Design of hard coastal structures against erosion
by Leo C. van Rijn www.leovanrijn-sediment.com, march 2013

1 Basic causes of erosion

Generally, coastal structures such as groynes, offshore breakwaters, artificial reefs are built to mitigate coastal erosion. Coastal erosion strongly depends on the type of coast (exposure, wave climate, sediment composition).

Erosion processes
Erosion of the coastline may be caused by:

- episodic storm-induced erosion of beach and dune zone (full or partial recovery may occur during post-storm conditions); in case of full recovery the beach and dune zone are stable and no measures are required;
- gradual, but systematic increase of the net sand transport due to curvature of coastline (gradients of longshore sand transport), or due to continuous cliff erosion of unconsolidated materials and/or due to spatial changes in wave climate, tidal currents or sediment composition;
- obstruction of the longshore transport due to the presence of natural barriers (headland, island, shoal, inlet, etc.) or artificial barriers (harbour breakwater, long groyne, land reclamation, artificial beaches for recreational purposes, etc.); sediment is accumulated on the updrift side and sediment is eroded on the downdrift side; some part of the sediment may eventually bypass the barrier;
- fluctuations or reductions in river sediment supply downdrift of river mouth;
- continuous offshore-directed transport due to presence of nearby deep channels or canyons trapping sediments; due to presence of headlands or man-made structures producing offshore-directed rip currents; due to increasing water level (lakes) causing continuous dune erosion;
- presence of nearby tidal inlet connected to back-barrier basin trapping sediments;
- mining/dredging of sand;
- erosion of spit heads and island heads due to inlet currents plus waves; spits are often instable and continuously changing their form; spits can be eroded rapidly if the supply of sand is interrupted by an updrift barrier; the net annual longshore transport rate updrift of an inlet may increase significantly towards the inlet due to increasing tidal velocities and due to reduced wave heights from opposite directions (in the lee of ebb shoals for opposite waves).

Factors favouring coastal erosion are:

- *exposure*: wave and current attack will be concentrated on headlands, capes and other protruding coastal forms (promontories); wave exposure is low for annual mean significant wave height at edge of surf zone (say, depth of 6 m) H_s<0.75 m; moderate for H_s between 0.75 and 1.5 m; and high for H_s>1.5 m;
- *high tides (spring tide), storm surge levels and storm intensity*: flooding, wave overtopping and breaching may occur;
- *persistent oblique wave attack*; wave-induced currents increase with increasing wave angle; net littoral drift will be relatively large in case of one dominant wave direction;
- *unconsolidated sediments*: low sandy coasts can be relatively easily eroded; bluffs, cliffs and rock-type coasts are more erosion resistant;
- *absence of nearshore bars/banks/shoals*; relief is important for offshore dissipation of energy (wave breaking);
- *presence of nearby sinks*; trapping of sediment by inlets, back-barrier basins (lagoons), ebb shoals, offshore sand banks, harbour basins, deep navigation channels, offshore canyons, etc.

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;
- to bring the coastline at a more seaward position by reclaiming land from the sea.

2 Coastal variability

Coastal variability is not the same as coastal erosion, which is herein defined as the permanent loss of sand from the system. Variability is the oscillating behaviour around a positive or negative trend line (see Figure 8.1.7).

Shoreline erosion and shoreline management requires a workable definition of the shoreline position. Often, it is defined as the low, mean or high water line during calm conditions. This definition may give rather variable results that strongly depend on the beach morphology. Therefore, it can be better defined as the line obtained by volumetric integration of the intertidal beach zone or the vertical zone between the dune foot (at about 3 m above MSL) and the -3 m line. Shoreline variations/fluctuations can be determined by using filtering methods and trend/periodicity analysis.

Maintaining of the shoreline at a certain position ("holding the line") requires the determination of the basal shoreline at a pre-selected year. Preferably, historical data (say 10 to 20 years) should be available to determine the basal line by regression over this period to eliminate variations.

Shoreline variations (LW-line, HW-line, dunefoot-line) generally are variations around a systematic trendline (chronic erosion or deposition; see Figure 2.1); the trendline may be caused by natural (autonomous) processes or related to man-made structures. Several time scales can be identified (see **Stive et al., 2002**):

- long-term variations (centuries): changes in relative sea level; climate changes in tidal range and wave heights, availability of sediment; long-term shoreline changes are in the range of 100 to 1000 m/century (1 to 10 m/yr);
- medium-term variations (years, decades): changes in wave climate and hence in wave-current conditions, migration of tidal channels and flats, sand bank migration, migration of inlets, closure of inlets, effects of coastal structures; shoreline changes can be up to 500 m over a period of 10 years (50 m/yr) near tidal inlets (attachment and detachment of shoals and banks; **Stive et al., 2002**); shoreline changes due to migrating sand waves (length of 1000 to 2000 m; longshore migration rates of 50 to 200 m/yr; see **Stive et al., 2002**) or other rhythmic features along an open coast are in the range of 10 to 100 m over 10 years;
- short term fluctuations (seasons; days to months): bar migration, sand wave migration, rip channels, beach fills, effects of coastal structures; the maximum local shoreline changes on the storm time scale (days) and on the seasonal time scale (summer-winter response) of the open shoreline are generally in the range of 1 to 50 m (5 m for Duck, USA; 10 m for Ogata, Japan; 20 m for Ajigaura, Japan; see **Stive et al., 2002**).

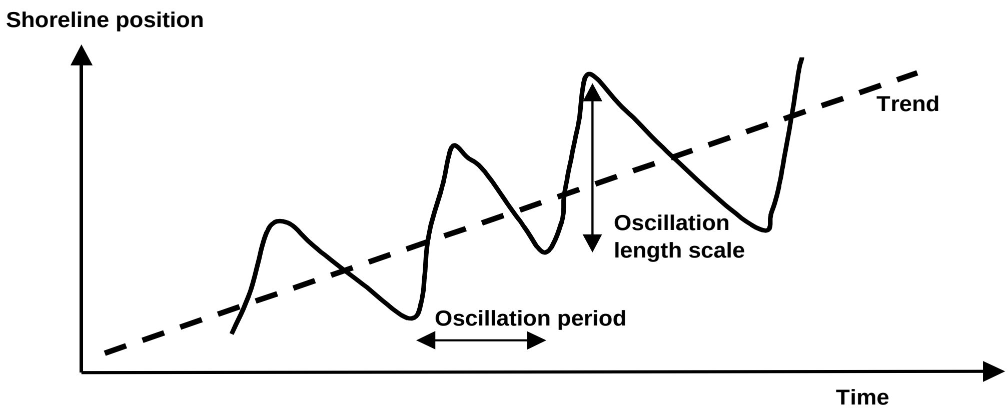

Figure 2.1 *Shoreline position as function of time*

Shoreline variations due to natural forcings are manifest at all time scales; shoreline variations due to human forcings typically operate at decadal and centennial time scales. Often, the oscillating component of the shoreline change (expressed in m/day) is much larger than the long-term change of the trendline. Spectral analysis (**Stive et al., 2002**) of time series 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 to 4 years) are also present, most probably associated with migrating sand waves. **List et al. (2003)** have found that the regionally-averaged beach slope becomes a few degrees (1 to 3) flatter (classic berm-bar profile response) during storm events for three sites along the east coast of the USA. Profile recovery (steepening) occurs during post-storm conditions. These locations were defined as short-term reversible hotspots (STRH). The maximum local shoreline change (erosion) over a coastal stretch of 70 km during pre-storm to storm conditions over 3 to 5 days was about 20 m; the maximum local shoreline change (accretion) during storm to post-storm conditions was also about 20 m. However, this type of symmetrical response did not always occur because many exceptions (non-STRH) were observed along the three sites with either no slope response or even a steepening response during storms (List et al., 2003).

3 Types of hard coastal structures

Coastal structures are generally built at locations where beach and dune erosion causes serious problems.
The decision to build a coastal structure should be based on a thorough analysis of the shoreline developments in the past and estimated developments in the future. The physical processes causing erosion should be properly identified, otherwise erroneous decisions may be taken. For example, it will not be very wise to build beach groynes if the shoreline recession is caused by the passage of the trough of a sand bank migrating along the coast. It should be realized that structures may ease the problem, but do not remedy the cause.

Coastal structures built around the world can be divided into:

- shore-parallel structures such as;
  - seawalls,
  - seadikes,
  - revetments,
  - artificial headlands,
  - detached breakwaters,
  - artificial reefs,
  - sea bottom protections (armouring of the shore);
  - artificial islands;
- shore-normal structures such as;
  - short and long groynes,
  - jetties,
  - harbour breakwaters;

The most basic function of **hard** structures is:

- to intercept and dissipate the energy of waves and currents and associated sand transport;
- to protect the shore against erosion;
- to protect the shore against sliding (bluffs, cliffs, dunes).

Hard structures should not excessively lead to:

- increased wave reflection:
- increased current velocities, set-up currents, vortex streets and turbulence intensities;
- increased longshore transport capacities;
- profile steepening and offshore transport;
- delayed or reduced beach and dune recovery;
- intensified shoreface scour, toe scour and lee-side erosion;
- destruction of natural bar/trough system.

Structures put out in the sea to modify the transport processes always involve acceleration and deceleration of water around the structure leading to scour and deposition. Generally, deposition occurs on one side (updrift) and erosion on the other side (downdrift). Often the erosion is solved by extending the structure along the erosive side in stead of solving the problem causing the erosion (blocking effect). Sand bypassing offers a possibility to supply erosive (downdrift) coasts with new material.
Structures that intercept the longshore transport, induce a consistent morphological response on three different time scales:
- initial stage; most of the longshore transport is blocked resulting in maximum beach accretion on updrift side and maximum erosion on downdrift side;
- intermediate stage; beach accretion slows down and bypassing of sediment increases gradually; lee-side erosion decreases;
- equilibrium stage; beach planform stabilizes, bypassing is maximum and lee-side erosion is minimum.

The time scale largely depends on the dimensions of the structure (length of breakwater/jetty/groyne or length and offshore distance of detached breakwater) and the magnitude of the net and gross longshore sand transport rates. Finally, the types of structures (hard and soft) in relation to coastal problems are presented in Table 3.1.

Coastal problem	**Type of structures (hard and soft)**
Safety	seawall (exposed sites) dune revetment dune enlargement offshore reefs and breakwaters (exposed sites)
Land reclamation	seawall (exposed sites) artificial dunes (sheltered sites) offshore reefs and breakwaters (exposed sites)
Recreational beaches	beach nourishment long groynes enclosing artificial beach/pocket beach (exposed sites) artificial headlands (sheltered sites) detached emerged breakwaters (sheltered sites) detached submerged breakwaters/perched beach (sheltered sites)
Mitigation of shoreline erosion and restoration of sand budgets	beach and shoreface nourishment offshore feeder berms detached submerged breakwaters (sheltered sites) groyne field (at sites with dominant longshore currents; near inlets)
Stabilisation of inlets	long jetties (exposed sites) detached emerged breakwaters sand bypassing methods (dredging)

Table 3.1 *Type of structures in relation to coastal problems*

4 Groynes

4.1 Types of groynes

Groynes are long (up to 300 m), 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 stabilize 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*.
Summarizing, groynes are built to serve three main purposes:

- to stabilize or widen the beach by trapping sand from the littoral drift; a groyne acts as a partial dam that intercepts a portion of the littoral drift; sediment material is impounded on the updrift side and erosion is caused on the downdrift side (often increased erosion due to generation of additional turbulence at the groyne tip); to reduce downdrift impacts these types of groynes should be relatively short and/or permeable and have relatively low crest levels;
- to stabilize the placement of beach fill material (nourished beaches); often these types of groynes are constructed to stabilize artificial beaches along heavily armored made-land shores without any updrift supply of sediment (no or almost zero longshore drift; sediment-starved shores; **Chrzastowski, 2004**);
- to prevent the movement of littoral material out of an area (at terminus of littoral cell close to an inlet or other sediment sink); these groynes should be relatively long and and have high crest levels above the prevailing high tide levels and normal wave run-up levels;.

Groynes can also be applied to deflect tidal currents from the shoreline and/or to stabilize relatively deep tidal channels at a more offshore position. These structures are known as *inlet groynes or current groynes*.
Generally, groynes can only retard (factor of 2 to 3) the shoreline erosion, but cannot stop the erosion completely. Groynes are most effective at sites with a predominant longshore current. Groynes are not effective at sites with almost normal wave incidence. Groynes can be an effective defence system in quiescent waters (bays; lake shores; estuaries) and in combination with beach fills along open coasts. Nowadays, the design of groyne fields along open coasts is nearly always combined with the placement of beach fills inside the cells to widen the beaches and to reduce downdrift impacts (see **Kana et al. , 2004** and **Shabica et al., 2004**).
A detailed overview of groynes is given by **Fleming (1990),** by **Kraus et al. (1994)** and by many authors in a Special Issue of the Journal of Coastal Research **(SI 33, 2004).** Detailed examples are presented by **Van Rijn (1998)**.
Two main types of groynes can be distinguished:

- *impermeable, high-crested structures*: crest level above MHW; sheet piling or concrete structures, grouted rock and rubble-mound structures (founded on geotextiles) with a smooth cover layer of placed stones; these types of groynes are used to keep the sand within the compartment between adjacent groynes; the shoreline will be oriented perpendicular to the dominant wave direction within each compartment (saw-tooth appearance of overall shoreline);
- *permeable, low-crested structures*: crest level between LW and HW lines to reduce eddy generation at high tide; pile groynes, timber fences, concrete units, rubble-mound groynes, sand-filled bags are used; permeability can increase due to storm damage; these types of groynes are generally used on beaches which have slightly insufficient supplies of sand; the function of the groynes is then to slightly reduce the littoral drift in the inner surf zone and to create a more regular shoreline (without saw-tooth effect).

Straight groynes, T-head, L-shaped and Y-shaped groynes have been built along many coasts; the latter three types of groynes are designed to reduce wave energy into the compartments and to prevent/diminish the generation of rip currents near the groyne heads. Measures should be taken to prevent wave reflection and production of turbulence (vortex streets) near the T-, L- and Y-shaped groyne heads. Round headed groynes are more attractive in conditions with strong currents and low waves (inlets).

The performance of groyne fields is positive, if there is:

- seaward shift of the shoreline;
- growth of the submarine terrace;
- steepening of the profile near the groyne tip;
- more fine sand fractions inside the groyne cell;
- seaward shift of outer shoals.

4.2 Hydraulic and morphodynamic effects

The groynes can act as support points for the beach inside the compartments. The beach line will be aligned with the incoming wave crests of the dominant wave direction (Figure 4.1Left). The beach level on the updrift side of the groyne root (near dune toe) can be substantially higher than at the downdrift side of the root, resulting in a step-type difference in elevation across each groyne. A curved bay-type beach (Figure 4.1Right) will be formed in a variable wave climate. The added beach width increases with the offshore length of the groynes.

Groynes have no effect on longshore transport beyond their toes and sediment collected within the compartments will be lost during storms due to offshore transport and the generation of circulation cells (rips). Scour holes formed at the tip of the groynes will deepen the bed locally, which will lead to an increase of the wave height and associated sand transport capacity.

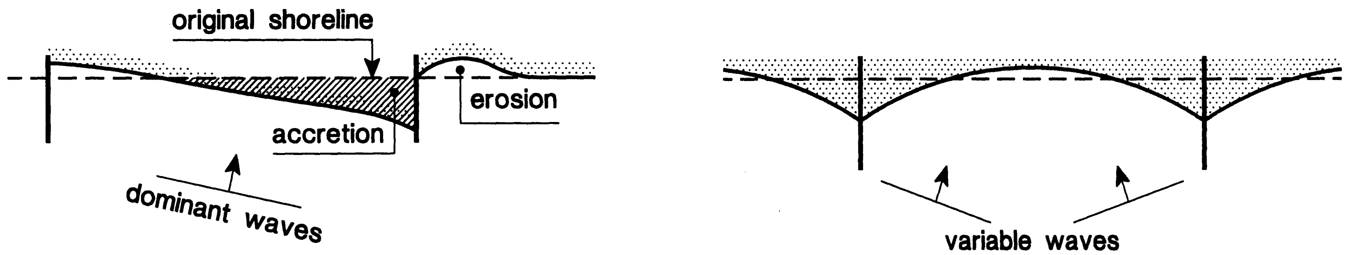

Figure 4.1 *Shoreline position inside groyne compartments*
Left: one dominant wave direction
Right: variable wave directions

The hydraulic and morphodynamic aspects of groynes can be summarized, as follows:

- reduction of wave height within the compartments in case of oblique waves, if the groynes are sufficiently long and high and at close spacing; the decay of incoming waves is insignificant for permeable groynes;
- reduction of wind- and wave-induced longshore currents and thus of littoral drift in the inner surf zone;
- deflection of wind-and tide-induced longshore currents to deeper water; contraction of flow around groyne heads leading to increase of velocities and hence to local scour; scour will be more severe for longer groynes;
- generation of circulation patterns (rip currents) within compartments of impermeable groynes; circulation patterns varying over the tide can be set up due to diffracted waves around the groyne heads, due to transmitted waves over the submerged crests and reflected waves off the groynes; these effects depend on the wave angle, the crest level and the ratio of spacing and length; rip currents are not generated along highly permeable groynes (pile screens);
- production of additional turbulence, depending on geometry, materials and dimensions of groynes;
- realignment of beach between the groynes; erosion on the updrift side and accretion on the downdrift side; erosion will diminish due to bypassing of sand around the tip of each groyne tip (saw-tooth alignment); the beach line has a curved shape in a variable wave climate, see Figure 4.1Right;
- lee-side erosion downdrift of the terminal groyne.

4.3 Design aspects and effectiveness

Design considerations (littoral processes, groyne characteristics and structural characteristics) are:

- ***environment:***
  - composition of beach materials and substrata,
  - dominant drift direction (build-up of material in the vicinity of existing barriers/structures),
  - nearshore bars and banks,
  - steepness of beach profiles,
  - wave climate (construction materials to be used),
  - longshore currents (tide-, wind- and wave-induced);
- ***crest level:***
  - the crest level near the dune toe should be slightly lower than the local beach;
  - the crest level near the groyne tip should be slightly higher (0.5 m) than the mean low waterline MLW;
  - the crest level should be roughly 1 m above the local sea bed so that the longshore drift (dominant bed load) is blocked under moderate waves;
  - sand is permitted to pass over the structure and to nourish downdrift beaches under storm waves;
  - the required beach level will determine the crest level in case of artificial beach or beach fill;
  - generally the crest level at the groyne root is about 0.5 m above the required summer beach level;
  - the crest level of submerged groynes should 0.5 to 1 m above the subaerial portion of the nourishment profile; the submerged crest depth should be designed variable or in 2 lines to hold the submerged part of the beach profile; submerged groynes are mainly effective during low wave energy conditions (**Gomez-Pina, 2004**; **Aminti et al., 2004**)
  - the crest width of rubble-mound structures should not be smaller than 3 m and at least at 0.5 m above MSL to allow the passage of heavy equipment for construction and repair;
- ***groyne length from shoreline (L) and spacing (S):***
  - the tip should be within the surf zone to allow sand to pass around it;
  - the spacing should be 2 to 4 times the groyne length to prevent the generation of rip currents and associated excessive erosion between the groynes; the spacing should decrease with increasing wave angle;
  - maximum groyne length is roughly determined by the mean low water spring line in tidal environments;
  - steep beaches and beaches with rather oblique wave attack require more closely spaced groynes;
  - nourished and artificial beaches generally have relatively high and long terminal groynes;
  - if severe lee-side erosion is to be expected, the terminal groynes generally are shorter (tapered) and more permeable to create a smooth transition to the unprotected beach downdrift;
  - the groyne root should run into the dune over some length or properly attached to a revetment (if present) to prevent outflanking or damage by local erosion.

  Practical values for L and S are:

  U.K.: L and S are about 60 m for shingle beaches, S/L between 0.5 and 1.5;
  L is about 100 m and S is about 130 m for sand beaches; S/L between 0.8 to 3;

  Holland: L between 100 to 200 m and S is between 200 to 400 m, S/L between 2 and 4.

The effectiveness of groynes is strongly related to the degree of blocking of the littoral drift and this depends on:

the groyne length in relation to the width of the surf zone (about -6 m to MSL);

- the spacing and overall geometry;
- the crest height, the mean sea level and the tidal range;
- the permeability;
- the beach material (sand, shingle or mixture).

Modern design of groyne fields along severely eroding beaches should include the placement of beach fills inside the groyne cells to obtain a wider beach and to mitigate downdrift erosion!!!

The effectiveness of groynes is most strongly related to the length of the groynes with respect to the width of the surf zone. As construction costs increase with the length of a groyne and given the limited effectiveness of beach groynes, it will be most economical to build relatively short groynes focussing on the reduction of the littoral drift in the inner surf zone during moderate wave conditions. Furthermore, the effectiveness of groynes during storm events with high sea levels is relatively low.
Long groynes (to -6 m) with a spacing of 1 to 3L can be used at the end of the littoral system (just before inlet), in case of severe erosion just updrift of the inlet.

The effectiveness of groynes also depends on the permeability of the groynes. This parameter has a strong effect on the generation of rip currents and associated offshore-directed sand transport. Low-crested groynes can act as permeable groynes, because sand can pass over the groyne, when it is submerged.

The effectiveness of groynes is also related to the type of beach material: sand (0.06 to 2 mm), mixed gravel/sand or gravel (>2 mm). Gravel beaches are much steeper (between 1 to 5 and 1 to 10) than sandy beaches. Gravel transport is less current-dominated and the beach material tends to be moved by the direct action of the waves. Gravel is moved up the beach in the direction of the uprush of the breaking waves or down the beach in the direction of the downrush in a zig-zag movement in case of oblique-incident waves. Net transport rates along gravel beaches are typically of order of 10,000 m^3 per year. Net longshore transport rates along sandy beaches are of order of 100,000 m^3 per year. A gravel beach can be re-oriented and drawn down rapidly by storm waves, but little of the gravel is drawn out beyond the breaker line and smaller less steep waves tend to restore the beach profile rather quickly (within days). A sand beach continues to flatten under prolonged storm waves. Sand moves offshore and the beach requires much longer time to return to a normal profile.
Relatively short groynes can be used along relatively steep gravel beaches. In mixed sand/gravel environments the groyne fields should consist of alternatingly short and longer groynes; short groynes can be used to deal with the upper gravel beach and longer groynes (every second or third) to control the lower sand beach.
In tidal environments the groynes generally run to the mean low water spring line under a sloping crest. Bypassing of littoral drift will take place in sandy environments.

Groynes (without beach fills) in erosive environments may initially lead to additional dune erosion, because the beach profiles between the groynes have to adjust to a new situation. The sand trapped between the groynes may come from the dune zone. This effect can be reduced by nourishment (pre-filling) of the compartments immediately after construction of the groynes. Groynes along a continuous beach should be pre-filled (beach fills) to bring the beachline quickly close to equilibrium conditions so that bypassing of sand is initiated immediately to prevent lee-side scour as much as possible.

Nairn and Dibajnia (2004) present an excellent discussion of the design aspects of the long groynes of the Keta sea defence system constructed between 2000 and 2003 in eastern Ghana, Africa. The project consists of a revetment and six long groynes (length of 190 m and spacing of 750 m) together with 2.6 million m^3 of beach fills in the groyne cells to protect a 5 km reach of rapidly eroding shoreline (d_{50} of 0.6 mm at upper part of beach with slope of 1:7 down to –2 m and d_{50} of 0.11 mm beyond the –2 m contour) against unidirectional low-steepness swell-type waves in the range of 0.5 to 2.5 m. Erosion rates are in the ranges of 2 to 7 m/year due to severe gradients of the longshore transport rates; the net longshore transport is in the range of 300,000 to 500,000 m^3/yr. A typical phenomenon is the presence of large bulges of sand moving along the shore resulting in shoreline variations of 20 to 40 m over a period of weeks. Straight, L-head and T-head groynes were studied; straight groynes were found to be the most cost-efficient solution, as shown in Figure 4.2. To promote bypassing of sand around the groyne tips (and to minimize the downdrift impact), pre-filling of the cells updrift of groynes 3 to 7 was applied (see Figure 4.2). The design focussed on three parameters: the average beach angle in the cells (determined from predominant wave direction), the updrift setback distance which is the distance between the groyne tip and the beachline near the updrift groyne (estimated from similar projects found in the literature to be about 30 m) and the beach shape on the downdrift side of the groynes (determined from physical model tests). Based on these studies, the length of the groynes was set to about 190 m (well within the dynamic surf zone to allow bypassing of sand). The spacing was based on a ratio of 1 to 4. The recommended beach fill material had a d_{50} of 0.4 mm (d_{10} of 0.1 mm and d_{90} of 2 mm) to match the grain size variations in the surf zone as much as possible (coarse sand in upper part and fine sand in lower part of profile).

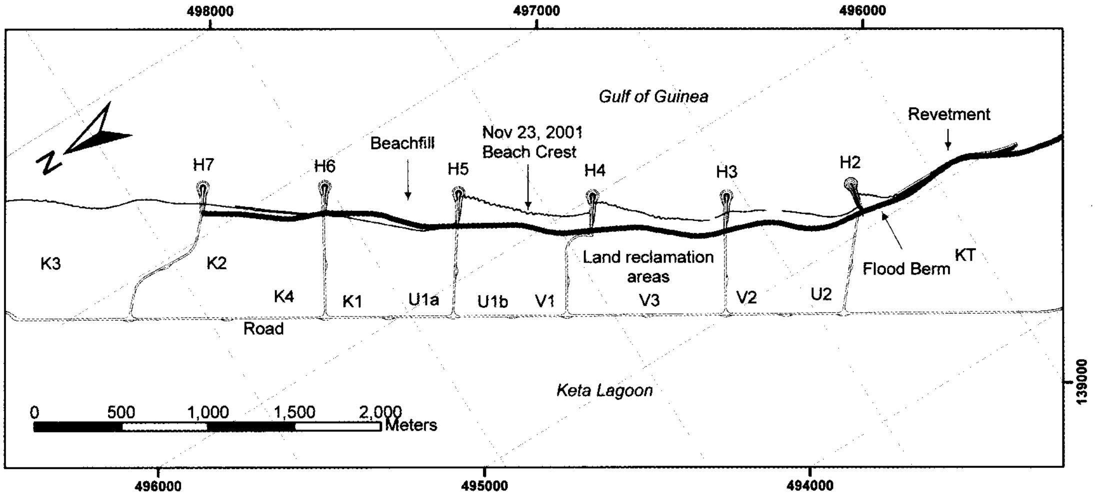

Figure 4.2 *Keta sea defence system, Ghana*

Based on various local input conditions (available quantities and size of rocks, available roads, etc.), the construction scheme was designed as follows: begin at the middle of the groyne field, then construct the updrift group followed by the downdrift group. Post-project monitoring showed rather good agreement of predicted and measured longshore transport rates. The predicted shoreline evolution within the cells showed good agreement with measured beachlines. Scour around the groyne tips did not occur, but accretion near the groynes was the dominant feature which resulted in a reduction in overall rock quantities.

Galgano (2004) has evaluated the effectiveness of a groyne system along Bethany beach (medium sand), Delaware Atlantic coast (USA). The groynes were poorly constructed (several renovations were required) and were too short (original length of 70 m) to be truly effective. In combination with regular beach fills it was possible to reduce the beach erosion considerably.
Lessons to be learned:

- groyne fields should be infilled immediately after or during groyne construction;
- construction should begin at the groyne furthest downdrift and commence updrift;
- groynes should be relatively short and low to maintain downdrift beaches;
- groynes should never be spaced more closely than 1:1;
- groyne fields must be constructed to operate throughout the entire littoral cell.

Kana et al. (2004) have evaluated groynes along Edisto beach, Folley beach and Pawleys Island beach, South Carolina (USA). Based on their experiences, management guidelines are given for groyne installations:

- groynes should only be considered for open coast sites where the average erosion rate exceeds 2 m/yr; sites with low erosion should favour non-structural erosion control solutions;
- groynes should be constructed from downcoast to upcoast and cells nourished (beach fills) to capacity (50% of difference between groyne profile and average beach profile applied over the length of the cell);
- downcoast impacts are best mitigated by concomitant nourishment and by terminating groyne fields at the downcoast ends of primary littoral cells;
- groyne fields should be sized and nourished to provide a stabile beach over the full design period (50 years or so);
- groyne profiles should follow the natural beach profiles and incorporate a berm section to control the dry beach, a beachface section and a low-tide section;
- groynes should be scaled to extend across the active littoral zone in normal conditions;
- stability of rubble mound groynes can be improved by grouting the voids;
- detailed beach surveys should be an integral part of any groyne management plan.

Basco and Pope (2004) have proposed some basic guidelines for the functional design of groins, as follows:

- groynes should only be applied if longshore transport processes are dominant;
- always include a beach fill to avoid erosion of downdrift beaches;
- agree on a minimum dry beach width (minimum distance between mean high waterline and the design shoreline) during normal storm events as a measure of success;
- use an initial groyne length (from tip to design reference shoreline/baseline) of 2 to 3 times the longshore spacing;
- use shoreline model to estimate shoreline changes around the groyne field;
- use model simulations to iterate on the design of the minimum dry beach width;
- use tapered ends of the groyne field (reduce length of terminal groynes) to minimize impacts on adjacent beaches;
- establish field monitoring effort for evaluation of performance.

Gomez-Pina (2004) has evaluated the aesthetical aspect of groynes. His guidelines are:

- well-integrated terminal groynes can be very efficient in holding beach fills;
- crest heights should be in the range of 0.5 to 1 m above the subaerial part of the nourishment profile; the submerged crest depth should be designed variable or in 2 lines to hold the submerged part of the beach;
- minimizing crest dimensions is important for social acceptance of groynes.

4.4 Applications of groynes

Groynes generally are cost-effective, if:

- the beach material is coarse sand and or gravel; short, high-crested impermeable groynes are usually sufficient;
- the wave energy climate is low to moderate and the tidal range is relatively small (micro-tidal);
- short, low-crested groynes are combined with regular beach fills in case of fine sand beaches; beach fills promote bypassing of sand to nourish downdrift beaches; sand beaches without beach fills in tidal conditions are extremely difficult to control with groynes; fine sand beaches are relatively flat and refraction will cause the wave crests to become parallel to the beach promoting offshore transport processes during storm events (reducing effectiveness);
- there is a sufficient supply of sediment by longshore drift and offshore transport is minor or absent;
- the beach head/dune toe is relatively stable (onshore/offshore transport processes are weak), otherwise outflanking may occur due to erosion around the root of the groynes;
- lee-side erosion is acceptable; initial beach fills between the groynes can promote bypassing around groyne heads reducing lee-side erosion.

Groynes may be applied in the following situations:

- along a beach with sand supply from updrift (oblique waves) and slightly increasing longshore transport rates (due to increased wave heights or shoreline curvature), causing minor erosion over a relatively short distance and where minor lee-side erosion is acceptable;
- at the end of the littoral system, such as immediately updrift of inlets (minimum lee-side erosion);
- along the shoreline of inlet throats (inlet groynes), where currents are strong;
- downdrift of harbour breakwaters to shift the zone of lee-side erosion in downdrift direction;
- for stabilization of recreational beaches downdrift of harbour breakwaters (lee-side erosion will continue); for stabilization of beach fills by high, impermeable terminal groynes (increase of lifetime);
- updrift of relatively short harbour breakwaters to spread accretion over greater beach length.

For example, an erosive beach can be temporarily widened and stabilized by a local groyne field. The beach can, however, still be eroded by offshore transport, if the profile is too steep. Regular beach fills will be required to improve the beach. This may be a reasonable solution, only if lee-side erosion is acceptable.

Groynes can also be built on the accreting updrift side of a relatively short harbour jetty (small fishery harbour) to spread the supply of sand to the jetty over a larger area, thus preventing the sand from passing along the tip of jetty into the harbour entrance.
Relatively long groynes in combination with sea bed protection can be used to protect the head of an island and/or spit against the eroding power of strong tidal currents (plus waves) passing the inlet. Groynes should be constructed initially without the presence of the bed protection. Regular monitoring of the sea bottom near the groynes should be carried out to determine the proper moment for dumping of stones for bed protection.

Simple pile groynes (**Raudkivi and Dette, 2002**) are a very cost-effective solution for sheltered beaches in micro-tidal conditions (Baltic Sea; Mediterranean). **Dette et al. (2004), Poff et al. (2004)** and **Trampenau et al. (2004)** evaluate the performance of permeable wooden pile groynes which are often used in non-tidal and micro-tidal conditions (Florisa, USA; Germany, Poland). For example, about 2000 wooden pile groynes have been installed along the Baltic Sea coasts of Germany and Poland. An important advantage of permeable pile groyne fields is that the shoreline remains continuous and does not display the characteristic saw-tooth shoreline behaviour of permeable groyne fields.

Design considerations of permeable pile groyne fields are:

- groyne length should extend to the seaward flank of the most inner breaker bar; spacing should be between 1:1 and 1:2; piles should be driven into bed over about 60% of their length;
- crest level should be about 0.5 m above the mean sea level up to the waterline; on the beach the crest level should also be about 0.5 m above the beach face (to be effective during high tide) yielding a sloping crestline;
- crest level should extend landward of the MHWL to prevent flanking (erosion of upper beach);
- permeability should not be larger than 40%; variable variability is preferred with 20% over inner 1/3L, 25% over middle 1/3L and 30% over louter 1/3L; wave reflection, wave diffraction and development of rip currents are greatly suppressed using this scheme; longshore current remains well-ordered and shoreparallel (see Figure 4.3);
- double-width groyne is more effective than two single groynes spaced far apart;
- wooden pile groyne suffer from borers in marine conditions reducing their lifetime;
- permeable groynes should not be used in combination with beach fills.

Beach groynes (without beach fills) are not effective along:

- steep reflective high-energy sand coasts (normal wave incidence);
- macro-tidal sand coasts;
- sand coasts of short length, where the littoral drift can not develop (short bays and lagoons);
- sand coasts with a small net transport (two large oppossing littoral drift volumes).

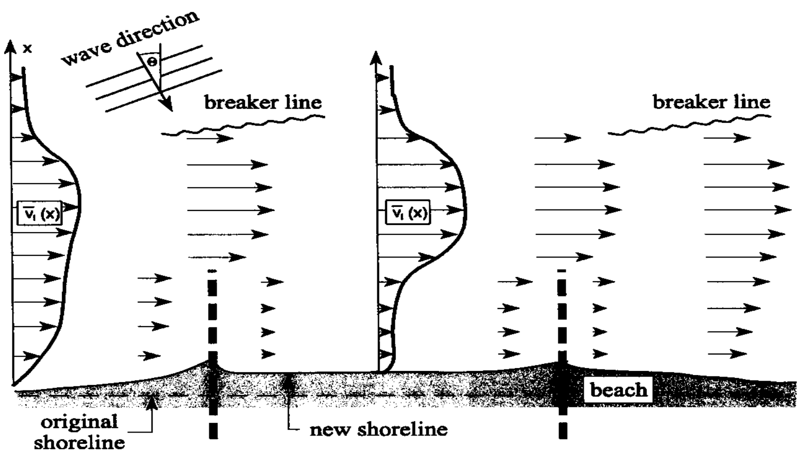

Figure 4.3 *Flow pattern near permeable wooden groyne field (Trampenau et al., 2004)*

4.5 Application of shoreline model for groynes

The **LONGMOR-model** (**Van Rijn, 1998**) has been used herein to compute the shoreline changes near three groynes (cross-shore length of 250 m; width of 25 m; spacing of 250 m). The active layer thickness of the coastal profile is assumed to be 8 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 tanβ= 0.01 (slope from waterline to 8 m depth contour).

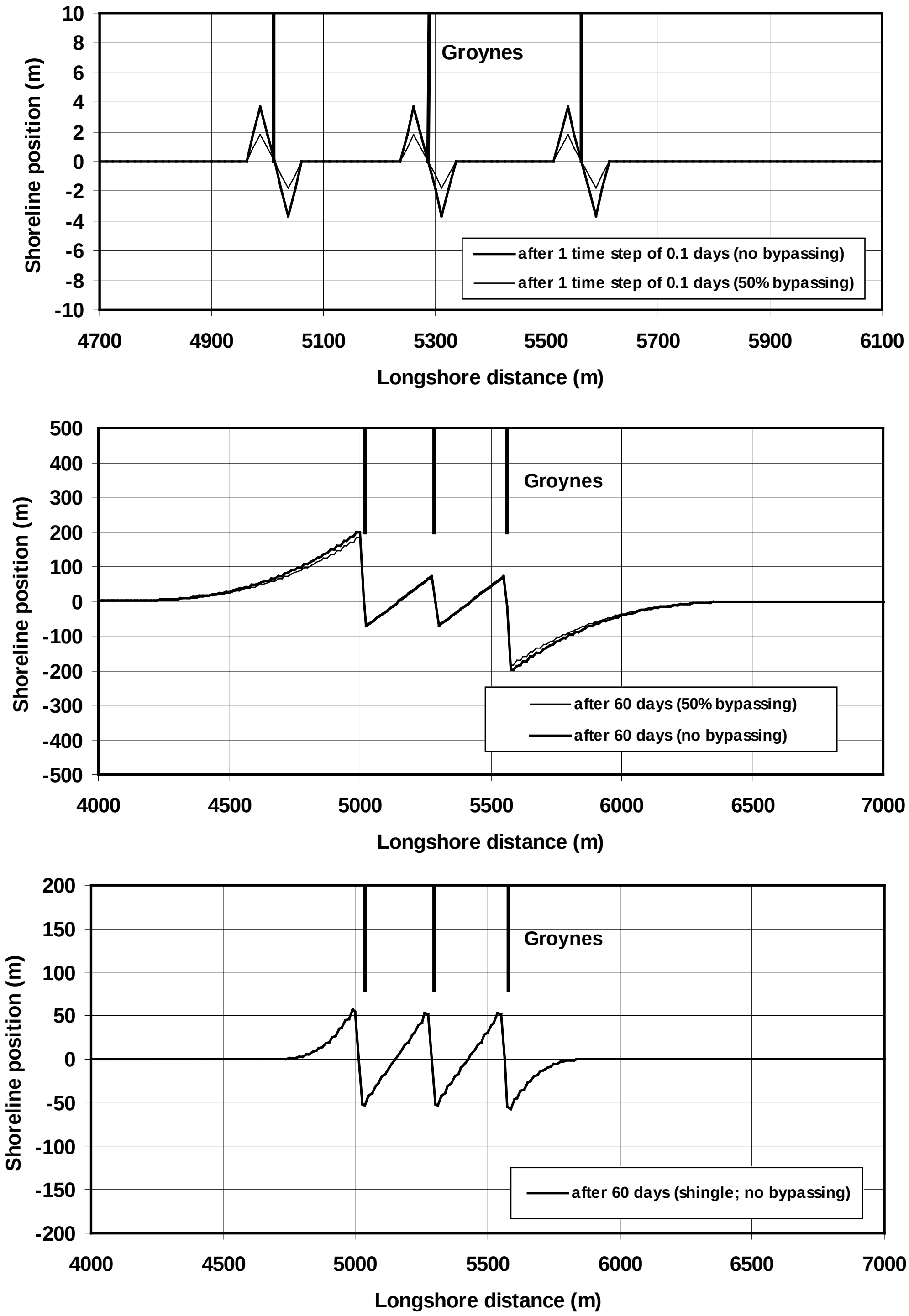

Figure 4.4 *Computed shoreline position near groynes; offshore wave incidence angle of 30° degrees*
Top: after 1 time step of 0.1 day; beach of sand (0.2 mm)
Middle: after 60 days; beach of sand (0.2 mm)
Bottom: after 60 days; beach of shingle (20 mm)

The tidal longshore velocities in the surf zone are assumed to be zero. The local wave breaking coefficient is assumed to be γ_{br}= 0.6. The longshore grid size is 12.5 m and the time step is 0.1 days. The shoreline changes over a period of 60 days have been determined using a wave climate with offshore waves of $H_{s,o}$= 2 m, T_p= 7 s and an offshore wave incidence angle of 30°. The longshore transport rates have been computed by using the method of Van Rijn.
Two computations using the **LONGMOR**-model have been made: (1) without bypassing (longshore transport rate is set to 0 at location of groynes) and (2) with 50%-bypassing (longshore transport rate is set to 50% at location of groyne). Figure 4.4Top shows the shoreline position after 1 time step of 0.1 day. As can be observed, the shoreline changes are twice as large for the case without bypassing. Figure 4.4Middle shows the shoreline changes after 60 days with accretion on the updrift sides and erosion on the downdrift sides of the groynes. The updrift longshore transport rate is about 7500 m^3/day for 0.2 mm-sand resulting in an updrift accretion area of 60x7500/8= 55000 m^2. The assumption of 50%-bypassing has almost no effect on the shoreline on the time scale of 60 days, because the longshore transport rates are reduced significantly as the shoreline builds out updrift of the groynes (wave incidence angle to local shore normal reduces). The bypassing rate is 50% of a relatively small quantity at the location of the groynes and has therefore not much effect on the shoreline changes. Figure 4.4Bottom shows a similar computation for a beach of shingle (20 mm). The shoreline changes are considerably smaller (factor 4). The updrift longshore transport rate is about 735 m^3/day for 20 mm-shingle resulting in an updrift accretion area of 60x735/8=5500 m^2 (factor 10 smaller than for sand case).

5 Jetties and harbour breakwaters

5.1 Introduction

Many natural inlets have been modified by building structures like jetties to minimize maintenance dredging of deepened channels or to stabilize the inlet against migration, see Figure 5.1. Detailed examples are presented by **Van Rijn (1998)**.

Jetties are long, narrow, dam-like structures that are built more or less perpendicular to the coast to prevent the shoaling and migration of inlet channels or navigation (approach) channels and to protect the channel entrance against storm waves.

Jetties generally extend through the surf zone to beyond the outer breaker line; jetties may increase the tidal current velocities in the inlet channel, resulting in deepening of the channel. The ebb shoal may be pushed to a more offshore position. Sand accumulation will take place on the updrift side and erosion on the downdrift side of the inlet; the latter may be on a relatively large scale as jetties are usually relatively long. Mechanical bypassing of sand may be required to reduce downdrift erosion.

Significant deepening of the inlet channel through the bars and shoals on the seaward side will alter the sediment bypassing mechanism and thereby the response of the downdrift beaches. Lee-side erosion will increase with deepening of the channel, as more sediment will move into the back-barrier basin and less sediment will be bypassed to the downdrift beach.
Other problems are channel deposition and migration of shoals within the inlet and scour near the jetty tips. Inlets without jetties may also suffer from bank erosion (often related to the migrational tendencies of the inlets).

This type of erosion can be mitigated in various ways (see Figure 5.2):

- construction of jetties and or groynes;
- beach nourishment on one or both sides of the inlets;
- inlet channel bank protection by dumping stone material or by channel bank sand nourishment;
- soft sand dams at the inlet entrance (sand buffer).

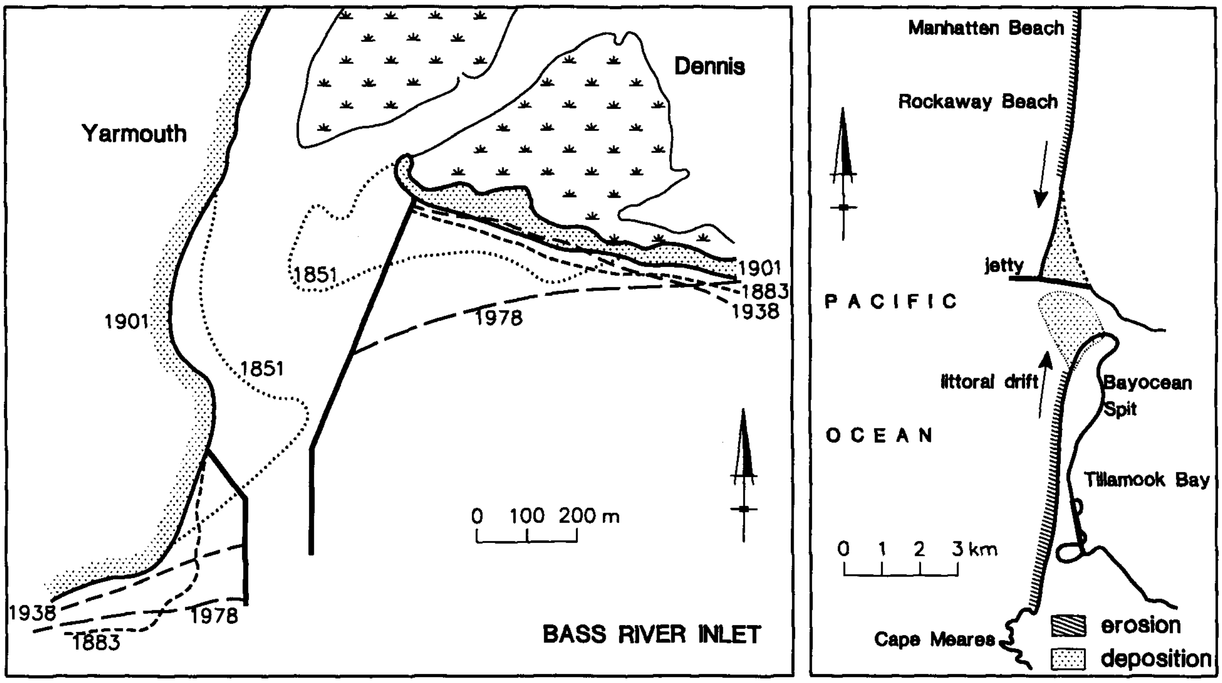

Figure 5.1 *Example of jetties and shoreline changes near jetties in USA*

Jetty construction is done:

- to interrupt the longshore sand transport and thereby to reduce deposition in the inlet channel; jetty length depends strongly on updrift longshore transport (bypassing around tip should be prevented/reduced as much as possible over a long period);
- to increase the current velocities in the channel and thereby to increase the channel depth;
- to bring the channel entrance to deeper water beyond the ebb delta and shoals, but if possible not so deep that the offshore shoal will be inactive and the natural bypass system is eliminated; sand should be able to reach the downdrift beaches;
- to stabilize the inlet channel.

Harbour breakwaters are structures perpendicular and/or oblique to the shore (Figure 5.3), forming a shield to harbours and boat anchorages and protecting a portion of the shoreline from the waves (blocking of incident wave energy). They are generally attached to the shore at one or both ends, often with a gap for boat entrance and extend outward through the surf zone. If the jetty or breakwater is not long enough, sand will be carried along the tip of the jetty into the entrance of the harbour forming a localized spit in the entrance.

The jetties and breakwaters should be impermeable (sand tight) structures to prevent uncontrolled passage of water and sand through the structure.

Harbour breakwaters are built:

- to protect the harbour basin against wave-current penetration;
- to intercept the longshore sand transport and thereby to reduce deposition in the harbour basin.

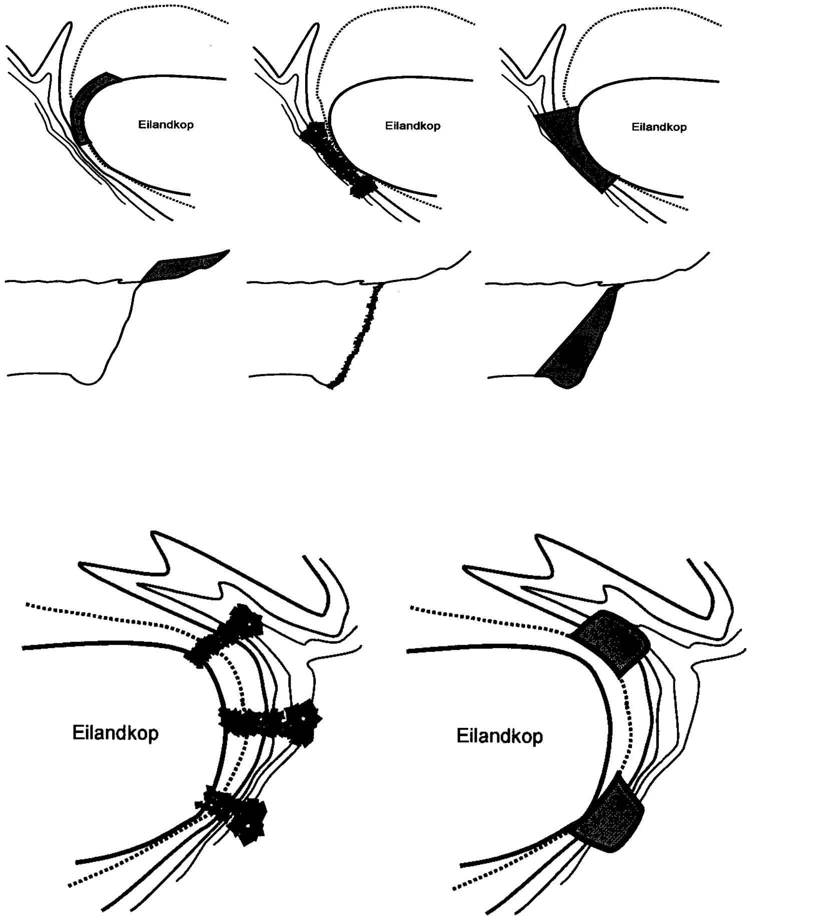

Figure 5.2 *Mitigation measures against erosion near inlets*
Top: Beach nourishment (left); channel bank protection (middle); channel bank nourishment (right)
Bottom: Hard stone groynes (left); soft sand dams (right)

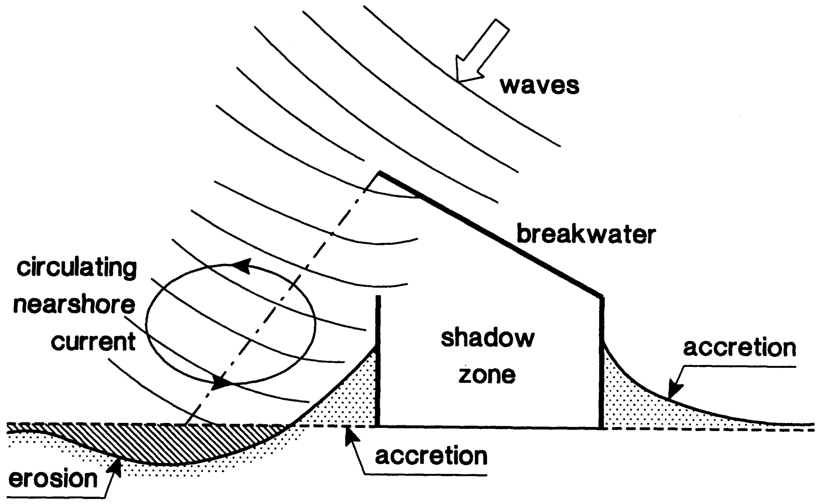

Figure 5.3 *Harbour breakwaters*

5.2 Hydraulic and morphodynamic effects

The hydraulic and morphodynamic effects of jetties and harbour breakwaters are:

- deflection of tidal longshore currents to deeper water;
- elimination of cross-shore currents in the navigation channel;
- contraction of flow around the tip of the jetty/breakwater and generation of relatively deep scour holes at tip;
- creation of a downdrift circulation zone with a length of roughly 5 times the jetty/breakwater length; production of turbulence in lee zone;
- creation of diffraction zone in the lee of the harbour breakwater; differential wave breaking causes longshore gradients of wave-induced set-up generating a complicated longshore current pattern with recirculating flow close to the structure;
- training of tidal flows through the inlet;
- increase of tidal velocities in the inlet to deepen the channel and to push the ebb shoal to deeper water;
- blocking of wave-induced currents and associated longshore sand transport;
- accumulation of sand over short distance on both updrift and downdrift sides of jetty/breakwater (near-field accretion); erosion on both sides (variable waves) farther away from jetty/breakwater (far-field erosion);
- generation of bypassing shoals downdrift of the tip of the jetty.

The characteristic effects of the construction of a harbour or an inlet with jetties along a dynamic coast with one dominant wave direction can be summarized, as follows:

- accumulation of sand on updrift side of harbour and (lee-side) erosion on downdrift side;
- bypassing of sand around the tip of the breakwater into the entrance of the harbour, if breakwater length is insufficient; groynes may be built to reduce bypassing;
- construction of seawall on downdrift side to stop erosion and to protect shore;
- extension of harbour breakwater in combination with construction of detached breakwater on updrift side to stop deposition of sand in harbour entrance.

The characteristic time scale of deposition/erosion along the shoreline is about 20 to 50 years.

The zone downdrift of the jetties is characterized by (Figure 5.1):

- near-field accretion over short distance (0.5 to 1 times the barrier length L);
- near-field erosion (over 0.5 to 1 L) due to redistribution of local beach sediments (wave shadow zone and generation of leeside circulation cell);
- transition zone without minor or zero erosion;
- far-field erosion (3 to 5 L) due to the re-establishment of the interrupted littoral drift; the erosional front moves downdrift at a rate of about 0.2 to 0.5 km per year.

The shoreline changes in the direct vicinity of the structure are determined by the gross transport rates (sum of oppositely-directed transport components), whereas the shoreline changes farther away from the structure are related to the net longshore transport rate (difference of both components). The gross transport rate can be as large as 1 million m^3 per year, whereas the net transport may be zero in conditions with symmetric wave incidence. An inlet in combination with the back-barrier basin can act as a substantial sink of sand due to the sand-importing capacity of the system; the sand may be eroded from the beaches on both sides of the inlet. Estimates of the gross transport rates are required to evaluate the overall morphological system.

5.3 Design aspects and effectiveness

Generally, *double* jetties are constructed, one on each side of the inlet. Double jetties should run more or less parallel; the area between the jetties should be narrow to prevent shoaling and meandering of the channel. This will lead to reduced hydraulic resistance and relatively high current velocities in the channel. The accumulation of sand on both sides of the jetties is a typical phenomenon in a variable wave climate.
A *single* jetty may be considered in case of one dominant wave direction. A single jetty is only feasible, if there is nearly unidirectional longshore transport. Such an inlet may silt up if there are waves and associated sand drift from the other direction.
The efficiency of jetties depends primarily on their length in relation to the cross-shore width of the littoral system. The tip of the jetties should not be too far outside the surf zone, otherwise the natural bypass system can not function and lee-side erosion will occur requiring downdrift beach nourishments. It may be more economical to design relatively short jetties and to carry out maintenance dredging in the channel entrance rather than to do downdrift beach nourishments to eliminate lee-side erosion due to relatively long jetties.
Preferably, the jetties should be high-crested impermeable structures perpendicular to the shore to divert the sediment offshore in the direction of the ebb shoals, where dredging can be carried out to maintain the entrance channel. The jetty width at the crest should be at least 5 m and the crest level of the jetty should be at least 0.5 m above MSL to allow passage of heavy land-based equipment for construction and repair.

Weir-type jetties are jetties with a low crest elevation at mean tide level, so that water and sand can be carried over the structure by wind-, tide- and wave-induced forces. This can only be successful, if there is a well-designed sand trap directly downdrift of the weir and regular maintenance dredging is applied. Generally, the weir with a length between 100 and 500 m is located in the nearshore section of the jetty, where the littoral drift is largest. A favourable effect of a weir-jetty is that dredging operations can take place in the lee of the jetty, improving operational efficiency.
The effectiveness of a weir-jetty in combination with a sand trap is limited, because it will only trap a certain portion of the suspended load transport in the nearshore area. The littoral drift in the outer surf zone will not be trapped. Problems will arise, if the sand trap behind the weir-jetty is not functioning properly or maintenance dredging is not sufficiently taking place. In that case there will be an abundance of fine sediments within the area between the jetties. Shoals and spits will develop close to the channel and it will be difficult to control the channel position.
Weir-type jetties should not be constructed in high-energy environments. Many weir-type jetties constructed in the past have been replaced by high-crested structures.

5.4 Sand bypassing

A remedial measure against the interception of longshore transport by a barrier (headland, jetty, breakwater, deep channel) is the mechanical bypassing of sand. The basic objectives are:

- reduction of sand accumulation on updrift side to prevent passing of sand around the tip of the barrier causing deposition into the entrance of the channel/harbour; minimizing maintenance dredging;
- reduction of lee-side erosion effects; minimizing beach nourishments or defence constructions on downdrift side.

The bypassing system consists of four elements:

- the intake point (sand trap),
- the removal equipment,
- the bypassing line and
- the disposal point.

A hopper dredger combines all elements sailing between the intake point and the disposal point. Maximum (100%) bypassing of sand generally is not feasible and the deficit is often taken from a borrow area and carried to the downdrift beaches (beach nourishments).

Bypassing solutions are:

- sand trap updrift of structure in combination with ship-based or land-based removal equipment (floating pumping lines, drag lines, buried fluidizers to bring sand to a central recovery point, hopper-dredgers, etc.); the sand trap may be in the lee of a detached breakwater in combination with a floating dredge or a movable pumping platform (small jack-up platform); the trap should be wide and deep enough to intercept most of the littoral drift; supply is often maximum in the nearshore zone and swash zone; land-based removal equipment is only suitable for a relatively narrow surf zone; a fixed land-based plant has a maximum capacity in the order of 50,000 to 100,000 m^3/yr (boom length is limiting factor); bypassing up to 10,000 m^3/yr should be done by truck transportation and dumping; hopper-dredgers can not always dredge close to the shore, although shallow-water hopper dredgers are available;
- sand trap downdrift of structure (weir-type jetty); removal by hopper dredgers; the downdrift dumping site can be in the direct vicinity of the inlet in case of one dominant wave direction; the dumping site must be farther away in case of a variable wave climate to prevent rapid recirculation of the sediments; tracer experiments can be done to determine the sediment transport pathways;
- sand trap in the entrance of the channel, where most of the sand will settle due to reduction of wave height (overdepth of 1 to 2 m, excluding sounding tolerance of 0.2 m and dredging tolerance of 0.5 m, resulting in total overdepth of about 2 to 3 m); occasional removal by hopper dredger depending on deposition rates;
- a sophisticated solution for large-scale bypassing is the jet-pump system operating updrift of the Nerang River entrance in Australia; this system consists of 10 jet pumps deployed at 30 m intervals from a pier extending through the surf zone; the water and sand is pumped into a gravity flume delivering the material to a buffer hopper, from where it is pumped through a buried pipeline to the downdrift side (capacity between 5,000 and 10,000 m^3 per day); it has functioned reasonably well, but clogging of the intake points remains a problem (regular cleaning); jet pumps make a crater with a diameter of about 20 m and a depth of about 5 m; an efficient system requires movable pumps or multiple pumps, because the craters will fill slowly by sand transport due to wave-current action; seabed fluidizers (inclined pipes with small holes to generate vertical flow) can help to bring the sediment to the craters.

6 Detached breakwaters and reefs

6.1 Introduction

A **detached breakwater** is herein defined as a hard structure with a crest width of the order of the local water depth. The crest may be positioned above the still water level (emerged) or below the still water level (submerged).

A **reef** is herein defined as a relatively wide, submerged structure in the shallow nearshore zone. Soft sand reefs are also known as nearshore mounds or nearshore berms.

Detached breakwaters (Fig. 6.1) are built as offshore barriers parallel (occasionally obliquely positioned) to the shore protecting a section of the shoreline by forming a shield to the waves (blocking of incident wave energy). There can be many variants in the design of detached breakwaters, including single or segmented, emerged (crest roughly 2 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. Three basic types have been used: (1) rubble mound with trapezoidal cross-section of rock or conctre units, (2) prefabricated units of triangular shape of concrete and (3) flexible membrane (geotextile) units constructed of sand-filled containers. Low submerged breakwaters can also be used as sills to support the seaward toe of beach fills (perched beaches). Detached breakwaters can be constructed as a single structure for localized shore protection or as multiple breakwaters with gaps between the segments for large-scale beach protection. Detached breakwaters can be designed in such a way that not all longshore transport is blocked by the structure, reducing potential lee-side erosion of adjacent beaches.

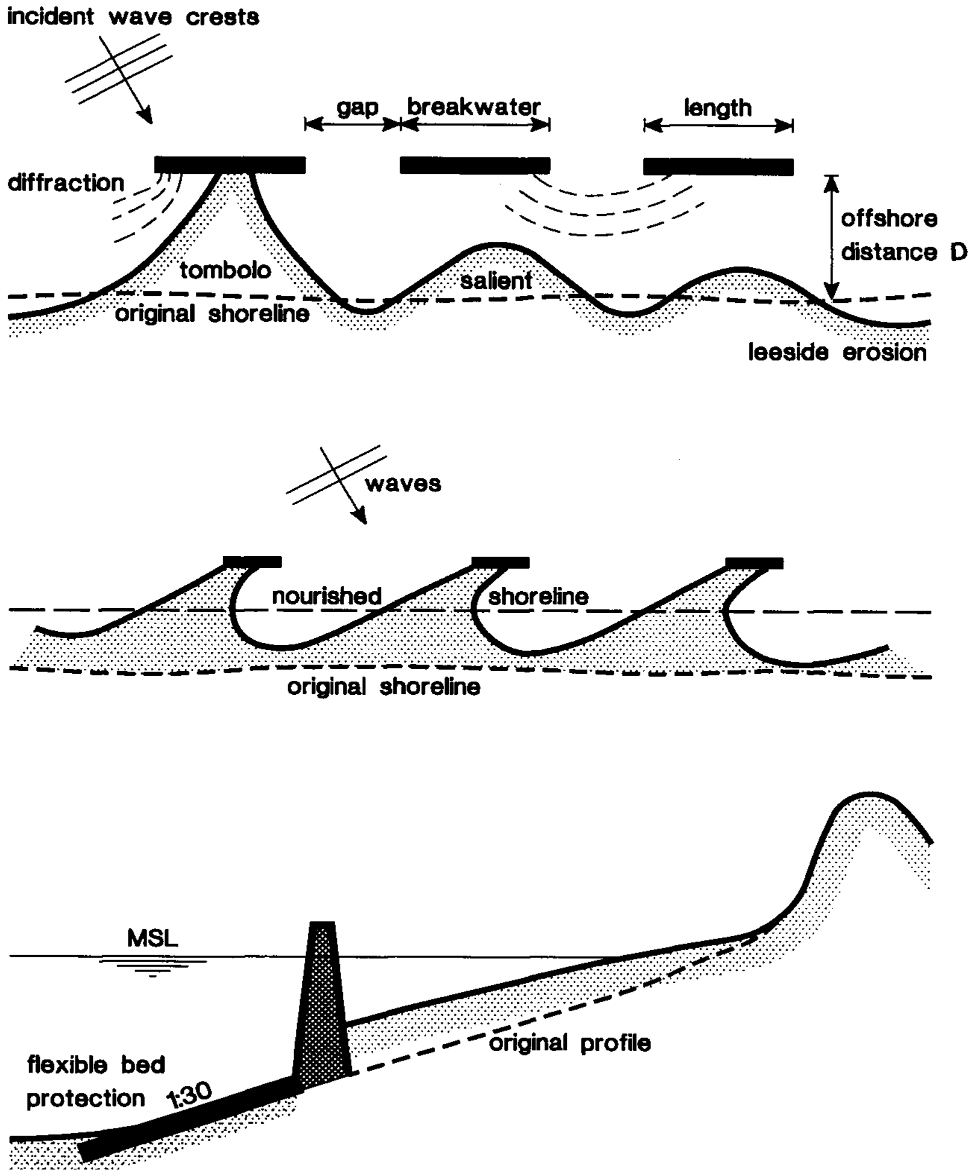

Figure 6.1 *Shoreline near detached breakwaters*
Top: Tombolo and salient behind detached breakwater
Middle: Artificial headlands and pocket beaches
Bottom: Cross-shore profile near detached breakwater

Artificial headlands (Fig. 6.1) are structures consisting of armoured shoreline sections with embayments in between (pocket beaches). Segmented breakwaters (headland control) will act as control points between which the beach will be modified into a small curved beach. Segmented breakwaters connected to the shore by tombolos can be seen as artificial headlands. The bay geometry can be determined by applying the empirical formulae of **Hsu and Silvester (1990)** and **Silvester and Hsu (1993)**. According to this method, the downdrift beach of the bay is aligned with the wave crests of the dominant wave direction; the updrift beach of the bay is aligned with the diffracted wave crests, yielding a static or dynamic (bypassing of sand around tip) equilibrium.

In the equilibrium situation the incoming waves will diffract and refract in such a way that they will break simultaneously around the whole beach line. As a result the shoreline (headlands) can be maintained in a more advanced position.
Artificial headlands can be used to protect recreational beaches, spit heads and island heads against wave-current attack.

Reviews of available data and design methods are given by **Rosati (1990) and Chasten et al. (1993)**.
Rosen and Vajda (1982), **Hsu and Silvester (1990), Suh and Dalrymple (1987) and Berenguer and Enriquez (1988)** presented relationships to predict the equilibrium beach lines behind detached breakwaters.

Liberatore (1992) and **Lamberti and Mancinelli (1996)** give detailed information of the experiences with emerged and submerged breakwaters along the Italian coasts.

6.2 Hydraulic and morphodynamic effects

Basic characteristics are (Figure 6.2):

- wave energy at the shoreline is reduced (breaking and reflection at breakwater); some of the incoming wave energy will arrive in the lee zone by:
  - diffraction around tips and through gaps;
  - transmission through breakwater;
  - overtopping of submerged breakwater;
- diffracted and transmitted waves will continue to propagate to the shoreline in the lee zone but the longshore transport capacity in the lee zone will be substantially reduced;
- sand moving along the shore is trapped behind the structure resulting in local deposition of littoral sands within the protected lee of the breakwater; seaward outbuilding of the beach;
- recirculation cells may be generated by gradients in wave set-up along the shore carrying sand toward the lee zone;
- longshore currents toward the tip points will be generated (suppressing the generation of recirculation cells) if there is considerable transport of water through the (permeable) structure and over the structure (submerged breakwater) resulting in erosion of sand from the lee zone by these currents and by rip currents through gaps between segmented breakwaters;
- in case of normal-incident waves, the diffracted waves will transport sand from the adjacent beaches into the lee of the structure until the shoreline is so aligned that the waves break parallel to the shoreline and the longshore transport becomes zero everywhere yielding a symmetrical salient or tombolo pattern; the shoreline will erode on both sides of the structure;
- in case of oblique-incident waves a system with alongshore currents and transport is generated, which may remain in function if a salient is formed; the shoreline near the structure will adjust in such a way that the smaller waves behind the structure can transport the same amount of sand as the larger waves updrift and downdrift of the structure; the salient shoreline causes the smaller diffracted waves to break at a more oblique angle.

Crest elevation determines the amount of wave energy transmitted over the top of the breakwater. High crest elevations preclude overtopping by all but the highest waves, whereas low crest elevations allow frequent overtopping. Occasional overtopping of a nearshore breakwater by storm waves can prevent tombolo formation or remove a tombolo once it has formed. Submerged breakwaters allow almost continuous passage of the lower waves.

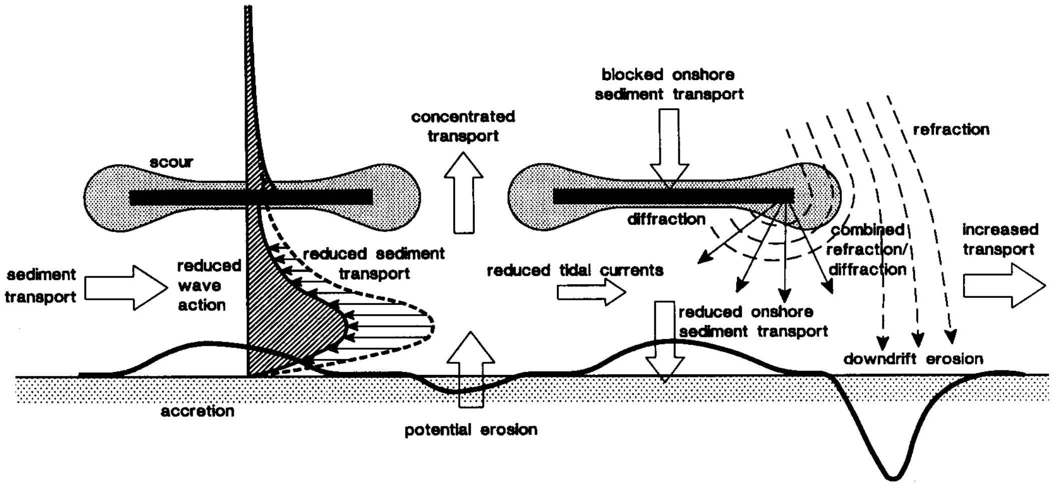

Figure 6.2 *Hydrodynamic and transport processes near detached breakwaters*

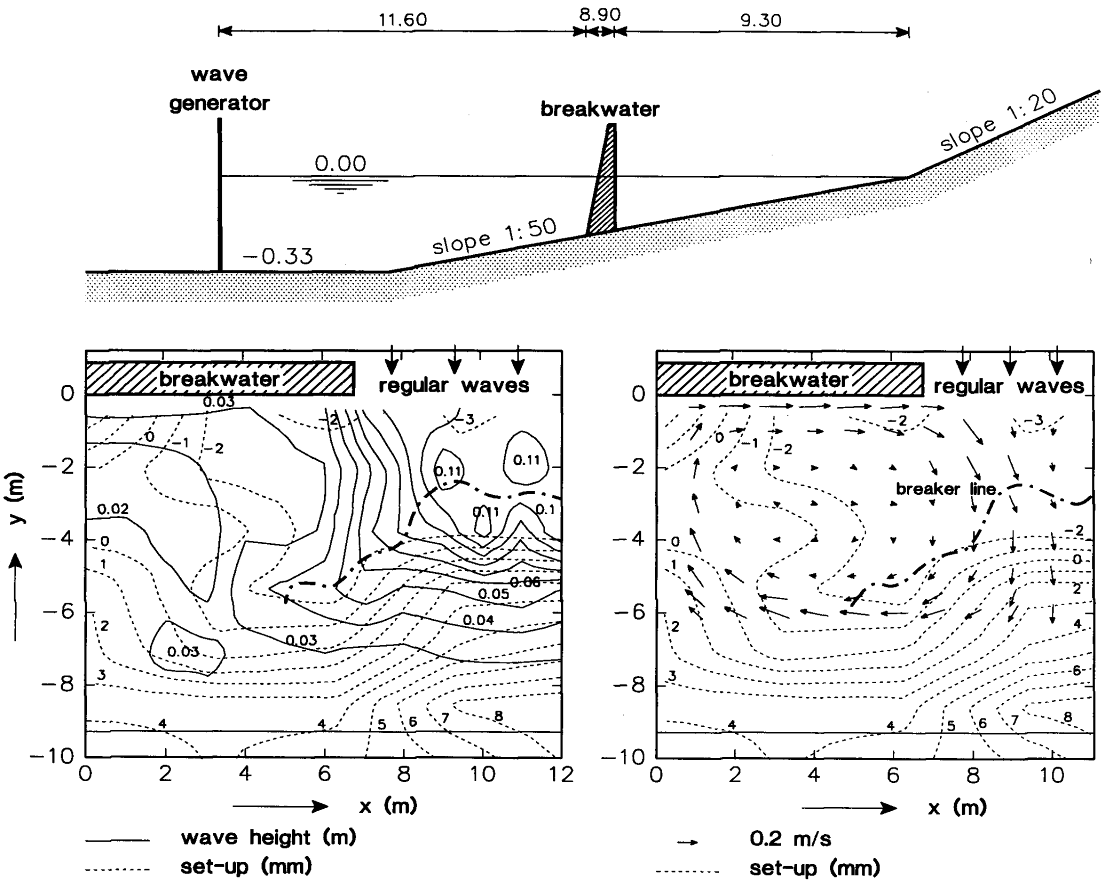

Figure 6.3 *Wave height, set-up and currents behind a detached breakwater in a laboratory basin with regular waves (H_o= 0.075 m, T= 1.7 s)*
Left: wave height, set-up and breaker line. Right: set-up, breaker line and currents.

Wave heights, set-up and circulation currents around a detached breakwater have been studied by **Hamm et al. (1995)** and **Mory and Hamm (1997)**. The experiments were carried out in a 3D wave basin. Regular and irregular waves were generated in a depth of 0.33 m. The sea bottom was a concrete floor consisting of three sections: a section with constant depth of 0.33 m, a section with an underwater plane sloping bottom (1:50) and a section with a plane sloping beach of 1:20 (see Figure 6.3). The mean water levels were obtained by measuring the mean piezometric levels in stilling wells connected to tappings in the sea bottom. Current measurements were performed by using an electromagnetic current meter and an immersible Laser Doppler meter. Measured results of wave height, breaker line, set-up and currents (at mid-depth) for regular waves (H_o= 0.075 m, T= 1.7 s) are shown in Figure 6.3. The most important results are:

- wave heights are much reduced in the lee of the breakwater due to the effect of diffraction;
- the set-up gradient is clearly related to the relatively large reduction of the wave height landward of the breaker line; the set-up contours near the shore remain parallel to the breaking line;
- a circulation cell is present in the lee of the breakwater; maximum velocities are about 0.3 m/s; the breaking line is the limiting line between the currents in the surf zone and a wide eddy cell with low velocities; the circulation is driven by wave breaking in the surf zone; the set-up gradients produce the pressure gradients required for eddy rotation;
- the current does not vary much over the depth in the zone between the breakwater and the breaker line; 3D effects are present in the surf zone;
- wave height and set-up contours are much smoother for irregular waves, the latter breaking at different depths; current velocities inside the circulation core are relatively large for irregular waves and almost zero for regular waves.

6.3 Design aspects and effectiveness

General design guidelines for offshore breakwaters refer to the design wave length and breakwater layout **(Rosati, 1990; US Army Corps of Engineers, 1992)**. The breakwater length (L) should be at least 2 times the design wave length and the gap length (L_{gap}) should be smaller than the design wave length. The offshore position (D) should be based on the desired shoreline pattern.
The shoreline may show a bulge-like pattern (*salient*) with the mean shoreline at a more seaward position or the shoreline may advance to the breakwater position (*tombolo*; shoreline connected to breakwater), depending on geometrical scale (L= breakwater length, L_{gap}= gap length, D= distance to original shoreline, and crest level below water surface, see **Figure 6.1**), wave climate and sand availability (**Pope and Dean, 1986**). Shoreline erosion may take place in the lee of the gap between the breakwaters.
Based on analysis of the available data sets, the following general rules can be derived for **emerged** breakwaters.

Depositional patterns
L/D>3; *permanent tombolo*;

- the breakwater length should be larger than the gap length (L/L_{gap}>1) to form a tombolo; increasing this ratio, increases the amount of energy transmitted through and over the segments while decreasing the diffraction effects; no erosion opposite to the gap will occur for L_{gap}/D<0.8;
- a tombolo behind a breakwater with L= 200 m, D= 200 m in a depth of 3 m can be formed in 1 to 3 years;
- tombolos will be formed if the structure is placed close to the shore well within the breaker zone or if the longshore transport rate is relatively large (abundance of sand);
- tombolos will eventually function as a groyne blocking the longshore transport;
- tombolos will lead to rip currents carrying sand to deeper water due to water piling up in the basin between adjacent tombolos under breaking waves;
- tombolos will lead to severe lee-side erosion in conditions with dominant oblique wave attack (if $Q_{s,passing}<Q_{s,downdrift}$);

L/D=2 to 3; *permanent or periodic tombolo* (if depth at breakwater location<3 m) **or *well-developed salient*** (depth>3 m); periodic tombolos are removed during storm conditions (highly variable wave climate);

L/D=1 to 2; *well-developed salient to incipient tombolo*;

- salients create less lee-side erosion due to bypassing of sand;
- salient formation can be promoted by allowing sufficient wave energy in the lee area by increasing offshore distance (depending on magnitude of littoral drift) and gap length, by reducing crest level (wave overtopping, wave diffraction is less behind a submerged breakwater generally leading to a wider salient) and by increasing the permeability of the structure (wave transmission);

L/D=0.5 to 1; *weak to well-developed salient*;

L/D=0.2 to 0.5; *incipient to weak salient*;

L/D<0.2; *no effect.*

Erosional patterns

L_{gap}/D<0.8; no gap erosion;

L_{gap}/D=0.8 to 1.3; minor erosion opposite to gap;

L_{gap}/D>1.3; major erosion opposite to gap.

Based on analysis results of detached breakwaters, the following features can be formulated:

- they are often constructed as segmented structures to protect eroding coastlines (headland erosion, see Fig. 6.1); wide gaps will cause a pattern of salients and embayments; shorter gaps produce more uniform salients;
- they are often constructed to protect recreational beaches along micro-tidal coasts with a moderate wave-energy climate;
- they are not so much constructed along meso- and macro-tidal coasts;
- they are sometimes used for local protection of the coastline directly downdrift of harbour barriers (shift of lee-side erosion to farther downdrift);
- they often require beach fills in the lee zone and beach nourishment on the downdrift side;
- they are not used to protect the coast near tidal inlets;
- they are generally constructed as rubble-mound structures; sand-filled bags have been used along the Italian coast;
- submerged structures appear to be less successful than emerged structures; hydrodynamic and morphodynamic processes near detached submerged breakwaters are not sufficiently known and proper design methods are not available; empirical/practical information is essential.

Differences between emerged and submerged detached breakwaters with respect to their impact on the shoreline can be summarized, as follows:

Emerged detached breakwaters

- substantial wave reduction;
- generation of circulation cells near tips, bringing sand into the lee zone (tombolo and salient formation)
- substantial lee-side erosion
- deterioration of visual scenery and water quality in lee zone (stagnant water);
- relatively high construction cost;
- successfully applied in micro-, meso- and macro-tidal conditions against erosion of headlands and recreational (artificial) beaches; beaches behind the structures are generally relatively wide (well-developed salients and tombolos);

Submerged detached breakwaters and artificial reefs

- only effective in trapping sand for salient formation if crest level is relatively high (0.5 to 1 m below MSL) in micro-tidal regime; beaches are relatively small (compared to those behind emerged breakwaters); effectiveness will reduce considerably if crest level is lowered due to settlement of structure;
- generation of longshore currents and rip currents through gaps (if present) due to transport of water over the crest, which may result in erosion of sand from the lee zone; this effect increases with decreasing crest level;
- erosion at seaward toe of structure and along the tips of the breakwaters bordering the gaps; bed protections are required over the full length of the gaps;
- lee-side erosion effects are often less serious;
- no remedy for storm-induced beach and dune erosion; submerged breakwaters are not very effective during storm conditions with relatively large storm surge levels;
- breakwater not visible for boats;
- relatively low cost;
- successfully applied in low-energy micro-tidal environments (Mediterranean coasts).

Maintenance of detached breakwaters generally is relatively high, because of settlement of the structures due to scour and the necessity of floating equipment for repair.
Many detached breakwaters have been built in micro-tidal conditions with weak tidal currents (about 4,000 in Japan). Tidal currents may lead to the generation of a tidal channel between the breakwater and the shoreline; waves may pass over the breakwater during flood (plus surge in case of storm conditions) with high water levels reducing the effect of wave energy dissipation by the breakwater.

Submerged sills and perched beaches

A detached breakwater can also be used as a submerged sill structure at the toe of a beach fill (known as perched beach, see **Figure 6.4**). The beach fill (sand pumped into the enclosure landward of the sill) creates a widened and heightened profile (the perch) confined at the seaward side by the sill structure. The sill creates a discontinuity (step) in the cross-shore profile so that the profile on the landward side of the sill is higher than the profile on the seaward side. Wave breaking and wave overtopping at the sill often lead to scour immediately landward of the sill, which can be suppressed by constructing a local bottom protection (apron) as a cover over the sand bed. A major drawback of the sill structure is the blocking of onshore transport to the beach zone during mild wave conditions. Hence, the natural feed of the beach by sand from the offshore zone is severely cut off. Regular maintenance fills are required as storm waves will erode the beach zone and carry the sediments offshore, whereas onshore transport is cut off by the sill.
A low-cost solution for mild wave conditions is the construction of a sill made of nylon sand bags.

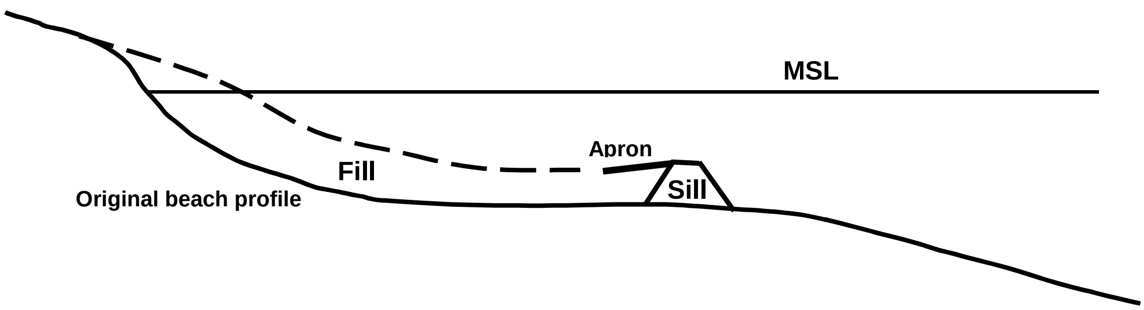

Figure 6.4 *Perced beach solution will submerged sill*

Gonzalez et al. (1999) have presented an equilibrium beach profile model for perched beaches based on Dean's equilibrium profiles ($y=ah^{2/3}$). They have found that the ratio of the depth (d) above the sill and the depth just in front of the sill (h_e) should be quite small ($d/h_e<0.2$) to obtain sufficient bed elevation (about 20%) on the landward side of the sill. Their method showed rather good agreement with data from laboratory experiments.

Headland breakwaters
An attractive solution to obtain a long recreational beach seaward of the shoreline in mild wave conditions is the design of a series of segmented (emerged) breakwaters in combination with tombolo-type beach fills anticipating for the generation of small embayments behind the gaps, see Figure 6.1Middle. This approach is also known as headland control involving the creation of fixed points along the coast, in order that bays are formed by the waves.

7 Seawalls, seadikes and revetments

7.1 Introduction

Seawalls, seadikes and revetments (Figure 7.1) are shore-parallel structures armouring the shore to protect the land behind it against episodic storm-induced erosion and/or long-term chronic erosion by the sea. In the latter case they usually are not very effective, because these types of structures hardly change the longshore transport gradients. Erosion of the beach and shoreface in front of the structure can continue and lee-side erosion usually is produced. Furthermore, beach and shoreface erosion will ultimately lead to an increased wave attack intensifying the transport capacity and hence intensified erosion (negative feed-back system).

A *seawall* is a vertical (or almost) retaining wall (massive structure) with the purpose of coastal protection against heavy wave-induced scour; it is not built to protect or stabilize the beach or shoreface in front of or adjacent to the structure. Thus, chronic erosion due to gradients of longshore transport will not be reduced.

A *seadike* is a sand dike (artificial sand dune) protected on both sides by armour layers and filter layers to prevent erosion of sand. Groynes are often constructed to reduce scour at the toe of the dike by deflecting nearshore currents. Seabed protection may be necessary in case of strong tidal currents passing the structure (seadike protruding into sea).

A *revetment* is an armour protection layer (consisting of light to heavy armour layer, underlying filter layer and toe protection) on a slope to protect the adjacent upland against wave scour by current and wave action.
Detailed examples are presented by **Van Rijn (1998)**.

Problems related to seawalls and revetments are:
- scour at the toe of the wall; the maximum scour depth is the scour depth below the position of the original sand surface (before the presence of the structure);
- lee-side erosion; the erosion along the downdrift side can be dramatic; shoreline retreat values up to 300 m extending over kilometers have been observed;
- accelerated erosion of the beach and nearshore profile in front of the seawall (steepening);
- delayed recovery of the profile;
- accelerated erosion of adjacent beach sections (up- and downdrift beaches) resulting in a more exposed position of the wall and narrowing of the beach in front of the wall by accelerating longshore currents around the protruding wall; passive and active erosion should be distinguished; the former being the natural erosion before construction of the wall (ultimately resulting in a more exposed position of the seawall) and the latter being the additional erosion caused by the presence of the wall;
- failure of the revetment/seawall may increase dune erosion locally (extra turbulence).

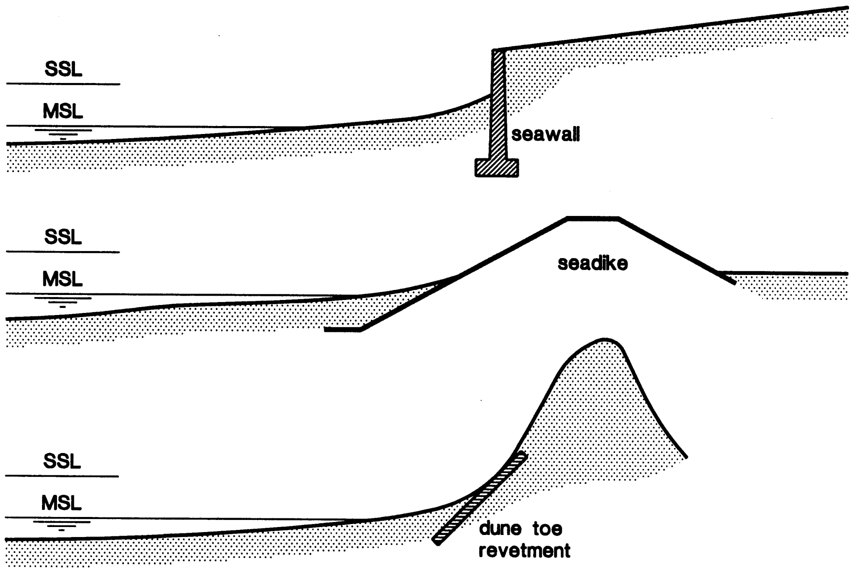

Figure 7.1 *Seawall, seadike and revetment*

7.2 Design aspects and effectiveness

Seawalls and seadikes

These structures (vertical, concave or sloping) are built along a limited section of the shoreline as a last defence line against the waves, when natural beaches and dunes are too small or too low to prevent erosion due to high waves. Erosion of downdrift shores may be accelerated by wave reflection from the structure.
Basic design characteristics are:

- the design of a seawall/seadike is an "end of the line" solution, if no other solution helps to solve the problem of erosion and/or flooding (high surge levels);
- seawalls/dikes are expensive structures, that have to be maintained continuously;
- in case of severe lee-side erosion, the seawall/dike sections eventually become headland-type capes protruding into the sea; bottom protection in front of the wall may be necessary to eliminate scour by the currents accelerating around the structure;
- reflective vertical or near-vertical seawalls cause relatively large scour depths at the toe; scour is less in front of a dissipative rubble-mound seawall;
- seawalls in the backshore with a beach in front of it give better performance than those without a beach; the impact of the wall is strongly dependent on its position with respect to the low water line; erosion is minimum if the seawall is built as far landward as possible (landward of level of maximum run-up during storm event); erosion is maximum if the seawall is built at a location seaward of the low water line sothat waves will reflect and or break against the wall.

Revetments

Revetments are not so massive as seawalls, because they are designed to reduce scour and erosion during a storm event rather than to stop it. Dune erosion above the revetment is allowed to occur. Revetments may be either impermeable or permeable structures, serving to retain (preventing slides) and to stabilize (preventing/reducing toe erosion) bluffs, cliffs and dunes. Most revetments do not significantly interfere with longshore transport processes.

Basic design characteristics are:

- the slope of the revetment should be smaller (say 1 to 3) than that of the dune toe to obtain a smooth transition (low reflection);
- the toe (at MSL) of the revetment should be trenched several meters below the dune toe level (say at +3 m to MSL) to reduce failure due to scour during storm events; horizontal toe protection over a length of about 2 to 3 m is necessary;
- the top of the revetment should be several meters above the dune toe level (+3 m above MSL); the revetment should be high enough to prevent overtopping by high waves under normal conditions; the wave run-up should be reduced as much as possible;
- light dune toe revetments can be covered in summer by sand to improve visual scenery;
- maintenance of storm damage is required.

Raudkivi and Dette (2002) propose to use membrane-type filter cloths within the dunebody to retain the sand during wave attack. Full-scale tests in a large wave flume in Germany showed minimum dune erosion. This solution was succesfully used at the west coast of the island of Sylt (Germany).

8 Discussion and conclusions

The characteristics of various types of beach protection solutions have been summarized:

- beach nourishments;
- groynes;
- emerged offshore (detached) breakwaters;
- submerged offshore (detached) breakwaters;
- perched beaches with submerged sills;
- headland breakwaters;
- stable reef berms and active feeder berms (shoreface nourishment);
- seawalls, seadikes and revetments.

An overview of the most realistic options is given in Table 8.1.

The selection of the most appropriate type of structure depends on many parameters: the basic objectives of the project (mitigation of shoreline erosion, the creation of a recreational beach or both), the tidal regime, the wave climate and the local availability and composition of sediments. The construction of a feeder berm is problematic if sand is not available in sufficient quantities in the surrounding of the project site. For example, sand material suitable for beach nourishment can not easily be found at most Italian sites. Offshore breakwaters require relatively high capital investments (approximately 2 to 2.5 million Euro per km) 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). The construction of feeder berms requires less initial investments (approximately 0.5 to 0.75 million Euro per km), but the costs of regular maintainance of the berm (every 5 years or so) are relatively large. The total costs of both approaches (breakwaters plus supplementary nourishment and feeder berms) should be compared and evaluated. **Pluijm et al. (1994)** have shown that the mitigation of shoreline erosion by full-scale nourishment is somewhat cheaper than that based on the construction of detached breakwaters for a meso-tidal, open coast.

Groynes are generally constructed at sites with dominant longshore currents (near tidal inlets) to reduce the erosive effects of these currents at the beach face, to retain the beach sand between the groynes and to extend the lifetime of beach fills. Short groynes can be very effective at coarse-grained (gravel) beaches with oblique wave incidence.

TIDE	**WAVE CLIMATE**	
	Mild **(Sheltered beach)**	**Severe** **(Exposed open beach)**
Micro-tidal	*Erosional:* - beach nourishment - feeder berm close to shore plus initial beach nourishment - emerged breakwater with initial beach nourishment (tombolo planform) - submerged breakwater (segmented with salient planform or continuous sill) with regular local beach nourishment - groynes near tidal inlets (longshore currents) - revetments to protect dune toe against erosion *Recreational:* - stable reef berm plus initial local beach nourishment - perched beach with submerged sill enclosed by groynes; occasional maintenance nourishment - headland breakwater plus initial beach nourishment (tombolo planform)	*Erosional:* - regular beach nourishments - feeder berm at edge of surf zone plus initial beach nourishment; regular maintenance of feeder berm - emerged breakwater with initial beach nourishment (salient planform); bypassing of longshore transport - submerged breakwater (segmented with salient planform or continuous sill) with regular beach nourishment; bypassing of LST; no remedy for storm-induced beach and dune erosion - groynes near tidal inlets (longshore currents) and regular beach nourishments - seawall to protect against flooding during storms *Recreational:* - stable reef berm plus regular local maintenance beach nourishment - perched beach wih submerged sill and local bed protection (apron) landward of sill; cover layer of coarse sand (1 m thick); enclosed by groynes; regular maintenance nourishments are required - headland breakwater plus initial beach nourishment (tombolo planform) and regular maintenance nourishment
Meso- and macro-tidal	*Erosional:* - feeder berm close to shore plus initial beach nourishment - emerged breakwater with initial beach nourishment (salient planform) - groynes near tidal inlets (longshore currents) with occasional beach nourishment - revetments to protect dune toe against erosion *Recreational:* - stable reef berm plus regular local maintenance beach nourishment	*Erosional:* - feeder berm at edge of surf zone plus beach nourishment; regular maintenance of feeder berm - emerged breakwater with regular beach nourishment (salient planform); bypassing of LST - groynes near tidal inlets (longshore currents) with regular beach nourishment *Recreational:* - stable reef berm plus regular local maintenance beach nourishment - seawall to protect against flooding during storms

Table 8.1 *Alternative solutions for protection of erosional and or recreational beaches*

Detached breakwaters are generally constructed along straight or slightly curved beaches or along pocket beaches suffering from structural erosion. Detached breakwaters can also be built on the updrift side of a relatively short harbour jetty (small fishery harbour) to spread the supply of sand to the jetty over a larger area, thus preventing the sand from passing around the tip of the jetty into the harbour entrance. Some supplementary beach nourishment may be required to deal with local erosion (opposite to gaps) in the case of emerged breakwaters. Substantial beach nourishment may be required along the downdrift shoreline of an emerged breakwater scheme.
Submerged breakwaters often require substantial supplemetary beach nourishments to deal with local erosion problems. The downdrift erosion usually is less than that in the case with emerged breakwaters, As the lee-side erosion effect on the downdrift coast is less severe for submerged breakwaters, these structures offer a good solution as a transition from a protected beach with emerged breakwaters to the unprotected downdrift beaches. The transition can also be effectuated by gradually increasing the gap lengths of emerged breakwaters.

Finally, it is noticed that submerged breakwaters offer an environmentally friendly solution, but it should also be realized that submerged obstacles may be dangerous for fishing boats, yachts and other recreational activities.

Groynes
Groynes may be applied in the following situations:
- along a beach with minor erosion over a short distance and where minor lee-side erosion is acceptable;
- at the end of the littoral system (near inlets);
- along the shoreline of inlets;
- updrift and downdrift of jetties and harbour breakwaters.

For example, an erosive beach can be temporarily widened and stabilized by a local groyne field. The beach can, however, still be eroded by offshore transport, if the profile is too steep. Regular beach fills will be required to improve the beach. This may be a reasonable solution, only if lee-side erosion is acceptable. Groynes can also be built on the accreting updrift side of a relatively short harbour jetty (small fishery harbour) to spread the supply of sand to the jetty over a larger area, thus preventing the sand from passing along the tip of jetty into the harbour entrance. The crest level of groynes should be slightly higher than the mean low water line or mean sea level in non-tidal conditions to allow passage of equipment for maintenance. The required beach level will determine the crest level in the case of an artificial beach fill. The spacing should be set to a value between 200 and 400 m for sandy beaches. The length of the groynes should be between 100 and 200 m. The initial length should be designed as small as possible. Based on monitoring results of the functioning of the groyne field, the length of the groynes can be increased, if necessary.
Relatively long groynes in combination with seabed protection can be used to protect the head of an island and/or spit against the eroding power of strong tidal currents (plus waves) passing the inlet. Groynes should be constructed initially without the presence of a bed protection. Regular monitoring of the seabottom near the groynes should be carried out to determine the proper moment for dumping of stones for bed protection.

Emerged detached breakwaters
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.
Based on the results observed along various micro-tidal USA-sites (**Stone et al. 1999; Mohr et al. 1999 and Underwood et al. 1999**) 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.

The design procedure can be, as follows:

- select the breakwater lengths and offshore distance;
  - L/D ratio should be about 0.8 to 1.2 for salient formation with bypassing of longshore transport (LST);
  - breakwater length should be of the order of 100 to 200 m;
  - offshore distance should be of order 100 to 200 m, depending on profile steepness (depth not larger than about 2 to 4 m), tidal range and wave climate; closer to shore along micro-tidal, sheltered beaches and somewhat further offshore along meso-tidal, exposed beaches (Winterton, UK; structure should not be situated in the highly variable 3D beach zone); relatively close to shore along macro-tidal beaches (Elmer beach, UK);
- select gap length;
  - L_{gap}/L ratio should be in the range between 0.8 and 1 to prevent shoreline erosion opposite to gap;
  - use a value of 1 for initial design (breakwater length can be extended later if shoreline erosion is too large);
- optimize design (gap length and crest level) by mathematical and physical modelling;
  - focus on set-up currents in lee of breakwater and in gap area;
  - focus on shoreline erosion opposite to gap;
  - tune layout for salient or tombolo formation, depending on wave climate;
- initiate field pilot project including monitoring programme for fine tuning of design;
  - construct multiple segmented breakwaters with varying lengths and gap lengths;
  - use small-scale beach nourishment at erosion spots opposite to gaps (if necessary); continuous nourishment will ultimately lead to tombolo formation;
  - increase length of breakwaters, if shoreline erosion is too large.

Properly designed emerged breakwaters offer an effective approach for beach protection. Negative points are the degradation of the visual scenery, whereas the water and sediment quality may also be degradated (slimy beach surface) if the water exchange behind the breakwater is hampered.

Submerged detached breakwaters

Submerged breakwater schemes may be built as segmented structures or as long, continuous structures. Gaps between the structures are not strictly necessary for water exchange (water quality). Some gaps may be required for passage of boats. A continuous submerged sill (small breakwater) can also be constructed as the seaward closure of a beach nourishment, known as perched beach solution. A long continuous breakwater may however lead to relatively large alongshore currents in the lee of the breakwater, because the water carried over the structure by the waves can not easily return in seaward direction and is instead redirected alongshore enhancing the natural longshore currents and increasing alongshore sediment transport (**Dean et al. 1997**). These latter effects are extremely dangerous in the case of a relatively long, single, submerged breakwater.

As only few of these structures have been built, the knowledge and experience are rather poorly developed. A basic problem is the wave breaking and wave overtopping generating a continuous supply of water over the submerged breakwater during windy conditions with associated generation of offshore-directed set-up currents (carrying sediments) from behind the breakwaters through the gaps and alongshore currents at the end sections. Submerged breakwaters are no remedy for storm-induced beach and dune erosion, because they are not effective in conditions with relatively large storm surge levels.

Based on the results presented by **Liberatore (1992)**, **Sawaragi (1992)** and **Tomasicchio (1996)**, it can be concluded that the crest level is of utmost importance. **Liberatore (1992)** points to the fact that initially low and narrow (3 to 5 m) barriers have been used in Italy, whereas higher and wider (10 to 20 m) structures have been adopted in later designs. **Sawaragi (1992)** discusses the morphological response of a submerged breakwater with a crest level at 2 m below MSL. The shoreline did not show any significant accretion, but erosion was also not observed. The shoreline behind nearby emerged breakwaters of similar dimensions showed substantial accretion. **Tomasicchio (1996)** discusses the morphological results of submerged breakwaters with relatively high and low crest levels. Shoreline accretion was observed behind the breakwater with crest level at 0.5 m

below MSL. Significant shoreline erosion occurred behind the other breakwater with crest level at 1.5 m below MSL. Hence, to be effective, the crest level of a submerged breakwater should be relatively high (not lower than 0.5 to 1 m below MSL). **Lamberti and Mancinelli (1996)** show some examples of cases, where emerged breakwaters have been replaced by submerged ones resulting in the disappearance of tombolos and the generation of shoreline recession.
A dramatic example of the morphological failure of a submerged breakwater is presented by **Dean et al. (1997)**. After constructing a single submerged breakwater off a section of the Florida coast at Palm Beach in 1992, the shoreline erosion was eventually so excessive that in 1995 it had to be removed to enable the re-stabilisation of the shoreline. The erosion was largely attributed to the generation of set-up currents behind the breakwater. **Stauble and Tabar (2003)** show that the open coast placement (detached, shoreparallel) of a reef structure is not very effective based on the evaluation of three projects along the Florida Atlantic Ocean shoreline. Large settlement can occur after storm events; individual units may be dislodged and scattered.

The design of a submerged breakwater requires detailed mathematical and physical modelling aimed at a better understanding of the hydrodynamic and morphodynamic processes with a focus on the suppression of set-up currents behind the breakwater. Field pilot projects including monitoring should be initiated to determine the optimum layout and dimensions of the structures (see **Stauble and Tabar, 2003**). The effect of local beach nourishments inshore of the breakwaters to mitigate local erosional effects should also be studied. In most cases substantial beach nourishment inshore of the breakwater scheme will be required. The nourishment volume should however not be too large, otherwise the construction of a submerged breakwater scheme including nourishment is not cost-effective compared with the placement of a feeder berm.

Perched beaches with submerged sill
A detached breakwater can also be used as a submerged sill structure at the toe of a beach fill (known as perched beach). The beach fill creates a widened and heightened profile (the perch) confined by the sill. The sill creates a discontinuity (step) in the cross-shore profile so that the profile on the landward side of the sill is higher than the profile on the seaward side. Wave breaking and wave overtopping at the sill often lead to scour immediately behind the sill (landward side), which can be suppressed by constructing a local bottom protection (apron) as a cover over the sand bed. Regular maintenance fills are required as storm waves will erode the beach zone and carry the sediments offshore, whereas onshore transport to the beach is cut off by the sill. If possible, the beach fill should be made of coarse materials with a cover layer of relatively coarse sand (0.3 to 1 mm). A perched beach may be attractive to create a recreational beach at a sheltered site (in mild wave conditions). **Stauble and Tabar (2003)** show that the most successful reef-type structures are perched-beach solutions when the narrow-crested reefs are used to create a closed cell perched-beach at the seaward end of a groyne compartment. The reef is able to prevent offshore transport of sand from within the compartment as long as the reef is attached to the adjacent groyne or jetty tip.

Headland breakwaters
These types of structures offer a good solution to create a recreational beach plain in mild wave conditions (sheltered beaches in microtidal environments). Initial large-scale beach nourishment is required to construct the tombolo planform. The design is relatively straightforward. The parameters to be selected are: beach width at the base of the mainland, initial breakwater length and offshore distance (see Figure 8.5.7). The length of the breakwaters should be designed conservatively so that they can be extended if shoreline erosion inshore of the breakwaters is too large (based on field monitoring). It is more cost-effective to increase the breakwater length (if necessary) than to initially construct breakwaters which are too long.

Stable reef berms and active feeder berms
An alternative solution for a ***hard*** detached breakwater scheme can be the placement of a ***soft*** nearshore feeder berm consisting of sandy material in water depths of 3 to 5 m (micro-tidal conditions) and 4 to 6 m in meso-tidal conditions. The crest width should be of the order of 10 times the water depth above the crest (say 20 to 30 m); the crest level should not be lower than 2 to 3 m below MSL. The berm will act as a reef filtering

out the larger erosive waves by wave breaking during storm events. The shoaling waves will promote onshore transport of sediments to the beach in the lee of the berm. Regular maintenance of the feeder berm is required to keep the feeding process in tact. The alongshore behaviour of a feeder berm will strongly depend on the local dominant wave direction. The placement of a feeder berm generally is a relatively cheap (if sand is available) and environmentally friendly solution to mitigate shoreline erosion. However, it may take a few years to clearly notice beach improvement due to the delayed response of the beach to the presence of a berm at the edge of the surf zone. Often, supplementary beach nourishments are required for recreational purposes.
A stable reef berm is generally constructed in deeper water (10 to 15 m) with the objective to act as a wave filter for the larger erosive storm waves. The construction of a stable reef berm is relatively cheap when it is made of sand dredged from a nearby navigation channel (maintenance dredging).

Hard versus soft solutions

A basic question to be answered for a given site is: can we maintain the shoreline at its present location and hold the sea back? The traditional approach is the 'hard' solution or the 'wall' solution. This approach is always feasible, but the costs to both our Economy and to Nature itself often are enormous? The construction of hard structures is always costly and it always results in a coastline consisting of walls, breakwaters and groynes, often for ten's of kilometres as present along many touristic coasts around the Mediterranean. Soft solutions (in harmony with nature) by dune restoration (including sand retaining plantations) and artificial nourishments both protect the shorelines and rebuild the beaches by bringing new sand (shingle or gravel) in the nearshore system. It is also costly when the materials have to be brought in from considerable distance at sea and the nourishments may have to be repeated many times as the sea tends to reerode the dumped materials.
A philosophy which is recently coming up, is: let nature have its way (**Pilkey et al., 1998; Charlier, 2004**). This approach only is an option for less populated areas, where the loss of land is not a real issue. It can not be applied when maintaining a beach or coastal zone is an economic necessity.
The formidable task for coastal managers and engineers is to find the most feasible, economic and sustainable solution (hard, soft or both) for each site of interest, taking the lessons of history into account.

9 References

Aminti, P. et al., 2004. *Evaluation of beach response to submerged groin construction at Marina di Ronchi, Italy, using field data and a numerical simulation model. Journal of Coastal Research, SI 33, p. 99-120*

Basco, D.R. and Pope, J., 2004. *Groin functional design guidance from the coastal engineering manual. Journal of Coastal Research, SI 33, p. 121-130.*

Berenguer, J. and Enriquez, J., 1988. *Design of pocket beaches, the Spanish case, p. 1411-1425. 21st ICCE, Malaga, Spain*

Charlier, R.H., 2004. *Hold the sea back-is it sustainable ? Retrospective and Projection. Journal of Coastal Research, Vol. 19, No. 4, p. 875-883*

Chasten, M.A. et al., 1993. *Engineering design guidance for detached breakwaters as shoreline stabilization structures. T.R. CERC 93-19, U.S. Army Corps Eng., Vicksburg, USA*

Chrzastowski, M.J., 2004. *History of the uniquely designed groins along the Chicago lakseshore. Journal of Coastal Research, SI 33, p. 19-38*

Dean, R.G. et al., 1997. *Full scale monitoring study of a submerged breakwater, Palm Beach, Florida, USA, p. 291-315. Coastal Engineering 29,*

Dette, H.H. et al., 2004. *Permeable pile groin fields. Journal of Coastal Research, SI 33, p. 145-159*

Fleming, C.A., 1990. *Guides on the uses of groynes in coastal engineering. CIRIA, London, England*

Fleming, C. and Hamer, B., 2000. *Successful implementation of an offshore reef scheme, p. 1813-1820. 27th ICCE, Sydney, Australia*

Galgano, F.A., 2004. *Long-term effectiveness of a groin and beach fill system: a case study using shoreline change maps. Journal of Coastal Research, SI 33, p. 3-18*

Gomez-Pina, G., 2004. *The importance of aesthetic aspects in the design of coastal groins. Journal of Coastal Research, SI 33, p. 83-98*

Hamm, L., Mory, M. and Peronnard, C., 1995. *Hydrodynamic measurements around a detached breakwater in a 3D wave basin. Report 52184 R10, Sogreah, Grenoble, France*

Hsu, J.R.C. and Silvester, R., 1990. *Accretion behind single offshore breakwater, p. 362-380. Journal of Waterway, Port, Coastal and Ocean Engineering, Vol. 116, No. 3*

Kana, T.W. et al., 2004. *Management and engineering guidelines from groin rehabilitation. Journal of Coastal Research, SI 33, p. 57-82*

Kraus, N.C. et al., 1994. *Modern functional design of groin systems, p. 1327-1342. 24th ICCE, Kobe, Japan*

Lamberti, A. and Mancinelli, A., 1996. *Italian experience on submerged barriers as beach defence structures, p. 2352-2365. 25th ICCE, Orlando, USA*

Liberatore, G., 1992. *Detached breakwaters and their use in Italy, p. 13.1-13.23. Short course on design and reliability of coastal structures, 23rd ICCE, Venice, Italy*

List, J.H., Farris, A.S. and Sullivan, C., 2003. *Large-scale response of foreshore slope to storm events. Coastal Sediments, Florida, USA*

Mohr, M.C., Pope, J., and McClung, J.K., 1999. *Coastal response to a detached breakwater system; Presque Isle, Erie, USA, p. 2010-2025. Coastal Sediments, Long Island, USA*

Mory, M. and Hamm, L., 1997. *Wave height, setup and currents around a detached breakwater. Submitted to regular or random wave forcing, p. 77-96. Coastal Engineering Vol. 31*

Nairn, R.B. and Dibajnia, M., 2004. *Design and construction of a large headland system, Keta Sea defence project, West Africa. Journal of Coastal Research, SI 33, p. 294-314*

Pilkey, O., et al., 1998. *Evaluation of alternative or non-traditional shoreline stabilization devices. Journal of Coastal Research, SI 26, p. 269-272*

Poff, M.T. et al. ,2004. *Permeable wood groins: case study on their impact on the coastal system. Journal of Coastal Research, SI 33, p. 131-144*

Pope, J. and Dean, J.L., 1986. *Development of design criteria for segmented breakwaters. 20th ICCE, Taipei, Taiwan*

Raudkivi, A.J. and Dette, H.H., 2002. *Reduction of sand demand for shore protection, p. 239-259. Coastal Engineering 45*

Rosati, J.D., 1990. *Functional design of breakwaters for shore protection: empirical methods. TR CERC 90-15, U.S. Army Corps Eng., Vicksburg, USA*

Rosen, D.S. and Vajda, M, 1982. *Sedimentological influences of detached breakwaters, p. 1930-1949. 18th ICCE, Capetown, South Africa*

Sawaragi. T., 1992. *Detached breakwaters. Short course on design and reliability of coastal structures, 23rd ICCE, Venice, Italy*

Shabica, C. et al., 2004. *Evolution and performance of groins on a sediment-starved coast; the Illinois shore of Lake Michigan, North of Chicago, 1880-2000. Journal of Coastal Research, SI 33, p. 39-56*

Silvester, R. and Hsu, J.R.C., 1993. *Coastal Stabilization: Innovative concepts. Prentice-Hall, Englewood Cliffs, New Yersey, USA*

Stone, G.W., Wang, P. and Armbruster, C.K., 1999. *Unanticipated response to detached, segmented breakwaters along Raccoon Island, Louisiana, p. 2057-2072. Coastal Sediments, Long Island, USA*

Stauble, D.K. and Tabar, J.R., 2003. *The use of submerged narrow-crested breakwaters for shoreline erosion control, p. 684-722, Journal of Coastal Research, Vol. 19, No.3*

Stive, M.J.F. et al. 2002. *Variability of shore and shoreline evolution, p. 211-235. Coastal Engineering 47*

Suh, K. and Dalrymple, R.A., 1987. *Offshore breakwaters in laboratory and field, p. 105-121. Journal of Waterway, Port, Coastal and Ocean Engineering, Vol. 113, N0. 2*

Tomasicchio, U., 1996. *Submerged breakwaters for the defence of the shoreline at Ostia field experiences, p. 2404-2417. 25th ICCE, Orlando, USA*

Trampenau, T. et al., 2004. *Hydraulic functioning of permeable pile groins. Journal of Coastal Research, SI 33, p. 160-187*

Underwood, S.G., Chen, R., Stone, G.W., Zhang, X., Byrnes, M.R. and McBride, R.A., 1999. *Beach response to a segmented breakwater system southwest Louisiana, p. 2042-2056. CS, Long Island, USA*

US Army Corps of Engineers, 1992. *Coastal groins and nearshore breakwaters, Engineering Manual. Report EM 1110-2-1617*

Van Rijn, L.C., 1998. *Principles of Coastal Morphology. Aqua Publications, The Netherlands (WWW.AQUAPUBLICATIONS.NL)*