SMALL COASTAL STORMWATER OUTLETS

PHASE 2: DESIGN GUIDELINES

by

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EXECUTIVE SUMMARY

This project on compiling guidelines for small to medium sized coastal outlets entails two parts, namely: Phase 1: Literature survey; and Phase 2: Design guidelines (this report). Recommendations on the construction of small stormwater outlets are also given. Coastal processes and marine information relevant to the design of these stormwater outlets are briefly described.

The design approach regarding location and layout of outlets should be to “work with nature”. To reduce potential conflict with other beach usages, the outlet structure should be as small and unobtrusive as possible, yet be functional. Public access along the beach must be maintained. Typical site selection criteria for a small coastal outlet have been tabulated. Where possible, a number of different smaller outlets should be combined into one large outlet for hydraulic, cost and aesthetic reasons. Allowance should be made in the design of stormwater outlets for: (1) potential direct wave impacts; (2) coastal flooding or inundation; (3) a long-term shoreline eroding trend (if present); (4) short-term shoreline variations; (5) scour; (6) wind-blown sand; and (7) other aspects.

An outlet structure can be protected by different permanent methods, namely: (1) concrete structures; (2) rock protection; (3) sand bags; (4) grout and block mattresses; (5) gabions and Reno mattresses; and (6) other methods. Design methods are presented for these protection measures. Properly designed and well-built concrete and rock structures have the longest expected life, while mattresses, geotextile and gabion structures will not last as long. For *concrete structures*, it is recommended to use: high quality concrete, the beach profile and the nature of the seabed to reduce cost. For *rock protection*, the rock properties, design approach, wave run-up, required armour rock size, toe design, geotextiles, a typical cross-section and general comments on rock design are presented. Large *sand bags* are the best suited geotextile containers for longer-term protection. Failure mechanisms for sand bag protection have been listed. A double layer geotextile for the bags, a flexible toe, attention to 3-dimensional aspects and physical model tests are recommended. A typical cross-section of a sand bag revetment is given. Requirements for concrete *block and grout mattresses,* conceptual design methods for this type of protection and the placement of geotextiles are discussed. The longevity of *gabions and Reno mattresses* in the sea is normally limited and therefore their marine use is generally not recommended. *Other methods* include exfiltration, building a large pier to support the outlet, and to incorporate the whole outlet in a groyne or seawall. Some of these options are very expensive.

Guidelines have been presented with regard to water quality. In general, pollution should rather be prevented from reaching the seashore by applying “best management practices”. If the contamination of stormwater cannot be prevented, the effluent should be treated, people and animals should be kept away from the impact zone of the pollution and/or the stormwater outlet must be designed to minimise the impact zone. Applicable legislation has also been addressed briefly.

Construction guidelines are provided regarding using favourable weather optimally, comments on the use of marine concrete, effects of sand transport and the protection offered by cofferdams.

KLEIN STORMWATERUITLATE AAN DIE KUS. FASE 2: ONTWERPRIGLYNE

SAMEVATTING

Hierdie projek om riglyne vir klein stormwateruitlate aan die kus op te stel, bestaan uit twee dele, naamlik: Fase 1: Literatuuuroorsig; en Fase 2: Ontwerpriglyne (hierdie verslag). 'n Aantal aanbevelings vir die konstruksie van klein stormwateruitlate word ook gegee. Kusprosesse en mariene inligting vir die ontwerp van hierdie uitlate word kortliks bespreek.

Die ontwerpsbenadering oor posisie en uitleg van uitlate behoort "werk met die natuur" te wees. Om konflik met ander strandgebruike te verminder, moet die uitlaatstruktuur so klein en onopsigtelik as moontlik wees, maar tog funksioneel. Openbare toegang na die strand moet behou word. Tipiese kriteria vir terreinkeuse is getabelleer. Waar moontlik, moet 'n aantal kleiner uitlate in een groter uitlaat saamgevat word vir hidrouliese, koste- en estetiese redes. In die ontwerp van stormwater-uitlate, moet daar voorsiening gemaak word vir: (1) 'n potensiële direkte golfaanslag; (2) vloede en oorstroming; (3) 'n langtermyn-tendens in kuslynerosie (indien teenwoordig); (4) korttermyn-kuslynwisselings; (5) uitskuring; (6) waaisand; en (7) ander aspekte.

'n Uitlaatstruktuur kan met verskillende metodes beskerm word, naamlik: (1) betonstrukture; (2) rotsbeskerming; (3) sandsakke; (4) bryvullings- en blokmatrasse; (5) korfskanse en Reno-matrasse; en (6) ander metodes. Ontwerpmetodes vir hierdie beskermingmaatreëls word voorgelê. Behoorlik ontwerpte en goed geboude beton- en rotsstrukture het die langste verwagte lewensduurte, terwyl matrasse, geotekstiel- en korfskansstrukture nie so lank sal hou nie. Vir *betonstrukture,* word aanbeveel dat hoë-gehaltebeton, die strandprofiel en die aard van die seebodem gebruik word om koste te bespaar. Vir *rotsbeskerming* word rotseienskappe, die ontwerpbenadering, golfoploop, die vereiste grootte van die bewapeningsrots, toonontwerp, geotekstiele, 'n tipiese dwarssnit en algemene kommentaar oor rotsontwerp voorgelê. Groot s*andsakke* is die geskikste geotekstielhouers vir langer-termynbeskerming. Swigtingsmeganismes vir sandsakbeskerming word gelys. 'n Dubbellaaggeotekstiel vir die sakke, 'n buigsame toon, aandag aan 3-dimensionele aspekte en fisiese modeltoetse word aanbeveel. 'n Tipiese dwarssnit van sandsakbeskerming word gegee. Vereistes vir *betonblok- en bryvullingsmatrasse*, konseptuele ontwerpmetodes en die plasing van geotekstiele word bespreek. Die lewensduur van *korfskanse en Reno-matrasse* in die see is normaalweg beperk en dus word mariene gebruik gewoonlik nie aanbeveel nie. *Ander metodes* sluit in eksfiltrasie, die bou van 'n groot pier om die uitlaat te ondersteun en om die hele uitlaat binne-in 'n strandhoof of seemuur te plaas. Sommige van hierdie opsies is baie duur.

Riglyne oor watergehalte word gegee. Oor die algemeen moet besoedeling eerder verhoed word om die kuslynte bereik deur "beste bestuurspraktyke" toe te pas. As die besmetting van stormwater nie verhoed kan word nie, behoort die uitvloeisel behandel te word, mense en diere moet weggehou word van die impaksone van die besoedeling en die stormwateruitlaat moet ontwerp word om die impaksone te minimaliseer. Toepaslike wetgewing word ook kortliks aangespreek.

Konstruksieriglyne word gegee vir die optimale benutting van goeie weer; kommentaar oor die gebruik van mariene beton; effekte van sandvervoer; en die beskerming deur kofferdamme.

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1. INTRODUCTION

Stormwater usually drains into nearby rivers, lakes, pans and wetlands. For municipalities situated on the coast, most stormwater usually drains to the sea. Comprehensive manuals for stormwater management on land have been published (for example, Georgia Stormwater Management Manual (2001) and the Stormwater Management Manual for Western Australia (2007)). The same applies for coastal engineering: manuals are available (for example, CEM (2006), CIRIA, CUR, CETMEF (2007) and Kamphuis (2012)). However, only limited literature and guidelines are available for stormwater outlets at the interface between land and sea, namely: the shoreline. At the shoreline, not only the *land factors* such as ground slope and runoff, but also *coastal factors* like waves, currents and sediment transport affect stormwater drains.

Large stormwater flows discharging to the sea are conveyed with large outfall structures; for example, a pier (a pipe on or under a deck supported by piles). Large stormwater flows normally drain large catchments within cities, and thus the required funding is usually available or can be motivated for in such cases. The result is that large stormwater outlets are usually designed to withstand coastal forcing as well as the land factors. However, in the case of small and medium outlets, limited funds are usually available despite the fact that the problems can still be considerable. Limited funds often lead to small and inadequate outlets on the shoreline, which result in damage to outlets costing relatively large sums of money to repair.

The ultimate aim of this project is to supply general guidelines which should be used during the design of small to medium sized outlets situated near or on the shoreline. Although guidelines for the *construction* of small to medium stormwater outlets will also be briefly described, the emphasis is on *design* guidelines. To achieve this ultimate aim, the project has been divided into two parts, namely:

- **Phase 1: Literature survey**. The objective was to compile guidelines available in international literature and to make sure that these guidelines are applicable to South African conditions. It was concluded that the limited guidelines that were available for small coastal stormwater outlets (summarised in the Phase 1 report; Schoonees and Theron, 2016) are insufficient for coastal design. Therefore, Phase 2 of the project has been undertaken.
- **Phase 2**: **Guidelines for stormwater outlets**. In this phase of the project, existing guidelines for small stormwater outlets will be presented and augmented by guidelines focusing on the coastal factors influencing the design of stormwater outlets and the problems caused by these factors. Phase 2 (this report) addresses the question: “How should the design of coastal stormwater outlets be undertaken?” If the number of failures of coastal stormwater outlets and the associated costs are considered, it is clear that extensive design guidelines are required. (Note, that for clarity and convenience, a few figures and sections of text from the Phase 1 report have been repeated in this Phase 2 report.)

Therefore the aim of this report, which only covers Phase 2 described above, is to compile a set of guidelines for small and medium coastal stormwater outlets applicable to South African conditions (of which the outlets around Mossel Bay serve as an example). Only small to medium sized outlets, defined here to be outlets having diameters of less than 1000 mm or rectangular (or square) outlets smaller than 1000 mm by 1000 mm will be considered. Furthermore, these outlets have to be located near or on the shoreline between the low tide line and the landward extremity of sea-storm impacts.

Although the emphasis is on the coastal factors impacting on stormwater outlets, mitigation measures for the land factors have also been briefly addressed (refer to the Phase 1 report; Schoonees and Theron, 2016). These mitigation measures for land factors (widely called best management practices; BMPs) influence stormwater and thus also contribute to the marine design of these outlets. For example, it is considerably less expensive to reduce the stormwater flow on land with these best management practices than having to construct a much larger stormwater outlet on the beach.

The guidelines provided in this report focus on coastal and marine factors, such as taking into account coastal processes like winds, waves, currents, tides, sediment transport, erosion/accretion and scour. The dispersion of contaminated stormwater in the nearshore zone (including the surf zone) will only be described briefly. The land factors, which include the hydraulic and structural design of stormwater infrastructure and outlets (such as the slope of pipelines or outlets, invert levels, flow capacity or discharge) are not included in the scope of this study. Numerous hydraulic manuals are available for this purpose.

Clearly, all engineering works have to be undertaken within the law. As a result, the most important applicable legislation will be presented briefly. However, it falls outside the scope of this report to discuss, in detail, the different applicable South African laws.

The guidelines in this report are necessarily of a general nature. In certain cases, other measures may be more appropriate or more important based on local conditions and the characteristics of the particular site. IMESA, Stellenbosch University, IWEESU and the authors take no responsibility for the application of the guidelines in this report. A competent civil engineer needs to assess all factors and take full responsibility for his/her design and construction of a stormwater outlet. However, it is hoped that the guidelines in this report will serve as an important aid for designers and local authorities, enabling comprehensive and cost-effective design and construction of small stormwater outlets.

The layout of this report is as follows:

- Chapter 2 is a brief description of the relevant **coastal processes** that should be considered in the design and construction of small coastal stormwater outlets.
- Chapter 3 deals with the **coastal information** required for the design of small coastal outlets.

- In Chapter 4, **guidelines** for the design and construction of small and medium coastal stormwater outlets are presented.
- The **summary, conclusions and recommendations** end off the report (Chapter 5).

2. PHYSICAL COASTAL PROCESSES

2.1 General

Coastal processes relevant to the design of small coastal stormwater outlets are briefly described in this chapter. These processes include wave processes, water-levels, currents and circulation, sediment transport and the dilution and dispersion of effluent.

2.2 Wave processes

2.2.1 Wave characteristics

Ocean waves are usually characterised by wave height, wave period, wave direction and wavelength. The wave height is the vertical distance from the wave trough to the adjacent crest of the wave. The wave period is the time that it takes for one complete wave (two wave crests) to pass a fixed point. The direction from which the wave is coming (the wave direction) is indicated by a wave orthogonal; that is, a line perpendicular to the wave crests. The wavelength is the horizontal distance between adjacent wave crests (or between adjacent wave troughs).

In deep water, the shape of a wave is typically sinusoidal whilst the wave crests becomes more peaked with flatter troughs in shallow water.

2.2.2 Wave generation

Waves are generated by wind blowing consistently over water (the ocean in this case). Wind waves or seas, are waves caused by wind at the place and time of observation; that is, waves locally generated by wind. In contrast, swell waves are wind-generated waves that have travelled out of the generating area. Swell waves generally are more regular and have longer periods and flatter crests than wind waves.

A wave spectrum of a wave train (or series of waves) is a statistical representation of a particular wave field (in deep or in shallow water) and describes the distribution of wave energy as a function of frequency (invert of the wave period) and wave direction.

For the South African coastline, most of the swell waves are generated either in: (1) the southern ocean during the passage of low-pressure weather systems; or (2) by cyclones along the Northern KwaZulu-Natal coast having migrated southwards from the Mozambique Channel. The semi-permanent subtropical high-pressure cells off the west and east coasts is responsible for the higher-frequency wave conditions on the west coast. Cut-off low (COL) systems normally consist of a low-pressure cell blocked by two high-pressure cells on either side. COL systems are relatively common but during rare stationary periods can result in extreme storms along the south-east and east coasts. Together, all these different waves form the deep-sea wave climate.

2.2.3 Nearshore waves and wave transformation

Nearshore waves directly attack marine structures like breakwaters and coastal stormwater outlets. The nearshore wave climate, which also plays an integral role in the generation of nearshore currents and sediment transport, is dependent on the deep-sea wave conditions and the nearshore wave processes (CSIR, 1995; WSP, 2013).

Deep-sea waves are transformed when moving from deep water to the coast. This wave transformation results in the nearshore wave climate. When deep-water waves approach a coastline, a number of processes occur (the wave transformation). The most important of these processes during wave transformation are: refraction, shoaling, diffraction and breaking (CSIR, 1995; WSP, 2013; CEM, 2006).

Refraction, which takes place when a wave travels at an angle to the bottom contours, includes the change in the orientation of the wave crest towards alignment with the bottom contours (CSIR, 1995; WSP, 2013). This happens because the portion of the wave advancing in shallower water moves more slowly than the portion which is still advancing in deeper water (CSIR, 1995; WSP, 2013).

Shoaling is the process by which the wave height changes when a wave travels from deep water to shallow water, irrespective of its direction of travel (CSIR, 1995; WSP, 2013).

Diffraction is the phenomenon by which energy is transmitted laterally along a wave crest. This effect is particularly noticeable when part of a series of waves is interrupted by a barrier such as a breakwater or rocky headland (CSIR, 1995; WSP, 2013).

The positions along the shoreline, at which the waves first start breaking due to the reducing depth, together form what is called the breaker line (CSIR, 1995; WSP, 2013). The zone between the breaker line and the shoreline is called the surf zone. Usually, waves start breaking at a depth of approximately 1.28 times the wave height at that location; that is, H_{bs}/d_b= 0.78 (or 0.6 to 1.1; H_{bs}= breaking significant wave height and d_b= depth at breaking).

There are different types of breaking of which plunging, spilling and a combination of plunging and spilling breaking are the most important (CEM, 2006). The type of breaking is determined by the surf similarity parameter, which contains the nearshore seabed slope, the wave height and the wavelength

(CEM, 2006). The dissipation of wave energy varies considerably for the different types of wave breaking and thus the forces on marine structures can also fluctuate significantly depending on the type of breaking.

Wave run-up is the rush of water up a structure, rocky slope, or a beach following from the breaking of a wave (CSIR, 1995; WSP, 2013). Run-up is dependent on, among other factors, the beach slope, wave characteristics, permeability and the roughness of the slope. Calculations of wave run-up includes wave set-up (refer to Section 2.3.4).

2.2.4 Exposure to wave action

Islands, capes, rocky headlands and large sand spits provide protection against wave action. Furthermore, bathymetric features like channels, deeper zones, submerged shoals and shallow reefs also affect the nearshore wave regime, resulting in an increase in wave height in certain locations and a decrease in wave height in other areas.

Wiegel (1964) classified beaches as either "exposed", "moderately protected" or "protected". In undertaking a coastal vulnerability assessment, Theron (2016) categorised the coastline into four categories with regard to wave exposure. (Theron's fifth category included the hazard of a river mouth breaking open and/or the meandering of the river.) In this study, the exposure to (only) wave action is classified as (Theron, 2016):

- **Category A: Fully protected**. The site is located such that the surrounding coastal features protect it from all or almost all wave conditions. Typically, such a site is in a sheltered part of a bay or directly in the lee of an island or large rocky headland.
- **Category B: Moderately protected**. Coastal features provide partial sheltering against wave action; especially, against the dominant wave conditions. The site is more protected than exposed.
- **Category C: Moderately Exposed**. Coastal features provide limited sheltering against waves; for example, sheltering is given for only a few wave conditions and/or partial protection against the dominant waves. The site is more exposed than protected and is typically situated along the middle and exposed portion the shoreline of a large bay.
- **Category D: Fully exposed**. The site is on an open coastline which does not offer any sheltering against waves.

2.3 Seawater-levels

2.3.1 General

Coastal water-levels are influenced by a variety of astronomical and meteorological/oceanographic factors. At times, these factors interact in a complex way to elevate water-levels significantly above or below normal levels. Elevated water-levels may intensify damage to coastal structures due to increased incidence of larger waves approaching and breaking closer to the beach. These may result in increased beach erosion, as well as an increased threat to coastal development. Elevated water-levels may also cause the inundation of low-lying, coastal areas and areas around estuaries.

2.3.2 Tides and datum levels

Tides are usually one of the most important mechanism influencing water-levels. Tides are the periodic rising and falling of seawater that results from gravitational attraction of the moon, sun, and other astronomical bodies acting upon water bodies on the rotating earth (WSP, 2013). Semi-diurnal tides (as occur in South Africa) result in, on average, two high tides and two low tides per day with an associated tidal range. The tidal range varies from month to month depending on the alignment of the astronomical bodies.

The different tidal levels along the South African coastline are as follows (SANHO, 2017):

Table 2.1: South African tidal levels (SANHO, 2017)

Abbreviation of the tidal level	**Description of the tidal level**
HAT	Highest Astronomical Tide (HAT is the highest predicted astronomical tide under average meteorological conditions over a full 18.6-year nodal cycle and is not reached every year.)
MHWS	Mean High Water Spring (The average height of the high water occurring at the time of spring tides.)
MHWN	Mean High Water Neap (The average height of the high water occurring at the time of neap tides.)
MLWN	Mean Low Water Neap (The average height of the low water occurring at the time of neap tides.)
MLWS	Mean Low Water Spring (The average height of the low water occurring at the time of spring tides.)
LAT	Lowest Astronomical Tide (LAT is the lowest predicted astronomical tide under average meteorological conditions over a full 18.6-year nodal cycle and is not reached every year.)

The datum level of hydrographic surveys by the SA Navy is Chart Datum (CD), which corresponds to LAT. Land-levelling Datum (LLD) is also called Mean Sea Level (MSL). Refer to Section 3.6 for the tidal levels at different locations around the SA coastline. The height that CD is below MSL varies around the SA coastline (SANHO, 2017); refer to Table 2.2. For example, at Port Nolloth, CD is at -0.925 m to MSL (Table 2.2).

Table 2.2: Heights (m) that CD is lower than MSL along the SA coastline (SANHO, 2017)

Location	Height (m) that CD is lower than MSL (From 1 Jan 2003 and onwards)
Port Nolloth	0.925
Saldanha Bay	0.865
Cape Town	0.825
Simon's Town	0.843
Hermanus	0.788
Mossel Bay	0.933
Knysna	0.788
Port Elizabeth	0.836
East London	0.716
Durban	0.913
Richards Bay	1.015

2.3.3 Storm surge

Storm surge consists of mainly two components, namely: (1) wind set-up; and (2) a variation (rise or drop) in the water-level because of low or high barometric (air) pressure respectively. Storm surge may also significantly increase local water-levels. Strong wind, associated with a storm occurrence, blowing over the surface of the sea towards land may result in elevated water-levels at the beach (wind set-up; WSP, 2013; CEM, 2006)). In addition, storms are often accompanied by a low pressure system which contributes to a rise in water-levels (inverse barometric set-up; CEM, 2006).

2.3.4 Wave set-down and set-up

When waves approach the shoreline, there is a drop in the *average* water-level at the breaker line compared with the still-water level, called wave set-down (CEM, 2006). However, the average water-level rises above the still-water level from the breaker line to the shoreline, which is called wave set-up (CEM, 2006). Thus, wave set-up near the shoreline is the rise in the *average* elevation of the water surface (especially at the shoreline) due to onshore mass transport of the water by wave action alone. The degree of set-up depends on the type, size and period of the breaking waves, as well as on the beach slope (CEM, 2006).

2.3.5 Sea-level rise

The Intergovernmental Panel on Climate Change (IPCC) found that the average temperature of the atmosphere and the ocean has increased (warmed), resulting in the melting of snow and ice and the rising of the sea level (IPCC, 2013). The main reason for this warming is human influence.

The following conclusions based on Church *et al*. (2013) in IPCC (2013) can be drawn:

- Accurate measurements during the period 1993 to 2010 indicate a global average rate of sea-level rise of 3.2 mm/year.
- The rate at which the sea level rise will rise in future will very likely exceed the rate observed from 1971 to 2010 due to increased warming of the ocean and further melting of glaciers and ice sheets.
- The rate at which the sea level rises, varies from location to location on the globe. Processes such as the variability in ocean circulation, cause these differences in sea-level rise. In a few areas, the sea level actually dropped.
- Based on graphs presented, the rise in sea level around the South African coast is similar to the global average sea-level rise.

After correcting the tidal record between 1970 and 2003 for Durban, Mather (2007) analysed the tide data and concluded that the rate of sea-level rise for Durban is 2.4 mm/year to 2.7 mm/year. These rates, which are similar to the global average rate, refer to a linear rise of sea-level over time.

Theron (2016) concluded that an appropriate scenario for sea-level rise by 2100 is between 0.85 m to 1 m. This scenario agrees with the predictions of Church et al. (2013) in IPCC (2013). It is recommended in this report that a sea-level rise of 1 m be allowed for over the next 100 years. This best estimate should be revised in the near future when newer IPCC findings become available.

2.4 Currents

2.4.1 Wave-driven currents and nearshore circulation

Figure 2.1 illustrates typical nearshore (current) circulation on a long straight beach, which consists of the following currents:

- Mass transport of water outside the surf zone.
- Flow of water in the bores (broken waves) and undertow.
- Longshore currents.
- Rip currents.

Mass transport of water is the slow net flow of water movement in the direction of the waves outside the surf zone (CSIR, 1995; WSP, 2013; CEM, 2006). As a result, water enters the surf zone.

In the surf zone, there is a strong flow of water in the bores (broken waves) towards the shoreline in the direction of the waves. This flow occurs especially in the upper layers of the water near the sea surface (CEM, 2006). At the same time, there is a weak net flow of water in a cross-shore direction towards the breaker line close to the seabed (this is the undertow; CEM, 2006).

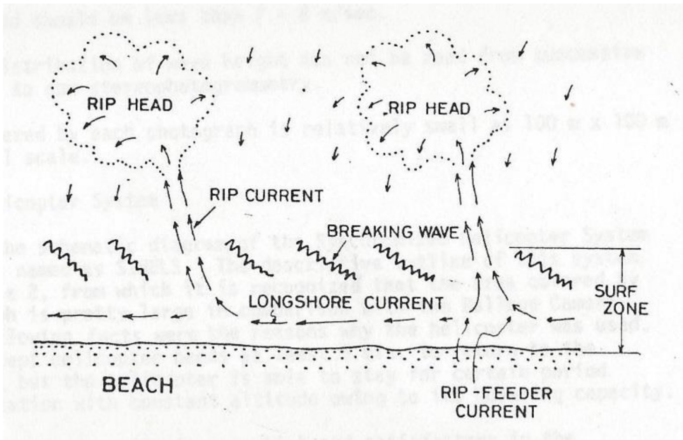


Figure 2.1: Schematic diagram of a typical nearshore current system (from Inman and Bagnold, 1963)

A longshore current is the flow of water parallel to the shoreline (CEM, 2006; Figure 2.1). Waves approaching the coastline obliquely and/or a longshore gradient in breaking wave height (resulting from a rocky reef or breakwater) mainly generate longshore currents in the surf zone (Komar, 1983). Longshore currents are the strongest in the surf zone (close to the breaker line) and their velocity diminishes quickly outside the surf zone (Komar, 1983; CEM, 2006).

Rip currents, which are strong surface currents flowing seawards in a narrow zone, take water out of the surf zone (Figures 2.1 and 2.2; CEM, 2006; CSIR, 1995; WSP, 2013). Outside the surf zone, the velocity of the rip current quickly reduces in the rip head (Figure 2.1). A part of the longshore current (and the other flows) feed the rip current and some of the longshore current bypasses the rip current to continue alongshore to the next circulation cell (Inman and Bagnold, 1963). A series of nearshore circulation cells is formed by the above.

Along irregular shorelines, the nearshore circulation can be completely different to the circulation along a straight coast. For example, the location and heights of rocky reefs and sand banks can significantly influence local flow patterns.

2.4.2 Currents generated by other mechanisms

Winds and tides also generate nearshore currents (CEM, 2006). Except in river mouths and at the entrance to lagoons or harbours, tidal currents are usually weak along the South African coastline. Note that the effect of the South African ocean currents, the Agulhas and Benguela Currents, are virtually always very small close to the shoreline and especially in the surf zone.

Figure 2.2: A rip current

2.4.3 Stratification

Stratification is when different layers of water having different densities, flow over each other in the water column (from the seabed to the water surface; CEM, 2006). The flow in the different layers can be in different directions. The different densities usually occur because of varying salinity and/or temperature of the water in the layers. Good mixing by waves and/or winds can break up stratification.

An example of stratification can be found in estuaries (upstream of river mouths) where colder, saline water enters the estuary close to the seabed from the sea whilst fresh, warmer water flows out to sea at the surface. Another example is at Saldanha Bay where the warmer surface water flows out to sea because of wind action and the rotation of the earth. Upwelling occurs when this water is replaced by cold, dense water entering Saldanha Bay close to the seabed.

2.5 Sediment transport

2.5.1 General

Sediment transport in the nearshore region is usually categorized as longshore or cross-shore sediment transport (CSIR, 1995; WSP, 2013). On an exposed beach, aeolian (wind-blown) sediment transport also plays a role. In general, sediment is very rarely moved by only one mode of transport; longshore, cross-shore and aeolian sediment transport occurs simultaneously (CSIR, 1995; WSP, 2013). Even on a long straight beach, the current circulation pattern (including rip currents) and the associated sediment transport patterns are very complex. Furthermore, marine sediment transport is dependent on wave and tide conditions with the result that it changes continually, not only in direction and rate, but also in the location at which it takes place in the nearshore zone (CSIR, 1995; WSP, 2013).

2.5.2 Longshore transport

When waves that advance towards the coast reach the nearshore zone, sediment (predominantly sand) is stirred up (CSIR, 1995; WSP, 2013). Although non-breaking waves also move sediment, most of the sand is transported inside the surf zone where wave breaking is the primary agent for suspending sand and moving sand along the bottom (CEM, 2006). Longshore currents can usually not entrain sediment on their own; however, sand stirred up by breaking waves is transported alongshore by these currents. Along a fully exposed beach, most of the longshore sediment transport occurs from about +2 m to mean sea level (MSL) and to depths of less than about 8 m to 10 m to MSL (WSP, 2013). Along fully protected beaches, longshore transport usually occurs up to depths of about 3 m to 4 m to MSL (WSP, 2013).

Depending on the environmental conditions, sediment is transported alongshore both upcoast and downcoast. The net longshore transport is the difference between the upcoast and downcoast transport rates. The gross transport is the sum of the (absolute values of) upcoast and downcoast transport rates.

If the longshore sediment transport is interrupted by an obstruction, such as a groyne or a breakwater, accretion will occur on the updrift side and erosion on the downdrift side (WSP, 2013). The latter (erosion) is due to the fact that the sand that previously fed the downdrift beach is trapped and thereby prevented from reaching the downdrift beach. At the same time, sand is still moved away from downdrift beach.

Longshore transport rates are usually determined by measuring accretion and erosion rates adjacent to coastal structures and at sand spits, sedimentation of harbour entrance channels or test pits, or by using sand tracers or by different kinds of samplers and traps (Schoonees and Theron, 1993).

2.5.3 Cross-shore transport

Cross-shore sediment transport is usually a swift process whereby a beach is eroded near the water-line during a storm (CSIR, 1995; WSP, 2013). The sand is transported seawards and deposited in deeper water where it forms an underwater bar on which the storm waves break (CEM, 2006). When the sea calms down again, sand is slowly transported back to the beach, thus re-establishing approximately the original beach profile if no net loss of sand has occurred. Most of the transport occurs in depths less than 10 m to MSL and, typically, insignificant volumes of sand are transported cross-shore in depths greater than 10 m to 15 m to MSL along exposed beaches (WSP, 2013).

2.5.4 Aeolian transport

Aeolian or wind-blown sediment transport refers to sediment that is moved by wind action. If the wind speed gradually increases as the wind blows over a flat expanse of sand, then at the point of incipient motion (or threshold of transport), a significant number of sand grains will be transported by the wind. As the wind speed increases further, more and more sand will be transported. Sand grains move in a variety of modes: by rolling (or creep), saltation (moved in a series of jumps in the wind direction), and suspension (Bagnold, 1954). Saltation is the most important mode in which sand grains are transported (Horikawa, 1988). Most of the sand is transported close to the ground.

Optimum conditions for wind-blown sand transport are the availability of dry, loose sand, strong winds, no salt binding the sand surface, no vegetation, and a long wind fetch, that is, a long expanse of sand over which the wind can blow (CSIR, 1995; WSP, 2013). These conditions are assumed when transport rates are predicted and therefore, potential aeolian transport rates are predicted. Fluctuations in the wind speed over time, as well as the grain size of the sediment also affect the transport rate (Horikawa, 1988).

Usually, the rate of aeolian sediment transport is orders of magnitude lower than the wave-driven (longshore and cross-shore) transport rate along a fully exposed coast (CSIR, 1995; WSP, 2013).

2.6 Effluent, water quality and dilutions

2.6.1 Effluent and water quality

Effluent is the flow of wastewater entering a river, lake or the sea. In this particular case, the effluent is the stormwater that is discharged to and disposed of in the sea. Substances like salt and nutrients dissolve in the rain water while sediment and debris are entrained and suspended in the effluent. It is well-known that stormwater can be contaminated; for example, after flowing over large parking areas or through informal settlements with poor sewerage systems.

Stormwater flowing into the sea is usually buoyant because it is fresh water entering salt water that has a higher density. However, in the case of brine released from a desalination plant into the stormwater system, the effluent can be a dense effluent. Thus, buoyant effluent will tend to rise to the water surface whilst dense effluent will tend to sink to the seabed.

It is important that the constituents (substances) in stormwater be determined because the detrimental effects vary for different constituents (DWAF, 2004). Passive, conservative substances such as suspended solids, are substances that do not change or react as time progresses. On the other hand, microbiological organisms die off over time. Furthermore, chemical and biological processes can change the concentrations of certain substances over time.

South African Water Quality Guidelines for: (1) the natural environment; and (2) for recreational use can be obtained from DWAF (1995) and DEA (2012) respectively. These documents cover the acceptable ranges of and/or maximum allowable concentrations of physico-chemical properties such as salinity and temperature and constituents including copper, nitrates and petroleum hydrocarbons. Furthermore, DWAF (2004) provides general guidelines for the disposal of land-derived water containing waste to the sea.

2.6.2 Dilution and dispersion of effluent

2.6.2.1 General

Dilution of effluent is the process of reducing the concentration of the constituents by mixing the effluent with water, usually from the body of water (typically, the sea) into which the effluent is discharged. Dilution is defined as follows (DWAF, 2004):

The required dilution for a particular constituent in an effluent to adhere to the maximum allowable concentration

$$= (C_E - C_A) / (C_G - C_A)$$

Where C_E=concentration of the constituent in the effluent

C_A= ambient (background) concentration of the constituent in the receiving water (the river, lake or sea)

C_G= maximum allowable concentration of the constituent (not to be exceeded in the river, lake or sea) as given in the SA Water Quality Guidelines.

For each constituent, the required dilution has to be determined. The highest of these dilutions is then the required dilution for the effluent as a whole.

The total dilution of an effluent from the outfall to any point of interest is (Toms and Fijen in Lusher, 1984) =

Initial dilution x Secondary dilution x Dilution resulting from chemical and biological processes (including die off of organisms).

The initial dilution can be defined as the dilution achieved by the entrainment of surrounding water into the discharged effluent through the water column at the discharge point(s) of the outfall (DWAF, 2004). For example, for a buoyant effluent, the effluent jets out of and rises from the discharge point or port (usually at the seabed) to the water surface forming a plume of effluent (DWAF, 2004). Note that stratification can contain a rising buoyant plume to the lower water layer (DWAF, 2004) with the result that the plume will not reach the water surface (an advantage) but with lower dilution (a disadvantage). For a dense effluent there is no buoyant entrainment if it is discharged at the seabed. If a dense effluent is discharged at the water surface, then there is entrainment as the effluent sinks to the seabed.

Secondary dilution is the dilution of the plume of effluent after initial dilution as it is transported away by current and wave action from the outfall towards any point of interest (DWAF, 2004). Dispersion is the movement and spreading of the effluent away from the outfall. The dispersion varies both spatially and over time because currents and waves are different in different locations and can fluctuate over time because conditions vary.

2.6.2.2 Calculating dilutions

Initial dilution

Different analytical methods (DWAF, 2004; and Toms and Fijen in Lusher, 1984) and models (for example, CORMIX (Jirka *et al*., 1996)) can be applied to compute initial dilutions. For coastal outlets, the initial dilution is normally low because of low discharge velocities and shallow water.

Secondary dilution

For the calculation of secondary dilutions, a distinction has to be made for discharge in: (1) the surf zone (Case A); and (2) the sea outside the surf zone, and in rivers and lakes (Case B). Case A is almost always the relevant case for small coastal outlets in South Africa because they discharge close to the shoreline and waves are usually present. If no waves are present, Case B applies such as when it is very calm in the sea or when the outlet discharges to a lake, river or large dam.

Case A (In the surf zone)

The distribution, transport and dilution of passive, conservative substances when introduced to the surf zone (at a relatively straight shoreline) are complex processes and the prediction of the ultimate behaviour of such a waste stream is not easy. The mixing and transport are dependent on various physical processes such as wave action, wind shear, tides and ambient currents. The two important mixing mechanisms are: (1) the breaking wave and its bore, causing rapid diffusion normal to the shoreline over the surf zone; and (2) the advective process due to longshore and rip currents. Inman *et al.* (1971) relate the mixing process in the surf zone to "nearshore circulation cells", that is, the eddy or circulation "cell" between two rip currents (also refer to CSIR, 1995 and DWAF, 2004). These circulation cells produce a continuous interchange between adjacent cells in the surf zone and the offshore zone. A waste stream from the land (Q_e in Figure 2.3) is carried along the shore and mixed with the offshore waters by the rip currents (Figure 2.3).

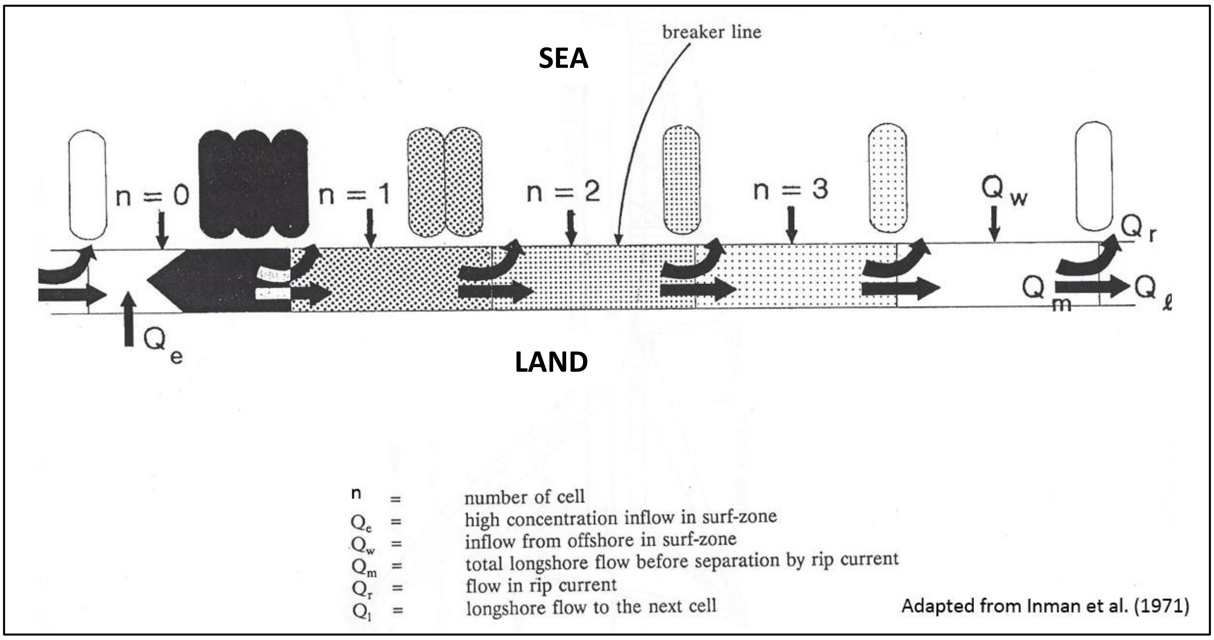


Figure 2.3: Mixing in nearshore circulation cells on a straight beach (Inman et al, 1971; CSIR, 1995)

When a substance or potential pollutant is introduced into the surf zone it will quickly be diffused by the turbulence in the bore (the broken wave) until it has a uniform concentration over the width of the surf zone (Inman et al, 1971). Further dispersion can then be considered to be one-dimensional as further dilution will only be in an offshore direction and longshore advection becomes the paramount process in the further dispersion of the waste field (Figure 2.3; Inman et al, 1971).

Inman et al (1971) solved the diffusion-advection (partial differential) equation whilst allowing for a source/sink term. They postulated that the longshore dilution ratio (R_l) is determined by the longshore and rip currents as follows (Figure 2.3; CSIR, 1995):

$R_l = Q_l/Q_m$

Where Q_l= Longshore flow to the next cell

Q_m= Total longshore flow in the previous cell before separation by the rip current

Therefore the rip current flow $Q_r = Q_m - Q_l$

Inman et al (1971) found from field measurements that the value of R_l fluctuates between 0 and 0.5. R_l approached 0.5 for longshore current velocities exceeding 0.4 m/s. Prototype results showed R_l to vary between 0.38 and 0.48 for cases during which the longshore current velocities were approximately 0.5 m/s (CSIR, 1995).

The concentration in the n-th cell (Figure 2.3) can be given by (Inman et al, 1971; CSIR, 1995):

$C_n = C_{n-1}\,(Q_l/Q_m)$

Or $C_n = C_0\,(Q_l/Q_m)^n$

With C_0= initial concentration

Inman et al (1971) derived the following equation in terms of the longshore distance from the injection point (discharge location) and the length of the circulation cells, namely:

$C_n = C_0\,(Q_l/Q_m)^{y/Yr}$

And y= distance alongshore from the injection point

Yr= length of the circulation cells (which is the spacing between rip currents)

The mixing averaged over the surf zone in the cross-shore direction and the concentration of the effluent as discharged into the sea at the outfall (C_e) enables the calculation of the initial concentration as follows (Inman et al, 1971; CSIR, 1995):

$C_0 = C_e / v_x$

Where v_x= cross-shore mixing coefficient.

Based on prototype measurements, Inman et al (1971) determined an equation for v_x, namely:

$v_x = 5.22\, c_2\, H_{bs}\, X_b / T_p$

Where H_{bs}= significant breaker wave height

X_b= surf zone width

T_p= peak wave period

The values of c_2 are listed in Table 2.3 below:

Table 2.3: Values of the coefficient c_2

Beach slope	**c_2**
0.09 to 0.12 (1/11.1 to 1/8.3	0.1
0.014 to 0.02 (1/71 to 1/50)	0.2

For each site, it is required to measure the rip current spacing (Yr) and the surf zone width off aerial photographs and satellite images. Data from around the world (including Durban and Richards Bay), indicate that Yr varies between 1.5 X_b and 8 X_b, with a best estimate range of 3 X_b and 4 X_b (Sasaki and Horikawa, 1975; Huntley and Short, 1992; CSIR, 1995).

The applicability of the above method has to be assessed if the shoreline is irregular. For example, mixing in nearshore circulation cells and the effect of rip currents could be the dominant mechanisms for dilution of the effluent. However, note that on a rocky coast, the detailed configuration of the rocky areas (including heights) has a determining influence on the flow patterns and therefore, also on the dilution of the effluent. Thus, the method of Inman et al (1971) will not be applicable in areas with irregular rocky areas. In such a case detailed modelling will have to be undertaken.

Case B (Outside the surf zone; rivers, lakes)

Secondary dilution for Case B occurs because turbulence and eddies along the boundaries of the moving plume of effluent entrain water, resulting in mixing of new water with the effluent. Thus, the effluent is further diluted.

Numerous analytical and empirical methods and numerical models exist for this purpose of determining the secondary dilution (Fischer et al, 1979; DWAF, 2004; Botes and Taljaard, 1996; and Toms and Fijen in Lusher, 1984). The Delft3D modelling suite by Deltares and the Mike modelling suite by the Danish Hydraulic Institute have successfully been used in the past to simulate the transport and dispersion of effluent. Most of these methods and models solve the diffusion-advection differential equation.

3. COASTAL INFORMATION FOR DESIGN OF STORMWATER OUTLETS

3.1 General

The first and very important step of designing coastal structures is to obtain a good understanding of the coastal processes and dynamics at the project site as well as in the larger "regional" context. Coastal structures should be designed by studying all available information regarding the geography, coastal dynamics, beach characteristics, wave regime, long-term shoreline evolution, eustatic rise in sea level, aeolian sediment dynamics and the characteristics of the foredune (should such a dune exist). Essential information for designing a coastal structure includes good topographical data of the shoreline and backshore areas, such as can be provided by means of, for example, a conventional topographical or LiDAR survey. Investigation of all these aspects aid the long-term sustainability of projects and help to keep the littoral active zone free from impacts due to unwisely sited infrastructure or developments.

The coastal processes that need to be considered and information required for design and construction include:

- Location of the site
- Bathymetry and Topography
- Nature of shoreline and seabed
- Winds, Waves, Currents
- Seawater-levels
- Sediment transport: longshore, cross-shore, aeolian
- Environmental issues
- Effluents and water quality; dilution and dispersion
- Conflicting beach usages

In terms of the data required for the design of coastal outlet structures, some of the most onerous requirements are: coastal topography, inshore wave conditions, historic shoreline changes, and potentially inshore bathymetry, although the coarser SAN bathymetry data is mostly sufficient.

3.2 Conflicting beach usages and environmental issues

Many of the problems in coastal areas relate to escalating conflicts between development (and associated infrastructure), and environmental protection and management of natural resources (Celliers *et al,* 2009; Mead *et al*, 2013). Coastal infrastructure, including outlets, also have to consider other important aspects, namely ecological and social issues. Ecological concerns refer to ecologic, biodiversity, environmental conservation aspects, and related integrated coastal management (ICM) requirements. Due to reductions and losses of ecologically important zones in coastal areas, increased provisions are required to compensate or mitigate these impacts. This entails determining environment buffers required adjacent to sensitive areas to maintain a functional coastal ecosystem under present and future conditions. Such buffer zone widths depend on factors such as the importance and functional area requirements of the ecological zones and social aspects. Provisions for ecological issues should be informed by an environmental assessment based on the South African National Biodiversity Institute (SANBI) biophysical sensitivity layers. Critical Biodiversity Areas can be identified from biodiversity maps, which include information on proposed management of the area. Such assessments should be conducted by (or in consultation with) biodiversity specialists or ecologists specializing in the coastal domain. The goal is a win-win (or "no regrets") approach which serves both man and nature in enhancing both sustainable coastal infrastructure and serving environmental needs, which ultimately enables sustainable coastal development and tourism and long-term social benefits.

Social issues include consideration of and making provisions for: Legal and zoning aspects, Public access, Aesthetic features, Heritage, and the many other human coastal usages such as recreation, sport, fishing, mariculture, boating, etc. Appropriate provincial authorities (and e.g. heritage bodies, etc.), can provide reports and maps indicating issues and features related to the above that are located within the coastal zone. These social components include some amorphous aspects and typically require wider consultation to resolve the issues. Thorough assessment of available information (e.g., reports, maps, etc.) together with such consultation should in most instances suffice. Ideally, or in special cases a site inspection accompanied by a relevant specialist is recommended in order to practically determine the buffer area that may be required for such aspects.

Before a coastal infrastructure project goes ahead, all the geophysical and biophysical components, as well as the socio-economic aspects have to be considered holistically. Ideally the infrastructure project should make provisions to allow space for the required buffer zones.

Related data required to assess potential conflicting beach usages include:

- Spatial Development Frameworks (from municipalities);
- Biodiversity maps provided by nature conservation bodies (e.g., Cape Nature) and SANBI;
- Cadastral boundaries, coastal protection zones, municipal/town planning zones, military/other special use areas, special management areas, etc.; and
- Reports and maps indicating heritage sites.
- Charts provided by the South African Navy Hydrographic office (SANHO) indicating port areas & jurisdiction, anchorage areas, navigation channels, etc.
- Google Earth imagery with all "layers" (points of interest and other features) "switched on". This is an easy way of obtaining a dearth of relevant information including potential conflicts within the coastal zone.

Websites that can provide useful information include:

http://mapservice.environment.gov.za/coastal%20viewer/

http://www.sanbi.org

3.3 Location of the site

The primary physical (abiotic) hazards to coastal infrastructure in South Africa from the sea are the following: extreme inshore seawater levels resulting in flooding and inundation of low-lying sites; direct (and indirect) wind and wave impacts; coastal erosion and underscouring of foundations and structures; and combinations of extreme events, such as sea storms during high tides, which will have the greatest impacts, and will increasingly do so as climate change-related factors (e.g. sea-level rise (SLR) set in Theron, 2016). Thus, the most important drivers of risk to South African coastal infrastructure from erosion and coastal flooding, are waves, tides and future sea-level rise. Critical factors which strongly affect the vulnerability of coastal structures are: physical elevation, coastal geology, distance of infrastructure to the sea (e.g., high water mark) and exposure to storm waves. Simply put, the further away from and higher above the sea, and more sheltered and stable the area, the better the site is in terms of structural safety.

The degree of protection of a site from prevailing wave energy is mainly determined by the site location, coastline configuration or shape and orientation and local bathymetry. Exposure to wave action of a site can be assessed based on the description provided in Section 2.2.4. This indicator accounts for the differing vulnerability to incident storm waves due to location (and other wave modification factors), ranging from fully exposed open coast sites to well sheltered locations, for example, within bays or on the leeside of headlands.

Coastal stability can be assessed by analysing historic shoreline changes (including high-water line, vegetation line, dune and vegetation areas and aeolian sediment pathways). Shoreline variations/"stability" and other important site characteristics can be detected from aerial photographs, ortho-photographs and satellite images, some of which can be obtained from the surveyor general, local and provincial authorities. To complement remote sensing techniques, careful use can be made of Google Earth images.

Coastal geography and geologic/geomorphology information can be obtained from geophysical data in geographic information system (GIS) format, online Department of Water and Sanitation (DWS) GIS data in layers, Agricultural Research Council, and some internet data sources.

3.4 Winds

Wind and barometric data is readily available from the South African Weather Service for many areas, including for nearby major airports. In areas near the nine SA commercial ports, coastal wind data may also be available from TNPA (Transnet National Ports Association of South Africa) or the CSIR.

Information on extreme winds in SA (e.g., to determine wind waves, setup or aeolian transport) is provided in Kruger et al (2013), as summarised in Figure 3.1.

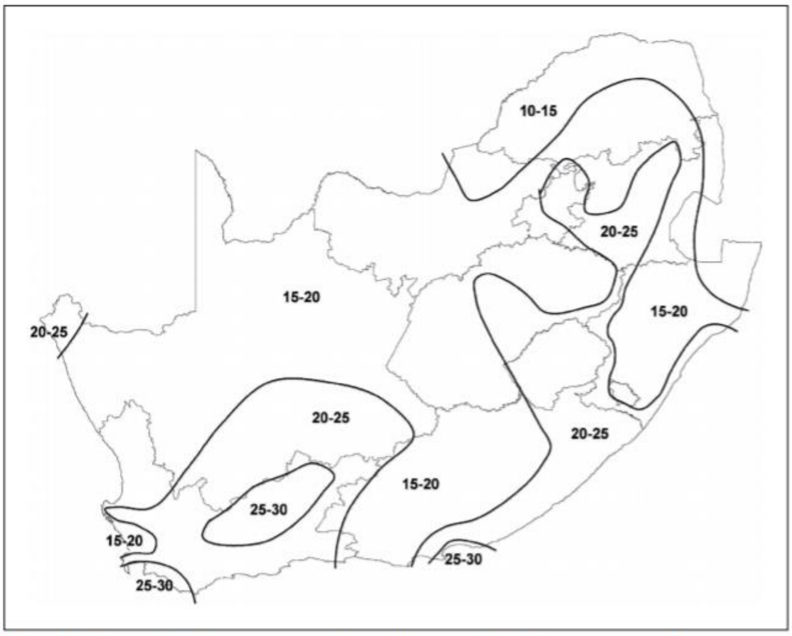


Figure 10 Proposed 1:50 year quantiles of annual maximum hourly mean wind speeds (m/s) (Kruger *et al* 2011)

Figure 3.1: Data on extreme winds is SA for design purposes (Kruger et al, 2013).

3.5 Waves

3.5.1 Accurate design wave conditions.

• Offshore wave climate - present and future.

The present offshore wave climate at deep sea locations around the South African coast can be determined by using NCEP hindcast wave data (NCEP, 2013), from the NOAA/NCEP WAVEWATCH III Global Model (Tolman et al, 2002). Alternatively, in some areas the wave climate may be derived from nearshore recordings off some of the major South African ports, potentially available from TNPA or the CSIR. Currently, an appropriate scenario for future wave climate off the South African coast is a 6 % to 10 % increase in wave height by 2100-2117, with the best estimate being a 6 % increase (Theron, 2016).

• Inshore wave climate - present and future.

To determine the inshore wave climate along the study area, mathematical wave modelling needs to be conducted using a state-of-the-art refraction model. Hydrodynamic wave modelling (SWAN, Booij et al, 1999) is, for example, used to transform the offshore wave data to inshore conditions. Potential climate change effects can be included by simply assuming a wave height increase in accordance with the scenario given above.

3.5.2 Approximate design wave conditions.

The elaborate procedure described above is usually only considered to be appropriate in some detailed investigations of large scale and expensive coastal projects.

Small coastal stormwater outlets will mostly be located landward of the spring low-tide line, while some might be located near the spring high-tide line. All of these locations imply shallow water depths even under extreme conditions resulting in high seawater levels at the outlet (discussed in Section 3.6). Thus, it can normally be assumed that the design wave will be a depth-limited wave, based on determining extreme water-levels, and the calculation of wave characteristics assuming a fixed wave height to water depth ratio. This approximate design procedure is explained in more detail in Section 2.2.3.

3.6 Seawater-levels

3.6.1 General

The coastal flooding elevation (or level) is here defined as the highest point that the seawater can reach at the shoreline, due to the effects of natural events such as tides, winds and storm waves, which may be exacerbated in the long term by processes such as sea-level rise. Therefore, to avoid impacts from coastal flooding, coastal infrastructure, where practical, needs to be located beyond this reach of the sea; thus, at higher elevations located further landward. Determining coastal flooding levels and consequently to what extent an outlet structure may be subject to flooding and the associated impacts, are therefore one of the primary components of designing coastal outlets. Coastal seawater-levels are briefly described in Section 2.3. A definition sketch of the various components leading to extreme inshore seawater levels (identifying the components of tide, barometric/hydrostatic setup, wind setup, wave setup, wave run-up and sea-level rise) is presented in Figure 3.2.

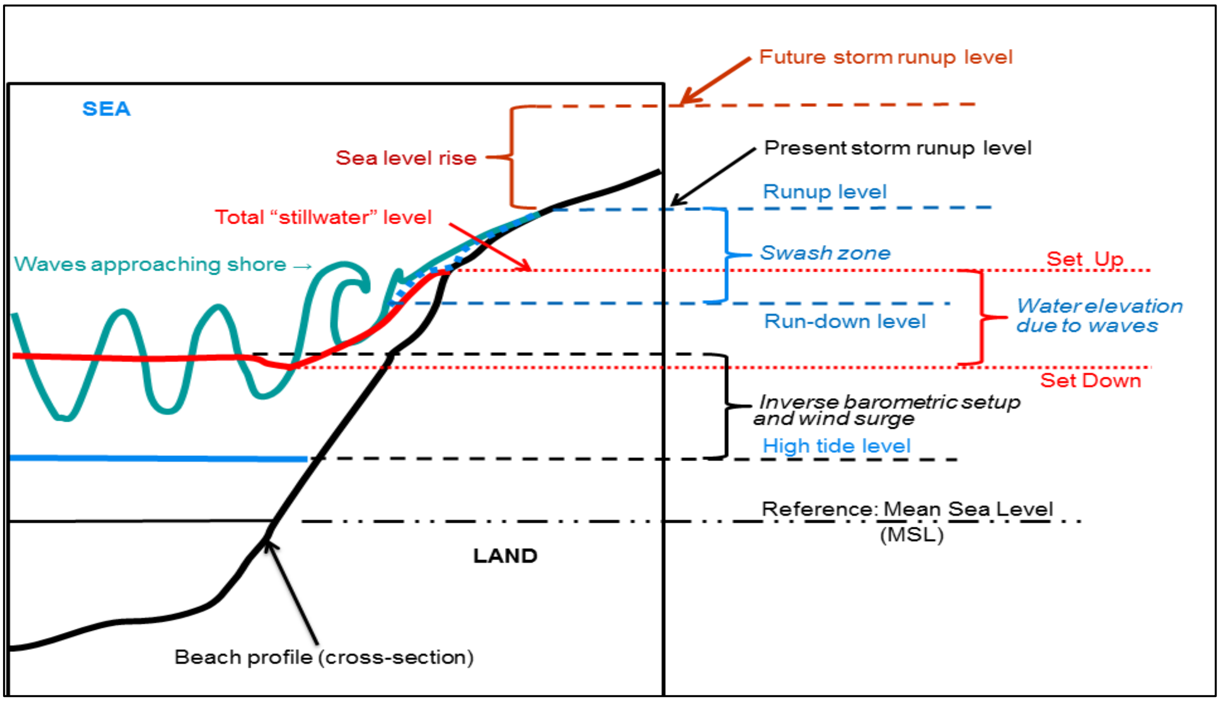


Figure 3.2: Definition sketch of the various components leading to extreme inshore seawater levels (adapted from Theron et al, 2012)

3.6.2 Extreme South African seawater-levels

The tidal elevations for the South African coast are summarised in Table 3.1.

Table 3.1: Summary of tidal levels (m to CD) around the South African coast (SANHO, 2017)

Location	LAT	MLWS	MLWN	MHWN	MHWS	HAT
Port Nolloth	0	0.28	0.78	1.40	1.91	2.25
Saldanha Bay	0	0.24	0.70	1.27	1.75	2.03
Cape Town	0	0.25	0.70	1.26	1.74	2.02
Simon's Town	0	0.24	0.73	1.29	1.79	2.09
Hermanus	0	0.27	0.75	1.29	1.78	2.07
Mossel Bay	0	0.26	0.88	1.46	2.10	2.44
Knysna	0	0.22	0.82	1.32	1.91	2.21
Port Elizabeth	0	0.21	0.79	1.29	1.86	2.12
East London	0	0.23	0.78	1.25	1.82	2.08
Durban	0	0.21	0.87	1.36	2.01	2.30
Richards Bay	0	0.27	0.97	1.48	2.11	2.47

Extreme South African seawater-levels excluding tides (thus, mainly due to wind and hydrostatic setup) have been analysed for all of the South African tidal stations, and the results (that is, residuals for various return periods) are summarised in Table 3.2.

Table 3.2: Extreme residual still-water level estimates (from Theron et al, 2014)

(a) Above CD	Return period in years							
	1	5	10	25	30	40	50	100
Saldanha Bay								
Residual sea level to CD (m)	1.25	1.38	1.40	1.43	1.43	1.44	1.44	1.46
Cape Town								
Residual sea level to CD (m)	1.21	1.34	1.36	1.38	1.38	1.39	1.39	1.40
Simon's Town								
Residual sea level to CD (m)	1.21	1.31	1.34	1.38	1.39	1.4	1.41	1.44
Mossel Bay								
Residual sea level to CD (m)	1.49	1.70	1.76	1.82	1.83	1.85	1.86	1.90
Knysna								
Residual sea level to CD (m)	1.44	1.64	1.68	1.72	1.73	1.74	1.75	1.77
Port Elizabeth								
Residual sea level to CD (m)	1.4	1.63	1.66	1.69	1.7	1.70	1.71	1.73
East London								
Residual sea level to CD (m)	1.29	1.42	1.46	1.51	1.52	1.54	1.55	1.58
Durban								
Residual sea level to CD (m)	1.33	1.51	1.56	1.63	1.64	1.66	1.67	1.72
Richards Bay								
Residual sea level to CD (m)	1.34	1.52	1.55	1.58	1.59	1.59	1.6	1.62

(b) Above mean sea level	Return period in years							
	1	5	10	25	30	40	50	100
Saldanha Bay								
Residual sea level (m)	0.38	0.52	0.54	0.56	0.56	0.57	0.58	0.59
Cape Town								
Residual sea level (m)	0.38	0.51	0.53	0.55	0.56	0.56	0.57	0.58
Simon's Town								
Residual sea level (m)	0.36	0.46	0.5	0.54	0.54	0.56	0.57	0.59
Mossel Bay								
Residual sea level (m)	0.55	0.77	0.83	0.89	0.9	0.92	0.93	0.97
Knysna								
Residual sea level (m)	0.65	0.86	0.9	0.94	0.94	0.95	0.96	0.99
Port Elizabeth								
Residual sea level (m)	0.56	0.8	0.83	0.86	0.86	0.87	0.88	0.89
East London								
Residual sea level (m)	0.57	0.71	0.75	0.8	0.81	0.82	0.83	0.86
Durban								
Residual sea level (m)	0.41	0.59	0.65	0.71	0.73	0.75	0.76	0.80
Richards Bay								
Residual sea level (m)	0.32	0.51	0.53	0.56	0.57	0.58	0.58	0.60

The recommended scenarios for SLR to be included in the seawater levels, for 2100-2117 are: 0.5 m (low), 1 m (medium), and worst-case of 2 m (high). The best estimate ("medium") scenarios for 2030 and 2050 are 0.15 m and 0.35 m, respectively (Theron, 2016).

Estimates are provided here of extreme values for realistic combinations of all the inshore seawater level components, as applicable to each South African coastal region. The South African storm surge levels thus calculated for each coastal area (i.e. combined mean high-water spring (MHWS) + wind, wave and atmospheric setup) for 1-in-10-year wave height and residuals (as an example) are indicated in Table 3.3. It should be noted that the estimates provided in Table 3.3 are based on relatively extreme events (1-in-10 year return period) and are applicable to open coast locations. The values are not applicable within estuaries and harbours or sheltered areas (in the lee of headlands or capes, or behind islands), where the wave setup phenomenon is mostly severely reduced.

Table 3.3: Calculation of South African open coast storm surge elevations (combined mean high-water spring (MHWS) + wind, wave and atmospheric setup for 1-in-10-year wave height and residuals; Theron, 2016).

Coastal area	Offshore wave height (1 in 10 year m)	Calculated extreme wave setup (m)	Tide (MHWS m to MSL)	Residual (barometric, etc. 1 in 10 year m)	Total combined inshore seawater level (m above MSL)			
				Year	**2013**	**2030**	**2050**	**2100**
				Sea Level Rise (m)	**0**	**0.15**	**0.35**	**1**
Orange River Mouth to Groen River Mouth								
	8.3	1.3	0.99	0.39	2.7	2.8	3.0	3.7
Groen River Mouth to Cape Columbine								
	10	1.6	0.89	0.72	3.2	3.3	3.5	4.2
Cape Columbine to Cape Agulhas								
	11.1	1.7	0.92	0.52	3.2	3.3	3.5	4.2
Cape Agulhas to Cape St Francis								
	10.7	1.7	1.17	0.86	3.7	3.8	4.0	4.7
Cape St Francis to Bashee Mouth								
	9.3	1.4	1.10	0.79	3.3	3.5	3.7	4.3
Bashee Mouth to Ponta do Uoro (South African border)								
	7.9	1.2	1.10	0.59	2.9	3.1	3.3	3.9

A first-order coarse storm surge level assessment for the South African coastal regions indicating the relative coastal flooding levels of the different South African coastal regions is provided in Theron (2016). A 1-in-50-year return period wave conditions along each coastal region is considered in combination with the other sea level setup effects as described in the foregoing. The results, which are only applicable in the open coast areas, are summarised in Figure 3.3. The storm surge scenario shown in Figure 3.3 is as follows: mean high-water spring (MHWS) + wind, wave and atmospheric setup along open coasts for 1-in-50-year wave height + 1 m SLR by 2100.

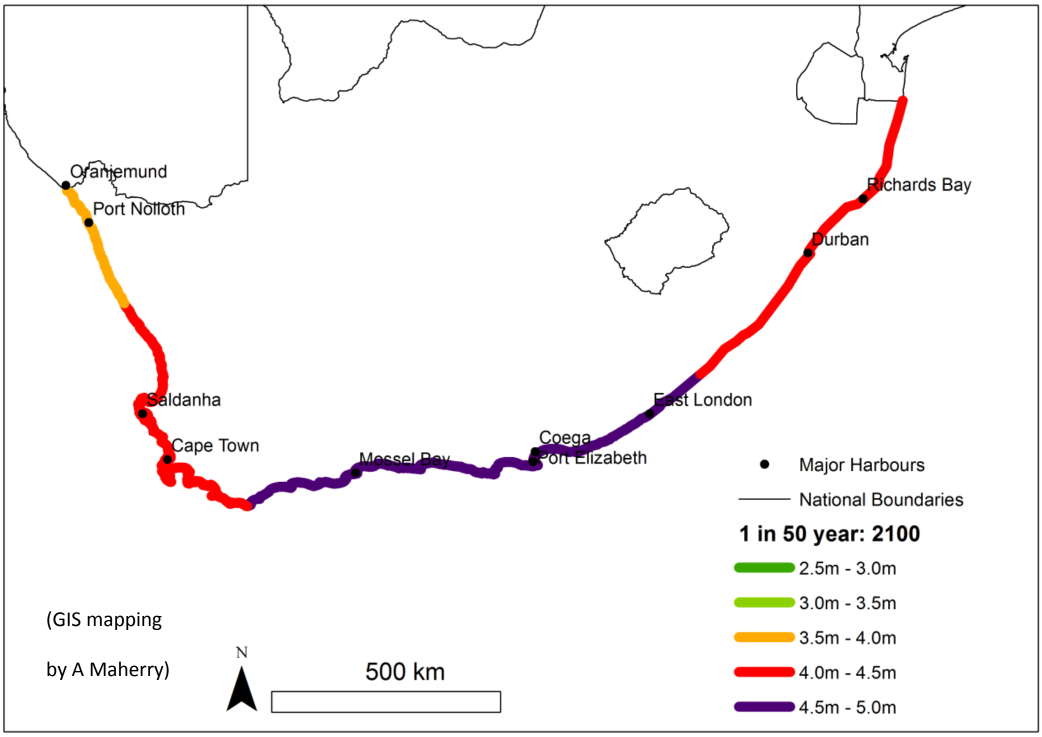


Figure 3.3: South African regional coastal storm surge elevations (i.e. excluding wave run-up) along open coasts for a 1-in-50-year wave return period and 1-m sea level rise scenario at 2100 (Theron, 2016).

Note that the foregoing results are extreme open coastal "storm surge" levels (not applicable in bays) and do not include wave run-up effects, as discussed in the following section.

3.6.3 Wave run-up levels

The wave run-up models of Nielsen and Hanslow (1991) and Mather et al (2011) *for beach areas* are the best of the available models and are adequate for application in South Africa. These models should, however, be used with certain adaptations as recommended in Theron (2016). Overall, the Nielsen and Hanslow model is the most suitable; the best results will be obtained with significant wave heights determined at about 20 m depth or less and then "reverse shoaled" to give the equivalent deep-water wave heights as input. (Reverse shoaling means that the wave height in the nearshore zone directly opposite the site is taken and the equivalent deep sea wave height is calculated by assuming: (1) parallel contours and Snell's Law; and (2) a wave incidence angle of 0 degrees (perpendicular wave attack); refer to CEM, 2006). The other inputs required for computing the wave run-up are the wave period, the beach slope, and the still water-level.

Where only deep-water heights are known, or where data is lacking on the beach slope, the Mather et al model can be applied for beach areas. The inputs required are the deep-water wave height, the distance to the 15 m contour and the still water-level. (The value of coefficient C should be set at 7.5

in open coast locations and even in semi-exposed locations. In well sheltered locations, the value of coefficient C should provisionally be set at 5.) Ideally, both methods can be used to compare and resolve or verify the results.

According to surveyed elevations (Smith et al, 2010), maximum run-up elevations on the open KwaZulu-Natal (KZN) coast near Durban during the March 2007 storm (which coincided with highest astronomical tide) reached up to about 10.5 m above MSL. The 2007 KZN storm was approximately a 1-in-10-year to 1-in-35-year event (Phelp et al, 2009). Maximum run-up elevations on the Cape Town coast during the 1 September 2008 storm (which was approximately a 1-in-10-year wave height event) reached up to about 7.9 m above MSL.

Wave run-up heights on rocky shorelines can be determined approximately by means of the method in the EurOtop Manual (EurOtop, 2007). Run-up on rock slopes of breakwaters and revetments is discussed in more detail in Section 4.2.3. Note that the run-up on natural rocky shores is not necessarily the same as on a uniform rock slope built by man. Clearly, the shape, slope, topography and the roughness affect the run-up on natural rock.

The results from the wave run-up models already include all the necessary sea-level and run-up components, thus directly yield the coastal flooding elevations.

3.7 Sediment

Sediment characteristics, especially grain size data and typology (mud/clay/silt/sand/gravel/cobble fractions) are required for the area where a coastal outlet is to be constructed. Fine sediment is, for examples, more susceptible to aeolian transport (that is, wind-blown sand problems) and erosion/scouring than coarser material.

Sediment data is not readily available for most of the SA coastline, but data for some beach areas are contained in Van der Merwe (2017). Such data may also be available from some coastal municipalities (e.g. Ethekwini) or the CSIR.

If suitable sediment data is not available, sediment grab samples of the study area should be collected. Samples should be collected from the neap low-tide line, the mid-tide line (MSL), the high tide line and one at about 100 m from the high tide line (or at the landward border of the beach where infrastructure (e.g. road/ parking areas) are closer than the 100 m). The samples must then be analysed for particle size distribution (by means of a sieve analysis or settling tube). This data will provide valuable information on the characteristics of the coastal sediments.

3.8 Topography

Topographic data

Sources of such information (determined in topographic surveys, aerial photogrammetry and LiDAR), include local authorities, provincial governments and surveying companies (land or air). LiDAR surveys provide detailed coastal topography of large areas, and the possibility to assess shoreline position and change (e.g. Stockdon et al, 2002). Previously, the use and availability of LiDAR data was a “privilege” usually not available in South Africa, but is now becoming much more prevalent locally (about 50% of the SA coast has been scanned). Minimum data standards and specifications for coastal LiDAR data, suitable for setback lines are provided in Lück-Vogel et al (2014).

If suitable topographic data is not available, topographic surveys of the shoreline should be undertaken. These surveys spatially should extend from the back-shore (including the frontal dune, if present) as deep as possible into the surf zone, i.e. from approximately 120 m inland of the high-water mark to -2 m MSL. Vertical accuracy should be better than 10 cm (ideally ~ 1 cm). Horizontal accuracy should be better than 1 m. The elevations must be related to an official datum level (MSL).

Dune slopes

In assessing dune stability for dunes of relatively low height (< 10 m from base to crest), it is usually sufficient to simply consider the angle of repose as the maximum potential stable slope angle. However, a conservative acceptable slope angle may be as mild as 1:6 under stable conditions and even as gentle 1:10 under unstable conditions (Theron, 2016). For dunes of more than 10 m in height, it is advisable to involve geotechnical engineers and to conduct a slip circle analysis for stability, especially if the slope is steeper than 1:6.

Beach slopes

Data on beach slopes is not readily available for most of the SA coastline, but data for some beach areas are contained in Van der Merwe (2017). Such data may also be available from some coastal municipalities (e.g. Ethekwini, Cape Town) or the CSIR. However, once suitable topographic data has been obtained for a site, it is a simple matter to determine beach slopes from the topography.

3.9 Bathymetry

Bathymetry data

To determine the water depth seaward of the project site, bathymetry data is required (including nearshore and ideally inshore data). Sources of such information/data include bathymetric data and charts provided by the South African Navy Hydrographic Office (SANHO), although detailed inshore (< 15 m depth) bathymetry is usually only available for large projects. Near harbours, TNPA could potentially provide more detailed and recent survey data. Web sites that can provide useful information include:

- http://mapservice.environment.gov.za/coastal%20viewer/
- https://webapp.navionics.com/#boating

Nearshore slope

The nearshore slope can be estimated based on the cross-shore distance from the shoreline (along the 0 m MSL contour) to the 15 m depth contour, which can be easily obtained from the SANHO bathymetric maps. A summary of the SA coastal bathymetry and analyses of inshore slopes along about 2/3 of the SA coast can be found in Theron (2016).

3.10 Nature of shoreline and seabed

Coastal features, shoreline types and coastal geomorphologic characteristics

The South African coastline is rugged and exposed, with few natural bays, and consists of long stretches of sandy beaches interspersed by rocky sectors. Theron (2016) sub-divided the SA coastline into five regions, on the basis of their morphological characteristics, general orientation and exposure to waves. A description of the geophysical shoreline characteristics within the 5 regions is given in Theron (2016). Two types of sandy coasts occur most commonly, one being the generally high energy, open shorelines often characterised by steeper slopes and more reflective conditions consisting of medium to coarse sand. The other is characterised by milder slopes and more dissipative conditions, often consisting of fine to medium sands, which is typically found in the more sheltered coastal embayments.

Shoreline types are generally classified as sandy, rocky or muddy (or a combination of these). The coastal zone of South Africa comprises various types of benthic substrate including several sandy, rocky and mixed substrata (Sink et al., 2012). As such, the South African coast contains no muddy shorelines, even inside the most sheltered coastal embayments. The only “coastal” areas where muddy shores are occasionally found, are those located inside some estuaries and lagoons.

Due to the diversity of the coastal characteristics, the sea will have varying effects or impacts on the infrastructure. For example, even if a particular hazard, say wave height (or wave energy) was similar along some coastal areas, the different coastal characteristics e.g. erodibility (i.e. geologic characteristics or simply hard/soft nature, etc.) will affect shoreline stability differently (Figure 3.4).

According to Tinley (1985), about 80 % of South Africa’s more than 3000 km of coastline is ‘soft’ (erodible sand, or a mix of sand and rock) and that significant parts of the sand dune coast along South Africa are eroding.

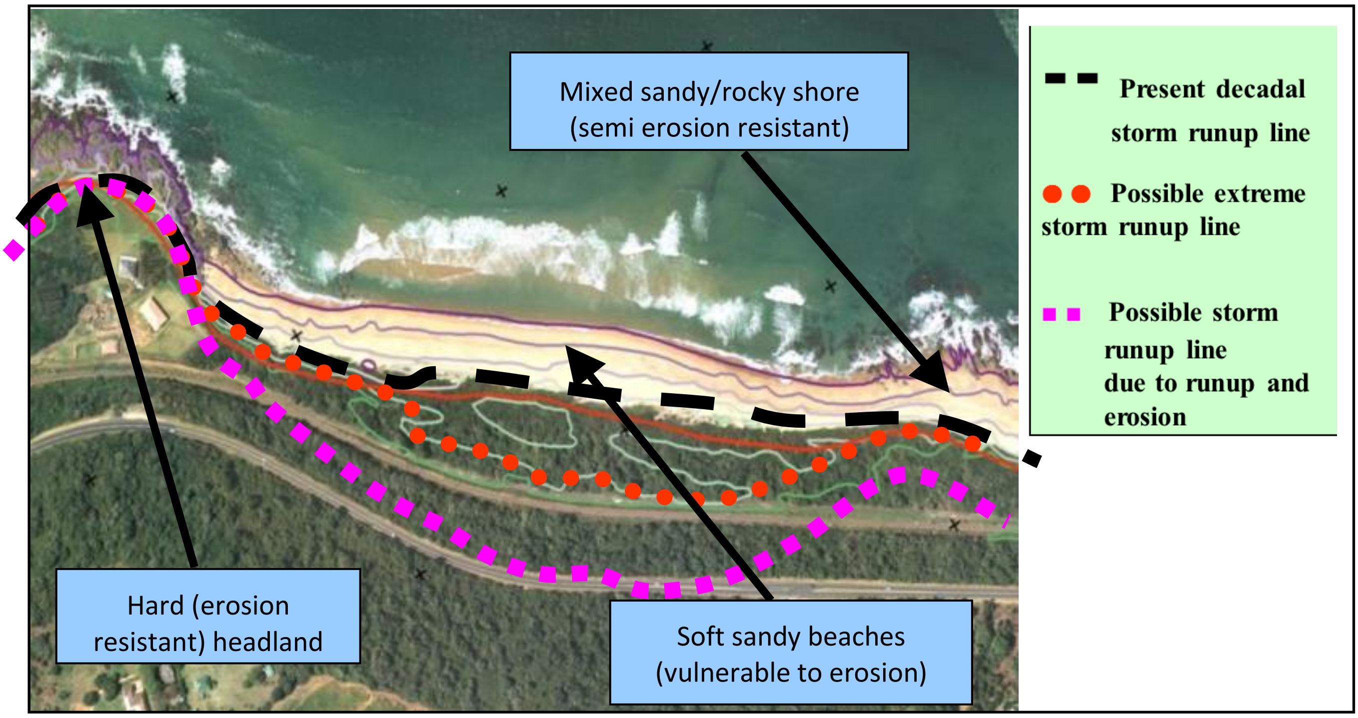


Figure 3.4: Varying coastal characteristics (e.g. erodibility) have varying effects on shoreline stability. (Imagery from Ethekwini Municipality)

Soft rocks (sedimentary) shores and erosive sea cliffs may be subject to significant coastal erosion within typical project design lifetimes. The best practical means of assessing the erosion potential for such shorelines, is to analyse historic recession of these shorelines by means of, for example, aerial photography spanning at least 30 to 50 years, but ideally going as far back as possible (which is typically in the 1930s or 1940s). In special cases or complex situations, the assistance of geotechnical experts or geotechnical engineers is required to provide more detailed assessment and recommendations. A practical means of obtaining a first estimate of the erosion potential of coastal bluffs, cliffs and rocky shores, is to assess the geologic and geomorphologic characteristics of the study area. The pertinent coastal geologic characteristics can be arranged in order of most erosion resistant to most erosion prone, as follows: hard rocks (igneous), "medium" hardness rocks (metamorphic), soft rocks (sedimentary), non-consolidated coarse sediment, non-consolidated fine sediments (Theron, 2016). The coastal geomorphologic characteristics can similarly be arranged in order of least to most erosion prone, as follows: mountains, rocky cliffs, erosive cliffs, sheltered beaches, exposed beaches, tidal (or other) flats (which usually consist of finer sediments, sometimes including some silt or clay fractions); dunes, and river mouths (Theron, 2016). In South Africa, tidal (or other) flats are mainly found within estuaries, due to our generally high energy coast.

Rock levels

Some South African shorelines consist of mainly sand and cemented coastal dune sand (e.g. along significant parts of the KZN coast). Thus, intermittent rocky outcrops (i.e. cemented coastal dune sand) are observed along such shorelines, while several other dune and more sandy beach areas are typically underlain by such rock at various depths below the sand surface level. Due to the partially rocky nature of the shoreline, the potential coastal erosion could in these areas be less severe than for a wholly “soft” (i.e. consisting of sand alone) shoreline. The potential erosion would be reduced, because of the presence of the rocky outcrops which effectively “pin” the shoreline in place, where these outcrops occur. The rocky substrate, seen along some parts of the coast, would thus tend to reduce erosion of the beach in these areas.

Founding conditions

Regarding suitable founding conditions and structural integrity, the general rule concerning the location of coastal outlets, is that the more “stable” the founding conditions are, the more suitable the site is for placement of such structures. Information on rocks levels can sometimes be obtained from geophysical GIS data where it is sufficiently detailed, and in some cases from online data. In-situ probing by means of simply hammering rods into the sand or air/water jet probing, and test pit excavations can be made to assess the founding conditions on a limited scale.

4. GUIDELINES FOR THE DESIGN AND CONSTRUCTION OF COASTAL STORMWATER OUTLETS

4.1 General

The guidelines have been classified into design and construction guidelines (Sections 4.2 and 4.3 respectively). However, the focus is on design guidelines. The most important applicable laws are listed under the engineering design guidelines.

It is important to note that the guidelines exclude: (1) the hydraulic design of the stormwater outlet; and (2) the structural design of the pipes or culverts and the stilling basin of the outlet. Information on these aspects can be obtained from, for example, SANRAL (2013).

It should also be noted that some of the applicable coastal stormwater outfall practices are multifunctional in that they address or alleviate several of the problems discussed in the Phase 1 report (Schoonees & Theron, 2016). Clearly, measures that reduce the flow rate and velocity of the outflow can reduce: (1) beach erosion; (2) scour and structural damage; (3) impacts on beach usage and aesthetics; and (4) reduce pollution problems. Therefore, *dune infiltration systems and infiltration basins* are generally good options (depending on site specific circumstances and limitations). The terrestrial practices that can be applied to reduce flows (including: rain tanks; swales; permeable paving, bioretention ponds and basins, exfiltration trenches, constructed wetlands, underground water storage (Figure 4.1), etc.) have already been discussed. In the coastal domain, *exfiltration trenches* are recommended (for example, Chin (2004), Price (2011); Price et al (2013) and NCSU (no date)). However, the applicability of these measures at a particular site needs to be investigated based on circumstances such as available space, the groundwater table, type of soil, and available hydraulic head.

Apart from the design and construction of small stormwater outlets, maintenance plays a key role in the integrity and functionality of outlets over their lifetimes. Therefore, it is recommended that:

- Regular **inspections** be carried out of stormwater outlets (say, monthly), as well as after floods and storms. Photographs and brief notes should be taken and stored for future use.
- **Maintenance** must be undertaken timeously because it is cost effective to repair in time rather than to wait for failure, necessitating the expensive reconstruction of the outlet(s). By keeping records of the nature of and spending on maintenance, informed decisions can be made on the need for replacement of outlets in future.
- The design should be such as to minimise maintenance of stormwater outlets. However, there should be a **budget** for regular maintenance. The approach to maintenance and the accompanying budget have to be refined over time to optimise the approach for each particular site.

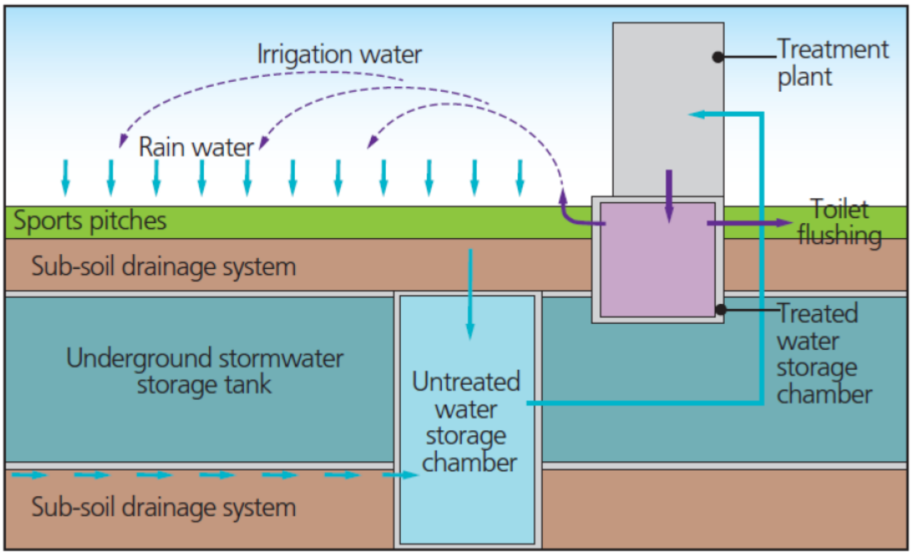


Figure 4.1: Happy Valley underground stormwater storage scheme yielding 220 000 m^3/year, Hong Kong (Luk et al, 2016)

Water-levels, waves and currents are continuously changing in the sea. It is important that the design wave condition (significant wave height; wave period and wave direction) be determined as the most severe condition that will occur during lifetime of the stormwater outlet. This means that a wave breaking directly on the outlet structure will normally be the most extreme condition. As a result, the critical condition is the water-level for which the wave breaking on the outlet structure will occur. This could be a low water-level, depending on the exact location of the outlet structure in the surf zone. On the other hand, higher water-levels make higher wave heights possible. Thus, different combinations of water-level and wave conditions have to be considered.

Because small coastal stormwater outlets will almost always be located in shallow water, it can be expected that the design wave will be a depth-limited wave. In turn, this means that extreme water-levels have to be determined, followed by the calculation of the wave characteristics. (This calculation procedure is explained in more detail in Section 2.2.3.) The designer has to evaluate the joint probability of extreme water-levels and wave conditions. However, one must not be unrealistic; for example: use highest astronomical tide level (which occurs once in 18.6 years) together with the 1 in 100 year wave height. This means a condition that will theoretically happen once in 18.6 x 100= 1 860 years. A range of peak wave periods covering at least 10 s to 18 s, has to be assumed. All feasible wave directions have to be considered.

The end of the pipe or culvert at the stilling basin of the outlet should be covered by a sturdy, stainless steel grid (screen) to prevent people and medium to large animals from entering the conduit. Clearly, the effect of this grid on the hydraulic characteristics of the stormwater system should be checked and be acceptable. If a flap valve is required to limit or prevent water from flowing into the outlet, it is strongly recommended that this flap valve be placed in the first manhole from the sea. In this way, it will reduce the chances of damage of the flap valve because of continuous wave action, vandalism and theft. Furthermore, less maintenance on the flap valve will be required.

4.2 Design guidelines

4.2.1 Beach usages, aesthetics and location

Conflicting beach usages

To reduce potential conflict with other beach usages, the outlet structure should be as small as possible, yet be functional. Public access along the beach must be maintained (an example of hindered access is depicted in Figure 4.2); that is, it should be easy to cross or go around the outlet structure (Schoonees and Theron, 2016). Sharp edges and unnecessarily protruding structural elements (especially metal bars, etc.) should be avoided to reduce possible injuries, especially if the structure is submerged at times. If submerged structures cannot be avoided in recreational beach locations, they must be clearly demarcated even when submerged and warning signs must be put up.

Figure 4.2: Example of an outlet structure hampering public access along a beach (Photo: A Theron)

Conflict must also be avoided as far as possible with other "beneficial coastal zone uses"; that is, direct contact recreational activities (like swimming/bathing), indirect contact recreational activities (e.g., sunbathing), collection of filter feeders (e.g., shellfish), marine protected areas, harbour and industrial facilities, mariculture (e.g., abalone farms), and undeveloped and/or pristine coastal environments.

The discharge from stormwater outlets is sometimes highly polluted and can result in human health or environmental issues if the discharge location is too near any of the before mentioned locations of "beneficial coastal zone uses". Water quality issues are discussed in more detail in Section 4.2.5.

Applying the "working with nature" approach, it would for example, be preferable to discharge into a retention pond and soak-away landward of the backshore area rather than onto the beach or into the sea. In terms of environmental engineering design, it is also recommended to incorporate and hide the outlet into other or multi-functional coastal infrastructure such as jetties, piers, promenades, look-out platforms, seawalls, groynes, revetments, breakwaters, boat ramps, etc. (Schoonees and Theron, 2016).

Aesthetics and beach access

The general approach regarding the aesthetics of an outlet is that it should be an unobtrusive as possible. As a result, the beach will appear to be natural. To achieve this goal, an outlet should blend in with the surrounding area as far as possible (Schoonees and Theron, 2016). For example, rock covering the outlet and especially if placed irregularly, can make an outlet to be hardly visible on a rocky beach. By burying an outlet in the sand of a beach, it will not be seen most of the time. However, functional requirements (for example, prevention of blocking) will probably eliminate this option.

Outlets with regular and angular shapes and having bright colours will draw the attention to outlets. Rounded and irregular shapes will be less obtrusive, especially if the colour(s) of the outlet is similar to the background colours. Note that marine growth will generally cover the outlet below about the high-tide mark and thus, there is no point in colouring or painting an outlet that will be covered by marine growth.

Numerous outlets over a short longshore length of shoreline will make the coastal zone to appear artificial. Where possible, different smaller outlets should be combined into a larger outlet for hydraulic and cost reasons. The same principle applies for aesthetic reasons.

Outlets can be combined with coastal structures such as breakwaters, groynes and piers (Schoonees and Theron, 2016). A coastal structure that has more than one function will not only save costs but will, in total, have a reduced visual impact compared to more than one structure (for example, a groyne and an outfall compared to only a groyne).

If it is very important to reduce the visual impact of an outlet or a group of outlets, a landscape architect can be consulted to make the outlet(s) less obtrusive.

Furthermore, an outlet should, wherever possible, not obstruct people from using the beach (Figure 4.2; Schoonees and Theron, 2016). Means to safely walk across an outlet should be provided where necessary. This crossing point (for example, steps) can be higher up the beach (above extreme wave run-up) so that the railing at the crossing point will not be subjected to wave forces. Note that kelp, seagrass and other material can become entangled on railing so that when waves wash against and over the railing, large forces can be exerted on the railing and the structure of the crossing point. By providing a small, raised platform as a lookout area at the crossing point, the structure of the crossing point can have a dual function.

Location of outlet

Site selection criteria

Some typical site selection criteria for a small coastal outlet are summarised in Table 4.1 below. The particular circumstances and local site characteristics have to be considered in each case. Regarding the placement of an outlet along the shore, great emphasis must be put on locating rocky areas or areas where the beach sand is only a thin veneer overlying shallow bedrock. It is certainly best practice to anchor coastal structures firmly to the bedrock where possible, especially where underlying rock occurs at relatively shallow depth beneath the ground surface. If the bedrock is located deeper down, end bearing piles can be considered to support structures, especially when larger structures are considered.

Table 4.1: Summary of some site selection criteria for a small coastal outlets.

Technical Criteria	Environmental Criteria
• Easy site access for construction purposes.	• Minimum impacts on coastal habitats and terrestrial biodiversity.
• Proximity to suitable connections points on landward storm-water conveyance system (the closer the better).	• Good dispersion, dilution and assimilation of effluent and minimum impacts on marine ecology.
• Favourable geotechnical aspects, especially good founding conditions.	• Minimal impacts on heritage resources.
• Land zoning, ownership, etc. These issues need to be addressed upfront, as they can derail the whole project.	• Avoid close proximity to swimming areas, conservation and protected zones; the further away, the better.
• Low exposure to ocean energy – the more sheltered the site, the better.	• Least possible impacts on landscape, natural scenery and “sense of place”.

Elevation of end of outlet structure

The outlet structure should ideally not be situated at an unnecessarily low contour level (i.e. low elevation relative to sea level). Such low positions may be exposed to wave action especially when sea-storms coincide with high spring-tides. This results in higher wave loads and substantial underscouring of such outlets can also occur.

Relocating outlets from depressions in which they may be located to higher elevations in order to prevent outlet blockage through marine sand inundation is sometimes an option. However, this could necessitate realignment of existing pipelines in such cases, which may sometimes be considered not to be a practical or economically favourable alternative.

A major factor leading to poor drainage performance of outlets is insufficient hydraulic gradient between the outlet and landward part of the stormwater system (which can be exacerbated by low elevation backshore areas and infrastructure located there). This implies placing the outlet at a low enough elevation to ensure that the hydraulic gradient is sufficient (i.e. meets the normal criteria employed in the design of the landward part of the stormwater system). This principle of placing the outlet at a lower elevation is therefore in conflict with the previous two recommendations, which urge for higher elevations. A careful balance of these matters must therefore be achieved to ensure the optimum design elevation. It is equally important to design coastal outlets to withstand wave action and to be operational despite a variation in seawater-levels, wave run-up, sand transport and changes in seabed levels (beach profile variations), as discussed in the rest of this chapter.

Regarding the cost of outlet construction and ease of potential future structural maintenance, outfalls with invert levels above high water spring tide (HWST), would usually be constructed in the dry. This implies normal land construction methods, and much lower costs than outlets located at lower elevations. This is because outfalls discharging below HWST will normally require a cofferdam for construction purposes, while for outfalls with invert levels below LAT (lowest astronomical tide level), a coffer dam and a jetty for deployment of a crane will almost certainly be required, which implies further major cost increases.

Measures for large elevation changes

Storm-water systems draining areas that are located at relatively high elevations above sea level are sometimes so constructed that the discharge falls directly only the beach or dune slope, in extreme cases even falling vertically through several metres. This often results in erosion, undercutting and damage of the outlet structure, even for small pipes (e.g. Figure 4.3).

Figure 4.3: Example of damaged outlet in Tergniet area (MVD Consulting Engineers, 2011).

Further examples of such erosion and underscour from the Mossel Bay – Rheebok area are shown in Figure 4.4.

Figure 4.4: Eroded embankments and foundations due to stormwater outflows (Photos: Mossel Bay - Rheebok, MVD Consulting Engineers, 2011).

Ideally, the water needs to be conveyed to the beach level in a controlled manner to discharge at a lower elevation preventing excessive erosion or underscour at the outlet point. This can be achieved by constructing a vertical chamber (which can be similar to a manhole) with an internal stilling basin at the bottom, or a stepped and inclined structure which drops the water in a controlled way while dissipating the energy (for example, Figure 4.5). It is important to note that the hydraulic and structural design of these structures must follow established codes of practice, such as, for example, covered in SANRAL (2013). (These landward design aspects are not part of the scope of this study.)

Figure 4.5: Example of a coastal outlet with structure (still incomplete) in which the water is brought down to nearer sea level in a controlled manner (Photo: A Theron).

If there is a remaining vertical drop at the end of the outlet that cannot be avoided, then the best option is that the discharge should fall onto natural rocky areas (e.g. Figure 4.6) or a properly designed rock/riprap structure (as described in Section 4.2.3). Alternatively, a small plunge pool (stilling basin) of the required dimensions and with a protected floor can be constructed below the outlet; a simple example of a "mini plunge pool" formed by riprap is shown in Figure 4.7. It is important for such structures to be properly designed with regard to hydraulic aspects, especially including strong and deep foundations designed for the extreme dynamics and scour as discussed in Section 4.2.2. Energy dissipating structures such as a rock apron, rock outlet basins and baffled outlets can also be applied in such situations to mitigate both scour and beach erosion problems, as described further in Sections 4.2.2 and 4.2.3.

Figure 4.6: Example of an outlet with structure which discharges onto rocks (Photo: A Theron).

Plate 13-1: Example of riprap at a stormwater outfall

Figure 4.7: Riprap (rock armour) preventing erosion and scour at outlet (Auckland Regional Council, no date).

4.2.2 Shoreline changes and scour

Horizontal shoreline changes

Long-term shoreline recession trends

Long-term shoreline changes in the vicinity of a planned outlet can be quantified by considering the variation in shoreline location at the site over an extended period. Analysis of vertical aerial photography is the usual means of doing this and such imagery (including, for example, suitable satellite images) is available for virtually the entire South African coast. Analyses of beach topographical surveys should be included if such data is available. The identification of pertinent features, processes and coastline changes (shoreline, vegetation line, etc.) from aerial photography also assists in understanding the study area.

If a significant eroding trend is apparent in the shoreline location, a conservative estimate of the erosion rate is extrapolated for the stipulated or chosen design lifetime, usually 50 or 100 years. This constitutes the horizontal distance that the outlet should ideally be placed landward of current sandy shoreline locations to provide for long-term shoreline erosion trends. Fortunately, there are only a few areas along the SA coast (e.g., Durban Bluff, north of the Port of Richards Bay) where the shoreline is known to be progressively receding. The following example (taken from Theron, 2016) is provided to illustrate how such provision (landward distance) is determined where a long-term eroding trend is found. In this analysis, aerial photographs were used to determine the shoreline location. Thus, the historic shoreline locations relative to a fixed reference line were determined, as shown in Figure 4.8. A significant long-term eroding trend in the shoreline location is apparent and this beach has retreated by about 13 m since 1937. The average regression is about 0.23 m/year, which means, for example, that an additional landward distance of 12 m should be provided for a 50 year planning horizon. (The assumption is thus made that that linear extrapolation of the historic trend calculated over the total record is representative of the expected future situation, i.e. the underlying cause of the shoreline regression will continue to have the same net effect in future).

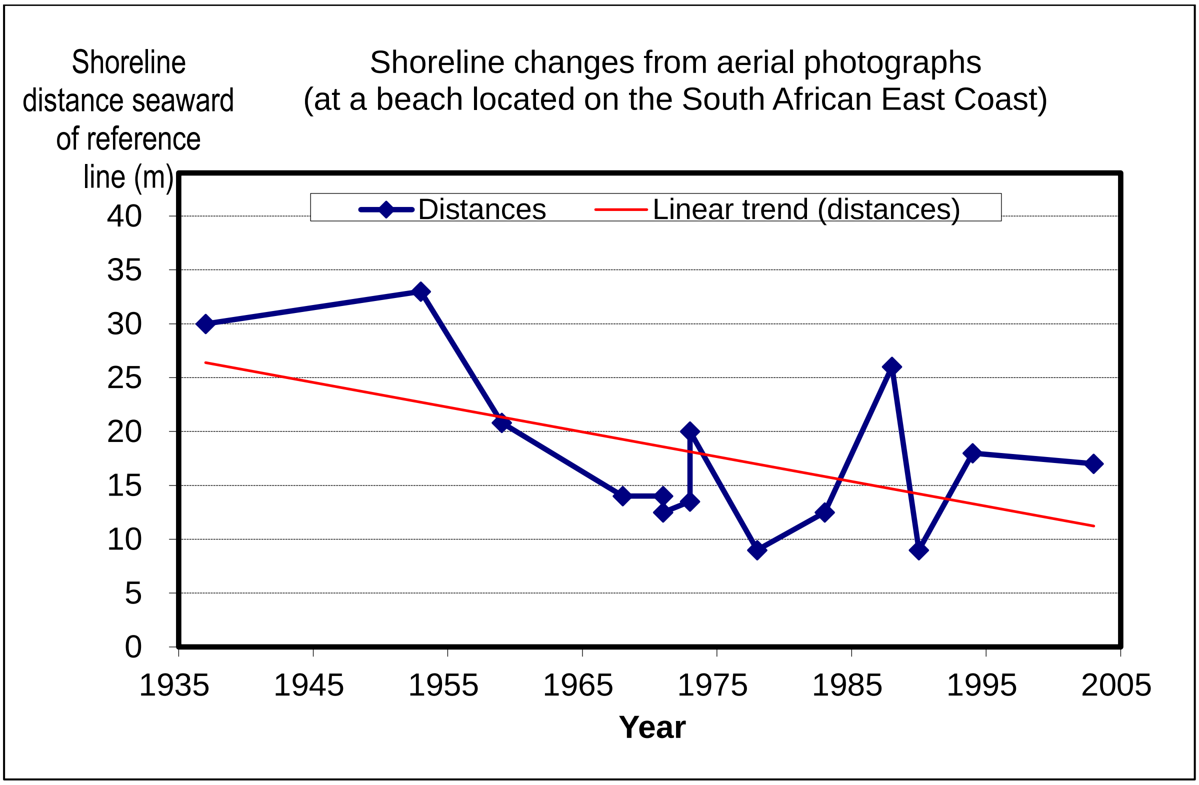


Figure 4.8: Long-term shoreline location changes from aerial photography.

The best practical means of assessing recession for soft rock and erosive cliff shorelines, is to analyse historic recession of these shorelines by means of, for example, aerial photography spanning at least 30 to 50 years. For soft rocks and erosive cliffs or bluffs of more than 10 m in height, it is advisable to involve geotechnical engineers and to conduct a slip circle analysis, especially if the slope is steeper than 1:6.

Short-term (storm) shoreline erosion

If repeated topographic beach survey data is available for the site, this can be analysed statistically to determine the short-term shoreline variability. Where no shoreline change data other than aerial photography is available, an initial estimate of short-term horizontal coastal erosion can also be made based on the imagery. (Aerial photographs, ortho-photographs and satellite images can mostly be obtained from the surveyor general, local and provincial authorities. To complement remote sensing techniques, careful use can be made of Google Earth images.) There should be virtually no area along the South African coast for which at least 10 different images are not available, which is recommended as the minimum number to provide some confidence in the analysis. The topographic beach survey data or imagery data can be used to predict the maximum short-term horizontal shoreline erosion (event) over a selected period (say 50 years). Theron (2016) demonstrated that the short-term shoreline variability of natural beaches in South Africa is mostly normally distributed. This finding can be used to predict the maximum landward movement, based on the normal statistical distribution (as described in Theron, 2016). The predicted shoreline variation (X) can thus be determined by the following equation:

$$\mathbf{X} = \mathbf{Z} \cdot \boldsymbol{\sigma} + \boldsymbol{\mu}$$

Where the probability of **Z** is given by the area under the normal curve,

μ is the mean, in this case of the shoreline variations, and

σ is the standard deviation, in this case of the shoreline variations.

In other words, the predicted shoreline variation (X), is equal to the Z value given by the chosen probability, multiplied by the standard deviation (σ), plus the mean (μ), both of the shoreline variations (Theron, 2016). The Z values can be read off standard statistical tables, such as Table IV published in Walpole and Myers (1978; p513); for example, for a chosen P of 0.95 (i.e. 95%), the Z values is 1.645. Assuming for the purposes of this example (taken from Theron, 2016) that the standard deviation was found to be 10 m, this would mean that the variation (offset from the mean) would not exceed 16.45 m (= 10 m x 1.645) for 95% of the time. This offset from the mean, taken in a *landward* direction, therefore constitutes the predicted erosion. Thus, in this example, the *erosion* would not exceed 16.45 m for 95% of the time. This constitutes the horizontal distance that the outlet should ideally be placed landward of the current shoreline location to provide for short-term (e.g. storm) shoreline erosion (in addition to providing for long-term recession). Typical horizontal shoreline variations in exposed South African beach locations are of the order of 30 m to 80 m (to perhaps 90 m in the most extreme cases, e.g., Figure 4.9 following a major storm), and progressively less than 30 m as the location becomes less exposed (more sheltered from direct wave impact).

Figure 4.9: KZN example of extreme shoreline erosion along an exposed location (Phelp et al, 2009)

Provision for future shoreline erosion due to sea-level rise (SLR).

The Bruun rule (Bruun, 1988) can be applied (with caution) to give a first estimate of possible erosion of 'soft' sandy beaches due to sea-level rise (SLR). The Bruun "rule" for the erosion (R) due to SLR is as follows:

$$R = \frac{S \times L}{H_d + H_b}$$

Where: S is the sea level rise in metres; L is the distance to the profile closure depth; H_d is the profile closure depth; and H_b is the height of the beach berm. Thus, the main parameters that are taken into account in Bruun's rule are schematically illustrated in Figure 4.10.

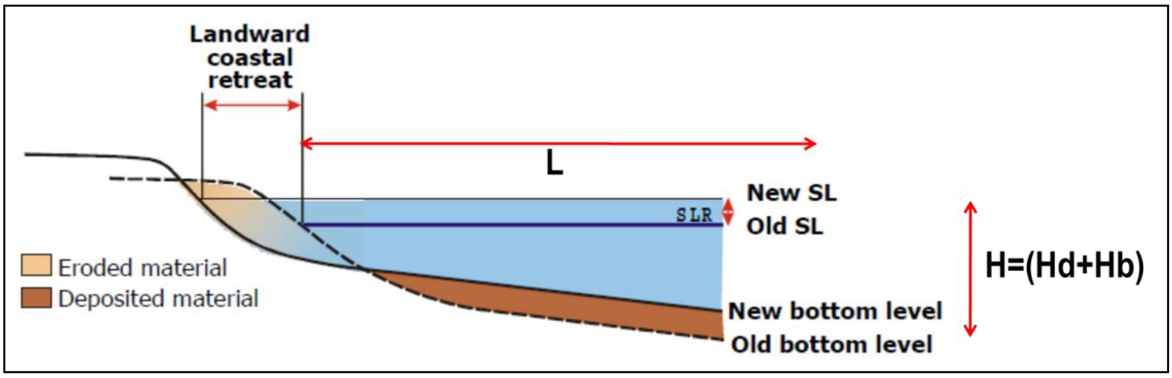


Figure 4.10: Schematic illustration of the Bruun model of profile response to rise in sea level showing erosion of the upper beach and nearshore deposition. (Adapted from Davidson-Arnott, 2003)

Hard, erosion resistant shores will generally show no noticeable erosion in response to sea level rise, but the high-water line will still move landward according to the slope above the present high-water line. This constitutes the horizontal distance that the outlet should ideally be placed landward of the current shoreline location to provide for potential future shoreline erosion (in addition to providing for long-term recession and short-term storm erosion).

Alternatively, beach profile modelling (for example, using SBEACH; Larson and Kraus (1989) and Larson et al. (1990)) can be undertaken to determine the effect of sea-level rise.

Vertical beach profile variations

In incidences where sufficient beach profile data is available, the vertical variations can be assessed directly from the observed profile changes. Typical vertical variations in very exposed South African beach locations (e.g. Figure 4.11) are in the order of 2 m (to perhaps 6 m in the most extreme cases, e.g. when a dune suffers major storm erosion), and progressively less than 2 m as the location becomes less exposed (more sheltered from direct wave impact).

Figure 4.11: SA example of vertical beach profile variation along an exposed location

Hard structures, including stormwater outlets that are located on sandy shorelines and positioned within reach of the sea, are also subject to scouring of the toe of the structure or underscouring of the foundations. Outlets structures (and especially the foundations) must be designed to cope with the maximum expected depth of such scouring in addition to the natural vertical beach profile changes discussed above, as illustrated in the concept sketches in Figures 4.12 and 4.13.

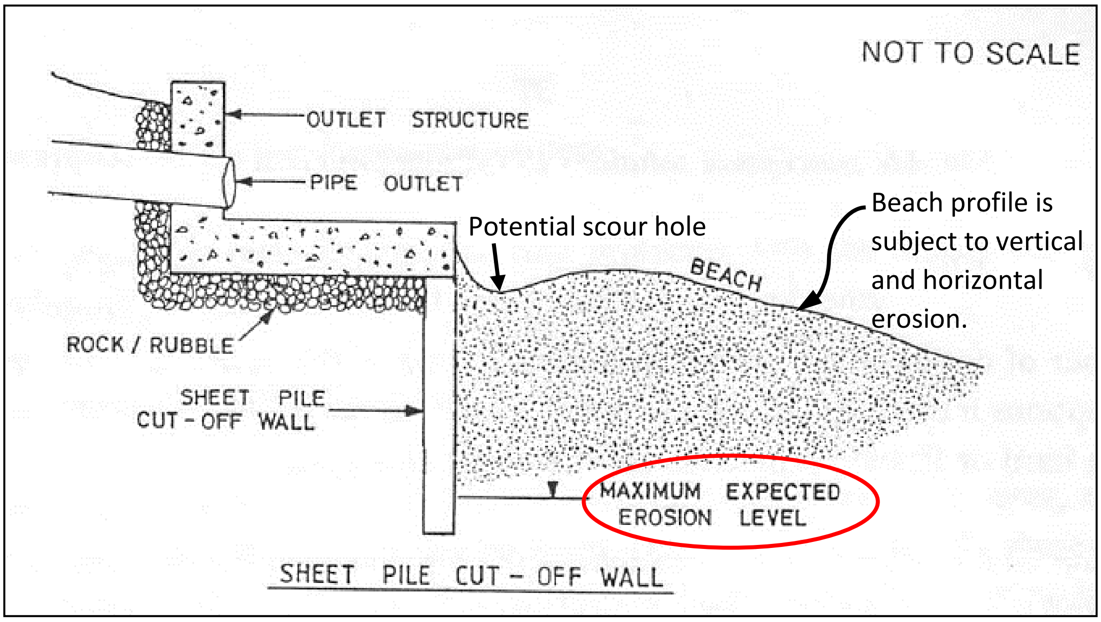


Figure 4.12: Conceptual stormwater outlet - sheet pile cut-off wall design (adapted from Lochner, Theron and Schoonees, 1993).

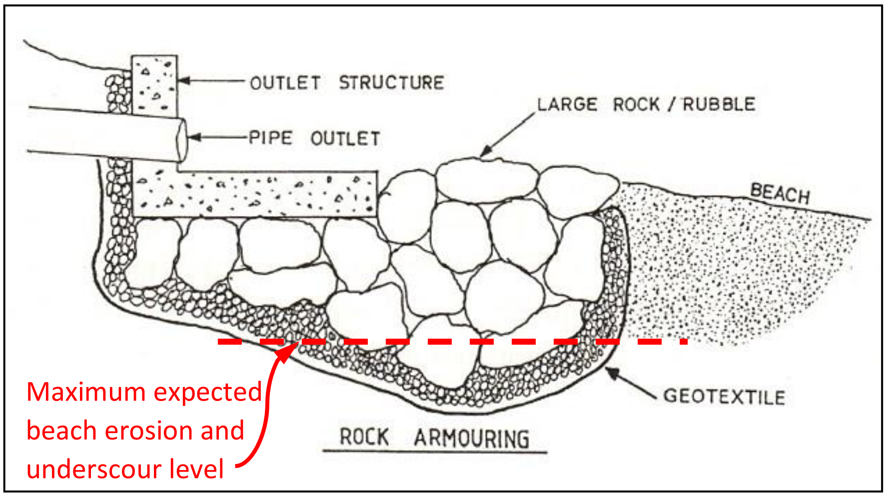


Figure 4.13: Conceptual stormwater outlet - armour rock design (adapted from Lochner, Theron and Schoonees, 1993).

In conjunction, the structure must also be designed for structural integrity, thus being able to span in between adjacent supports without relying on any support from the beach sand between spans. Figure 4.14 illustrates an example of an outlet structure that remained intact despite a significant drop in the beach level, owing to the supporting structure being founded deep enough to be stable under eroded conditions, as well as the ability of the structure to span in between piles without support.

Figure 4.14: Examples of outlet structures that remained intact despite a significant drop in the beach level (photo left: R Cox; photo right: R Kasserchun).

Design guidelines regarding coastal erosion and scour of coastal outlets are provided in the following section.

Scour

Stormwater outlet structures situated at low elevations relative to sea level may be exposed to wave action especially when sea-storms coincide with high spring-tides, resulting in substantial under-scouring of such outlets, for example, Figure 4.15.

Figure 4.15: Underscouring of an outlet located within reach of storm wave run-up.

A number of options exist to combat underscouring of stormwater outlets due to wave action. These options include locating (or in some instances relocating existing) outlets higher up on the beach away from the sea, or using rigid or flexible protection for the outlet structure as described in the following paragraphs (also refer to Section 4.2.3 for more detail):

Relocation could necessitate realignment of the pipeline to a flatter slope, which may be considered not to be a practical or economical solution in some instances. An outlet position higher up on the beach would also be more susceptible to aeolian sand inundation. The outlet structure should in any case be placed to reduce or eliminate beach erosion; for example, by placing the outlet on a nearby rocky area or at a headland.

Suitable *rigid protection measures* include a sheet pile cut-off wall (e.g. Figure 4.12) or an inclined (reinforced) concrete slab below and in front of the floor of the outlet. The concrete slab should be founded on rock fill or piles. Both the cut-off wall and inclined slab should go deeper than the maximum expected erosion level of the sand below the outlet structure, as illustrated in the concept sketch in Figure 4.12. Rigid protection measures are often difficult to implement successfully over the long-term in a dynamic beach and dune environment as they tend to increase beach erosion, thereby necessitating a deeper cut-off wall or concrete slab. They are also relatively expensive and are thus generally considered to be a less attractive option. Problems as a result of settlement or scour are less likely with a sheet pile cut-off wall than with an inclined concrete slab. Concrete block-type structures which rely on interlocking units for structural integrity are generally also not suitable under more severe direct wave attack or in highly dynamic coastal environments.

Flexible protection measures include rock armouring, gabions, Reno mattresses and flexible concrete structures such as, for example, Armorflex mats. While flexible concrete and wire basket / mattress structures are usually less expensive than rigid concrete structures, problems are often encountered with the longevity of such systems. Therefore the best practical and economical solution to the scouring problem is to use rock armouring (e.g. Figures 4.7, 4.13 and 4.16). Rock armouring is relatively cheap and easy to construct but is aesthetically less pleasing. In most cases, the required size and volume of rock are very important considerations.

Figure 4.16: Rock armour preventing erosion and scour from both wave action and stormwater runoff. (Photo: A Theron, False Bay)

Gabions and Reno mattresses will require significant maintenance and even if well protected against rust, they will eventually have to be replaced. If gabions and Reno mattresses are used they must be underlain by a filter layer and a suitable geotextile, and must also be solidly filled, preferably with rounded stone. Gabions have already been used extensively at outlets in the South African coastal zone, with mixed outcomes, often including failures (e.g., Figure 4.17).

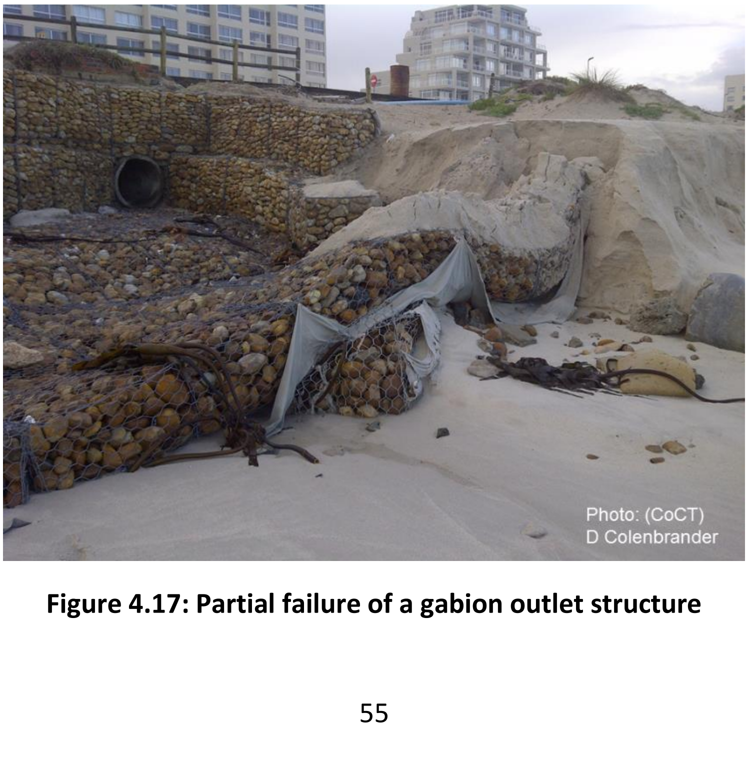


Figure 4.17: Partial failure of a gabion outlet structure

Properly designed rigid and flexible protection measures as described above would also be suitable to prevent the undercutting of outlets by stormwater runoff. Additional suitable remedial measures include making use of a small *plunge pool or stilling basin, or a flood retention pond with a soakaway* if space permits. It is important for coastal outlets to be properly designed with regard to hydraulic aspects. Energy dissipating structures such as a *rock apron, rock outlet basins* and *baffled outlets* (for example, Figures 4.18 – 4.20) can be applied to mitigate both scour (undercutting of the structure) and beach erosion problems. *It is equally important to design coastal outlets to withstand direct wave action (where outlets are located within direct reach of the sea), and to be operational despite a variation in seawater-levels, wave run-up, sand transport and changes in seabed levels (beach profile variations).*

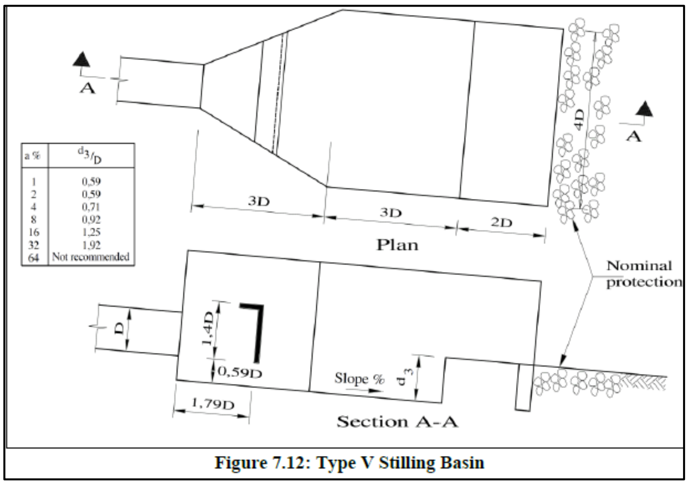


Figure 4.18: Sketch of concrete outlet structure (SANRAL, 2013)

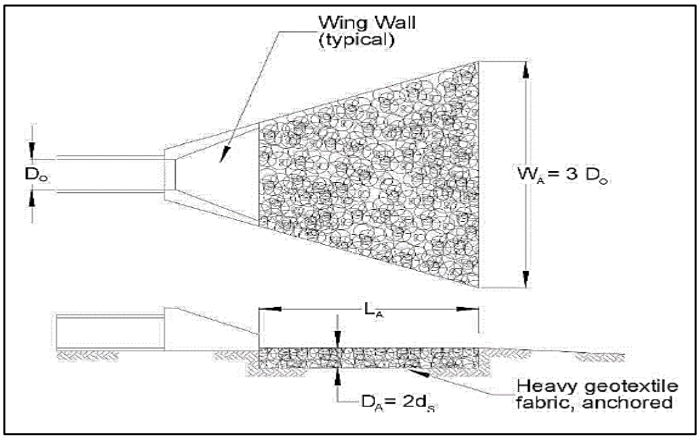


Figure 4.19: Sketch of a riprap (rock) outlet (from Hawke's Bay, 2009)

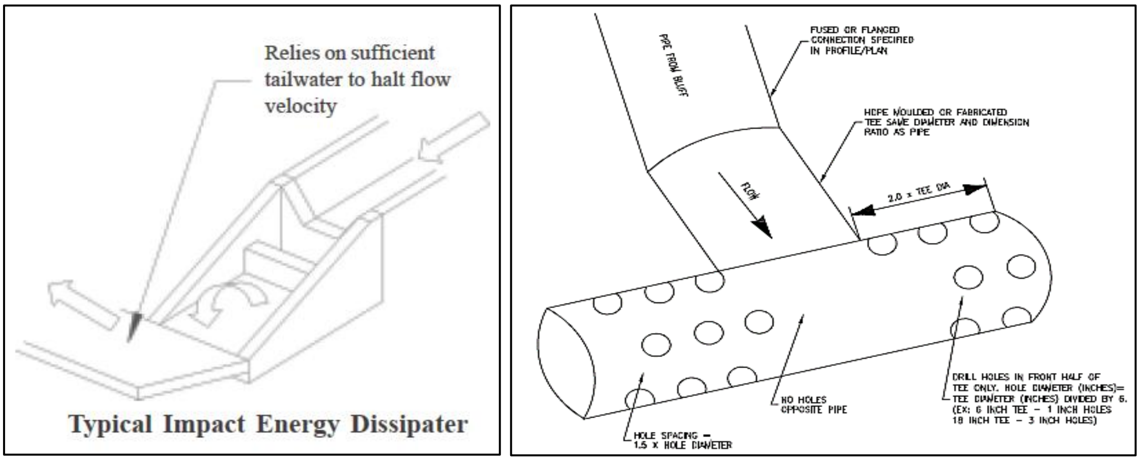


Figure 4.20: Sketch of energy dissipating outlet structures (Left: Auckland Regional Council, no date; Right: Portland Stormwater Management Manual, 2014)

Undercutting of an outlet can cause a plunge pool to be formed under the front (seaward end) of the outlet. This pool acts as a stilling basin which normally prevents erosion beyond a certain depth. If the structure remains intact and functional under these conditions, one option is not to take any further action. Alternatively, a plunge pool of the required dimensions and with a protected floor can be constructed below the outlet. Another alternative is to make use of a retention pond at the outlet with a soakaway seaward of the outlet. The pond should be large enough to catch the main bulk of a flood while the water soaks away into the sand, thus preventing any scouring of the outlet. Problems that may be encountered with a pond are water damming up in the pipeline and the relatively large space that is required. A disadvantage of both plunge pools and retention ponds is that they may result in polluted or stagnant pools of water forming, with associated health and aesthetic problems. If the volume of stormwater runoff is not too large for the area that is available, and the sand is well drained, a retention pond and soakaway is an attractive option, especially if this is incorporated into the design of the buffer dune.

To reduce scour around the sides of an outlet due to surface runoff from parking areas and roads, it is recommended that the surface runoff be prevented from accumulating at a single point behind the outlet. Instead, the surface runoff could be allowed to soak away over a larger area. However, this could lead to flooding during heavy rainstorms if the water cannot drain away fast enough. It is recommended that the surface runoff be channelled into shallow ditches between the parking area and the back of low hummock dunes (where such occur). Where these ditches do not naturally exist they should be constructed. Problems may occur if these ditches fill with sand (either washed in or blown in by wind).

A second alternative would be to collect the surface runoff in a drain at the parking area level and then channel the water onto the floor of the stormwater outlet through a pipe or a well-constructed open channel. This alternative is less favourable than the first, as it is more costly and increases the chances of scouring below the outlet.

In any event, the outlet should have adequately dimensioned and well-constructed back and side walls and a well-founded floor. Good quality materials should also be used as the structures are located in the aggressive coastal environment (low grade concrete should be avoided). These measures will protect the outlet and prevent damage to the concrete itself. In some instances protection of pipes (encasing) to increase structural strength and durability may be an option, but only if properly designed, including adequate foundation support and scour protection where required. Where possible, *nearby smaller stormwater pipes may be grouped together* at the backshore (or seawall) into one bigger outfall pipe by means of a more robust structure. However, this will also result in flow concentration, possibly higher velocities and potentially increased scour and erosion, which must be incorporated into the design.

Finally, pedestrian traffic over and around the outlet should be discouraged as this contributes to erosion by surface water runoff. Fencing can be used to prevent the slopes around the outlets from being disturbed by people, except if the outlet is already inside a fenced off dune area. A grid has to be placed in front of the opening of *large* stormwater outlets to trap litter and to prevent children and larger animals from entering the pipes.

Further specific design aspects to cater for coastal erosion and scour of outlet structures are included in Section 4.2.3. Best management practices can reduce the amount and velocity of the outflow and thereby reduce beach erosion, scour and structural damage. (Schoonees and Theron (2016) and Section 4.1).

4.2.3 Protection of the outlet

General

Different permanent methods exist with which to protect the outlet structure (pipes or culverts and the stilling basin of the outlet). These are mostly the same methods that are applied for general protection of coastal areas, but for protection of small outlets, the scale (or lateral extent) would just be more limited. The relevant protection methods include the following:

- Concrete structures
- Rock protection
- Sand bags
- Mattresses (grout and block)
- Gabions and Reno mattresses
- Other methods

Concrete structures

Concrete of a high quality and strength and low permeability should be used for marine works. Corrosion of reinforcing steel can be a problem in the aggressive coastal zone. Maximum corrosion normally occurs in the splash zone between the low water and high water marks. Cover for the reinforcement must, as far as possible, prevent sea water from infiltrating the concrete and reaching the reinforcement. Crack control for the concrete is thus very important. Sometimes, the concrete structures are painted with a bitumen mixture to reduce the chance of sea water reaching the reinforcement.

It is advantageous to build a concrete outlet in a location protected against wave and current action. Furthermore, concrete structures (including outlets) generally need a firm, stable foundation. To address both these issues (protection and a foundation), it makes good sense to use the nature of the seabed, the bathymetry and the local topography to one's advantage in design. On a rocky or a mixed rocky and sandy coast, the pipeline or culvert can be placed in a gully and either fixed to the rock with dowels or concreted over to cover the pipeline or culvert. By locating the stormwater outlet structure (stilling basin) in the lee of rock, wave forces and cost can be reduced significantly.

It is recommended to anchor coastal / marine structures firmly to the bedrock where possible, especially where underlying rock occurs above the water or at relatively shallow depths beneath the ground surface. If the bedrock is located deeper down, end bearing piles can be considered to support structures, especially when larger structures are considered. Adequate information of rock levels above and under the sand, gravel or cobbles are required. Furthermore, weathering of the bedrock needs to be assessed in order to ascertain the feasibility of using the bedrock as foundation. If the bedrock is deeper down than the expected extreme scour level, then a layer of stone (a screed layer) can be placed on top of the bedrock in order to obtain a level surface. For shallower bedrock, concrete placed over dowels (anchored to the seabed) should be used.

If it is not feasible to use the bedrock as foundation, then other protection measures such as by placing rock protection should be considered. This protection has to withstand the scour action of the local waves and currents.

If the outlet structure will be subject to significant wave and current forces, then the design should allow for these forces. Refer to CEM (2006) and CIRIA, CUR, CETMEF (2007) for more information on wave forces against coastal structures.

Rock protection

General

Usually, a rock revetment (rock slope or rubble-mound revetment) would be the cheapest option to protect the shoreline (Figure 4.21). The same applies for rock placed around an outfall to protect the stilling basin of a small to medium outlet against wave and current action (Figure 4.22). As a result, other options should be compared with a rock revetment in terms of functionality, lifetime and cost.

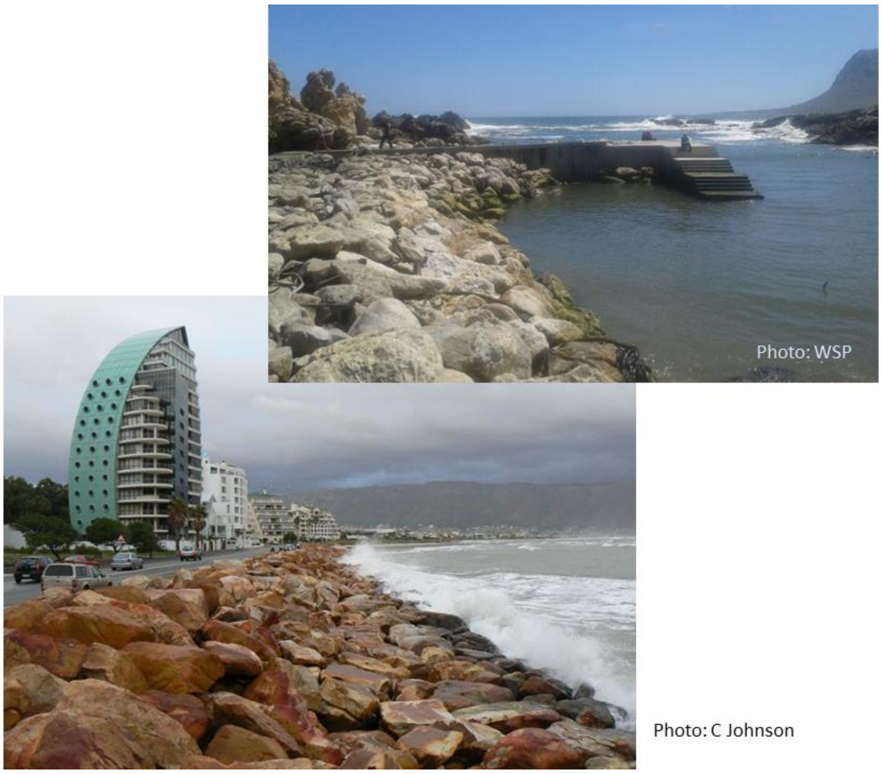


Figure 4.21: Typical rock revetments for general shore protection

Figure 4.22: Rock revetment protecting an outfall

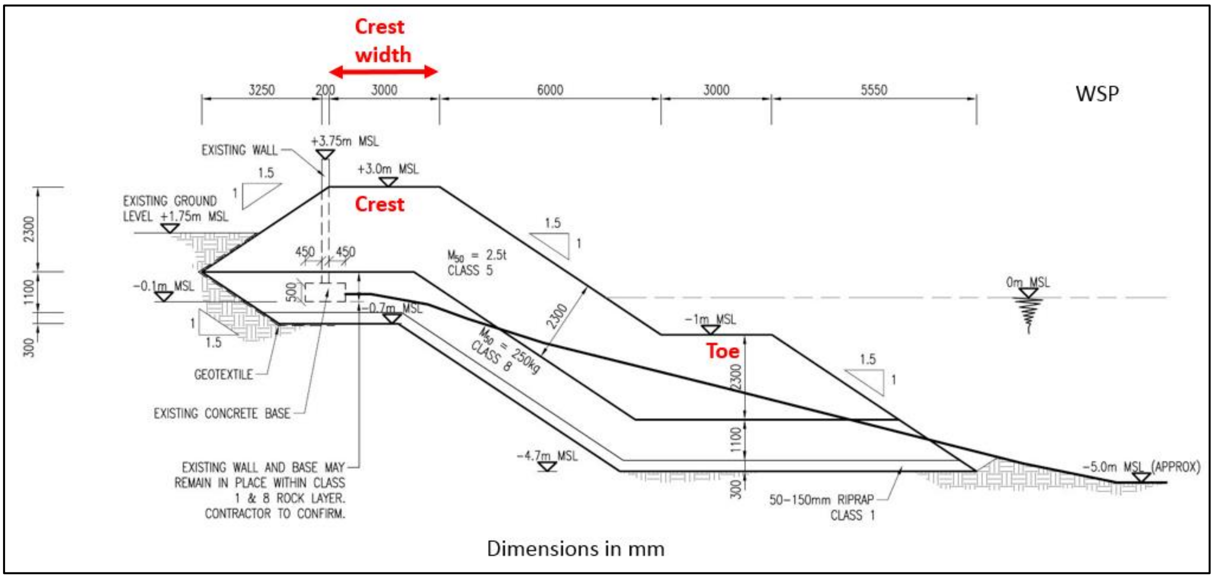


Figure 4.23: Cross-section of a typical revetment (WSP)

Figure 4.23 shows a typical cross-section of a rock revetment. Note the crest and toe of the structure (the outfall is not shown).

In this section on rock protection, it is assumed that the design wave and water-level condition(s) are known at the toe of the rock revetment. This design condition can be determined based on assuming depth-limited conditions and/or carrying out modelling of wave transformation. Apart from the design condition(s), basic knowledge of rock properties is required. Thereafter, the design approach will be briefly discussed, followed by a description of wave run-up and a design method to calculate the required rock size that will serve as the armour layer. Design of the toe of the revetment, characteristics of geotextiles and general comments conclude this section.

This section is only a brief summary of rock design in shallow water; more information can be obtained from The Rock Manual (CIRIA, CUR, CETMEF, 2007) and from the Coastal Engineering Manual (CEM, 2006).

Rock properties and definitions

Graded rock that adheres to a given size distribution is used in rock revetments. This rock is characterised by the median mass (M_{50}), which is the mass exceeded by 50 % of the rock in the distribution. The nominal rock size (D_{n50}), is the dimension of a cube that has the same mass as the median rock. The relationship is as follows:

$D_{n50}= (M_{50}/ \rho_{rock})^{1/3}$ ...(4.1)

Where ρ_{rock}= density of rock (approximately 2 650 kg/m^3 depending on the type of rock).

The minimum thickness of gravel, stone or rock layers is 300 mm. The thickness of rock layers can be calculated with the following equations, namely:

$t_a = t_u = t_f = n\ k_t\ D_{n50}$ ...(4.2)

Where $t_a = t_u = t_f$ = thickness of the armour, underlayer and filter layers respectively.

n= number of layers of rock (minimum 2; that is, a double layer)

k_t = layer thickness coefficient

n_v= volumetric porosity of the rock layer

Table 4.2 gives the values of these variables (k_t and n_v) for rock.

Table 4.2: Values of k_t and n_v (CEM, 2006)

Type of rock and number of layers	Layer thickness coefficient k_t (-)	Volumetric porosity n_v (-)
Smooth rock (n =2)	1.02	0.38
Rough rock (n = 2)	1.00	0.37
Rough rock (n ≥ 3)	1.00	0.40

For standard narrow gradings (on which the design methods are based), M_{85}/M_{15}= 2.5. (M_{85}= the mass of a given rock size so that 85 % of the rocks in the grading will have a mass less than M_{85}. Also: M_{15}= the mass of a given rock size so that 15 % of the rocks will have a mass less than M_{15}.) If one assumes a symmetric distribution, it means that:

M_{85}/M_{50}= 1.25 ...(4.3)

and that

M_{50}/M_{15}= 1.25 ...(4.4)

If rock have be used that do not adhere to a narrow grading (not recommended), it means that special physical model tests are essential to substantiate a design.

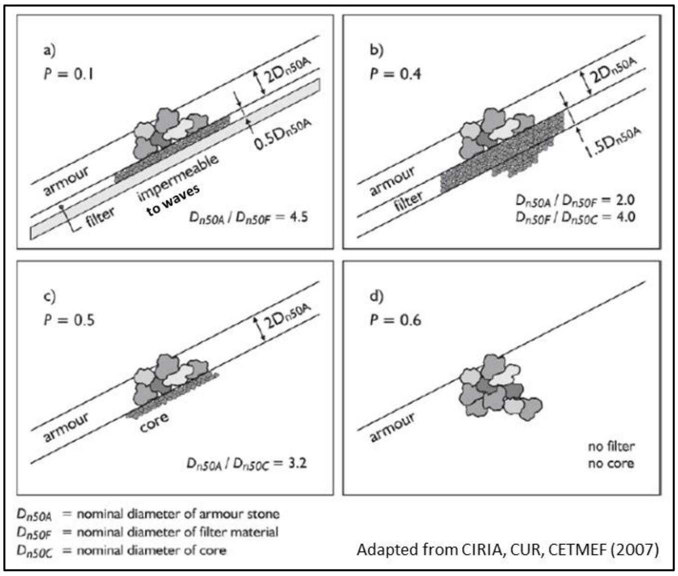


Figure 4.24: Wave permeability (notional permeability factor)

Rock structures have a wave permeability (P; or notional permeability factor) depending on the cross-section of the revetment (Figure 4.24; CIRIA, CUR, CETMEF, 2007). Note that P must not to be confused with the permeability to water for flow of water through porous material. For almost all revetments, P= 0.1 (Figure 4.24) because revetments have to contain soil or are built against concrete structures.

Rock revetments should have a crest (Figure 4.23). The minimum crest width (Figure 4.23) of a rock structure is 3 to 4 times D_{n50}. Typical rock slopes (tan α) for revetments and breakwaters range from 1: 1.5 (1 vertical distance: 1.5 horizontal distance; =1/1.5= 0.667) to 1: 3. Even though slopes of 1: 1.333 are used for certain modern armour units, slopes steeper than 1: 1.5 should not be used for revetments using rock as armour. The flatter the slope of a rock revetment, the more stable the rock will be because the dissipation of wave energy is distributed over a larger area. Smaller rock can therefore be used. However, considerably more rock on a flatter slope will be required.

The relationships between rock sizes in adjacent layers are as follows (CIRIA, CUR, CETMEF, 2007):

$M_{50,u}= M_{50,a}/10$ to $M_{50,a}/15$ …(4.5)

$M_{50,f}= M_{50,u}/10$ to $M_{50,u}/15$ …(4.6)

$M_{50,c}= M_{50,f}/10$ to $M_{50,f}/15$ …(4.7)

If these relationships are applied, the revetment should automatically adhere to geotechnical filter rules. It is recommended to use the factor 10 (and not 15) in the abovementioned equations because it is somewhat conservative. A further advantage is that, if the rock placed during construction deviates slightly from the specified rock, the rock should still be within acceptable limits.

The allowable damage to a rock slope has to be specified in design. Damage to rock slopes has been defined in various ways. For example, the percentage of rocks displaced from the crest to a depth equal to H_s (significant wave height) below the still water-level (CEM, 2006). Table 4.3 lists the different categories of damage.

Table 4.3: Categories of damage to a rock slope (CEM, 2006)

Category	Percentage damage (%)
Initial damage (Start of damage)	0 – 5
Intermediate damage	5 – 10
Failure (exposure of the underlayer)	> 20

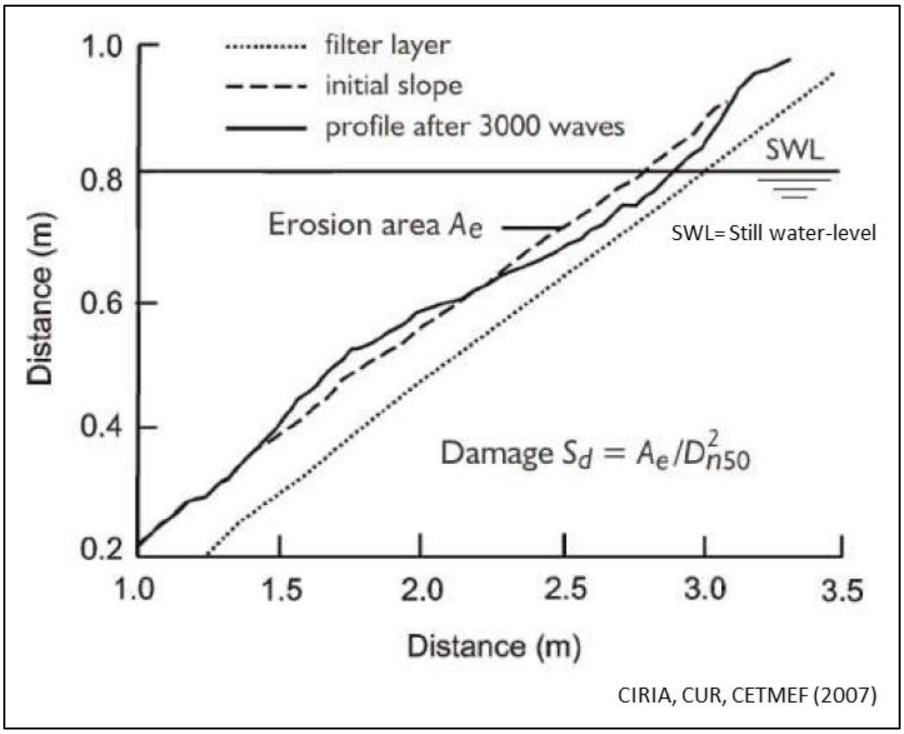


Figure 4.25: Definition of the damage factor (S_d) based on the erosion of a rock profile

An alternative method to define the damage to a rock slope is by means of the eroded area (in cross-section) on the slope, namely the damage factor (S_d). Refer to Figure 4.25 for the definition. According to CIRIA, CUR, CETMEF (2007), values of the damage factor S_d are presented in Table 4.4. Clearly, design of a rock slope should be for "start of damage"; that is, for S_d= 2 (Table 4.4).

Table 4.4: Design values of the damage factor (S_d) for different categories of damage to a double layer rock slope (CIRIA, CUR, CETMEF, 2007)

Rock slope	Damage level		
	Start of damage	Intermediate damage	Failure
1: 1.5	2	3 – 5	8
1: 2	2	4 – 6	8
1: 3	2	6 – 9	12

Design approach

A rock revetment can be designed to be either statically or dynamically stable. Statically stable means that no or virtually no movement of the rock is allowed. For dynamically stable structures, movement of the rock by waves is acceptable, thus letting the waves reshape the rock slope until equilibrium is reached. For small coastal outlets, it is recommended to design rock slopes to be statically stable because moving rock can act as projectiles and break other infrastructure, as well as being scattered over a beach and making it more dangerous and less functional.

Failure modes for a rock revetment include the following:

- Movement of the armour rock.
- Individually rocks breaking because of poor rock quality.
- Water overtopping the crest of the revetment resulting in scour landwards of the revetment.
- Rock from the toe of the revetment is moved away from the toe (seawards, up the slope or alongshore).
- The foundation fails geotechnically.
- Excessive scour in front of the toe.
- Rock or stone from the underlayer, filter and/or core is lost through the armour layer.
- The core slumps; for example, because it contained fines that have been washed out.
- The mass capping (concrete crown wall on top), if present, is moved by wave action.

The design approach for a rock revetment to protect a small coastal outlet can be summarised as follows:

- Decide on the usage of area in the lee of the revetment. For small coastal outlets, it should be unacceptable to allow significant wave action in the lee of the structure and overtopping.
- Choose rock as armour.
- Choose the revetment slope; usually 1: 1.5.
- Calculate the wave run-up and/or tolerable overtopping rate. EurOtop (2007) can be used for choosing tolerable overtopping rates and for computing the overtopping rate; however, for small coastal outlets, very little overtopping should be allowed so as not to damage infrastructure.
- Determine the crest height based on the wave run-up and/or the allowable overtopping.
- Choose the wave permeability (P).
- Choose the allowable damage (Use S_d= 2 for “start of damage”; Table 4.4)
- Compute the size of the armour rock.
- Calculate the thickness of the armour layer based on the size of the armour rock.
- Determine the size and thickness of:
  - Underlayer rock
  - Filter rock
  - Core rock (if applicable).
- Design the toe of the revetment.
- Decide on whether a mass capping (crown wall) is required on top of the revetment. Access to the outlet by crane or truck for maintenance and repair is the criterion in deciding whether a crown wall is necessary. A crown wall is expensive and normally it should not be necessary.

- Construction methods. End tipping is usually carried out to deliver rock for a revetment (refer to Section 4.3.2).
- The bearing capacity of the seabed soil must be sufficient to support the revetment. It may be necessary to excavate soil to reach a better foundation and/or use gravel as fill replacing the excavated soil.
- Rock and other construction material must be locally available. For rock, it is necessary to consider sizes and whether sufficient quantities can be obtained.

Wave run-up

It is customary (industry standard) to use the 2 % wave run-up limit ($R_{u\ 2\%}$); that is, only 2 % of the waves will run up the rock slope to a higher elevation than the chosen value. Equations for computing the wave run-up is presented here for two cases, namely: (1) impermeable rock slope with P=0.1 (Figure 4.24); and (2) permeable rock slopes for P=0.5 (Figure 4.24). For both these scenarios, the following characteristics are assumed:

- It is a rough and uniform slope. There are no berms in the slope.
- Irregular, long-crested waves attack the slope at right angles, resulting in the maximum run-up (that is, the wave direction is at 90 degrees to the revetment). The wave spectrum is a JONSWAP spectrum (CEM, 2006; a good assumption for South Africa).

The run-up for *impermeable* rock slopes is (CEM (2006):

$R_{u\ 2\%} / H_{is} = A\ \xi_{0m}$ for $1.0 < \xi_{0m} \leq 1.5$

$= B\ (\xi_{0m})^C$ for $\xi_{0m} > 1.5$

For *permeable* rock slopes, the run-up is (CEM, 2006):

$R_{u\ 2\%} / H_{is} = A\ \xi_{0m}$ for $1.0 < \xi_{0m} \leq 1.5$

$= B\ (\xi_{0m})^C$ for $1.5 < \xi_{0m} \leq (D/B)^{1/C}$

$= D$ for $(D/B)^{1/C} \leq \xi_{0m} < 7.5$

H_{is}= incident significant wave height at the toe (immediately seawards) of the revetment.

The values of the coefficients for 2 % wave run-up are as follows:

A= 0.96; B= 1.17; C= 0.46; D= 1.97

The surf similarity parameter (ξ_{0m}) can be computed using the following equation:

$\xi_{0m} = \tan\alpha / (2\pi\ H_{is} / gT_m^2)^{0.5}$

T_m= mean wave period= 0.83 T_p (approximately) for a JONSWAP spectrum (CEM, 2006).

T_p= peak wave period (available from wave recordings in South Africa)

Design method for armour rock size in shallow water

The following equation by Van Gent (CIRIA, CUR, CETMEF, 2007) can be used to calculate the required size of the armour rock on a straight section of a breakwater or revetment in shallow water:

Stability number for the armour rock=

$H_{is}/(\Delta\ D_{n50}) = 1.75\ (1/\tan\alpha)^{0.5}\ (1 + D_{n50,core}/D_{n50})^{0.667}\ (\ (S_d/\ (N^{0.5}\ )\ )^{0.2}$ ...(4.8)

Where H_{is}= incident significant wave height at the toe (immediately seawards) of the revetment

D_{n50}= nominal size of the armour rock

Δ_{rock}= relative density= $(\rho_{rock} - \rho)/\ \rho$

ρ_{rock}= density of the armour rock= approximately 2 650 kg/m^3

ρ= 1 025 kg/m^3 for sea water.

$\tan\alpha$= slope of the *revetment*

S_d= damage factor

N= number waves

$D_{n50,core}/D_{n50}$= ratio between the D_{n50} of the core rock versus the D_{n50} of the armour rock. For a revetment having only an armour layer and a core (two layers) $D_{n50,core}/D_{n50}$= 0.464 by using the relationships between rock sizes in adjacent layers with a factor of 10 (Equations (4.3) to (4.5)). For a three layer system (armour, underlayer and core), then $D_{n50,core}/D_{n50}$= 0.215 (again using a factor of 10 for different rock layers; Equations (4.3) to (4.5)). If soil is protected or a geotextile is applied (typically, the case for a revetment), then $D_{n50,core}/D_{n50}$= 0.

Note that the **Van Gent formula is only valid for shallow water; that is, if $h/H_{is} < 3$** (h= water depth just seawards of the toe of the revetment). This should be the case for small coastal outlets. If the application is outside of this range, then CIRIA, CUR, CETMEF (2007) should be consulted for alternative design formulae. It also needs to be checked that the wave height used is possible; that is, that $H_{is} \leq 0.78$ h (the depth-limited scenario).

The number of waves (N) is related to the duration of the design storm. Unless a detailed storm analysis has been carried out, the maximum number of waves should be assumed, namely N= 7500.

From the stability number of the armour rock (Equation (4.8)), D_{n50} of the armour rock can be computed, enabling the calculation of the median mass M_{50} of the required armour rock (Equation (4.1)). M_{85} and M_{15} can be calculated from Equations (4.3) and (4.4). The thickness of the armour rock layer is then computed from Equation (4.2). This is followed by the calculation of the sizes of the rock in the underlayer, filter layer and core (Equations (4.3) to (4.5)) and the respected layer thicknesses from Equation (4.2).

Toe design

The toe is a very important part of the rock revetment. Usually, rock toes are designed to be flexible; that is, limited rock movement is allowed whereby rock falls into the start of a scour hole but without this rock movement affecting the armour rock layer. The rock that has fell into the scour hole, prevents or limits further scour. If excessive erosion of the toe occurs, the armour rock will slide downward, damaging the structure, which could result in failure.

Figure 4.26 shows the cross-section of a rock toe that can been built on a rocky seabed (CIRIA, CUR, CETMEF, 2007). The toe is anchored in an excavated trench. Clearly, a natural trench or the roughness of the seabed can alternatively be used to hold the toe. Another possibility is to construct a toe beam that is anchored to the seabed by means of piles (CIRIA, CUR, CETMEF, 2007). This toe beam then prevents the toe rock from sliding seawards.

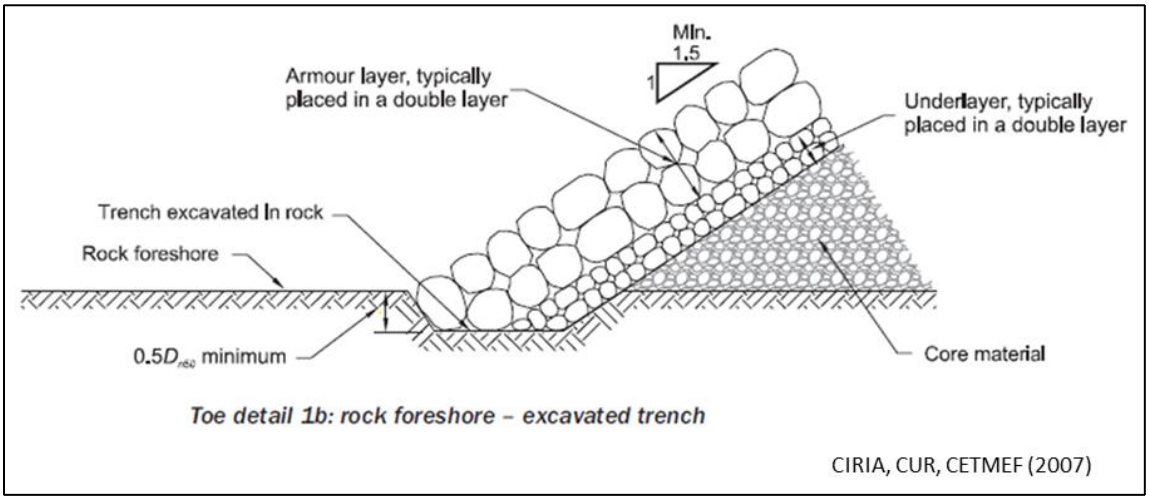


Figure 4.26: Possible toe configuration for a rocky seabed

CIRIA, CUR, CETMEF (2007) proposes a special toe for sandy seabeds and a severe scour potential Figure 4.27). In this particular case, the toe is built on the existing sand, which is easier than excavating a trench first. Typically, sandy South African shorelines have a severe scour potential because of the strong wave action.

Refer to CIRIA, CUR, CETMEF (2007) and CEM (2006) for more detail about toe configurations for different scenarios.

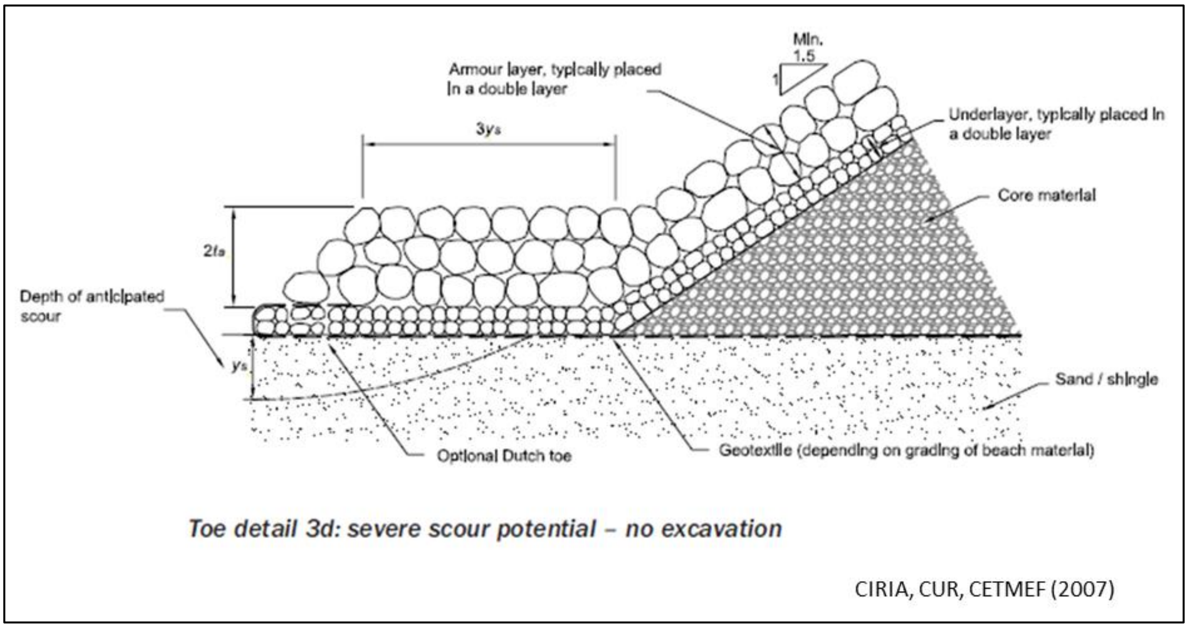


Figure 4.27: Possible toe configuration for a sandy seabed with a large scour potential

Geotextiles

Usually two types of geotextiles (filter fabrics) are applied in the marine environment, namely: (1) woven geotextiles; and (2) non-woven (or needle-punched) geotextiles. Knitted geotextiles are generally not used in marine structures (Ingold and Miller, 1988). Normally, woven geotextiles are stronger than needle-punched geotextiles but cannot stretch (strain) as much before tearing. Depending on the application, either strength or stretching may be the most important characteristic. Woven and non-woven are sometimes even used together as a composite geotextile (Rankilor, 1994).

PIANC (2011) lists a number of functions of geotextile. However, the filtration function is the most important for revetments; that is, to keep the soil in the embankment whilst allowing the water to freely seep out. The water permeability has to be maintained during the life of the revetment. Rankilor (1994) specifies for a geotextile to be sand tight, the following criterion has to be adhered to, namely:

$O_{90,\ geotextile}$ / $D_{90,\ soil}$ < 2.5 for a *woven* geotextile

and

$O_{90,\ geotextile}$ / $D_{90,\ soil}$ < 5 for a *non-woven* (needle-punched) geotextile

Where

$O_{90,\ geotextile}$ = the pore size of the geotextile so that 90 % of the pores are smaller than this size.

$D_{90,\ soil}$ = grain size of the soil so that 90 % of the grains are smaller than this size.

With regard to placement of a geotextile, it is important to prepare the soil properly by removing loose rocks, sticks, sharp objects and debris so that the ground is even and that the geotextile can lie flat. Note that the dumping of large rocks and stones (larger than, say, 50 kg) directly onto a geotextile can damage it. For larger rocks and stone, a filter layer (s) is (are) required on top of the geotextile to protect it against falling stones and rocks. Usually, the geotextile is rolled down the slope to place it. It may be necessary to initially place a stone or a small heap of gravel on each corner under the water to ensure that the geotextile stays in position until the correct filter layer can be placed.

PIANC (2011) gives information about different seams (joints) that can be used. It is recommended that seams be used to connect different strips of geotextiles. Overlapping without stitching is not recommended because, invariably, soil will be pumped out by wave and current action (Rankilor, 1994). A hand-held sowing machine can be applied on site to do the stitching.

A geotextile in marine applications has to withstand harsh conditions and remain stable under abrasion, puncturing by objects, chemical attack, ultra-violet light from the sun, etc. It is recommended that specifications about different characteristics and suitability for the application be obtained from manufacturers. Characteristics like the mass of geotextile per unit area, pore sizes, water permeability, puncture resistance, tensile strength, stretching (strain) during tensile loads, etc. (Ingold and Miller, 1988) should be provided by the manufacturers.

General comments on rock design

The planshape (or layout) of the revetment has to be considered carefully to assess possible vulnerable sections of the revetment and to evaluate the 3-dimensional effects of the revetment (including the effect on the adjacent shoreline). Vulnerable sections of the revetment are bends in and corners of the revetment and transition areas. Transition areas are locations at which the armour rock size changes from large rock to small rock or intersections of a concrete structure(s) with the revetment or with each other. Smooth transition areas are required. It may be necessary to increase the rock size at bends and corners, or, at least, use the larger rock in the grading from the loads received from the quarry at the bends and corners.

Sand bags

Geotubes (sand sausages; Longard tubes), sand envelopes and sand bags (or geotextile sand-filled containers) have been used for erosion control (Theron et al, 1994). Of these measures, sand bags are the best suited to protect small and medium coastal stormwater outlets, mainly, because of the relatively small scale projects, ease of construction and the versatility of the use of sand bags.

Large sand bags filled with sand, grout or concrete have been applied all over the world to combat coastal erosion. International examples include Mexico (Porraz, 1976), Australia (Jackson et al, 2005), the United States of America, the Netherlands and Germany (Saathoff et al, 2007). In South Africa, groynes built from large sand bags, together with sand nourishment were constructed in Langebaan (McClarty et al, 2006). Widespread coastal erosion occurred in March 2007 along the KwaZulu-Natal coastline. Sand bag revetments were widely used to protect the shoreline (Smith et al, 2007; Mather, 2007; Phelp et al, 2009). Figure 4.28 shows the application of the EnviroRock sand bags manufactured by Kaytech for general shore protection at

Langebaan and in KwaZulu-Natal. According to Kaytech, their EnviroRock^{TM} 3PL Geocontainers with a volume of 2.5 m^3 (Figure 4.29), when filled, are 3 m long by 1.5 m wide and weighing 4.2 tonnes with a fill density of 1700 kg/m^3. McClarty et al (2006) quote that similar bags have a mass of 3.5 t to 4 t (when filled) with a thickness (height when placed) of approximately 0.5 m.

Figures 4.28 to 4.31 depict stormwater outfalls supported by large sand bags.

Figure 4.28: Use of the EnviroRock sand bags in South Africa (Kaytech) for general shore protection: (a) sand bag revetment; and (b) a groyne

The aim of the use of sand bags in this section is to provide medium to long-term protection against wave and current action. For short-term usage, refer to Section 4.3.2. As a result, the sand bags (including its material and sown joints) have to be sturdy. In the rest of this section, it is assumed that the sand bags will be filled with sand and not grout or concrete. It is easier, require less equipment and it is considerably cheaper to use sand as fill.

Photos: A Theron

Figure 4.29: Use of the sand bags to support a small outfall along the KwaZulu-Natal coastline

Photos: D Phelp

Figure 4.30: A small outfall supported by sand bags (KwaZulu-Natal coastline)

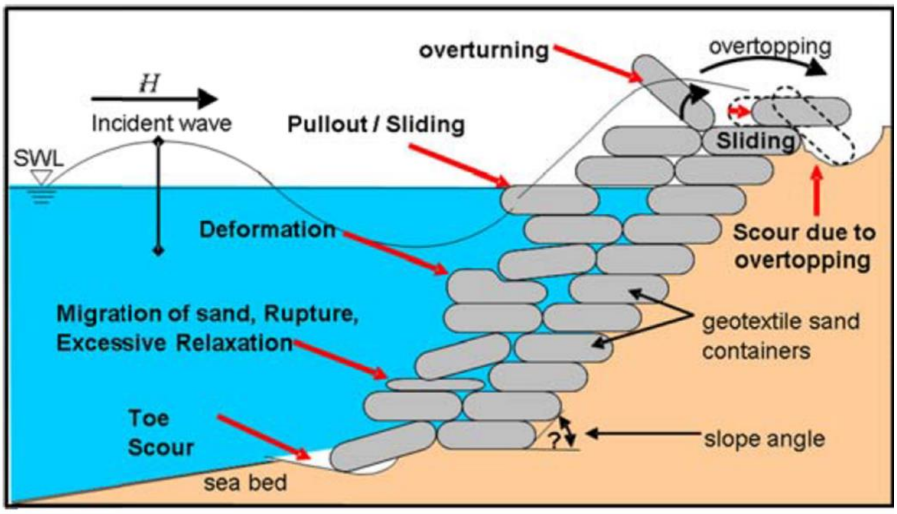
Figure 4.31: Shore protection (using sand bags) on the KwaZulu-Natal coastline, incorporating an outfall

The most important failure mechanisms can be listed as follows (Figure 4.32; PIANC, 2011; Dassanayake and Oumeraci, 2012):

- Tilting (overturning).
- Sliding.
- Deep geotechnical slip.
- Bearing capacity of the subsoil is too low.
- The sand inside the sand bags moves and causes large settlement.
- The seabed is scoured away at the toe and part of the structure falls into the scour hole.
- Wave overtopping results in scour at the back of the structure.

The design of a sand bag revetment have to take at least the following into account, namely:

- *Durability* of the geotextile and the material (fabric) of the sand bags.
- *Stability* of the structure.



From Dassanayake and Oumeraci (2012)

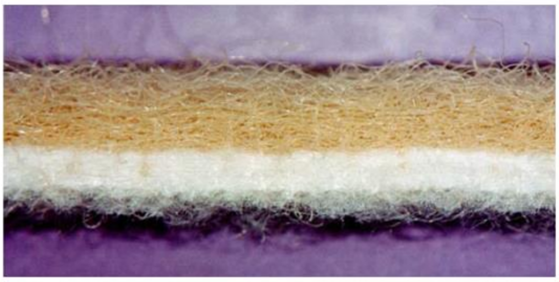
Figure 4.32: Failure mechanisms for a sand bag revetment

Regarding *durability*, a sand bag can, clearly, only be effective as shore protection as long as its contents remain in the bag. A significant advantage of sand bags is that the material (sand) for fill is usually available at the site. There are a number of ways in which the sand fill can be lost from the bags, namely:

- The bags can tear or burst open because of wave and current action. Partially filled bags are particularly at risk. Flapping of the material can lead to holes in the material.
- Abrasion of the bag fabric by sediment, floating debris, boats and pedestrians can cause failures.
- Degradation of the material of the bags because of ultra-violet light of the sun. Even though inhibitors are added to the fabric, eventually the material will disintegrate. As a result, the material need to be protected against sunlight.
- Vandals can cut and damage the bags.

To ensure the durability of the sand bag material, a double layer (Hornsey et al, 2009) geotextile has been developed in Australia (Figure 4.33). Apart from being stronger and resistant against abrasion, the outer layer protects the inner layer against ultra-violet light. Furthermore, sand grains are trapped in the outer layer, which makes it considerably more difficult to cut with a knife and thus, reduces vandalism. The material used for EnviroRock by Kaytech (2017) in South Africa have similar properties (Figure 4.33). Saathoff et al (2007) and Rankilor (1994) give a good description of the required characteristics and tests to be conducted of the geotextile to be used for making sand bags. Kaytech (2017) lists the characteristics and test for EnviroRock.

Figure 4.34 illustrates how the large sand bags were filled at Langebaan.



From Hornsey et al (2009)

Photo: Kaytech

Figure 4.33: Material used for large sand bags

Construction by Southern Oceaneering

Photo: Kaytech

Figure 4.34: Filling sand bags at Langebaan

A typical cross-section of a sand bag revetment is depicted in Figure 4.35. Note the double layer of sandbags (it is recommended) and that the sand bags have been placed (as is recommended) with their long axes parallel to the wave direction (and perpendicular to the shoreline). This is contrary to the placement orientation in Figure 4.28. The study by Baret (2013) confirmed that the recommended orientation of the sand bags in Figure 4.35 is the preferred one.

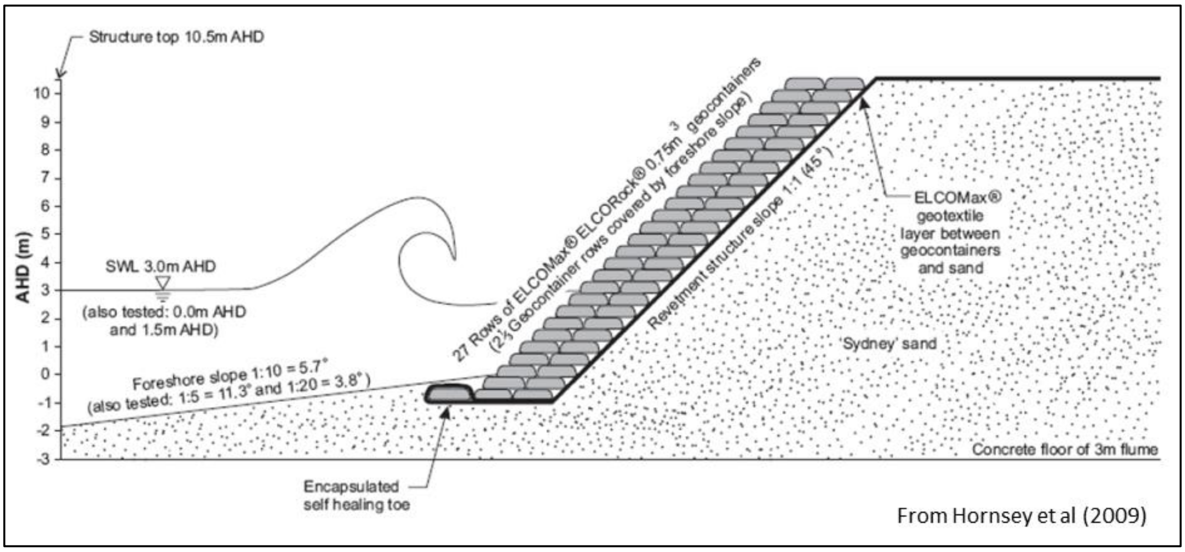


Figure 4.35: Typical cross-section of a sand bag revetment

The 2.5 m^3 sand bags are recommended whilst it is not recommended to use bags smaller than 1 m^3 or bags much bigger than 2.5 m^3. Sand bags should be filled to capacity (Hornsey, et al, 2009; Baret, 2013), but not be overfilled. This is to severely limit the movement of sand inside the sand bags. (Note that the German practice of filling sand bags to a capacity of approximately 80 % differ from the Australian (Hornsey et al, 2009) and South African approaches.) The bags should be closed by stitching. Furthermore, care should be taken during construction not to damage the sand bags. For example, old conveyor belts (not shown in Figure 4.28b) were later used to protect the placed sand bags from damage by the tracks of the crane and other vehicles (comment by Kaytech).

It is important to ensure that an outfall is fully supported along its whole length where it goes through the sand bag revetment. This support is required below, laterally and above the outfall to take the loads and essentially prevent or severely limit movement (including vibrations) of the pipeline in all directions.

Note that water from wave overtopping, if excessive, can cause damage to infrastructure and even be dangerous to pedestrians and cars (EurOtop, 2007). It is recommended that the sand bag revetment be built high enough to prevent significant wave overtopping. Nevertheless, attention should be given to the possible lateral flow of overtopped water during conditions exceeding the design condition. For example, placing a few bags in rows on top of the structure to form little groynes or spurs at regular intervals alongshore, should limit lateral flow and allow the overtopped water to drain through the structure.

The toe of the revetment is extremely important for the stability of the structure. It is recommended that two or more sand bags as flexible toe be placed directly on the sand seawards of the toe. The orientation of these bags should be alongshore; that is, the long axes of the bags should be parallel to the shoreline. The concept is that the toe is very flexible: these rows of bags can easily fall into a scour hole and is designed as such, thus, preventing further scour. This advice is based on experience at Oranjemund (Theron et al, 1999). Saathoff et al (2007) give similar advice about the toe of a sand bag revetment.

The layout (plan shape) of a sand bag revetment should be considered carefully and 3-dimensional aspects also have to be evaluated. The sand bag protection should enclose the spillway structure; or else, the revetment should continue sufficiently far landwards so that during erosion of the adjacent shoreline, the spillway structure is not attacked from behind (the land side). There should be no sharp corners because these corners are the most vulnerable sections of the sand bag revetment.

The best slope based on present research is 1: 1 (45 degrees; Figure 4.35). This was found by Recio (2007), Saathoff et al (2007), Recio and Oumeraci (2008), Hornsley et al (2009) and Baret (2013). Although a flatter slope will result in the dissipation of wave energy of a larger area, it is the friction between sand bags that is dominant in this particular instance. For a steeper slope of 1: 1, there is a large gravity force on the bags (self-weight) so that an adequate contact area between bags are obtained, ensuring appropriate friction between bags. Furthermore, the force to pull out a sand bag from the slope by the waves are larger because of the larger self-weight acting on the bags for a steep slope of 1: 1.

Figure 4.36 contains a design graph for 2.5 m^3 sand bags for different slopes (Hornsey et al, 2009). These results are based on 77 physical modelling tests at a large scale of 1: 13. It can be seen that the best slope of the revetment is 1: 1. It is not recommended to use slopes steeper than 1: 1. Furthermore, it is also evident that the wave period (apart from the slope) is important because the period affects the type of breaking. The sand bag slope is significantly less stable for longer period waves (Figure 4.36).

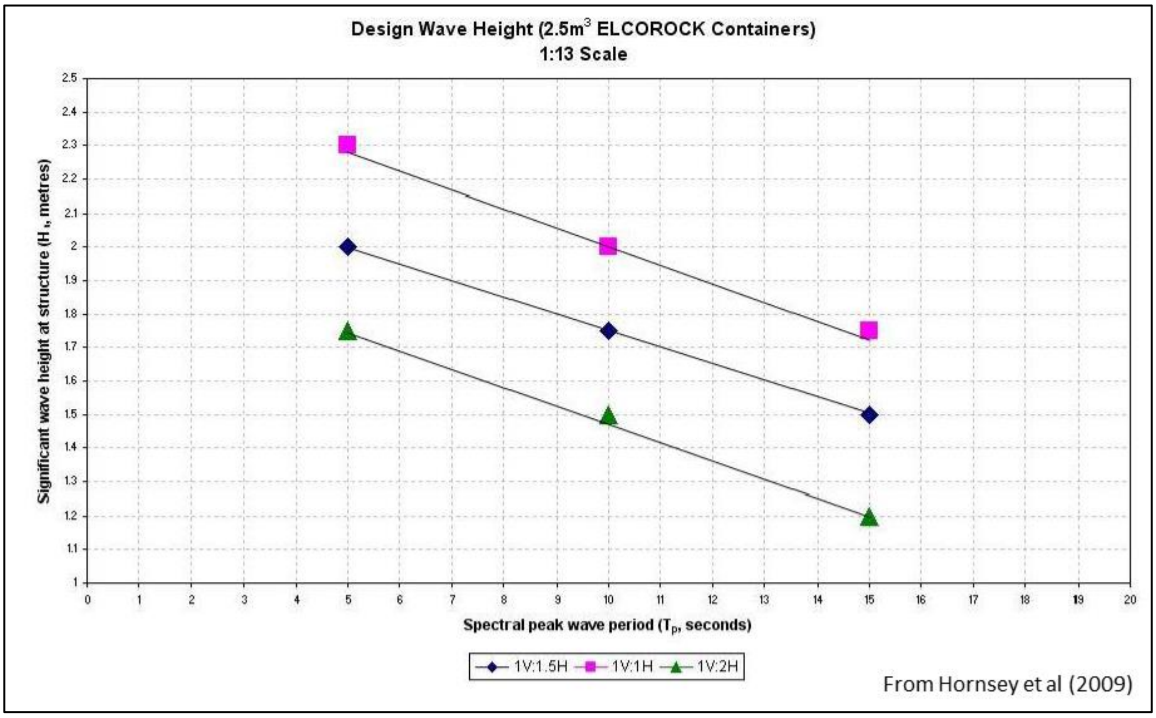


Figure 4.36: Design graph for 2.5 m^3 sand bags placed at a 1: 1 slope

It is recommended that the design procedure by Recio (2007) and Recio and Oumeraci (2008) be used for the design of sand bag revetments. In his physical model tests, Baret (2013) has found good agreement between the Recio and Oumeraci stability predictions (their design method) and the results from his physical model tests.

It is important to note that attention should be given to details of a sand bag revetment because experience has shown that failure can occur because of small issues regarding the design. It is recommended that physical model tests be conducted to confirm and optimise the design of a sand bag revetment.

Concrete block and grout mattresses

Concrete block mattresses are solid or perforated concrete blocks that are usually connected by means of wire rope or polyester rope (Figures 4.37a and b and 4.38). For marine applications, strong, polyester rope is preferred because corrosion will not be a problem. These mattresses are placed on a geotextile (filter fabric) that covers the ground, allows water to flow through the mattress and prevents or limits soil leaching out from underneath the geotextile. Sometimes the concrete blocks are cast directly onto the geotextile with fabric loops (woven into the geotextile) providing the connection between the geotextile and the concrete blocks (Figure 4.27).

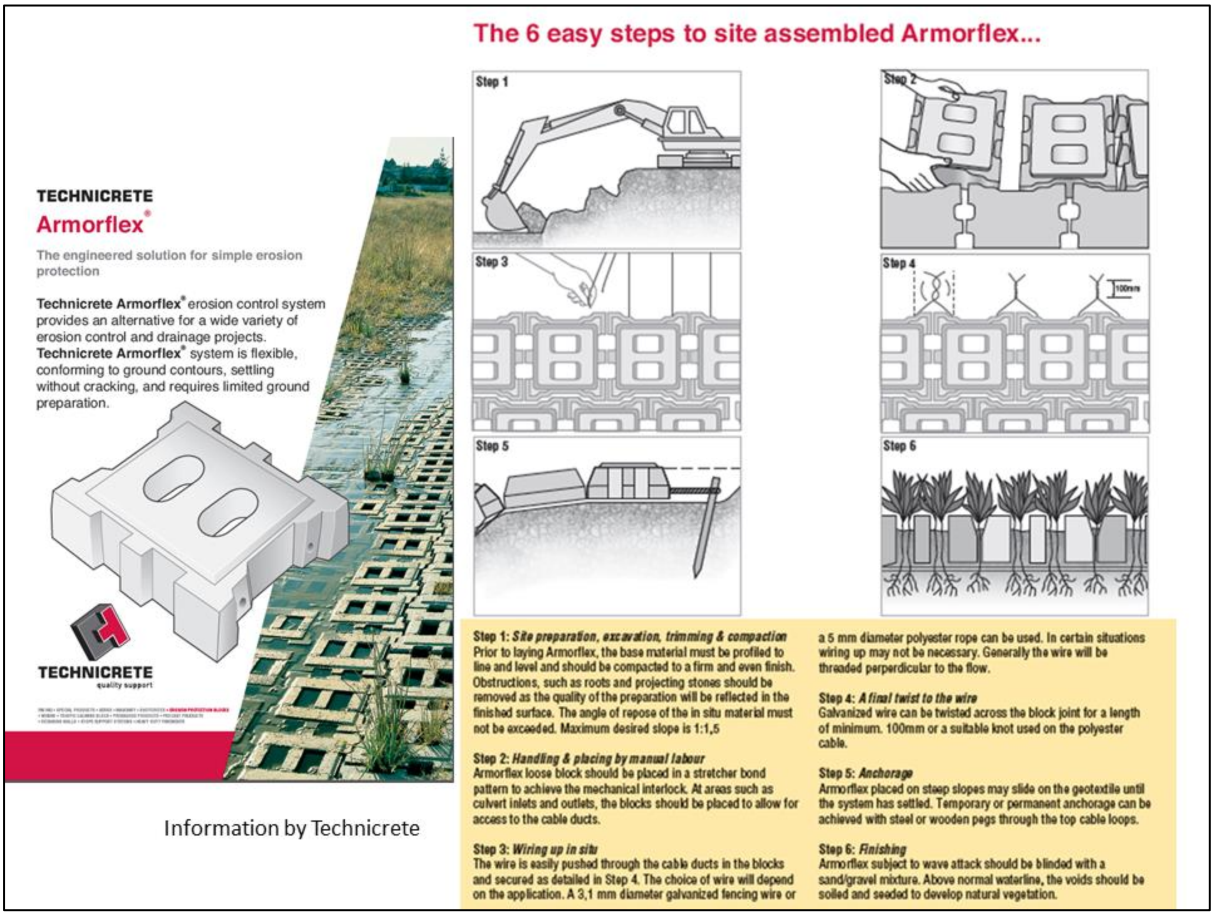


Figure 4.37a: Armorflex (Concrete Block Mattresses) by Technicrete

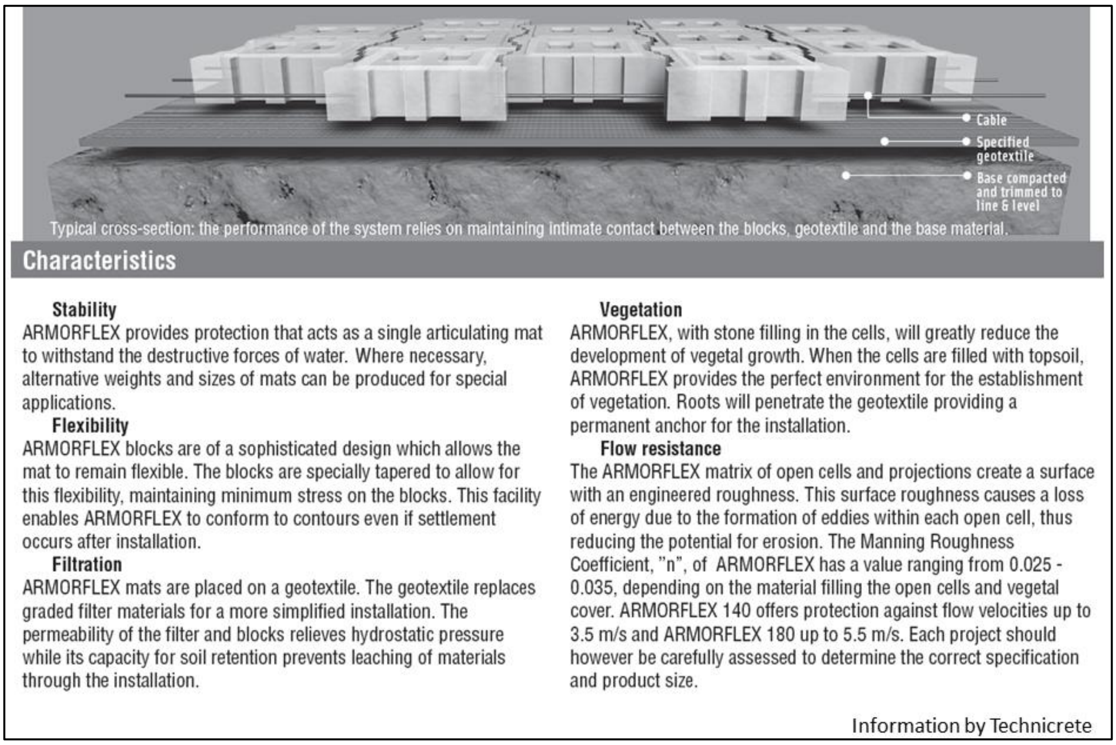


Figure 4.37b: Armorflex (Concrete Block Mattresses) by Technicrete

Photos: Maccaferri

Figure 4.38: Articulated Concrete Block Mattresses by Maccaferri

Photo: K Möller and CSIR

Figure 4.39: Mattress with concrete blocks cast directly onto the geotextile

Concrete block mattresses are flexible so that the mattresses can follow the ground contours and allow for settlement. It is customary to anchor the mattresses on steeper slopes by using stakes hit into the ground (Figure 4.37a). Perforated blocks can either be filled with gravel or coarse sand to restrict vegetation growth or filled with topsoil to stimulate plant growth. Pipelines are regularly protected by placing concrete mattresses over them (Figure 4.38).

A *grout mattress* (Figures 4.40 and 4.41) consists of two connected geotextile sheets, one on top of the other but closed at the edges (in the form of a duvet cover). At regular intervals in both directions, these two sheets are woven together in spots, thus forming permeable filter points or weep holes. After preparation of the soil, the grout mattress is laid out and sometimes fixed to the soil by means of stakes (especially on steeper slopes). The geotextile sheets act as formwork when the geotextile sheets are pumped full with grout from the lower elevations of the geotextiles sheets to the higher points. The weep holes remain intact, allowing water flow back and forth. Excess grout should not clog the weep holes. Soil should not be washed out from under the grout mattress. At the same time, the weep holes that force the grout into a rounded rectangular shapes throughout the mattress (Figures 4.40 and 4.41), also make the grout mattress flexible. Clearly, exposure to ultra-violet light from the sun will result in degradation of the geotextile. Therefore, the grout mattresses should preferable be covered.

Photos: D Lessard and WSP

Figure 4.40: Grout mattresses in the Middle East

Information and photos by Erosion Protection Systems

Figure 4.41: Grout mattresses in Australia by Erosion Protection Services

Proper design of concrete block and grout mattresses must include the following:

- Ensure that the mattresses will be stable under wave and current action during extreme conditions. This normally means that the thickness of the mattresses has to be calculated. Sliding downwards of all the mattresses must be prevented.
- The geotextile must initially be and remain functional during the design life of the structure: (1) the geotextile must be permeable; (2) it must not allow the leaching out of soil from underneath the mattresses; and (3) it must not not tear.
- The applicability of construction methods has to be considered. Furthermore, access to the site and space for materials and equipment need to be taken into account. Practical issues to be evaluated include joints and seams of the geotextile.

Rankilor (1994) presents a graph that can be used for conceptual design of *concrete block mattresses*. The stability number ($H/(\Delta_{concrete}D)$) is given a function of the surf similarity parameter, which defines the type of wave breaking.

Where D= required thickness of the concrete block mattress (minimum of 100 mm)

H= wave height (presumably the significant wave height at the structure)

$\Delta_{concrete}$= relative density= $(\rho_{concrete} - \rho)/ \rho$

$\rho_{concrete}$= density of the concrete in the mattress= approximately 2 400 kg/m^3

ρ= 1 025 kg/m^3 for sea water.

In the Rankilor (1994) graph, the stability number varies between 3.5 and 8.5 and the required thickness D can be computed. Note that the slope at which the concrete mattress is placed must be flatter than 1: 2.1 (25 degrees) and that a maximum wave height 1.2 m applies (Rankilor (1994). Furthermore, a thick geotextile and/or a granular underlayer should be used to reduce hydraulic pressure underneath the grout mattress (Rankilor (1994).

The Rankilor (1994) design method for *grout mattresses*, which should be considered a rule of thumb, is as follows:

$D = H / (5\Delta_{grout})$

Where D= required thickness of the grout mattress (minimum of 100 mm)

H= wave height (presumably the significant wave height at the structure)

Δ_{grout}= relative density= $(\rho_{grout} - \rho) / \rho$

ρ_{grout}= density of the grout in the mattress= approximately 2 000 kg/m^3

Rankilor (1994) states that a thick geotextile and/or a granular underlayer should be used to reduce hydraulic pressure underneath the grout mattress.

The design methods by Klein Breteler and Pilarczyk (1998), Rankilor (1994) and PIANC (1992, 2011) can be applied for the hydraulic stability of mattresses. Pilarczyk (2000) provides further information on the use of geotextiles while the data sheets of the suppliers of the mattresses present their specifications. The minimum required thickness of the mattresses can be computed as a function of the significant wave height, relative density of the material in the mattress and the type of wave breaking (Klein Breteler and Pilarczyk, 1998). The type of wave breaking (for example, plunging or spilling breaking) that will occur, is determined by the surf similarity parameter (CEM, 2006). Furthermore, Klein Breteler and Pilarczyk (1998) stressed the importance of the geotechnical properties of the foundation soil. In evaluating the stability of the mattresses against sliding and possible lifting of the mattresses, the following geotechnical conditions should be adhered to:

- Elastic storage. (Elastic storage in the foundation soil is related to the permeability and stiffness of the soil structure and the compressibility of the pore water.)
- Softening (liquefaction) of the foundation soil.
- A drop in the water-level.

In general, the requirements for a suitable geotextile should at least include the following (Ingold and Miller (1988), Rankilor (1994), Pilarczyk (2000) and PIANC (2011)):

- The geotextile should be very permeable to water, resulting in low pore-water pressures, especially during wave up- and downrush.
- The geotextile must have a high strength and should not stretch excessively.
- Sediment should be contained underneath the geotextile and should not wash out so as to maintain a stable foundation for the mattresses.

Note that the use of mattresses in the coastal zone in shallow water is restricted to calm conditions; that is, the maximum significant wave height should be less than 1 m. For higher waves, the required thickness of the mattress becomes too large and then either rock or armour units is a better solution.

Special attention must be given to fixing the mattresses to the outlet structure. The edges of the mattresses should be buried so that they are more stable whilst also making it more difficult for thieves and vandals to access the edges of the mattresses. In the case of concrete block mattresses without an inbuilt geotextile, the edges of the mattresses should be without geotextile (placed directly on the soil) in order to induce sinking of the edges during wave action. These strips of mattresses without geotextile should be far enough from the outlet structure so that the stability of the outlet structure is not detrimentally influenced by the local sinking of the mattresses along the edges.

Geotextile normally come in rolls with a given width. To cover the required area, different strips of geotextile have to be properly joined by either a strong, sown joint or by gluing the strips together. Overlapping of geotextile strips should be avoided if possible. Industrial sewing machines are available to easily carry out joining in the field. Where possible, the orientation of the geotextile strips should be so that the joints are parallel to the wave direction; that is, at right angles to the shoreline.

For grout mattresses, care have to be taken that grout is not lost through the geotextile. Furthermore, the geotextile should be filled to capacity throughout the mattresses. This will ensure stability and prevent flapping of a half-filled mattress.

Gabions and Reno mattresses

Gabions (Figure 4.42) and Reno mattresses are wire baskets packed with stone and fixed to each other to form a continuous stone wall and/or mattress.

The advantages of gabions and Reno mattresses include the following:

- The wire baskets are light and easy to transport to the site.
- The completed gabion structure is highly permeable and thus, water pressure is quickly relieved.
- Stone for filling the baskets can usually be sourced locally and cheaply.
- Unskilled labour can be used because the construction is normally easy and relatively quick.
- Construction is labour intensive; thereby, it will improve the lives of the local community.

Photos: K Schoonees

Figure 4.42: Gabions on the back beach (Southern Cape Coast)

The main disadvantages are:

- As long as the wire baskets are intact, the gabions and Reno mattresses act as a single structure. However, if the wire of the wire baskets break, the stone usually spills out and the structure fails at least locally. (This is similar to sand bags.)
- Stone can move inside the wire baskets and damage the wire in the process. However, if correctly and tightly packed, then this should not be a serious disadvantage.

The coastal zone (and especially the area between the low and high water marks) is a high corrosion area. To counteract the corrosion, the wire of gabions is protected against corrosion by both galvanising the wire and by covering the wire with plastic coating. However, abrasion by sediment (sand, gravel, pebbles and/or cobbles) and flotsam, together with pedestrian and dog traffic, removes the plastic cover and galvanising. For example, gabions in the intertidal zone, is subjected to oscillating water movement twice in approximately 12 seconds (every 6 seconds) as each wave passes. Ultra-violet light from the sun will also eventually result in the cracking of the plastic coating even though inhibitors have been added to the plastic to improve the lifetime of the plastic exposed to the sun.

Experience in Southern Africa has clearly indicated that the life of gabions in the sea is limited because of abrasion, corrosion, failure of the wire and the stone being washed out. However, note that hydraulic applications such as weirs and stilling basins in rivers and streams (in fresh water) have been very successful with long lifetimes.

Welsby and Motyka (1984) assessed the durability of gabions around the coastline of the United Kingdom. Their final conclusion is as follows:

"Opinion as to the lifespan of metal gabions on the foreshore is divided but the general consensus is that in areas subject to severe wave activity, gabions will succumb to rapid abrasion and as a result their lifespan can be as short as 2 or 3 years. On flat sand beaches subject to moderate or low wave activity the lifespan can be a decade or more. On the backshore, where gabion structures are not subjected to regular wave activity, they can be expected to have a considerably longer life."

To conclude then, the use of gabions and Reno mattresses in the sea is generally **not** recommended. However, if gabions and Reno mattresses are very seldom (less than once a year) exposed to wave and current action for a short time and buried for the rest of the period, they can be used in the coastal environment. Nevertheless, it should in these instances still be evaluated whether loose stone or rock will not be a better option.

Note that gabions can be very handy should a filter layer of rock be required to drain water away. That is, a gabion lined with a geotextile and filled with gravel can easily be placed around a structure to drain water away. This means that the gabion is used for: (1) ease of construction (and its strength is not required after deployment); and (2) to limit the volume of gravel.

Other methods

Placing the stilling basin of the outlet very high up on the beach so that waves will never reach it, makes it unnecessary to protect the outlet against wave action. The best management practices described in Schoonees and Theron (2016) such as infiltration, would have to be used to reduce or eliminate any outflow, even during floods. Typical consequences of an outlet discharging landwards of wave action, include a pool of stagnant water at the end of the outlet and potential aeolian sand transport problems.

By building a large pier or jetty to support the outlet above any wave action, is another possibility. Another option is to incorporate the whole outlet in a groyne, seawall or breakwater. These options are possible, but the abovementioned structures are very expensive and most probably only financially feasible if the structure is built for other purposes (rather than for the stormwater outlet).

4.2.4 Inundation and flooding

Practices to mitigate flooding and inundation from the sea

As mentioned in Section 4.2.2 (and in the Phase 1 report; Schoonees and Theron (2016)), flooding and inundation of inland and backshore areas result from blocked drains, when the fluvial stormwater runoff is greater than the design capacity of the stormwater system, and where poor drainage ability occurs (due to various factors). Regarding flooding related to the capacity of the stormwater system, this is clearly a terrestrial hydraulic design issue as addressed in readily available literature which is not included in the scope of this document.

Regarding backshore flooding, another problem that is identified is that when water-levels in the sea or an estuary are very high, the water can also back up into the stormwater pipes thus resulting in such flooding. Fitting of *duckbill valves* (for example, see Figure 4.43) to prevent both inflow and backflow of seawater, as well as ingress of marine sand is a good option to mitigate such problems. Simple metal flap valves fitted directly to the exposed end of pipe or on the outfall opening are prone to blockages resulting from sand build up against the outside of the flap in excess of what can be forced open and washed away by the stormwater runoff in the pipe. Such metal flaps are also

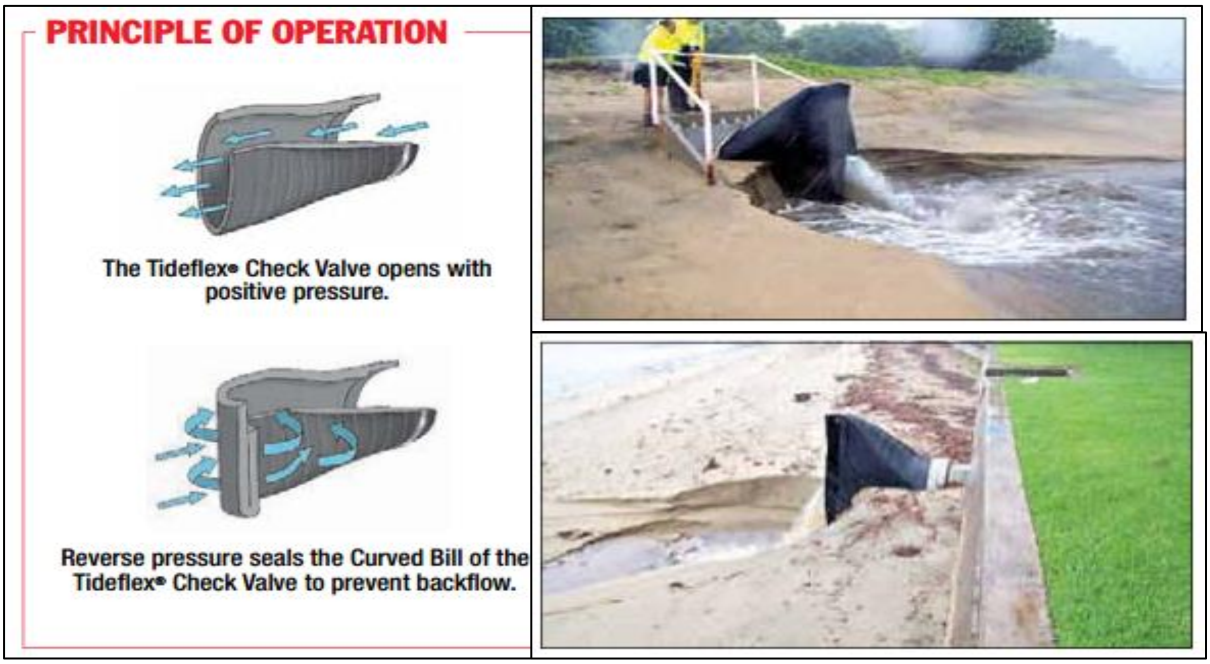


Figure 4.43: Duckbill valves (from Tideflex, 2016)

vulnerable to wave damage, vandalism and theft, as well as corrosion. Although duckbill valves are more expensive, they are much less vulnerable to such problems and are preferred. In terms of damages and expenses associated with flooding events, the cost of fitting duckbill valves may be offset relatively quickly.

Practices to mitigate sand inundation of the outlet

As alluded to in the previous paragraph, a major cause of blocked drains in the coastal zone, is sand inundation resulting from aeolian sediment transport or ingress of sand through wave and tidal action. Further recommendations for the alleviation of wind-blown sand problems and sand ingress from surface runoff are discussed here.

Three measures that can be employed to combat blocking of the outlets by wind-blown sand are to: (1) stabilise the sand; (2) relocate the outlet; and/or (3) adapt the outlet configuration. It is recommended that the best alternative would be to stabilise the sandy areas near the outlet with healthy, well-established dune vegetation (space permitting). The outlets therefore have to be incorporated into the buffer dune (where such exists or into the design of rehabilitated dunes). Stormwater outlets are often located within 1 m or 2 m of the edge of the parking areas at the back of the dune cordon. The buffer dune will trap wind-blown sand and therefore result in less sand accumulating in the mouth of these outlets. Relocating outlets from depressions in which they may be located to higher elevations in order to prevent sand inundation is sometimes an option, but may in other cases not be considered to be a practical or economically favourable alternative. This could necessitate realignment of existing pipelines in such cases.

Construction of an adequately wide outlet structure with back and side walls and floor will also reduce the amount of sand deposited at the mouth of the outlet pipe. The outlet structure can be specifically designed to increase the speed of the airflow over and around the structure, thus preventing or limiting sand from being deposited at the outlet. However, this alternative would require specialist design and possibly physical modelling, and is therefore considered to be impractical and too costly in many instances.

Besides fitting of duckbill valves to prevent ingress of marine sand, a sediment backflow control device as proposed in the Queensland Urban Drainage Manual (2013) could also be considered (Figure 4.44). This device is also designed to mitigate landward ingress of marine sand and to promote “self-clearing” (that is, scouring out of marine sand from the last pipe segment).

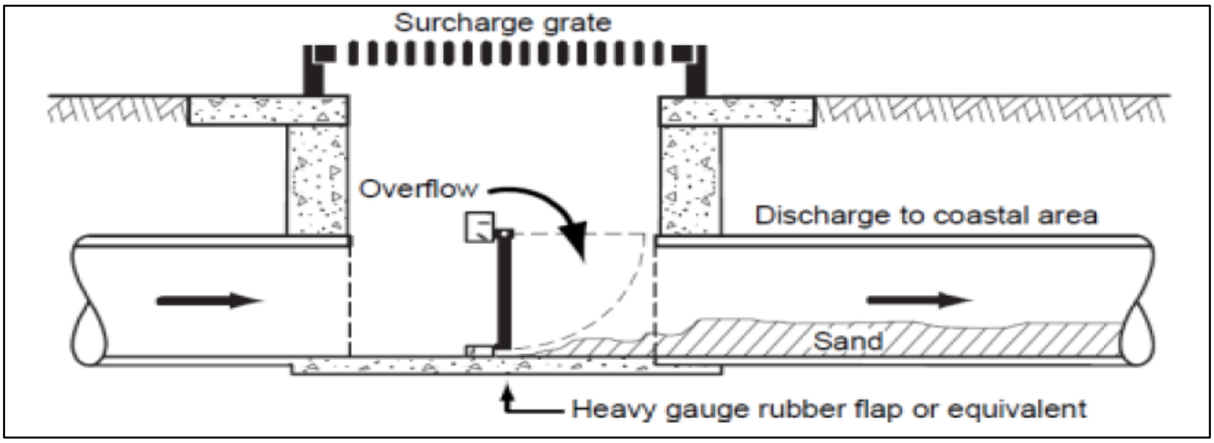


Figure 4.44: Sediment backflow control device (from Queensland Urban Drainage Manual, 2013)

Sand can also be washed into outlets from the surrounding area of outlets terminating on sandy slopes. The sand can be prevented from being washed into such outlets by preventing surface runoff from flowing around the outlet (methods are discussed in the previous section) and by providing adequate side and back walls which lead the water around the structure. Sand should also be prevented from entering the pipelines by installing closed manhole covers in active littoral areas near the shoreline. Open-slotted covers required for drainage of surface water should rather be located further inland.

4.2.5 Water quality

General

In general, prevention is better (and cheaper) than cure. The following guidelines and recommendations have been presented in Schoonees and Theron (2016) in this respect, namely:

> Pollution associated with stormwater outlets relates to mainly contaminated fluvial runoff resulting in pollution in the sea near the outlet, stagnant pools formed at the seaward end of the outlet (or ponding of polluted runoff), and debris and litter. All of these problems are ideally alleviated by following best management practices in the terrestrial domain (catchment) as discussed in Section 4.2 in Schoonees and Theron (2016). Ponding of runoff at the terminal end of the outlet can be prevented or reduced by means of effective outlet structures or by incorporating infiltration components in the outlet system. However, if polluted water emerges at the shoreline, it is very difficult to manage this further and it is far better and cheaper to address the pollution in the terrestrial domain.
>
> Similarly, litter generated inland should rather be dealt with upstream and on land (Schoonees and Theron, 2016). Litter trap structures and mechanisms require significant maintenance to prevent blockages or overspill of litter and are also susceptible to vandalism and theft of metal components (Schoonees and Theron, 2016).
>
> Low flow conditions induces settling of solids and pollutants in a stormwater system until a sudden increase in flow after rain results in a large additional pollution load (the so-called "first flush"). Fried (1975) as referenced in Schoonees and Theron (2016) mentions that street cleaning is cost effective to remove heavy metals and pesticides close to the source of contamination rather than treatment at the receiving end.
>
> Exfiltration involves the extraction of a part of the stormwater flow and letting the water infiltrate the soil by means of a perforated pipe and/or a trench lined with a geotextile and filled with rock or stone (extraction + infiltration= exfiltration). The first flush is usually treated in this way. Apart from reducing the flow rate at the outlet, it is also effective to decrease pollution levels, recharge aquifers and to limit the underground intrusion of seawater towards land. Refer to Schoonees and Theron (2016) for more detail.

Dispersion and dilution

If the contamination of stormwater cannot be prevented, one or more of the following measures can be taken, namely:

- Ideally, the **effluent should be treate**d to acceptable water quality standards before discharging into the coastal zone. However, this option is very expensive and usually requires continuous operation.

- **Keep people and animals** away from the polluted effluent that is discharged; that is, away from the so-called impact zone immediately around the discharge point at which the pollution levels can be unacceptably high. A distance of at least 100 m alongshore on either side of the discharge point should be regarded as the impact zone unless detailed studies have more accurately indicated the extent of the impact zone. Management measures in this category include:
    - Put up information boards (signs) telling beach users to warn them against bathing, swimming, water sports and the collection of mussels, oysters and other filter feeders as food in the vicinity of the stormwater drain.
    - Police these impact zones by means of lifesavers or other beach personnel.
    - Close parts of the beaches in during times of first flushes by means of signs and tape fences. Note that first flushes are less of a problem in winter rainfall areas because first flushes will occur at times when it usually will be too cold to swim. However, surfers, wind surfers and kite surfers use dive suits and do venture into the sea during winter.
    - Monitoring of water quality by analysing water samples on a regular basis is required in order to make informed decisions in this regard.
- The stormwater outlet must be designed so that maximum mixing and dilution are obtained and the impact zone is minimised. If possible, a stormwater outlet should not be placed on a bathing beach or alternatively, that the impact zone would be away from the bathing and swimming area.

4.2.6 Applicable legislation

The main legislation applicable is as follows:

- EIA Regulations promulgated under Chapter 5 of the National Environmental Management Act (NEMA) (Act 107 of 1998), National Environmental Management Amendment Act, No. 62 of 2008, the EIA Regulations (GN No. R. 982 of 4 Dec 2014, Listing Notice 1 (GN No. R. 983 of 4 Dec 2014), Listing Notice 2 (GN No. R. 984 of 4 Dec 2014), Listing Notice 3 (GN No. R. 985 of 4 Dec 2014), National Exemption Regulations (GN No. R. 994 of 8 Dec 2014), National Appeal Regulations (GN No. R. 993 of 8 Dec 2014)), and latest amendments (GNR 324-327, 350) from April 2017.
- Integrated Coastal Management Act (ICM) of Act No. 24 2008 and ICM Amendment Act (#36 of 2014) - the Department of Environmental Affairs is responsible for and to regulate the use of coastal waters, including the discharge of effluents from land based activities; coastal management lines, etc.
- Coastal Water Discharge Permits (CWDP) control wastewater disposal to coastal waters including estuaries (must get permit).
- Local Government Act: Municipal Systems, 2000. Spatial Development Frameworks (from Municipalities) – IDP/DPLG; Cadastral boundaries; Municipal/Town planning zones.
- National Water Act (1998) (DWS) - effluent disposal to sea/estuaries (must get license).

Policies on disposal of waste water:

- Operational Policy for the Disposal of Land-Derived Water Containing Waste to the Marine Environment of South Africa, (DWAF, 2004). The South African Water Quality Guidelines for Coastal Marine Waters provides recommended target values for a range of water quality constituents to prevent negative impacts on the marine ecosystem (DWAF, 2004).
- Assessment framework for the management of effluent from land based sources discharged to the marine environment (DEA, 2015).

Other legislation that may be applicable depending on the specific site:

- NEM: Biodiversity Act, No.10 of 2004.
- National Heritage Resources Act, No 25 of 1999.

4.3 Construction Methods and Guidelines

4.3.1 Typical construction methods

Note that some of the photos that follow depict the construction of larger marine structures; however, the principles for construction remain the same for smaller structures such as small stormwater outlets.

Construction of marine structures such as groynes, seawalls, ocean outfalls and breakwaters is normally carried out as follows:

- **Access from land**. Usually, either: (1) a temporary embankment (causeway) and/or a cofferdam; or (2) a jetty (pier; a deck supported by piles) with or without a cofferdam are used.
- **Access from the sea**. Floating equipment such as a jack-up platform, floating cranes and barges are used. As a result, a minimum water depth and calm weather are required to accommodate these floating equipment.
- A combination of construction with **access from both land and sea**.

The embankment giving access from land, is normally built from sand, gravel or small rock (the material) by end tipping the material from trucks (Figure 4.45) and then reshaping the placed material using excavators and/or bulldozers. This material has to be protected on the outside against wave action by means of larger rock, sand bags, sheet piles, etc. The top surface of the embankment has to be suitable to drive machines on and, if necessary, provide space for temporary storage of equipment and material.

Figure 4.45: An embankment constructed by end tipping (in the Middle East)

It is widely practised to use a cofferdam to protect the works area against wave and current action (Figures 4.46 and 4.47 from Schoonees and Theron, 2016; and Figures 4.48 and 4.49). A cofferdam is usually a wall (bund) of sand or gravel that is, in turn, protected from erosion by the sea. Alternatively, sheet piles can be used on their own (Figure 4.50). It is also customary to place sheet piles in conjunction with cofferdams for protection (Figures 4.51 and 4.52) and/or to limit seepage of water into the work area (Figure 4.47). The examples shown in the figures illustrate these same methods as applied for general protection of larger coastal areas, but for protection of small outlets, the scale (or lateral extent) would just be more limited.

Figure 4.46: Example of coastal construction works protected by a temporary sand wall and sand bag construction. (Photo: A Theron, False Bay, 2016)

Figure 4.47: A cofferdam with sheet piles and dewatering for large scale construction in the dry in the Middle East

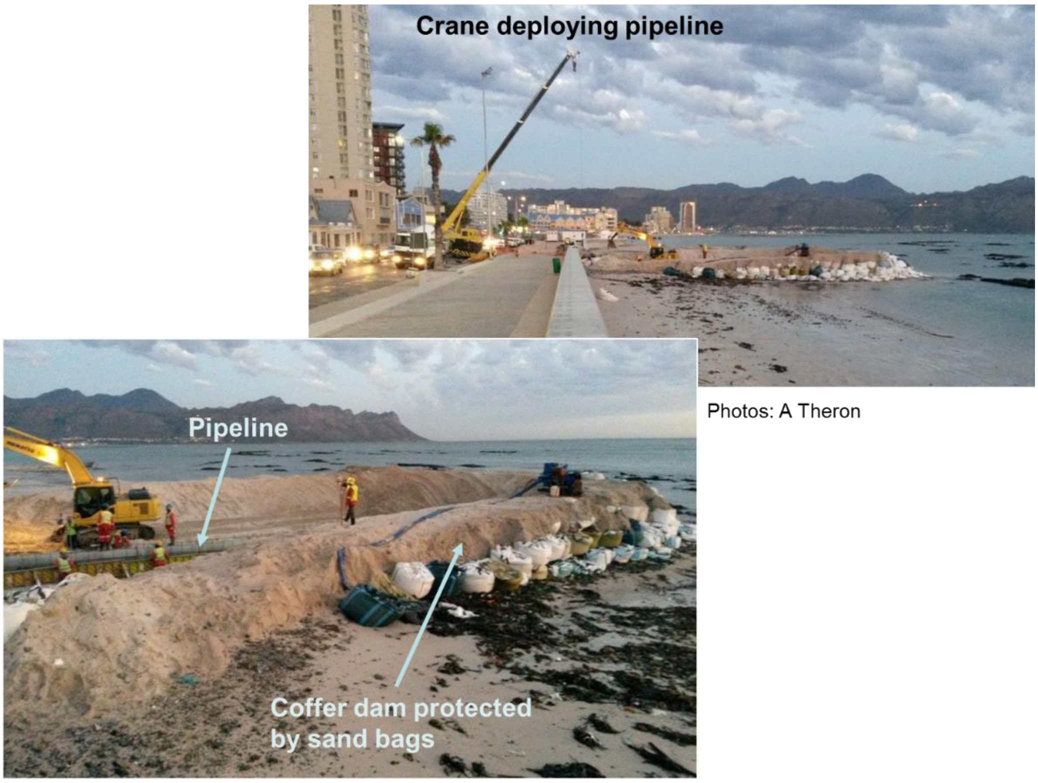
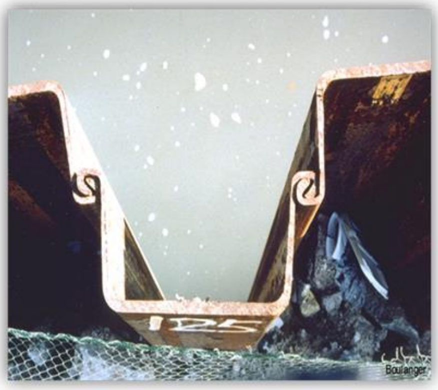


Figure 4.48: Placing a pipeline by a crane whilst protected by a coffer dam at the Strand

Figure 4.49: A coffer dam for the deployment of an outfall in False Bay

Photos: M Scott and Clough, Murray and Roberts



Sheet pile wall

Photo: A Bintliff (WSP)

Figure 4.50: Sheet piles and a sheet pile wall in the Middle East

The construction of a jetty normally starts by building an abutment to support a crane with a pile driver (Figures 4.53 and 4.54). Two piles are driven into the ground, one on each side of the abutment along the shoreline. Once this is completed, a connection piece is fitted on top of the two piles (Figure 4.53). Two parallel gangways are then placed from the abutment to each of the piles. The crane then moves forward onto the gangways that support the tracks of the crane (Figure 4.53). (Alternatively, rail can be used for the crane.) A further two piles are driven into the soil seawards of the first two piles. This followed by another connection piece between the piles and the placement of two more gangways. By repeating this process, the jetty is constructed so that work can be undertaken from above the waves. A grab can be fitted to the crane so that a trench for the pipeline or outlet can be excavated next and parallel to the jetty (Figure 4.54).

Wave conditions are generally severe around the South African shoreline, except in lakes, lagoons and a few protected bays. These conditions make working from the sea very difficult and expensive. Furthermore, the duration of suitable, calm weather is limited. Small and medium coastal outlets usually have to be located on the upper beach, in the tidal zone and very shallow water and thus, floating equipment is not suitable, mainly, because of a lack of water depth. As a result, small coastal outlets will almost exclusively be constructed using access from land. It is also unlikely that a jetty will used for a small outlet.

Jetty and sheet pile wall

Photos: A Theron

Figure 4.51: Sheet pile wall and jetty for installing a pipe in the surf zone near Swakopmund

Figure 4.52: Sheet pile jetty and sand bag protection at Durban

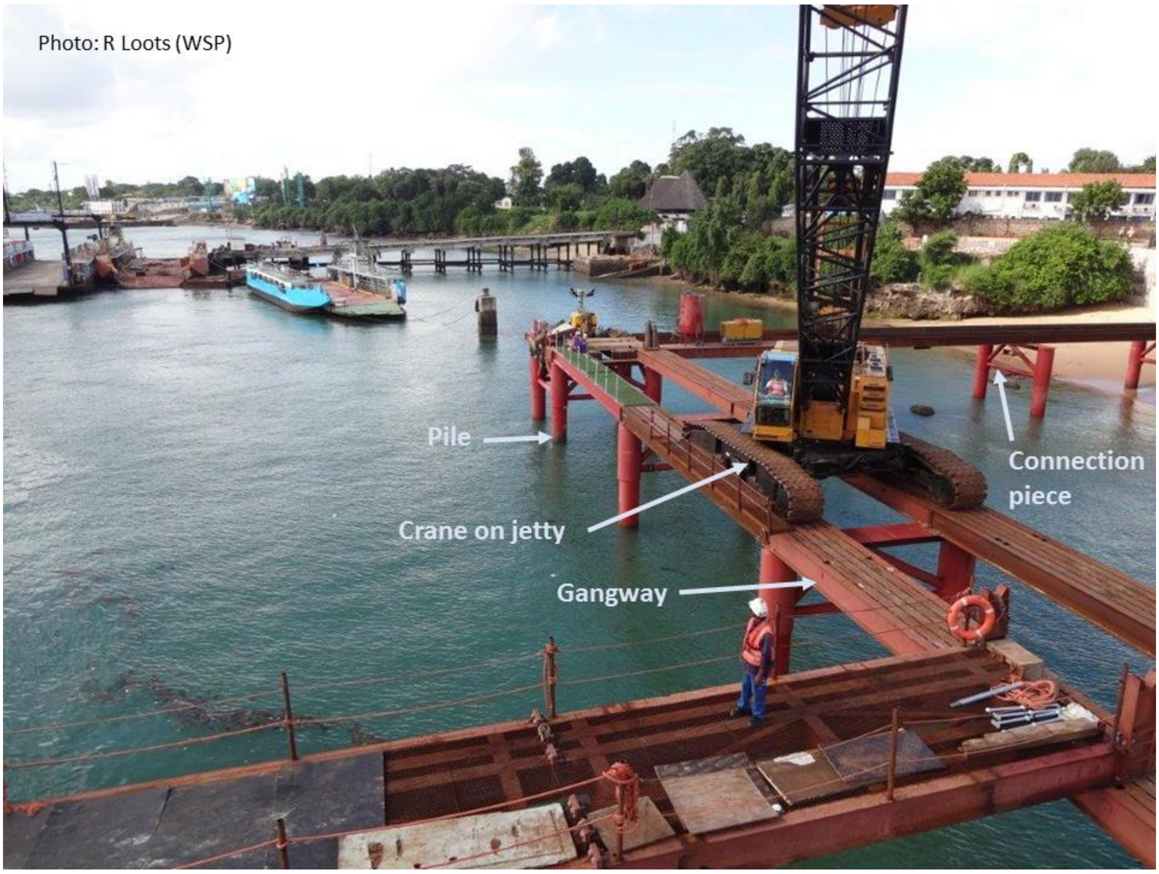


Figure 4.53: Crane on a jetty at Mombasa

Figure 4.54: Pipelines being placed north of Swakopmund (large scale construction)

4.3.2 Construction guidelines

The following general guidelines for construction of small to medium coastal stormwater outlets should be considered, namely:

- **Use favourable weather optimally** by limiting the construction period in the sea (Schoonees and Theron, 2016). Therefore, all equipment must be available and stored close to the construction site. Furthermore, precast concrete elements and other prefabricated sections should be used where possible and these elements should be ready for deployment close to the site. In this way, construction can progress quickly during calm and favourable sea and weather conditions.

  Construction can follow the water-level down as the tidal water-level starts dropping towards low-water spring tide (LWST). Note that for about 2 hours before and 2 hours after low tide, the water-level is reasonably close to low tide level. During this 4 hour period, construction above the low tide level can usually be undertaken with little interference from wave and current action and quickset cement can set.

- **Concrete** can be mixed at the work area or supplied by concrete trucks. Concrete can also be placed by using a bucket which is swung by a crane from the mixer to the placement location.

Alternatively, concrete can be pumped from land to an inaccessible working area by supporting the concrete supply pipeline on scaffolding (for example, over an uneven rocky zone).

Columns or supports for outlet structures can be fixed to a rocky bottom by placing a concrete manhole ring (or a very short pipe segment) on the seabed and then excavating the sand on top of the bedrock inside of the ring. Holes are then drilled into the bedrock into which dowels are fixed with epoxy in these holes. The concrete ring acts as formwork and as protection against wave action from the side when filling the ring with quick set or other concrete. It is important to protect the upper surface of the concrete in the ring by preventing the cement from being washed out of the concrete before the concrete hardens. Note that in some instances, sand can be used for this protection.

- If the work area protected by a cofferdam or the abutment leading up to a jetty forms an obstruction to the longshore **transport of sand** at the site, then accretion will form on the updrift side and erosion will occur on the downdrift side. Generally the net longshore sand transport is upcoast in South Africa. This means that usually the net longshore transport is north- to north-westbound on the West Coast and north-eastbound on the East Coast. (Inside bays, the direction of the net longshore transport can be different.) As a result, accretion will normally occur on the south or south-east side on the West Coast and on the south-west side on the East Coast. This accretion can result in re-excavation of sand while the downdrift erosion may endanger the stability of embankments and abutments.

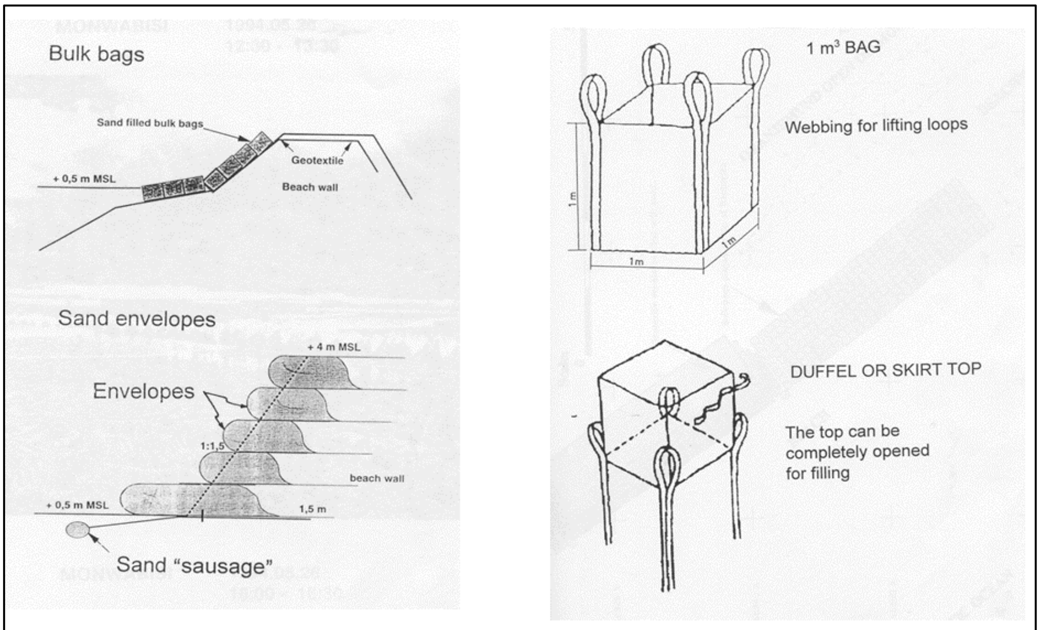
  Cross-shore sand transport will quickly erode a cofferdam and therefore, protection of a cofferdam is usually needed. Aeolian sand transport is usually only a nuisance during construction when the work area is on a dry beach or in the frontal dune. If a trench is excavated to build the stormwater outlet, then sand is usually transported into the trench at a considerable rate. Thus, it is recommended to close off the trench at the seaward end for as long as possible (Figure 4.54). A trench can be excavated by earthmoving equipment or by using a grab operated from a crane on a jetty.

- A temporary **cofferdam** can be a wall (bund) of sand; a sand wall protected by sand bags (Figures 4.55 and 4.56); sand envelopes and/or sand sausages (sand filled geotextile containers); a rock wall; or a sheet pile wall (Figure 4.50). In extreme cases, dewatering equipment can also be installed. Sheet piles also have the advantage of reducing seepage of water into the working area. Timely maintenance of such a temporary structure is important.

  A cofferdam only consisting of sand may be feasible if the works area is high up the beach so that wave run-up rarely reaches the cofferdam. Higher tidal levels may result in wave run-up reaching the cofferdam more frequently. Waves reaching the cofferdam regularly will quickly erode an unprotected cofferdam and as such, continuous maintenance will probably be necessary by placing new sand on the cofferdam as fast as the sea takes it away.

With regard to the use of sand bags, sand envelopes and sand sausages (Figure 4.55 and 4.56), it is recommended to apply sand bags. Note that the purpose of the sand bags here is to provide protection over the short term. Refer to Section 4.2.3 for the use of sand bags over the medium to long term. The sand bags are 1 m^3 bulk bags with duffel tops (Figures 4.55 and 4.56) that can be bought cheaply second hand. Geo-synthetic sand-filled containers (large sand bags; Luger *et al* (2006)) can also be used but they are significantly more expensive than bulk bags. It is **not** recommended to: (1) use sand bags smaller than 1 m^3; or (2) to place bulk sand bags on top of each other. These bulk bags can easily be filled, transported, placed and removed with an excavator. The lower rows of bags on the slope of the cofferdam to be protected, should be placed against each other and directly on the sand (Figure 4.55). Soon after placement, these bags will sink approximately 0.3 m into the sand, where after they will be very stable. It is recommended that the bags be closed using cable ties and that the lifting loops be tied together after placement (Figure 4.56). In this way, the bags will not lose their sand even if they topple over. If space allows it, the outer slope (slope towards the sea) of the cofferdam should be flatter so that the sand bags will be more stable. To remove the bags, the bag material can be cut and the material be pulled out (or picked up later along the shoreline). More information can be found in Theron *et al* (1994 and 1999) and Schoonees *et al* (1999).

Large stone and rock together with a geotextile can also be used effectively to protect the outer slopes of cofferdams. However, it is possible that the sea can transport the stone and rock from the cofferdam and deposit them all over the beach. This can restrict beach usage and be hazardous to beach users; thus, it must be ensured that the stone and rock will be stable. It is almost impossible to pick up all stone and rock that have been scattered over the beach. However, if the stormwater outlet is on a rocky shore or the sand thickness over bedrock is small, then the rock can be removed relatively easily (after completion of the construction as is normally required) even if the stone and rock have sunk into the sand. Flatter outer slopes of the cofferdam will be more stable than steeper slopes but require more space and stone and/or rock.



From CSIR (1997)

Figure 4.55: Bulk bags and sand envelopes

Photos: A Theron

Figure 4.56: A building temporarily protected by sand bags (Southern Cape Coast)

For outfalls with invert levels above high water spring tide (HWST; that is, in Categories 1 and 2; refer to Lindford (2015) in Schoonees and Theron (2016): Section 2.2.2), construction would usually be in the dry (normal land construction methods). For Category 2 and 3 outfalls (discharging between HWST and mean sea level (MSL)) a cofferdam will normally be required. For outfalls discharging between MSL and lowest astronomical tide (LAT), a cofferdam will probably be necessary. For Category 4 outlets (invert level below LAT), a coffer dam and a jetty for deployment of a crane above the wave action limit will almost certainly be required (as for large stormwater outfalls crossing the shoreline).

- Temporary structures like a jetty are normally designed for a 1 in 10 year event. This is considered a balance between the **risk of failure** and overdesign of a marine structure.

However, the contractor may wish to choose a relevant encounter probability such as the probability of a large storm to occur is less than (say) 10 % or 20 % and the lifetime of the structure and then use the accompanying return period (other than the 10 years mentioned above). More information about the encounter probability can be found in CEM (2006) and CIRIA, CUR, CETMEF (2007). The required height of a jetty should take into account the most extreme high water-level from factors like tides and storm surge, as well as the extreme wave height. This extreme wave height should take into account the highest significant wave height in the design storm; the maximum wave height in the spectrum during the peak of the storm (= approximately 1.8 x significant wave height (CIRIA, CUR, CETMEF, 2007)); and the part of the wave above the still-water level. Note that personnel and equipment on a jetty can be withdrawn to land (and safety) if a storm is predicted for the site. Wave predictions can be very valuable in this regard.

- As for other construction, **safety** is very important during marine construction. The public must be kept out of work areas. Furthermore, warning signs must be put up. It must be realised that changing water-levels, waves, currents and winds can result in dangerous conditions. Working in the water may require the use of divers requiring adherence to safety regulations and legislation.

The foregoing general guidelines for construction of small to medium coastal stormwater outlets should be adapted, taking into consideration site specific circumstances and limitations, the size of the outlet structure and the scope or magnitude of the project.

5. SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

5.1 General

This extended summary is meant for readers that need to obtain a reasonable overview of the whole report if they only read this chapter. Refer to the Executive Summary at the start for a brief summary.

Comprehensive manuals for (1) stormwater management on land; and (2) on coastal engineering have been published. However, only limited guidelines are available for coastal stormwater outlets.

This project on small to medium sized coastal outlets has been divided into two parts, namely:

- **Phase 1: Literature survey**. Guidelines available in world literature were compiled the applicability of these guidelines to South African conditions was investigated (Schoonees and Theron, 2016). It was found that existing guidelines were inadequate.
- **Phase 2: Design guidelines**. Existing guidelines for stormwater outlets are presented and augmented by additional guidelines focusing on the coastal factors influencing the design of stormwater outlets.

Therefore the aim of this report (Phase 2) is to compile a set of design guidelines for small and medium coastal stormwater outlets applicable to South African conditions (of which the outlets around Mossel Bay serve as an example). Furthermore, mitigation measures (widely called best management practices) for land factors have also been briefly addressed (refer to the Phase 1 report; Schoonees and Theron, 2016) because these practices affect the coastal design aspects. The land factors, which include the hydraulic and structural design of stormwater infrastructure and outlets, are not included in the scope of this study. Some recommendations on the construction of small stormwater outlets are also given. Furthermore, the most important applicable legislation is presented briefly.

The guidelines in this report are necessarily of a general nature. In certain cases, other measures may be more appropriate or more important based on local conditions and the characteristics of the particular site. IMESA, Stellenbosch University, IWEESU and the authors take no responsibility for the application of the guidelines in this report. A competent civil engineer needs to assess all factors and take full responsibility for his/her design and construction of a stormwater outlet.

5.2 Coastal processes and information for the design of stormwater outlets

Coastal structures should be designed by studying all available information regarding the geography, coastal dynamics, beach characteristics, wave regime, long-term shoreline evolution, eustatic rise in sea level, aeolian sediment dynamics and the characteristics of the foredune (should such a dune exist). The coastal processes that need to be considered and information required for design and construction include:

- Location of the site
- Bathymetry and topography
- Nature of shoreline and seabed
- Winds, waves and currents
- Seawater-levels
- Sediment transport: longshore, cross-shore, aeolian
- Environmental issues
- Effluents & water quality; circulation, dilution and dispersion
- Conflicting beach usages

Essential information for designing a coastal structure includes good topographical data of the shoreline and backshore areas, such as can be provided by means of, for example, a conventional topographical or LiDAR survey. Investigation of all these aspects aid the long-term sustainability of projects and help to keep the littoral active zone free from impacts due to unwisely sited infrastructure or developments. In terms of the data/inputs required for the design of coastal outlet structures, some of the most onerous requirements are: coastal topography, inshore wave conditions, historic shoreline changes, and potentially inshore bathymetry, although the coarser SAN bathymetry data is mostly sufficient. Potential sources or means of obtaining such data and information is provided in Chapter 3.

5.3 Guidelines

5.3.1 Beach usages, aesthetics and location

To reduce potential conflict with other beach usages, the outlet structure should be as small as possible, yet be functional. Public access along the beach must be maintained. Sharp edges and unnecessarily protruding structural elements (especially, metal bars, etc.) should be avoided to reduce possible injuries, especially if the structure is submerged at times. Conflict must also be avoided as far as possible with other "beneficial coastal zone uses": i.e. direct contact recreational activities (e.g. swimming), indirect contact recreational activities (e.g. sunbathing), collection of filter feeders (e.g. shellfish), marine protected areas, port and industrial facilities, mariculture (e.g. abalone farms), and undeveloped and pristine coastal environments. The discharge from stormwater outlets is sometimes highly polluted and can result in human health or environmental issues if the discharge location is too near any of the before mentioned "beneficial coastal zone uses".

Some typical site selection criteria for a small coastal outlet are summarised in Table 4.1, while some guidance regarding elevation of the end of the outlet structure is provided in Section 4.2.1. The particular circumstances and local site characteristics have to be considered in each case. The design approach regarding location, layout and detailed design, should be to "work with nature", and in terms of environmental engineering design, it is also recommended to incorporate and hide the outlet into coastal structures such as jetties, piers, promenades, look-out platforms, seawalls, groynes, revetments, breakwaters, boat ramps, etc. An outlet should be as unobtrusive as possible by making it blend in with the surrounding area. For example, by: (1) covering the outlet with irregularly placed rock; (2) burying it where feasible; (3) avoid regular and angular shapes; and (4) do not use bright colours. Because of marine growth, there is no point in colouring or painting an outlet below about the high-tide mark.

Numerous outlets over a short longshore length of shoreline will make the coastal zone to appear artificial. Where possible, different smaller outlets should be combined into a larger outlet for hydraulic and cost reasons. The same principle applies for aesthetic reasons. Outlets can be combined with coastal structures such as breakwaters, groynes and piers. A coastal structure that has more than one function will not only save cost but will, in total, have a reduced visual impact compared with more than one structure. A landscape architect can be consulted to make the outlet(s) less obtrusive.

An outlet should, wherever possible, not obstruct people from using the beach. Means to safely walk across or around an outlet should be provided where necessary. This crossing point can be higher up the beach so that the railing at the crossing point will not be subjected to wave forces. Note that kelp, seagrass and other material can become entangled on railing, thus resulting in large forces. By providing a small, raised platform as a lookout area at the crossing point, the structure of the crossing point can have a dual function.

5.3.2 Shoreline changes and scour

If a significant long-term eroding trend is apparent in the shoreline location, a conservative estimate of the erosion rate is extrapolated for the design lifetime, usually 50 (or 100) years. This constitutes the horizontal distance that the outlet should ideally be placed landward of present sandy shoreline locations to provide for long-term shoreline erosion trends. Fortunately, there are only a few areas along the SA coast (e.g., Durban Bluff and north of the Port of Richards Bay) where the shoreline is known to be progressively receding. Typical short-term horizontal shoreline variations in exposed South African beach locations are of the order of 30 m to 80 m (to perhaps 90 m in the most extreme cases), and progressively less than 30 m as the location becomes more sheltered from direct wave impact. This constitutes the horizontal distance that the outlet should ideally be placed landward of the present shoreline location to provide for short-term shoreline erosion (in addition to providing for possible long-term recession).

Typical vertical variations in very exposed South African beach locations are of the order of 2 m (to perhaps 6 m in the most extreme cases, e.g. when a dune suffers major storm erosion), and progressively less than 2 m as the location becomes less exposed (more sheltered from direct wave impact). Hard structures, including stormwater outlets that are located on sandy shorelines and positioned within reach of the sea, are also subject to scouring of the toe of the structure or underscouring of the foundations. Outlets structures (and especially the foundations) must be designed to cope with the maximum expected depth of such scouring in addition to the natural vertical beach profile changes discussed above. In conjunction, the structure must also be designed for structural integrity, thus being able to span between adjacent supports without relying on any support from the beach sand in between spans.

A number of options exist to combat underscouring of stormwater outlets due to wave action. These options include locating (or in some instances relocating existing) outlets higher up on the beach away from the sea, or using rigid or flexible protection for the outlet structure as described in Sections 4.2.2 and 4.2.3 (and further summarised in Section 5.3.3). *In any event, the outlet should have adequately dimensioned and well-constructed back and side walls and a well-founded floor. Good quality materials should also be used* as the structures are located in the aggressive coastal environment (low grade concrete should be avoided). These measures will protect the outlet and prevent damage to the concrete itself. In some instances protection of pipes (encasing) to increase structural strength and durability may be an option, but only if properly designed, including adequate foundation support and scour protection where required.

5.3.3 Protection of the outlet

General

An outlet structure can be protected by different permanent methods, namely: (1) concrete structures; (2) rock protection; (3) sand bags; (4) grout and block mattresses; (5) gabions and Reno mattresses; and (6) other methods.

Concrete structures

Concrete of a high quality and strength and low permeability should be used for marine works; for example, to limit corrosion of reinforcing steel. Adequate cover and crack control for the concrete (including painting with a bitumen mixture) are very important.

By locating the stormwater outlet structure in the lee of rock, wave forces and cost can be reduced significantly. Generally, concrete outlets need a firm, stable foundation. Thus, use the beach profile and nature of the seabed to reduce cost. On a rocky or a mixed rocky and sandy coast, the pipeline can be placed in a gully, fixed to the rock with dowels and/or concreted over to cover it.

Rock levels above and under the sediment are required. Furthermore, weathering of the bedrock needs to be assessed in foundation design. It is recommended to anchor marine structures firmly to bedrock where possible. If the bedrock is located deeper down, end bearing piles can be considered. If the bedrock is below the expected extreme scour level, then a stone screed layer can be place on top of the bedrock in order to obtain a level surface. For shallower bedrock, concrete placed over dowels (anchored to the seabed) should be used. If it is not feasible to use the bedrock as foundation, then other protection measures such as by placing rock protection should be considered.

Rock protection

General

Because a rock revetment is usually the cheapest option to protect the stilling basin of a small outlet from wave and current action, other options should be compared with a rock revetment in terms of functionality, lifetime and cost. Figure 4.23 illustrates a typical cross-section of a rock revetment. The design wave and water-level condition(s) at the toe of the rock revetment can be determined based on depth-limited conditions and/or by wave transformation modelling.

Rock properties

Equations for basic rock properties such as median mass and rock layer thickness have been presented. The minimum thickness of gravel, stone or rock layers is 300 mm. Design methods are applicable for rock with standard narrow gradings. If rock has be used that does not adhere to a narrow grading, it means that special physical model tests are essential to substantiate a design.

Rock revetments should have a minimum crest width of 3 to 4 times the nominal rock diameter. Typical rock slopes for revetments and breakwaters range from 1: 1.5 to 1: 3. Slopes steeper than 1: 1.5 should not be used for revetments using rock as armour. The flatter the slope of a rock revetment, the more stable the rock will be. However, smaller, but considerably more rock, will be required. The relationships between rock sizes in adjacent layers have been given; rock in a lower layer should be 10 to 15 times smaller than the rock in the layer above. It is recommended to use the factor 10 in the abovementioned equations because it is somewhat conservative. If the rock placed during construction deviates slightly from the specified rock, the rock should still be within acceptable limits.

Design approach

For small coastal outlets, it is recommended to design rock slopes to be statically stable. Failure modes for a rock revetment have been discussed. The design approach for a rock revetment to protect a small coastal outlet can be summarised as follows:

- Decide on the usage of the area in the lee of the revetment.
- Choose rock as armour and the revetment slope; usually 1: 1.5.
- Calculate the wave run-up and/or tolerable overtopping rate. For small coastal outlets, very little overtopping should be allowed so as not to damage infrastructure.
- Determine the crest height based on the wave run-up and/or the allowable overtopping.
- Choose the wave permeability (P).
- Choose the allowable damage. Use S_d= 2 (“start of damage”) so as to minimise maintenance.
- Compute the median size of the armour rock.
- Calculate the thickness of the armour layer based on the size of the armour rock.

- Determine size and thickness of the underlayer, filter layer and core rock.
- Design the toe of the revetment.
- Decide on whether a mass capping is required based on access to the outlet by crane or truck for maintenance. A crown wall is expensive and normally it should not be necessary.
- Construction methods. End tipping is usually carried out to deliver rock for a revetment.
- The bearing capacity of the seabed soil must be sufficient to support the revetment. It may be necessary to excavate soil to reach a better foundation and/or gravel to fill the excavated soil.
- Rock and other construction material must be locally available. For rock, it is necessary to consider sizes and whether sufficient quantities can be obtained.

Wave run-up

Equations for computing the wave run-up have been presented for impermeable and permeable rock slopes.

Design method for armour rock size in shallow water

The Van Gent equation (CIRIA, CUR, CETMEF, 2007) can be used to calculate the required size of the armour rock on a straight section of a revetment in **shallow water**. In this equation, the $D_{n50,core}/D_{n50}$ factor= 0 if a geotextile is applied (as for a revetment). If the outlet is in deeper water, then CIRIA, CUR, CETMEF (2007) should be consulted for alternative design formulae. It also needs to be checked that the wave height used is possible; that is, the depth-limited scenario. Unless a detailed storm analysis has been carried out, the maximum number of waves should be assumed; namely, N= 7500.

Toe design

Usually, rock toes are designed to be flexible; that is, limited rock movement is allowed whereby rock falls into the start of a scour hole but without this rock movement affecting the armour rock layer. A typical cross-section of a rock toe that can been built on a rocky seabed is illustrated in Figure 4.26. The toe is anchored in an excavated trench. Another possibility is to construct a toe beam that is anchored to the seabed by means of piles. A special toe has been proposed for sandy seabeds with a severe scour potential (Figure 4.27). Typically, sandy South African shorelines have a severe scour potential.

Geotextiles

Usually two types of geotextiles (filter fabrics) are applied in the marine environment, namely: (1) woven geotextiles; and (2) non-woven (or needle-punched) geotextiles. The filtration function of geotextiles is most important for revetments. The water permeability has to be maintained during the life of the revetment. A criterion for a geotextile to be sand tight has been specified.

Soil has to be properly prepared by removing loose objects so that the ground is even and that the geotextile can lie flat. Note that the dumping of large rocks and stones (> say, 50 kg) directly onto a geotextile can damage it. Therefore, a filter layer (s) may be required on top of the geotextile to protect it against falling stones and rocks. Usually, the geotextile is rolled down the slope to place it. It may be necessary to initially place stones or a small heap of gravel on the geotextile to keep it in position until the correct filter layer can be placed on top of it. Different seams (joints) for geotextiles have been listed. It is recommended that seams be used to connect different strips of geotextiles. Overlapping without stitching is not recommended (Rankilor, 1994). A hand-held sowing machine can be applied on site to do the stitching.

It is recommended that specifications about different characteristics and suitability of the geotextile for the application be obtained from manufacturers. Characteristics like the mass of geotextile per unit area, pore sizes, water permeability, puncture resistance, tensile strength, stretching during tensile loads, etc. should be provided (Ingold and Miller, 1988).

General comments on rock design

The layout of the revetment has to be considered carefully to assess possible vulnerable sections and to evaluate the 3-dimensional effects of the revetment (including the effect on the adjacent shoreline). Vulnerable sections of the revetment are corners of the revetment and transition areas. Smooth transition areas are required. It may be necessary to increase the rock size at bends and corners; or, at least, use the larger rock in the grading at the corners.

Sand bags

Large sand bags filled with sand, grout or concrete have been applied all over the world and in South Africa (Langebaan and KwaZulu-Natal) to combat coastal erosion. Of the measures using geotextile sand-filled containers, sand bags are the best suited to protect small coastal stormwater outlets in the long term. For short-term usage, refer to Section 4.3.2.

The most important failure mechanisms for sand bag protection have been listed. The design of a sand bag revetment must include the durability of the geotextile and the stability of the structure. Regarding *durability*, a sand bag can, clearly, only be effective as shore protection as long as its contents remain in the bag. A significant advantage of sand bags is that the material (sand) for fill is available at the site. There are a number of ways in which the sand fill can be lost from the bags, namely:

- The bags can tear or burst open. Partially filled bags are particularly at risk.
- Abrasion of the bag fabric by sediment, debris, boats and pedestrians can cause failures.
- Despite added inhibitors, the ultra-violet light of the sun can degrade the fabric of the bags.
- Vandals can cut and damage the bags.

It is recommended to use a double layer geotextile for the bags (Hornsey et al, 2009). Apart from being stronger and resistant against abrasion, the outer layer protects the inner layer against ultra-violet light. Furthermore, sand grains are trapped in the outer layer, which makes it considerably more difficult to cut with a knife and thus, reduces vandalism. The material used for EnviroRock by Kaytech (2017) in South Africa have similar properties. Saathoff et al (2007) and Rankilor (1994) give a good description of the required characteristics and tests to be conducted of the geotextile to be used for making sand bags. Kaytech (2017) lists the characteristics and tests done for EnviroRock.

A typical cross-section of a sand bag revetment is depicted in Figure 4.35. It is recommended to use a double layer of sandbags and to place the sand bags with their long axes perpendicular to the shoreline. Sand bags of 2.5 m^3 are recommended; however, do not use bags smaller than 1 m^3 or bags much bigger than 2.5 m^3. Sand bags should be filled to capacity, but not be overfilled. The bags should be closed by stitching. Sand bags should not to be damaged during construction. Old conveyor belts can be used to protect the placed sand bags from construction vehicles.

Excessive wave overtopping can damage infrastructure and even be dangerous for pedestrians and cars (EurOtop, 2007). It is recommended that the sand bag revetment be built high enough to prevent significant wave overtopping. Nevertheless, attention should be given to the possible lateral flow of overtopped water during conditions exceeding the design condition. For example, placing a few bags in rows on top of the structure to form little groynes or spurs at regular intervals alongshore, should limit lateral flow and allow the overtopped water to drain through the structure.

The best slope based on present research is 1: 1 (45 degrees; Figure 4.35). Figure 4.36 contains a design graph for 2.5 m^3 sand bags for different slopes (Hornsey et al, 2009). It is not recommended to use slopes steeper than 1: 1. It is further recommended that the design procedure by Recio (2007) and Recio and Oumeraci (2008) be used for the design of sand bag revetments. It is recommended that two or more sand bags as a flexible toe be placed directly on the sand seawards of the bottom of the slope (additional to the toe shown in Figure 4.35). The orientation of these bags should be alongshore.

The planshape and 3-dimensional aspects of a sand bag revetment should be considered carefully. The sand bag protection should enclose the spillway structure; or else, the revetment should continue sufficiently far landwards so that the outlet is not attacked by waves from behind (the land side). There should be no sharp corners in the sand bag revetment.

It is important to note that attention should be given to details of a sand bag revetment because experience has shown that failure can occur because of small issues regarding the design. It is recommended that physical model tests be conducted to optimise the design of a sand bag revetment.

Concrete block and grout mattresses

Concrete block mattresses are solid or perforated concrete blocks that are usually connected by means of wire rope or polyester rope. For marine applications, strong, polyester rope is preferred. These mattresses are placed on a geotextile that covers the ground, allows water flow through the mattress and prevents or limits soil leaching out from underneath the geotextile. Sometimes the concrete blocks are cast directly onto the geotextile with fabric loops. Concrete block mattresses are flexible so that the mattresses can follow the ground contours and allow for settlement. It is customary to fix the mattresses on steeper slopes by using stakes hit into the ground. Perforated blocks can either be filled with gravel or coarse sand to restrict vegetation growth or filled with topsoil to stimulate plant growth. Pipelines are regularly protected by placing concrete mattresses over them.

A *grout mattress* consists of two connected geotextile sheets, one on top of the other but closed at the edges. At regular intervals in both directions, these two sheets are woven together in spots, thus forming permeable weep holes. After preparation of the soil, the grout mattress is laid out and sometimes fixed to the soil by means of stakes (especially on steeper slopes). The geotextile sheets act as formwork when the geotextile sheets are pumped full with grout from the lower elevations of the geotextiles sheets to the higher points. The weep holes remain intact, allowing water flow back and forth. Excess grout should not clog the weep holes. The weep holes that force the grout into a rounded rectangular shapes throughout the mattress, also make the grout mattress flexible. Clearly, exposure to ultra-violet light from the sun will result in degradation of the geotextile. Therefore, the grout mattresses should preferably be covered.

Proper design of concrete block and grout mattresses must include the following:

- Ensure that the mattresses will be stable during extreme conditions.
- The geotextile must remain functional during its design life. The geotextile must be permeable; not allow the leaching out of soil from underneath the mattresses; and not tear.
- The applicability of construction methods has to be considered. Practical issues to be evaluated include joints and seams of the geotextile.

The Rankilor (1994) graph that can be used for conceptual design of *concrete block mattresses* has been presented. This graph enables the determination of the required thickness of the mattress. The slope at which the concrete mattress is placed must be flatter than 1: 2.1 (25 degrees) and that a maximum wave height 1.2 m applies (Rankilor (1994). Furthermore, a thick geotextile and/or a granular underlayer should be used to reduce hydraulic pressure underneath the grout mattress (Rankilor (1994).

The Rankilor (1994) conceptual design method for *grout mattresses* has been discussed. Rankilor (1994) states that a thick geotextile and/or a granular underlayer should be used to reduce hydraulic pressure underneath the grout mattress. The design methods by Klein Breteler and Pilarczyk (1998), Rankilor (1994) and PIANC (1992, 2011) can be applied for the hydraulic stability of mattresses. Pilarczyk (2000) provides further information on the use of geotextiles. Klein Breteler and Pilarczyk (1998) stressed the importance of the geotechnical properties of the foundation soil. In evaluating the stability of the mattresses against sliding and possible lifting of the mattresses, the following geotechnical conditions should be adhered to: (1) elastic storage; (2) softening (liquefaction) of the foundation soil; and (3) a drop in the water-level.

In general, the requirements for a suitable geotextile should at least include the following (Ingold and Miller (1988), Rankilor (1994), Pilarczyk (2000) and PIANC (2011)):

- The geotextile should be very permeable to water (especially for wave up- and downrush).
- The geotextile must have a high strength and should not stretch excessively.
- Sediment should be contained underneath the geotextile and should not wash out.

Note that the use of grout mattresses in the coastal zone in shallow water is restricted to maximum significant wave height less than 1 m. For higher waves, the required thickness of the mattress becomes too large and then either rock or armour units is a better solution.

Special attention must be given to fixing the mattresses to the outlet structure and also to the edges of the mattresses. These edges should be buried so that they are more stable whilst also making it more difficult for thieves and vandals to access the edges of the mattresses. In the case of concrete block mattresses without an inbuilt geotextile, the edges of the mattresses should be without geotextile. These strips of mattresses without geotextile should be far enough from the outlet structure so that the stability of the outlet structures is not detrimentally influenced by the local sinking of the mattresses along the edges.

Geotextile normally come in rolls with a given width. To cover the required area, different strips of geotextile have to be properly joined by either a strong, sown joint or by gluing the strips together. Overlapping of geotextile strips should be avoided if possible. Industrial sewing machines are available to easily carry out joining in the field. Where possible, the orientation of the geotextile strips should be at right angles to the shoreline.

For grout mattresses, care have to be taken that grout is not lost through the geotextile. Furthermore, the geotextile should be filled to capacity throughout the mattresses. This will ensure stability and prevent flapping of a half-filled mattress.

Gabions and Reno mattresses

Experience in Southern Africa has clearly indicated that the life of gabions in the sea is limited because of abrasion, corrosion, failure of the wire and the stone being washed out. The use of gabions and Reno mattresses in the sea is generally **not** recommended. However, if gabions and Reno mattresses are very seldom (less than once a year) exposed to wave and current action for a short time and buried for the rest of the period, they can be used in the coastal environment. Nevertheless, it should in these instances still be evaluated whether loose rock will not be a better option.

Note that gabions can be very handy to place a filter layer of rock to drain water away. That is, a gabion lined with a geotextile and filled with gravel can easily be placed around a structure. This means that the gabion is used for: (1) ease of construction (and its strength is not required after deployment); and (2) to limit the volume of gravel.

Other methods

Placing the stilling basin of the outlet very high up on the beach may make it unnecessary to protect the outlet against wave action. The best management practices described in Schoonees and Theron (2016) such as exfiltration, should be used to reduce or eliminate any outflow, even during floods. Typical consequences of an outlet discharging landwards of wave action, include a pool of stagnant water at the end of the outlet and potential aeolian sand transport problems.

By building a large pier or jetty to support the outlet above any wave action, is another possibility. Another option is to incorporate the whole outlet in a groyne, seawall or breakwater. These options are possible, but the abovementioned structures are very expensive and most probably only financially feasible if the structure are built for other purposes (rather than for the stormwater outlet).

5.3.4 Water quality

General

In general, prevention is better (and cheaper) than cure. The following guidelines and recommendations have been presented in Schoonees and Theron (2016) in this respect, namely:

- Pollution associated with stormwater outlets relates to: (1) pollution in the sea near the outlet; (2) stagnant pools formed at the seaward end of the outlet; and (3) debris and litter. All of these problems are ideally alleviated by following best management practices on land.
- Ponding of runoff at the terminal end of the outlet can be prevented or reduced by means of effective outlet structures or by incorporating infiltration components in the outlet system.
- Similarly, litter generated inland should rather be dealt with in the terrestrial domain. Litter traps or mechanisms require significant maintenance to prevent blockages or overspill of litter and are also susceptible to vandalism and theft of metal components.
- Street cleaning is cost effective to remove heavy metals and pesticides close to the source of contamination rather than requiring treatment at the receiving end.
- Exfiltration involves the extraction of a part of the stormwater flow and letting the water infiltrate the soil by means of a perforated pipe and/or a trench. The first flush is usually treated in this way. Exfiltration is also effective to decrease pollution levels, recharge aquifers and to limit the underground intrusion of seawater towards land.

Dispersion and dilution

If the contamination of stormwater cannot be prevented, one or more of the following measures can be taken, namely:

- Ideally, the **effluent should be treate**d to acceptable water quality standards before discharging into the coastal zone. However, this option is very expensive and usually requires continuous operation.
- **Keep people and animals** away from the impact zone. A distance of at least 100 m alongshore on either side of the discharge point should be regarded as the impact zone unless detailed studies have more accurately indicated the extent of the impact zone. Management measures in this category include:
    - Put up information boards telling beach users to warn them against bathing, swimming, water sports and the collection of mussels, oysters and other filter feeders.
    - Police these impact zones by means of lifesavers or other beach personnel.
    - Close parts of the beaches in during times of first flushes.
    - Monitoring of water quality is required by analysing water samples on a regular basis.
- The stormwater outlet must be designed so maximise mixing and dilution and to minimise the impact zone. If possible, a stormwater outlet should not be placed on a bathing beach or alternatively, that the impact zone would be away from the swimming area.

5.3.5 Applicable legislation

The main legislation applicable are the following:

- National Environmental Management Amendment Act (#62 of 2008), the updated EIA Regulations (2014), and latest (2017) amendments.
- Integrated Coastal Management Act (ICM Act #24 of 2008) and ICM Amendment (#36 of 2014)
- Local Government Act: Municipal Systems (2000).
- National Water Act (1998)

Policies on disposal of waste water, as well as other legislation that may be applicable depending on the specific site, are all listed in Section 4.2.6.

5.4 Construction guidelines

Wave conditions are generally severe around the South African shoreline. These conditions make working from the sea difficult and expensive. Furthermore, the duration of calm weather is limited. As a result, small coastal outlets will almost exclusively be constructed using access from land. Thus, either: (1) an embankment and/or a cofferdam; or (2) a jetty with or without a cofferdam will be used.

The following general guidelines for construction of small to medium coastal stormwater outlets (Schoonees and Theron, 2016) should be considered, namely:

- **Use favourable weather optimally** by limiting the construction period in the sea. Therefore, all equipment and material must be available and stored close to the construction site. Furthermore, precast concrete elements should be used where possible. In this way, construction can progress quickly during calm and favourable sea and weather conditions. Construction can follow the water-level down as the tidal water-level starts dropping towards low-water spring tide (LWST). Note that for about 2 hours before and 2 hours after low tide, construction above the low tide level can usually be undertaken with little interference from wave and current action and quickset cement can set.

- **Concrete** can be mixed at the work area or supplied by concrete trucks. Concrete can also be placed by using a bucket which is swung by a crane. Alternatively, concrete can be pumped from land to an inaccessible working area by supporting the concrete supply pipeline on scaffolding. Columns or supports for outlet structures can be fixed to a rocky bottom by: (1) placing a concrete manhole ring on the seabed; (2) excavating the sand inside of the ring; (3) drilling holes into the bedrock; and (4) fixing dowels with epoxy in these holes. The concrete ring acts as formwork and protection against wave action for quick set concrete. It is important to protect the upper surface of the concrete in the ring.

- A cofferdam or the abutment leading up to a jetty forms an obstruction to the longshore **transport of sand** at the site and therefore accretion will form on the updrift side and erosion on the downdrift side. This accretion can result in re-excavation of sand while the downdrift erosion may endanger the stability of embankments. Cross-shore sand transport will quickly erode a cofferdam and therefore, protection of a cofferdam is usually needed. Aeolian sand transport is usually only a nuisance during construction. If a trench is excavated for the stormwater outlet, then sand is usually transported into the trench at a considerable rate. Thus, it is recommended limit the inflow of sand from the sides and to close off the trench at the seaward end for as long as possible. A trench can be excavated by earthmoving equipment or by using a grab operated from a crane on a jetty.

- A temporary **cofferdam** can be: (1) a wall of sand; (2) a sand wall protected by sand bags or sand filled geotextile containers; (3) a rock wall; or (4) a sheet pile wall. Dewatering equipment can also be installed. Sheet piles will reduce the seepage of water into the working area. Timely maintenance of such a temporary structure is important.

A cofferdam consisting of only sand will normally only be feasible if the works area is high up the beach so that wave run-up rarely reaches the cofferdam.

It is recommended to apply sand bags (1 m^3 bulk bags) for short-term protection of a cofferdam. It is *not* recommended to: (1) use sand bags smaller than 1 m^3; or (2) to place bulk sand bags on top of each other. The lower rows of bags on the slope of the cofferdam to be protected, should be placed directly on the sand. It is recommended that the bags be closed using cable ties and that the lifting loops be tied together after placement. To remove the bags, the bag material can be cut and the material be pulled out (or picked up later along the shoreline).

Rock together with a geotextile can also be used effectively to protect the outer slopes of cofferdams. It must be ensured that the rock will be stable. Usually rock has to be removed after construction. This is easy on rocky or mixed rocky/sandy seabeds, but difficult on sandy beaches. Flatter outer slopes of the cofferdam will be more stable than steeper slopes but require more space and rock.

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