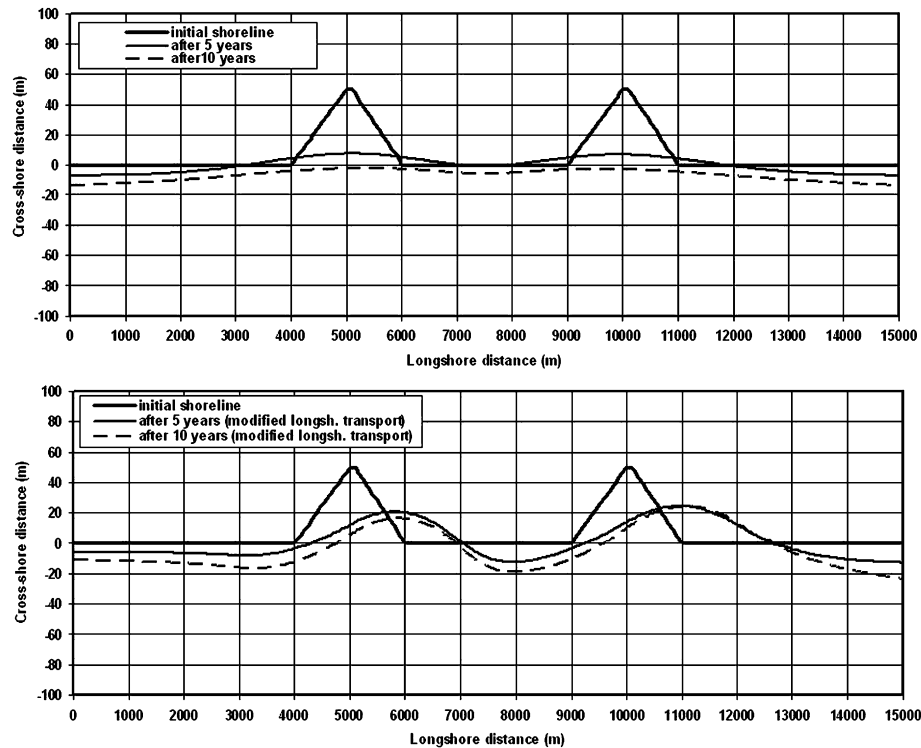


Fig. 12. Shoreline behaviour after 5 and 10 years of headland-type beach fills (net longshore transport from left to right); spacing of 5000 m. Top: standard longshore transport Bottom: longshore transport on updrift flank is 20% larger than on downdrift flank.

availability of adequate volumes of compatible sand at nearby (economic) locations. For example, sand material suitable for beach nourishment cannot easily be found at most Italian and Spanish sites along the Mediterranean. Hence, hard structures have often to be used to deal with erosion along these latter sites. This option will be explored hereafter focussing on groynes and detached breakwaters.

5. Controlling erosion by hard structures

Generally, coastal structures such as groynes, detached breakwaters and artificial submerged reefs are built to significantly reduce coastal erosion and to maintain a minimum beach width for recreation. Hard structures such as groynes and breakwaters are,

however, no remedy for storm-induced erosion of sandy dunes and soft cliffs during conditions with relatively high surge levels (+3 to +5 m above mean sea level). Seawalls and revetments have to be built to stop dune and cliff erosion completely. Usually, these latter structures are built in regions (along boulevards of beach resorts) where natural dunes are absent or have been removed for recreational purposes.

Fig. 15 presents a showcase of nearly all available coastal structures along a Mediterranean coastal section of about 5 km in Sitges (south of Barcelona, Spain). Open groyne cells with a spacing of about 500 m can be observed at the northern part of the beach and partly closed cells are present along the southern side of the beach. The harnessed solution with T-head groynes and detached

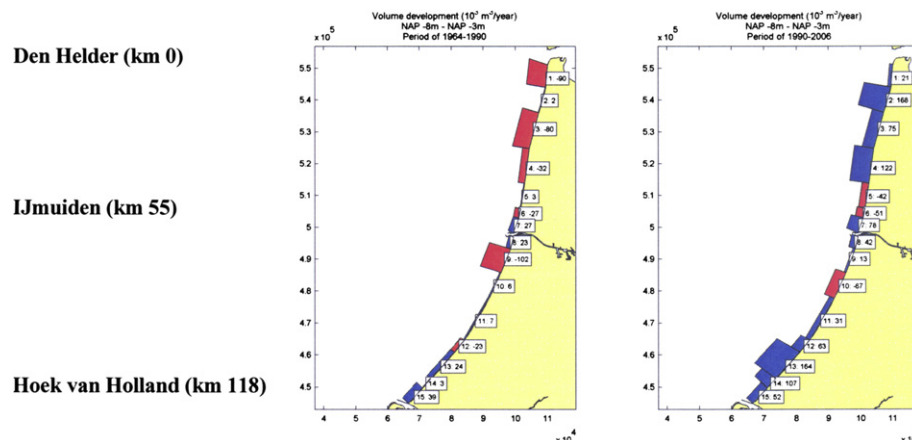


Fig. 13. Volume changes per year in zone between -8 m and -3 m (to MSL) depth contours (Red = erosion volume, -90,000 m³ in Section 1; Blue = deposition volume, 21,000 m³ in Section 1) Left: period 1964 to 1990 Right: period 1990 to 2006. (For interpretation of the references to colour in this figure legend, the reader is referred to the web version of this article.)

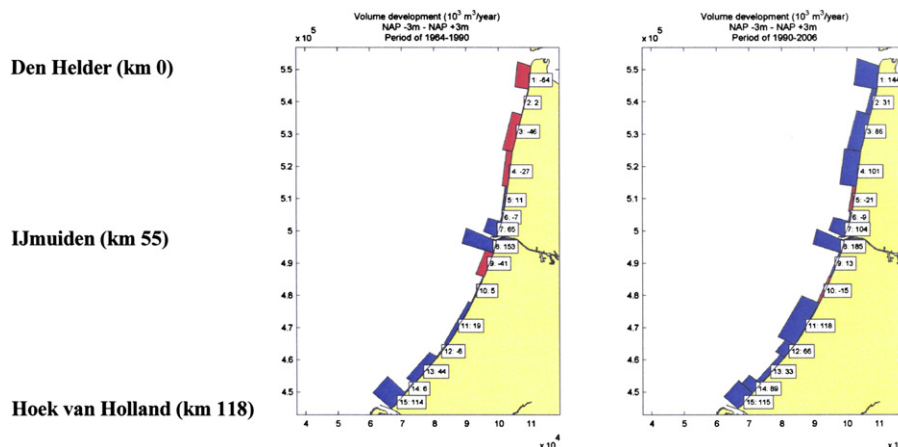


Fig. 14. Volume changes per year in zone between -3 m and $+3 \text{ m}$ (to MSL) depth contours. (Red = erosion volume, $-64,000 \text{ m}^3$ in Section 1; Blue = deposition volume, $144,000 \text{ m}^3$ in Section 1) Left: period 1964 to 1990 Right: period 1990 to 2006. (For interpretation of the references to colour in this figure legend, the reader is referred to the web version of this article.)

breakwaters on the southern side is necessary to retain the beach sand within the cells. Although this type of beach protection looks massive, it has been done nicely using natural rocks and the view from the boulevard is not unattractive. The beach offers opportunities to all types of beach recreation: swimming, surfing, fishing, etc for adults, youngsters and families with small children.

5.1. Groynes

Groynes are long, narrow structures perpendicular or slightly oblique to the shoreline extending into the surf zone (generally slightly beyond the low water line) to reduce the longshore currents and hence the littoral drift in the inner surf zone, to retain the beach sand between the groynes, to stabilise and widen the beach or to extend the lifetime of beach fills. A series of similar groynes (groyne field) may be constructed to protect a stretch of coast against erosion.

These structures are known as *beach groynes*. An overview of groynes is given by Fleming (1990), Kraus et al. (1994), US Army Corps of Engineers (1994), Van Rijn (1998, 2005) and many other

authors (Journal of Coastal Research SI 33, 2004). Groynes can also be applied to deflect tidal currents from the shoreline and/or to stabilise relatively deep tidal channels at a more offshore position. These structures are known as *inlet groynes* or *current groynes*.

Two main types of beach groynes can be distinguished:

- *impermeable, high-crested structures*: crest levels above $+1 \text{ m}$ above MSL (mean sea level);
- *permeable, low-crested structures*: crest level between MLW and MHW.

The differences in hydraulic behaviour of impermeable and permeable groynes have been elucidated by Dette et al. (2004), Trampenau et al. (2004) and Poff et al. (2004), based on laboratory test results and field observations. Impermeable groynes will tend to fully block the longshore current and transport over the entire length of the groynes. The longshore transport system is diverted seawards. Circulation and rip currents are generated due to set-up variations within the cell resulting in seaward transport of sediment on the updrift side of the groyne cells. The end result in a wave climate with one dominant wave direction is the typical saw-tooth bathymetry with scour channels near the groyne heads due to local rip currents. The saw-tooth effect increases with increasing groyne spacing.

In a severe wave climate with two dominant but adversary wave directions with respect to the shore normal, sediment erosion due to breaking waves, undertows and rips generally is dominant inside groyne cells along coasts consisting of fine sand (0.2 mm) resulting in narrow beaches (see Fig. 15; northern groynes at Sitges, Spain). Usually, large-scale erosion will develop down coast of the terminal groyne (lee-side erosion) if natural bypassing is absent. Erosion between the groynes and beyond the terminal groyne can only be mitigated by regular beach fills to stabilize the beach.

Groynes with permeability $<10\%$ largely act as impermeable groynes. Groynes with a permeability of about 30% – 40% essentially act as resistance to flow through the groynes without generation of circulation cells. Longshore velocities are reduced by about 50% over the entire length of the groynes. Velocities seaward of the groynes are much less than those with impermeable groynes. Natural bypassing of sediments is established resulting in a more continuous shoreline and large-scale lee-side erosion beyond the terminal groyne is suppressed. Wooden pile screens or pile clusters (single or double pile rows) offer an efficient solution with low construction and maintenance costs along micro-tidal coasts with



Fig. 15. Coastal protection by various hard structures at beach of Sitges, south of Barcelona, Spain.

mild wave climates (Baltic, Mediterranean, Florida Gulf coast). The highest groyne performance was found for a permeability in the range of 30%–40%. The groyne length should be slightly larger than the surf zone width (up to the landward flank of the inner bar trough) and groyne spacing should be equal to groyne length (Trampenau et al., 2004). Groyne height is about 0.5 m above mean sea level (minimum visual intrusion) in tide less basins and variable (sloping downward in seaward direction) in tidal basins. The maximum pile length above the sea bed is about 3 m. Initial beach fills should be used along relatively steep, eroding beach profiles. Using these types of permeable groynes, the longshore transport can be significantly reduced (factor 2 to 3). Favourable experiences are obtained along the German Baltic coast and along the West Florida coast (Naples Beach, Poff et al., 2004).

High-crested, impermeable beach groynes generally have lengths (L) between 50 and 100 m; crest levels beyond +1 m above MSL (mean sea level); groyne spacing (S) between 1.5 and 3 times the length of the groynes ($S = 1.5$ to $3L$), as shown by experimental research in a wave basin (Özölçer et al., 2006). If the groyne spacing is too large ($>3L$), the longshore current and transport will be re-established resulting in a very oblique or curved shoreline (saw-tooth shoreline) between the groynes. The cover layer of armour units should be well placed to obtain a smooth, visually attractive scenery.

High-crested, impermeable groynes should only be considered along exposed, eroding coasts of fine sand (0.2 mm), if the recession rates are exceeding 2 m per year. The length of the groynes should be smaller than the width of the surf zone during storm conditions ($H_{s,0} > 3$ m) to promote sufficient bypassing of sand. Along beaches of fine sand these types of groynes will only reduce beach erosion (factor 2 to 3), but not stop it completely, as the waves can easily propagate into the compartments. Regular beach fills are required to reduce the beach erosion to manageable quantities. At the north section of the Holland coast (The Netherlands) consisting of 0.2 mm sand, the beach groynes built around 1850 have reduced shoreline erosion from about 3 to 5 m/year to about 1–2 m/year. Since 1990, regular beach nourishments (between 200 and 400 m³/m/year) are used to completely stop shoreline erosion along the Holland coast.

Groyne crest levels should not be much larger than about +1 m to allow bypassing of sediment during high tide and stormy conditions to reduce lee-side erosion. High crest levels prevent sediment bypassing and are unattractive for beach recreation. Groyne notching by creating openings along the groyne in the most active longshore transport zone (swash zone and inner surf zone) has been applied along USA beaches to improve the sediment bypassing of existing groyne fields in combination with beach fills between the groynes (Rankine et al., 2004a,b; Donohue et al., 2004; Wang and Kraus, 2004).

Relatively long groynes at close spacing (narrow cells) may be very effective along swell dominated ocean coasts consisting of relatively coarse sand (0.3–1 mm) at the upper beach zone and finer sand in the lower beach zone. Regular onshore movement of coarse sand by near-bed transport may result in wide beaches between the groyne, as shown by the Miramar coast, Argentina.

The effectiveness of straight groynes can be substantially increased by using T- or L- head groynes, which may be designed along very exposed, eroding coasts to reduce the wave energy into the compartments and to prevent/diminish the generation of rip currents near the groyne heads, see the beaches of Miramar, Argentina; Longbrach-Bradley, USA and Sitges, Spain (Fig. 15). The length of the head should be of the same order of magnitude as the length of the groyne ($L_{\text{head}} \cong L$). Experimental research in a wave basin and field experience at a Turkish beach shows that T-groynes work well for $S/L = 1.5$ and $L_{\text{head}} \cong L = 60$ m (Özölçer et al., 2006). Measures should be taken to prevent wave reflection and the

production of turbulence (vortex streets) near these types of groynes. Round headed groynes are more attractive in conditions with strong currents and low waves (inlets).

5.1.1. Numerical modelling results

A numerical study on the design of straight and T-head groynes using the DELFT3D-model has been performed at Deltares/Delft Hydraulics (Eslami Arab, 2009). The model was operated in 3D mode using 11 layers. Waves were modelled using the SWAN-model including diffraction. The water level was constant (no tide). The results of tests NT4 and NT5 (two groynes) of Badiei et al. (1994) were used to validate the model. Qualitative agreement between measured and computed bed level changes was obtained. The validated model was used to compute the effect of various groyne fields on the nearshore morphology of a sandy coast ($d_{50} = 0.2$ mm). The offshore wave height (irregular waves) is $H_{s,0} = 1$ m with $T_p = 5$ s and offshore wave incidence angle of 15° . The width of the surf zone (L_b) is about 160 m. The groyne length L_g is 160 m. The groyne spacing S has been varied. In all runs there was accretion at the updrift side of the most updrift groyne and erosion at the downdrift side of the most downdrift groyne, see Fig. 16. The volumetric changes inside the cells vary in the range of $\pm 1\%$ of the maximum accommodation volume between the groynes. Minor trapping prevails for the runs with a groyne length equal to the width of the surf zone. The volumetric changes inside the groyne cells are related to the amount of sand (both bed load and suspended load) passing the line through the tip of the groynes. Onshore-directed transport processes (wave-related bed load and suspended load transport) occur due to wave-asymmetry effects. Offshore-directed transport processes occur due to undertow velocities and due to the horizontal diffusive effect. This latter effect of horizontal diffusive transport is caused by the circulation velocities and the gradients of the sediment concentrations. The concentrations between the groynes generally are larger than those outside the groyne cells due to the decreasing water depths and continuously breaking waves (fully saturated energy). Generally, this will result in diffusive transport from the zone with high concentrations inside the cells to low concentrations outside the cells. The actual suspended load passing the line through the tips of the groynes strongly depends on the magnitude and direction of the circulation velocities between the groynes and the concentration distribution. In Fig. 17 it can be seen that the computed circulation velocities (and hence the suspended load) vary from case to case (wide cells to narrow cells). These processes cannot yet be computed with high accuracy and thus the relatively small values of the computed accretion/erosion volumes inside the groyne cells are not very accurate. The computational results, however, do show that the potential trapping of sand inside the cells is extremely small. Hence, the capacity of the groyne cells to trap or retain sand is small. This is also evident from the results of the laboratory experiments of Hulsbergen et al. (1976) and Badiei et al. (1994).

The trapping of sand increases with decreasing width of the cells. The trapping of sand also increases with increasing offshore wave angle, because the wave shadow zone inside the cells (with lower wave heights and smaller sand concentrations) is more pronounced resulting in diffusive transport from outside to inside the cells.

A run with tidal water level variations and tidal velocities showed less trapping probably because the circulation velocities inside the cells were enhanced by the tidal velocities resulting in an increase of the offshore-directed diffusive transport. Furthermore, erosion is enhanced during ebb conditions with relatively small water depths between the groynes and relatively large circulation velocities.

Groynes will be more effective on beaches with coarser sand, where onshore bed load transport generally prevails. In that case

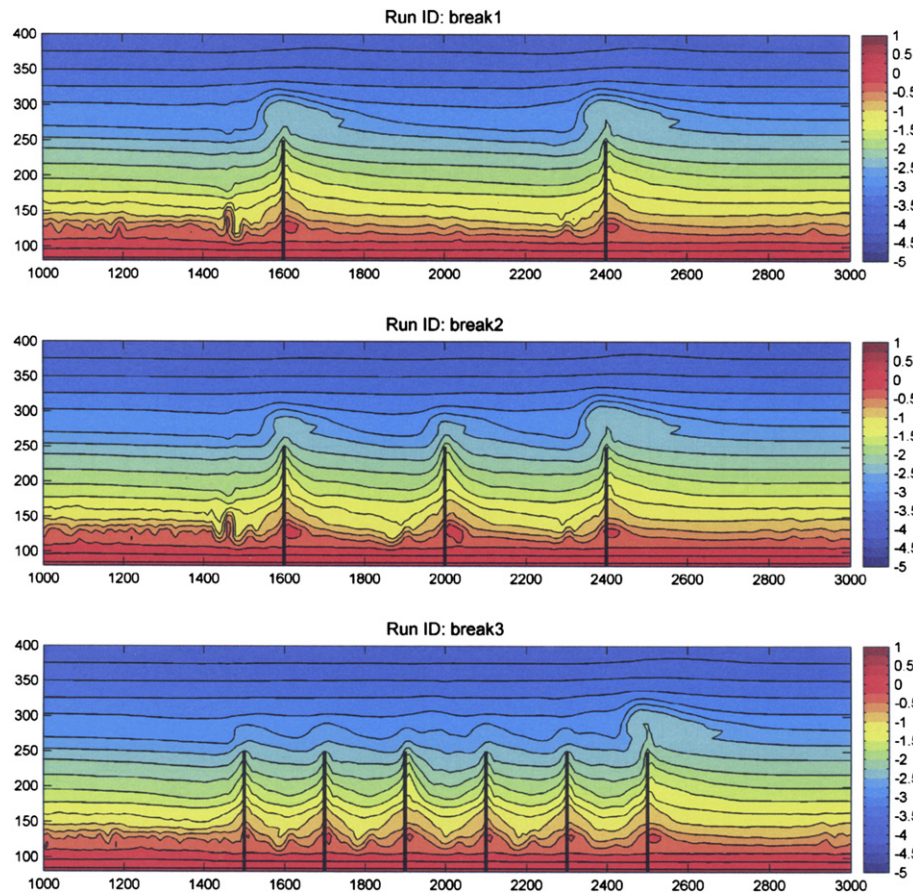


Fig. 16. Computed bathymetry after 45 days of Run B1, B2 and B3 (Net longshore transport from right to left $\leftarrow$) Top: Run B1: Angle = 15° . Length $L_g = 160$ m, Spacing $S = 800$ m, ($S/L_g = 5$, $L_g/L_b = 1$) Middle: Run B2: Angle = 15° . Length $L_g = 160$ m, Spacing $S = 400$ m, ($S/L_g = 2.5$, $L_g/L_b = 1$) Bottom: Run B3: Angle = 15° . Length $L_g = 160$ m, Spacing $S = 200$ m, ($S/L_g = 1.25$, $L_g/L_b = 1$).

the groynes usually are short (up to the low water line) to prevent the longshore movement of the coarse sediment particles in response to the wave climate (beach rotation/oscillation).

The numerical modelling study at Deltares (Eslami Arab, 2009) also included the performance of T-head groynes focussing on an existing case along the coast of Turkey (Özölçer et al., 2006). The characteristics of the two T-head groynes are: length of about 50 m beyond the still water line, spacing of 100 m, length of head of about 40 m. The offshore wave height is set to 1 m (period of 7 s; duration of 15 days; no tide). The offshore wave incidence angle is 12° . The sediment size is about 0.33 mm. Sensitivity runs show that the T-head groynes can be best modelled using the real dimensions of the groynes (not as thin walls) using vertical boundaries or sloping boundaries on both sides. The grid resolution should be rather high (2 m) to obtain accurate results. The roughness of the walls or slopes (often rock) should be represented by realistic values. Fig. 18 shows the computed results after 15 days with a constant wave height of 1 m. The computed bathymetry is in close qualitative agreement with the observed bathymetry after 6 months (including calm periods with no waves). The cell between the groynes is filled with sand due to trapping of bed load and suspended load from outside.

5.2. Detached breakwaters and reefs

A detached breakwater (Fig. 19) is herein defined as a hard shore-parallel structure protecting a section of the shoreline by forming a shield to the waves (blocking of incident wave energy).

The crest may be positioned above the still water level (emerged) or below the still water level (submerged) and has a width of the order of the local water depth. There are many variants in the design of detached breakwaters, including single or segmented breakwaters with gaps in between, emerged (crest roughly 1 m above high water line) or submerged (crest below water surface), narrow or broad-crested, etc. Submerged breakwaters are also known as reef-type breakwaters and are attractive as they are not visible from the beach. A reef (hard or soft) is a relatively wide, submerged structure in the shallow nearshore zone.

Low-crested structures are often constructed to increase the lifetime of beach fills along straight or slightly curved beaches or along pocket beaches suffering from structural erosion in micro-tidal conditions (Mediterranean). Sometimes, low submerged breakwaters are constructed as sills between the tip of groynes to support the seaward toe of beach fills (perched beaches); Italian coast near Carrara.

Submerged structures cannot stop or substantially reduce shoreline erosion (dune-cliff erosion) during storm conditions, as most of the waves will pass over structure to attack the dune or cliff front. Supplementary beach nourishments are required to deal with local storm-induced shoreline erosion (especially opposite to gaps). Downdrift erosion generally is manageable as longshore transport is not completely blocked by low-crested structures. A major problem of submerged breakwaters and low-crested emerged breakwaters is the piling up of water (wave-induced set-up) in the lee of the breakwaters resulting in strong longshore currents when the breakwater is constructed as a long

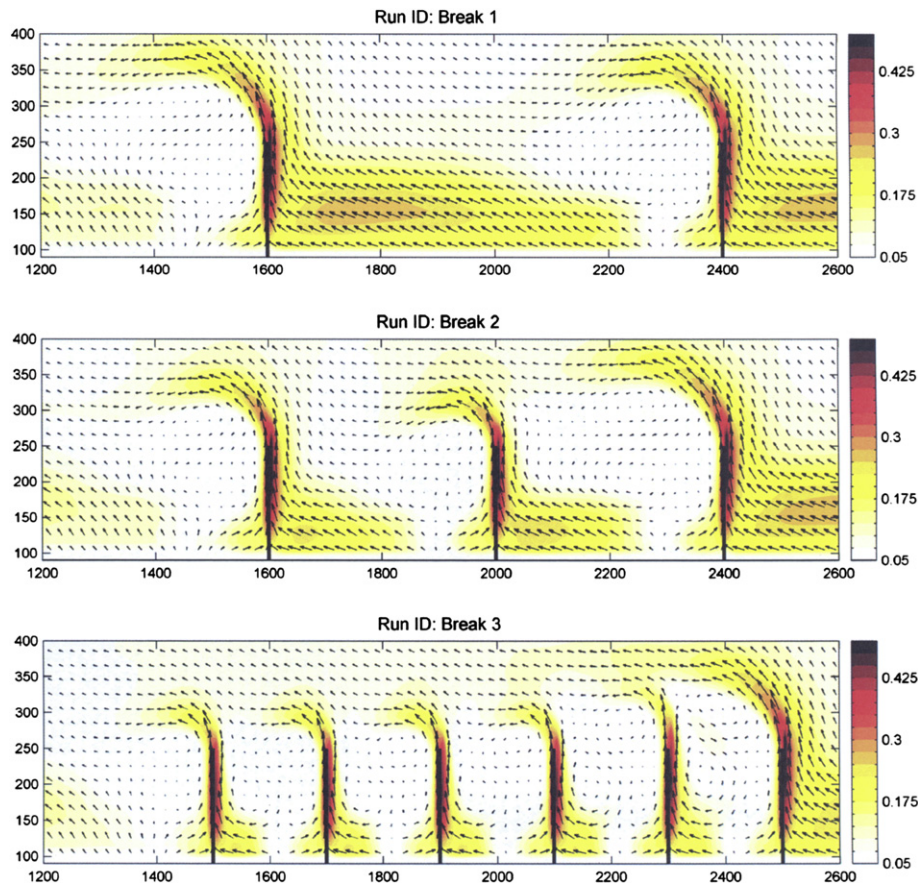


Fig. 17. Computed depth-averaged velocities at initial conditions of Run B1, B2 and B3 (longshore current from right to left $\leftarrow$) Top: Run B1: Angle = 15° . Length $L_g = 160$ m, Spacing $S = 800$ m, ($S/L_g = 5$, $L_g/L_b = 1$) Middle: Run B2: Angle = 15° . Length $L_g = 160$ m, Spacing $S = 400$ m, ($S/L_g = 2.5$, $L_g/L_b = 1$) Bottom: Run B3: Angle = 15° . Length $L_g = 160$ m, Spacing $S = 200$ m, ($S/L_g = 1.25$, $L_g/L_b = 1$).

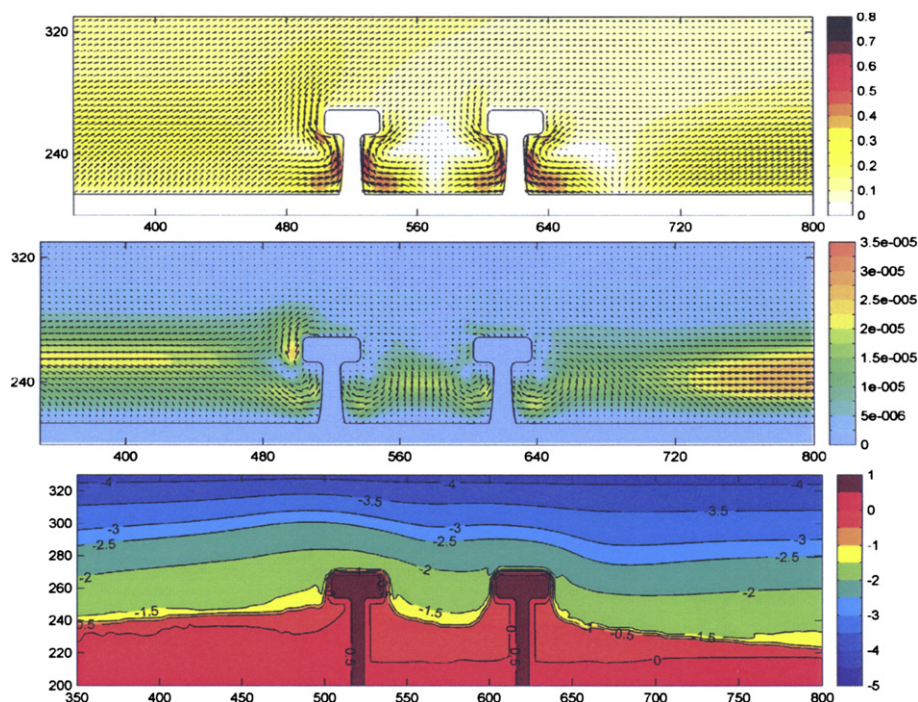


Fig. 18. Computed results for T-head groynes along coast of Turkey (longshore current and transport from left to right $\rightarrow$) Top: depth-averaged velocity vectors. Middle: transport vectors (in m^2/s) Bottom: bathymetry after 15 days (in m).

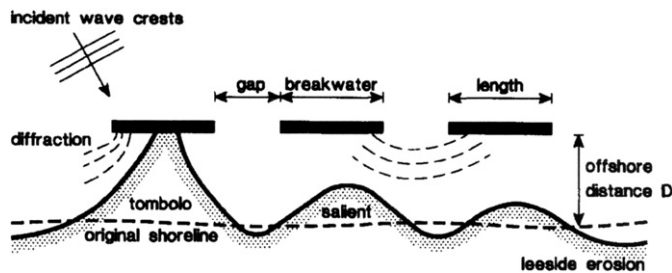


Fig. 19. Shore-parallel detached breakwaters.

uninterrupted structure (no gaps) or in strong rip currents through the gaps when segmented structures are present.

The wave filter effects depend on the mean water level conditions, the incident wave parameters and the structural geometry. The shoaling/breaking processes in front of a structure, which is frequently overtopped, increases the pumping of water fluxes over the detached breakwater. Resulting sediment fluxes and morphodynamic evolution are, therefore, a function of the wave and the circulation fields associated to the structure. Any attempt to understand and model the morphodynamic impact requires the explicit consideration of terms driving the pumping and circulation effects. Cáceras et al. (2005a,b) have studied various methods to deal with wave overtopping and enhanced mass fluxes and associated design difficulties for low-crested breakwaters. If a submerged or low-crested breakwater is not designed properly, additional negative morphological effects such as local scour and shoreline erosion may easily occur. For example, the submerged breakwater (consisting of precast interlocking units without gaps) built on the lower east coast of Florida (USA), approximately 7 km south of the entrance of the Port of Palm Beach was later (1995) removed because of excessive erosion problems in the lee of the breakwater (Dean et al., 1997; Browder et al., 2000). A review of submerged structures by Ranasinghe and Turner (2006) reveals that a majority of these structures have resulted in shoreline erosion in their lee. Their conclusion is that the use of these structures is likely to remain relatively limited.

Disadvantages of detached breakwaters are the relatively high construction and maintenance costs, inconvenience/danger to swimmers and small boats and aesthetic problems (visual blocking of horizon).

Most emerged breakwaters have been built along micro-tidal beaches in Japan, in the USA and along the Mediterranean. Few have been built along open, exposed meso-tidal and macro-tidal beaches. The crest should be in the range of +1 m to +4 m above MSL depending on tidal range to be effective against storm-induced shoreline erosion. The type of beach plan form in the lee of these structures strongly depends on dimensions and geometry (L = breakwater length, D = offshore distance to original shoreline, L_{gap} = length of gap between segments, see Fig. 19). The beach can be built out to the structure (permanent tombolo for $L/D > 1$) or not (salient for $L/D < 1$), if sufficient sediment is available; otherwise additional beach fills (nourishments) are required. Shoreline erosion generally occurs opposite to the gaps for $L_{\text{gap}}/L > 1.3$, but is minor for $L_{\text{gap}}/L < 0.8$. Emerged breakwaters cannot stop storm-induced erosion completely, as large storm waves will pass over the structure in conditions with high surge levels above the crest level. Emerged breakwaters can be built: (1) to increase the local beach width and to stop the beach rotation along recreational pocket beaches between two headlands and (2) to reduce chronic shoreline erosion and storm-induced erosion to acceptable limits in combination with regular nourishments to restore the original

shoreline. The ideal emerged breakwaters in terms of coastal protection are made of rock and built close to the shore with a high crest level and small gap lengths, but such a structure will largely block the horizon and is not attractive in terms of beach recreation.

Bricio et al. (2008) have analysed 27 detached breakwater projects along the northeast Catalan coastline (almost tide less) of Spain based on pre- and post-project aerial photographs. The offshore distance D is defined as the distance to the original shoreline (in the range of 80–234 m). The (emerged) breakwater lengths L are in the range of 57–236 m. Tombolos are present for $L/D > 1.3$ and salients for $0.5 < L/D < 1.3$. No information was available on additional beach nourishments.

Overviews of experiences are given by Rosati (1990) and Chasten et al. (1993). Liberatore (1992), Lamberti and Mancinelli (1996) and Lamberti et al. (1985) give information of the experiences with emerged and submerged breakwaters along the Italian coasts.

Based on the results observed along various micro-tidal USA-sites (Stone et al., 1999; Mohr et al., 1999 and Underwood et al., 1999), Mediterranean coasts (Cáceres et al., 2005a,b) and meso-tidal sites in the UK (Fleming and Hamer, 2000), it is concluded that the design of an emerged breakwater scheme is not a straightforward process, but rather an iterative process consisting of an initial design phase based on mathematical and physical modelling, the testing of the design by means of a field pilot project including a detailed monitoring programme and the fine tuning of the design by modification of breakwater lengths based on the field experiences.

5.2.1. Numerical modelling results

The basic effect of a detached breakwater on the hydrodynamics and the morphology can be very well computed by mathematical models, as shown by Fig. 20 (Deltares/Delft Hydraulics, 1997; see also Bos et al., 1996). The breakwater is situated on a plane sloping beach with a slope of 1:50. The length of the detached breakwater is $L = 300$ m. The offshore distance D has been varied in the range of 120–500 m.

Irregular waves normal to the shore (including directional wave spreading) with an offshore wave height of $H_{\text{rms}} = 2$ m and a peak wave period of $T_p = 8$ s have been used. The sediment diameter is $d_{50} = 0.25$ mm ($d_{90} = 0.35$ mm); the sediment fall velocity is $w_s = 0.031$ m/s. The bottom roughness is $s = 0.05$ m. The breaker line outside the breakwater region is about 200 m from the shore, where the water depth to the still water level is 4 m (no tide).

The computed wave height patterns show a significant decrease of the wave height in the lee of the breakwater. Pronounced circulation zones are generated in the lee of the breakwater due to variations of the set-up along the shore (relatively low set-up values in lee and relatively high set-up values on both sides of the breakwater). Maximum velocities are of the order of 0.5 m/s for a relatively large distance from the shore ($D = 500$ m; $L/D = 0.6$) to 0.8 m/s for a relatively small distance to the shore ($D = 120$ m; $L/D = 2.5$). The computed morphology shows the development of a double tombolo for an offshore breakwater distance of 120 m, a single tombolo for offshore distances in the range of 150–300 m and a salient for an offshore distance of 500 m. In the latter case large scour holes can be observed at the tips of the breakwater. These scour holes do not develop when tombolos are generated. The generation of a double tombolo with a dead water zone in between is not very realistic. In practise, this zone will be rapidly filled with sediment by longshore transport in the swash zone, which is not included in the mathematical model.

Overall, the model predicts tombolo-morphology for $L/D > 1$ and salients for $L/D < 0.6$. The model results are in excellent agreement with the data of Rosen and Vajda (1982), Harris and

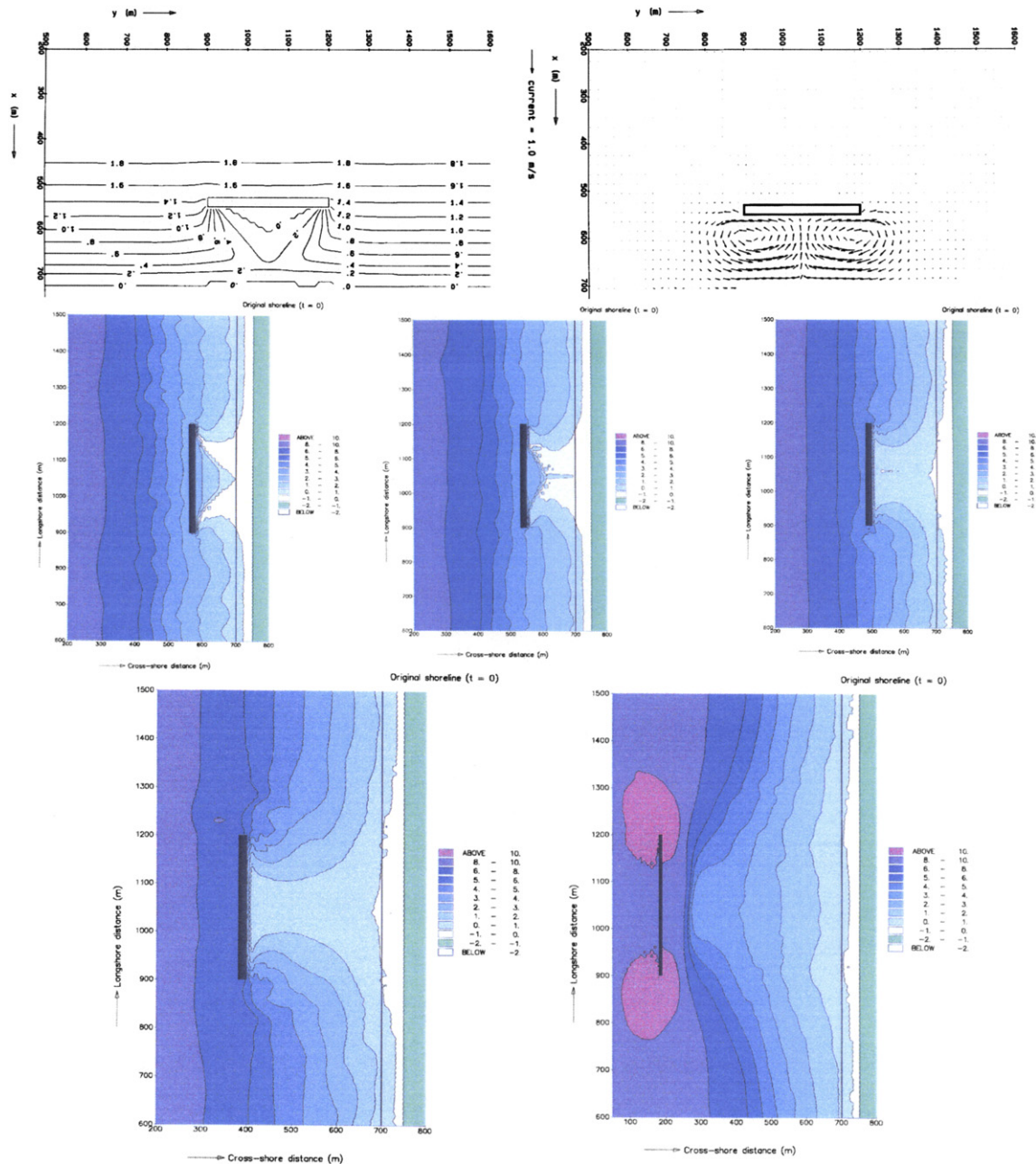


Fig. 20. Computed wave height, flow field and morphology of detached breakwaters. Top left, right: wave height and flow field (breakwater at 150 m from shore). Middle: morphology after 50 days (breakwater at 120, 150 and 200 m from shore) Bottom: morphology after 75 days (breakwater at 300 and 500 m from shore).

Herbich (1986) and others, which all are based on the development of tombolos for $L/D > 1$.

6. Longshore coastal variability due to hard structures

Both groynes and detached breakwaters are structures that are based on the compartmentalisation of the coast into a series of small-scale coastal cells, each with its own sediment budget. The idea is that as long as the sediment budget within each cell remains approximately constant, the erosion of the shoreline is minimum. These ideas will hereafter be explored by example computations for a groyne field along an eroding coastal section with a length of 15 km. Shoreline changes can be simply understood by considering the sediment continuity equation for the littoral zone (roughly the

surf zone) with alongshore length Δx , cross-shore length Δy_s and vertical layer thickness (h). The sand volume balance reads:

$$h(\Delta y_s / \Delta t) + \Delta Q_{LS} / \Delta x = 0 \quad (1)$$

with: y = cross-shore coordinate, x = longshore coordinate, y_s = shoreline position, h = thickness of active littoral zone layer, Q_{LS} = longshore transport rate or littoral drift (bed-load plus suspended load transport in volume including pores per unit time, in m^3/s). Basically, Equation (1) which is solved by the LONGMOR-model (Van Rijn, 1998, 2002, 2005) states that a coastal section erodes if more sand is carried away than supplied; vice versa coastal accretion occurs if there is a net supply. Equation (1) shows that the shoreline changes are linearly related to the assumed depth (h) of the active zone.

The example computation refers to chronic erosion along a schematised sandy coast. The local wave climate (North Sea wave climate) is assumed to generate a net longshore transport of about $375,000 \text{ m}^3/\text{year}$ at $x = 0$ and about $500,000 \text{ m}^3/\text{year}$ at $x = 15 \text{ km}$. Hence, a significant longshore transport gradient is imposed resulting in a chronic coastal recession of about 7 m in 5 years along this coastal section (see Fig. 21). This value is equivalent to an overall erosion of $630,000 \text{ m}^3$ (recession $\times$ profile height $\times$ length = $7 \times 6 \times 15000$) over 5 years along this section of 15 km.

The LONGMOR-model has been used to determine the consequences of creating coastal cells by means of long groynes (head-land type of groynes). Relatively large groyne spacing of 5000 and 1000 m are explored to demonstrate the effects. The cross-shore length of the groynes is about 100–150 m beyond the mean water line (MSL) down to the -5 m depth contour. The layer thickness or profile height of the dynamic littoral zone is assumed to be 6 m. The beach sediment is sand with $d_{50} = 0.2 \text{ mm}$ and $d_{90} = 0.3 \text{ mm}$. The local beach slope is assumed to be $\tan\beta = 0.05$ (slope of 1:20 from water line to -6 m depth contour). The blocking coefficient of the groynes is assumed to be 50% (and thus 50% bypassing of sediment). The local wave breaking coefficient is assumed to be $\gamma_{br} = 0.6$. The longshore grid size is 50 m and the time step is 0.01 days. The shoreline changes over a period of 5 years have been determined using a wave climate with offshore waves in the range of 0.5–4 m and offshore wave incidence angles of 30° and -15° with respect to the coast normal yielding a net longshore transport rate of $375,000 \text{ m}^3/\text{year}$ at $x = 0 \text{ m}$ based on the method of Van Rijn (2005). The CERC-method produces values which are twice as large; the Kamphuis-method produces values which are about 30% smaller (Van Rijn, 2005).

Fig. 21 shows the typical saw-tooth shoreline behaviour (spacing of 5000 m; $S/L = 25$ to 30) with accretion on the updrift side and erosion on the downdrift side of the groynes for a wave climate with one dominant direction. The maximum local erosion is of the order of 60 m after 5 years, which is much larger than the original chronic erosion of about 7 m after 5 years. Smaller shoreline recession values can be observed inside a groyne field with a spacing of 1000 m ($S/L = 5$ to 6), see Fig. 22. The maximum shoreline recession on the downdrift side of the last (terminal) groyne is not affected by the groyne spacing and is of the order of 80 m after 5 years. Smaller spacing are required to reduce the erosion within the compartments to acceptable limits, but smaller spacing are not very cost-effective (higher construction costs). In both cases (Figs. 21 and 22) the blocking coefficient of the groynes is assumed to be 50%. Larger blocking coefficients (up to the maximum value of 1; complete blocking) will result in larger

shoreline recession values at the downdrift groynes. The results also linearly depend on the assumed depth (here $h = 6 \text{ m}$) of the active zone. Shoreline erosion will be much less in a milder wave climate (Mediterranean). Shoreline erosion will be more symmetric in a multi-directional wave climate.

It is concluded that the implementation of a groyne scheme with relatively wide compartments leads to an increase of the variability of the local shoreline with maximum recession values much larger than the initial shoreline recession in the case of a wave climate with one dominant direction. Regular artificial beach restoration within each cell by nourishment (in the range of 100,000 to $500,000 \text{ m}^3$ after 5 years) will be required to keep the lee-side erosion within acceptable limits, which is not very attractive from a management perspective. Furthermore, straight groynes are not very efficient in cross-shore direction, as the erosion of the shoreline by cross-shore transport processes (wave-induced undertow) can carry on freely.

Another example is the shoreline erosion on both sides of an offshore reef. To protect the boulevard and beaches of a major city at the Holland coast (beaches of 0.2 mm sand), the effectiveness of an offshore reef with a crest level at 1 m below mean sea level and a length of 3 km at a depth of 10 m (1.5 km from the coast) was explored by using the DELFT3D modelling system. The local wave climate has two dominant directions: south-west and north-west. The net longshore transport being the sum of two large, but opposite values is quite small (order of $100,000 \text{ m}^3/\text{year}$ to the north; from left to right), see Fig. 23 Top. However, when an offshore reef is present, the net transport at the southern beach side ($x = 7 \text{ km}$) of the reef increases enormously because the longshore transport from the opposite direction is largely blocked resulting in a net longshore transport of about $400,000 \text{ m}^3/\text{year}$ at $x = 7 \text{ km}$. At the northern beach side ($x = 10 \text{ km}$) of the reef the net transport of $100,000 \text{ m}^3/\text{year}$ to the north in the old situation is turned into a net transport of $300,000 \text{ m}^3/\text{year}$ to the south in the new situation. These significant transport variations over a length of about 3 km lead to relatively large shoreline variations after 5 years (see Fig. 23 Bottom): accretion of about 150 m in the middle of the reef zone and erosion of about 75 m on both sides of the reef zone. Basically, sand from both sides is carried into the middle lee zone of the reef. The reef functions well in terms of protection of the beach and boulevards against wave attack in the lee of the reef but serious side effects (erosion) are introduced which have to be mitigated by nourishment.

It is concluded that these types of shoreline structures seem to make things worse by introducing excessive longshore variability and local erosion and may therefore be unattractive to control beach erosion if alternative solutions are available (nourishment).

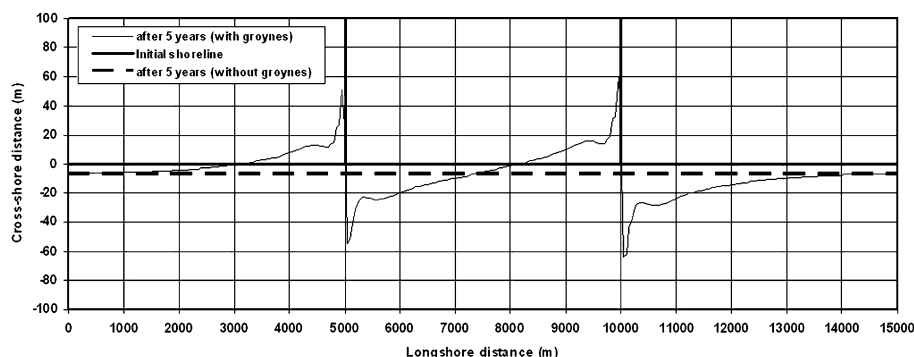


Fig. 21. Shoreline behaviour after 5 years based on imposed longshore transport gradient with and without groynes (net longshore transport from left to right); 2 groynes with spacing of 5000 m.

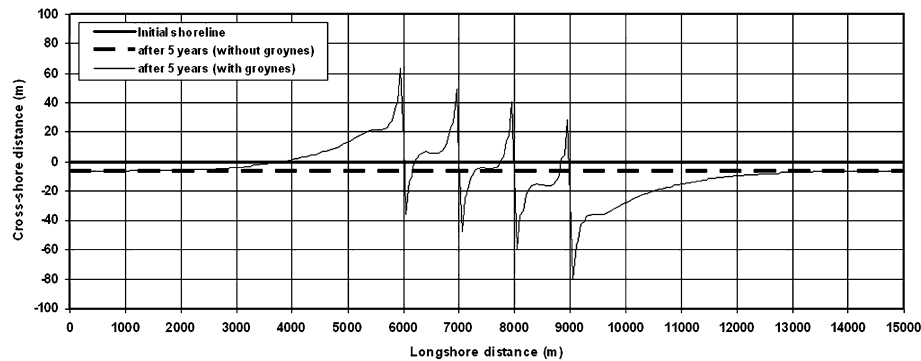


Fig. 22. Shoreline behaviour after 5 years based on imposed longshore transport gradient with and without groynes (net longshore transport from left to right); 4 groynes with spacing of 1000 m.

An exception is the erosion near a tidal inlet. In that case the most downdrift groyne can function as a long jetty preventing that the longshore drift moves into the inlet. Another exception is the erosion along a beach where nearby sand sources are very scarce. In that case a very harnessed solution consisting of relatively small cells with T-head groynes and detached breakwaters (see Fig. 24) can be used to maintain a minimum beach for recreation and to reduce coastal erosion. The compartments should be small to contain the sediments within the compartments. When

a boulevard is present, the back-beach needs to be protected by revetments and seawalls. Such a solution is expensive and requires relatively large maintenance budgets in a severe wave climate, but may be necessary when regular beach nourishment by (scarce) sandy material is not economically feasible.

The cross-shore length of the groynes should be as small as possible (of the order of 50–100 m, see Fig. 24) to reduce on the construction coasts and to minimise the effect on the longshore transport and thereby to minimize the lee-side erosion effects.

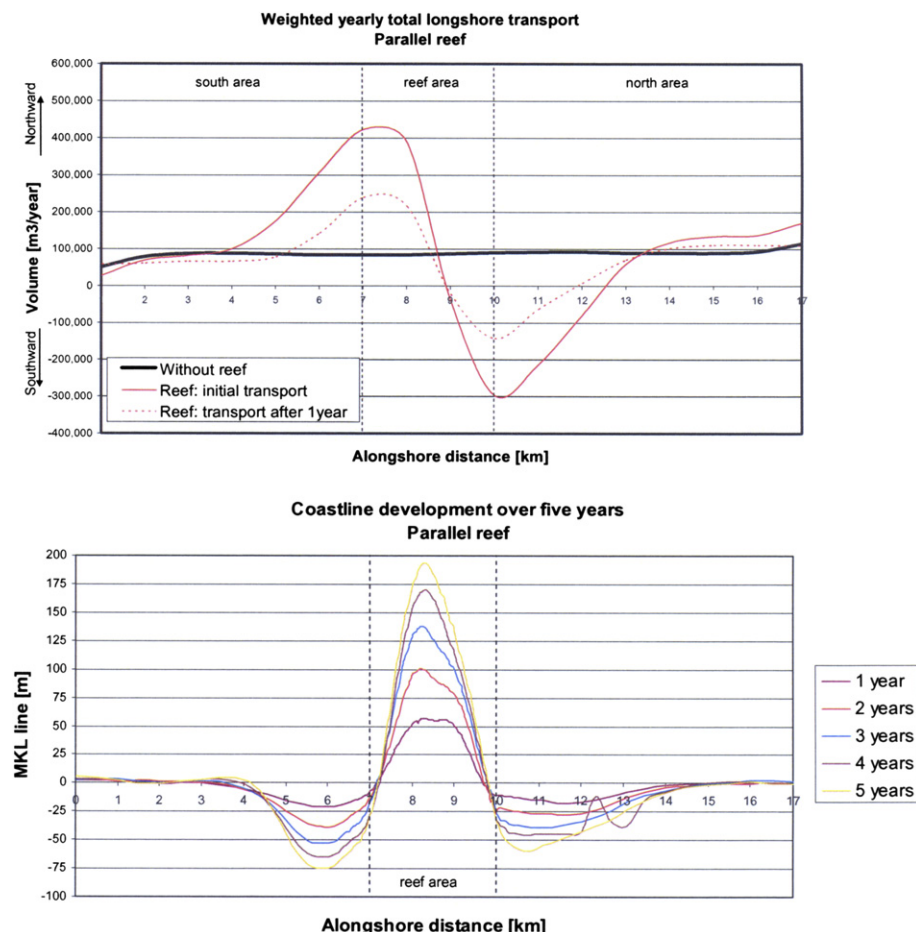


Fig. 23. Net longshore transport (Top) and shoreline variation (Bottom) in lee of offshore reef (Van der Hout, 2008).

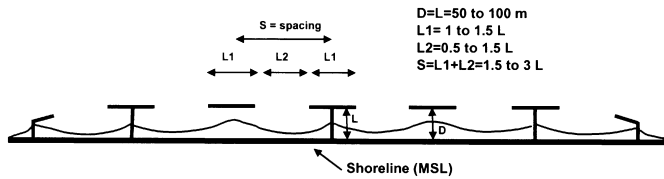


Fig. 24. Coastal protection by T-head groynes and detached breakwaters.

Generally, the crest level near the groyne tip should be slightly higher (0.5–1 m) than the mean sea level (MSL) and the crest level near the dune toe should be slightly lower than the local beach.

7. Summary, evaluation and conclusions

Coastal erosion strongly depends on the type of coast (exposure, wave climate, surge levels, sediment composition, beach slope). Coastal erosion has both cross-shore and long shore components. Dune erosion during extreme events with high surge levels up to 5 m mainly is a cross-shore process bringing the sediments from the immobile dune front into the mobile littoral system. Computed dune erosion values are of the order of 20 m³/m for a surge level of 1 m and up to 200 m³/m for a large surge level of 5 m.

If there is a substantial loss of sediment over a period of 5 years or so, it maybe considered to nourish the area with a sediment volume equal to the observed volume loss. Sand nourishment is the mechanical placement of sand in the nearshore zone to advance the shoreline or to maintain the volume of sand in the littoral system. It is a soft protective and remedial measure that leaves the coast in a more natural state than hard structures and preserves its recreational value. The method is relatively cheap if the borrow area is not too far away (<10 km) and the sediment is placed at the seaward flank of the outer bar (shoreface nourishment) where the navigational depth is sufficient for hopper dredgers. Beach nourishment is about twice as expensive as shoreface nourishment and even more if lifetimes are very short (1–2 years).

Beachfills are mainly used to compensate, local short-term erosion in regions with relatively narrow and low dunes (in regions of critical coastal safety) or when the local beach is too small for recreational purposes. Shoreface nourishments (also known as feeder berms) are used in regions of relatively wide and high dunes (relatively safe coastal regions) to maintain or increase the sand volume in the nearshore zone with the aim to nourish the nearshore zone on the long term by natural processes (net onshore transport).

Practical experience of the Holland coast shows that large-scale erosion can be stopped by massive beach and shoreface nourishment over long periods of time (20 years). This approach is only feasible if sufficient quantities of sand are available and the dredging and dumping costs are acceptable (about 10–15 million Euro per year or 100 to 150 Euro/m coastline per year for the Holland coast with a total length of about 100 km). Although sand nourishment offers an attractive solution in terms of coastal safety and natural values, it may not be the cheapest solution because of the short nourishment lifetimes involved (regular renourishments every 2–5 years). In regions where sand is not easily available, it should be assessed whether hard structures may offer a more cost-effective solution to deal with chronic erosion, particularly if rock is available at nearby locations.

Generally, hard coastal structures such as groynes, detached breakwaters and artificial reefs are built in urban areas to significantly reduce coastal beach erosion and to maintain a minimum beach for recreation. Preferably, these types of hard structures

should be built at locations (downdrift of protruding coastal section) where the transport gradient is almost zero at the transition from increasing to decreasing longshore transport to prevent lee-side erosion. Hard structures such as groynes and breakwaters are, however, no remedy for dune erosion during conditions with relatively high surge levels (above the dune toe level). Seawalls and revetments have to be built to stop dune erosion completely for reasons of coastal defence in urban areas.

Computational results show that the implementation of a groyne scheme leads to an increase of the variability of the local shoreline with maximum recession values much larger than the initial shoreline recession in the case of a wave climate with one dominant direction. Artificial beach restoration within each cell by dredging will be required regularly to keep the lee-side erosion within acceptable limits, which is not very attractive from a management perspective. Furthermore, straight groynes are not very efficient in cross-shore direction, as the erosion of the shoreline by cross-shore transport processes (wave-induced undertow) can carry on freely.

Groyne structures seem to make things worse by introducing excessive longshore variability and local downdrift erosion and are therefore not attractive to control erosion if alternative solutions are available (nourishment). An exception is the erosion near a tidal inlet. In that case the most downdrift groyne can function as a long jetty preventing that the longshore drift exits into the inlet.

Another exception is the erosion along a beach where nearby sand sources are very scarce. In that case a very harnessed solution consisting of relatively small cells with T-head groynes and detached breakwaters can be used to maintain a minimum beach for recreation and to reduce coastal erosion in urban areas. When a boulevard is present the back-beach needs to be protected by revetments and seawalls. Such a solution is expensive and requires relatively large maintenance budgets in a severe wave climate, but may be necessary when regular beach nourishment by (scarce) sandy material is not economically feasible.

Hard structures (groynes, detached breakwaters) require relatively high capital investments plus the continuous costs of maintenance works (storm damage, subsidence, scour problems, redesign, etc.) and costs of supplementary beach nourishments to deal with local erosion problems (opposite to gaps and along the downdrift side). Indicative figures are given in Table 1. The construction costs of rubble-mound groynes with a length of 200 m (spacing of 600 m) are about 1 million Euro. Adding interest and maintenance costs, this will be about 3–5 million Euro over a lifetime of 50 years or about 100–150 Euro perm coastline per year. The construction of detached breakwaters is considerably larger in the range of 200–300 Euro per m coastline per year. The use of soft shoreface nourishments requires less initial investments, but the costs of regular maintenance of the feeder berm (every 3–5 years) have to be added resulting in annual costs of about 100–200

Table 1
Investment cost of shoreline protection measures.

Type of structure	Construction + maintenance costs over 50 years (in Euro per m coastline per year)
Straight rock groynes	50–150
Rock revetments	100–200
Shoreface nourishments (every 5 years)	100–200 (if sand is easily available)
Seawalls	150–300
Beach fills (every 3 years)	200–300 (if sand is easily available)
Submerged breakwaters	200–400
Emergent breakwaters	250–500

Euro per m coastline per year. Beach nourishments are more expensive (200–300 Euro per m coastline per year) and even more if the lifetime is only 1–2 years.

Given all uncertainties involved, it may be concluded that goynes are relatively cheap and emerged breakwaters are relatively expensive structures. Since groynes are not very effective at most sites, the real choice generally is between nourishment and detached breakwaters. As the costs of these alternatives are of the same order of magnitude, other decisive factors (morphological impact, aesthetics, available building materials) should be taken into account based on detailed studies. It should also be realised that the design of detached breakwaters is a rather complicated process consisting of mathematical and physical modelling, the testing of the design by means of a field pilot project including a detailed monitoring programme and the fine tuning of the design by modification of breakwater lengths based on the field experiences.

An important trend that can be seen recently at touristic beaches is the gradual change from small-scale cells (groyne fields; 100 m scale) to larger scale cells (headland-type groynes with beach fills in between; 1 km scale) to accommodate high-quality beach recreation requirements, see Gómez-Pina (2004).

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