# SECTION 6:
# SCIENTIFIC AND ENGINEERING ASSESSMENT

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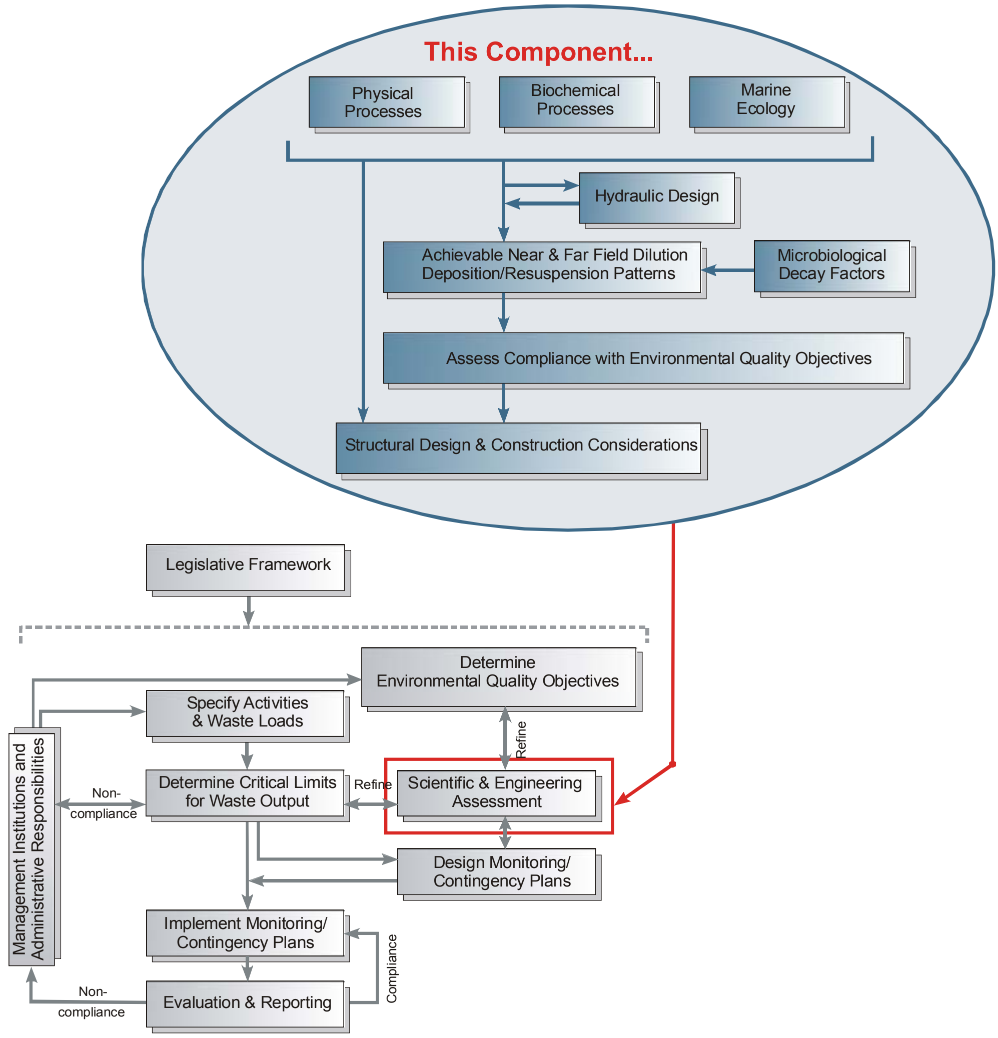


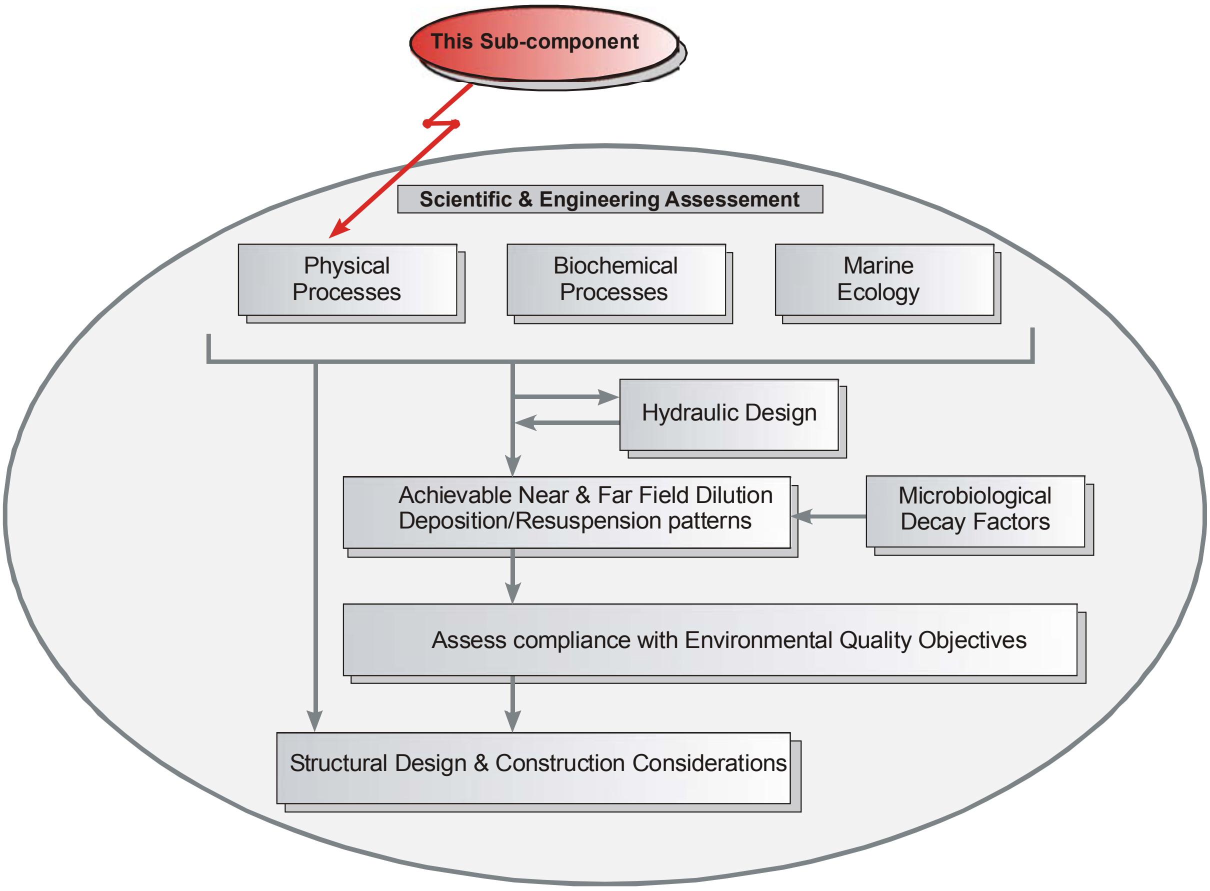

***PURPOSE:***

***The objective of this component is to refine the environmental quality objectives and establish if a waste disposal practice will comply with the environmental quality objectives set for a particular marine environment. This is achieved through the following steps:***

- ***Characterise physical and biogeochemical processes and the ecological functioning***
- ***Conduct the hydraulic design of the (offshore) outfall based on preliminarily required dilution estimates and taking into account the characteristics of the waste load (volume and composition)***
- ***Determine the achievable near and far field dilution and the deposition/re-suspension patterns of the proposed wastewater discharge. Also assess possible synergistic or cumulative effects, taking into account other anthropogenic influences in the study area.***
- ***Assess for compliance with environmental quality objectives. Where non-compliance with environmental quality objectives is evident, the hydraulic design of the outfall needs to be adjusted. Where compliance cannot be achieved through adjustment of the hydraulic design, either the critical limits for the waste load need to be reduced (e.g. through additional pre-treatment prior to discharge) or the environmental quality objectives need to be re-defined (i.e. only in extreme situations, e.g. in cases where economic/social gains justify such environmental sacrifice).***
- ***Define the structural design and the construction considerations of a marine outfall to meet requirements as determined by the above***

***Where applicable, a distinction is made between requirements for* Pre-assessments *and for* Detailed Investigations *(with reference to* Section 3.1: Licence Authorisation Process*).***

## 6.1 PHYSICAL PROCESSES




***PURPOSE:***

***The purpose of this component is to gain an understanding of the hydrodynamic and geophysical characteristics and processes in the study area by:***

- ***Producing a geo-referenced map of physical features, such as coastline configuration, topography, bathymetry and geological characteristics of the sea bottom***
- ***Assessing hydrodynamic processes (i.e. currents, water column stratification, water temperature variability and turbulence) for a range of environmental conditions (i.e. for various tides, waves, winds and air-sea fluxes as experienced in the marine environment).***

***An additional purpose is to provide a basis to be used in the hydraulic design, as well as the assessment of achievable near and far field dilutions and deposition/re-suspension patterns of particles.***

### 6.1.1 Overview

#### *i. Bathymetry*

A bathymetric survey is carried out to provide water depth contours, indicating the slope of the sea-floor and irregularities such as protruding reefs and offshore sandbars (Figure 6.1).

During a bathymetric survey, seawater depths at a large number of sites are determined using an echo-sounder operated from a survey boat. Depths are recorded as the boat travels at predetermined parallel lines perpendicular to the coast. In order to obtain a review of the area, lines spaced about 100 m apart are adequate whereas along the pipeline route itself, line spacing of 25 m or less is required. Corrections for tidal height and swell interference must be undertaken. The use of an integrative survey software package, providing accurate position fixing, bathymetric data and corrections for tide/swell, is necessary for the production of accurate contour plots and profiles.

***NOTE:***

***Anthropogenic perturbations of marine water and sediment quality are usually perceived to be the result of biogeochemical modifications. However, developments that modify circulation dynamics, such as harbour structures and marina developments, can also modify sediment and water quality characteristics of the marine environment.***

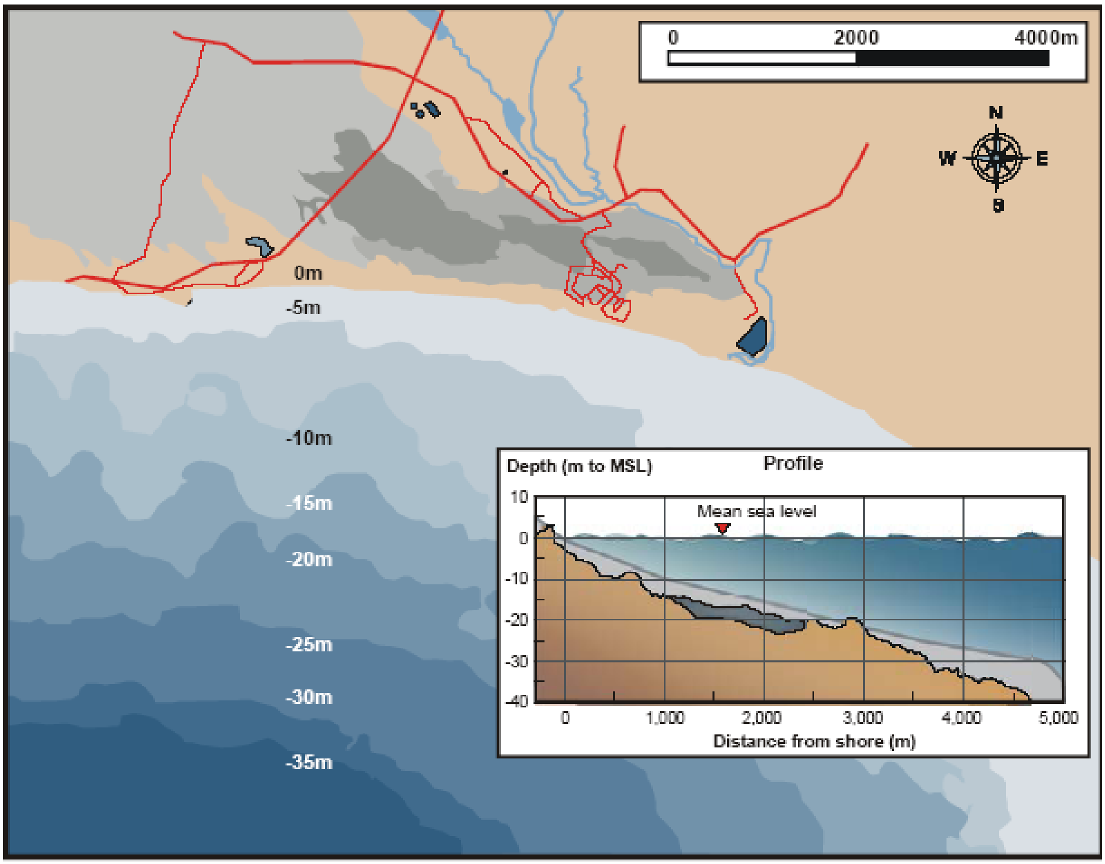



***FIGURE 6.1: Example of bathymetric contour map and typical profile***

### *ii. Seabed physiography*

The physiography of the seabed not only determines the method and cost of construction of the outfall but also indices the type of biological communities that may be present along the proposed outfall route based on the nature of the seabed type. For example, biological communities generally are more diverse on rocky reefs than on sandy seabeds. Reefs thus should be avoided if at all possible.

A side scan sonar survey is conducted to provide a graphic picture (‘aerial photo’) of the seabed in order to define the location of reefs, gravel and boulder beds and sandy areas, and the height and direction of sand waves and ripples as well as obstructions such as wrecks, anchors or other pipelines or cables (Figure 6.2).

A side scan sonar transmits a very narrow sound beam to either side of a sound source unit (normally called a “side scan fish”), towed at a predetermined depth behind a survey vessel. Projections and irregularities on the seabed, such as reefs, sand waves and wrecks, reflect the sound back to the sensing transducers on the unit and the magnitude of sound energy reflected is recorded on a continuous graph. As the unit is towed along a co-ordinated survey line, the seabed is mapped in a strip up to 250 m wide on either side of the unit’s path. Diver observations complement the sonar records by the ‘calibration’ of the side scan images with collected bed material. Probing by divers will confirm the depth of sand cover over rocky material. Underwater videos and photographs can also complement the survey, not only for the planners and designers, but also to enable the public to study real observations along the outfall route and conditions prior to, and after, construction of the outfall.

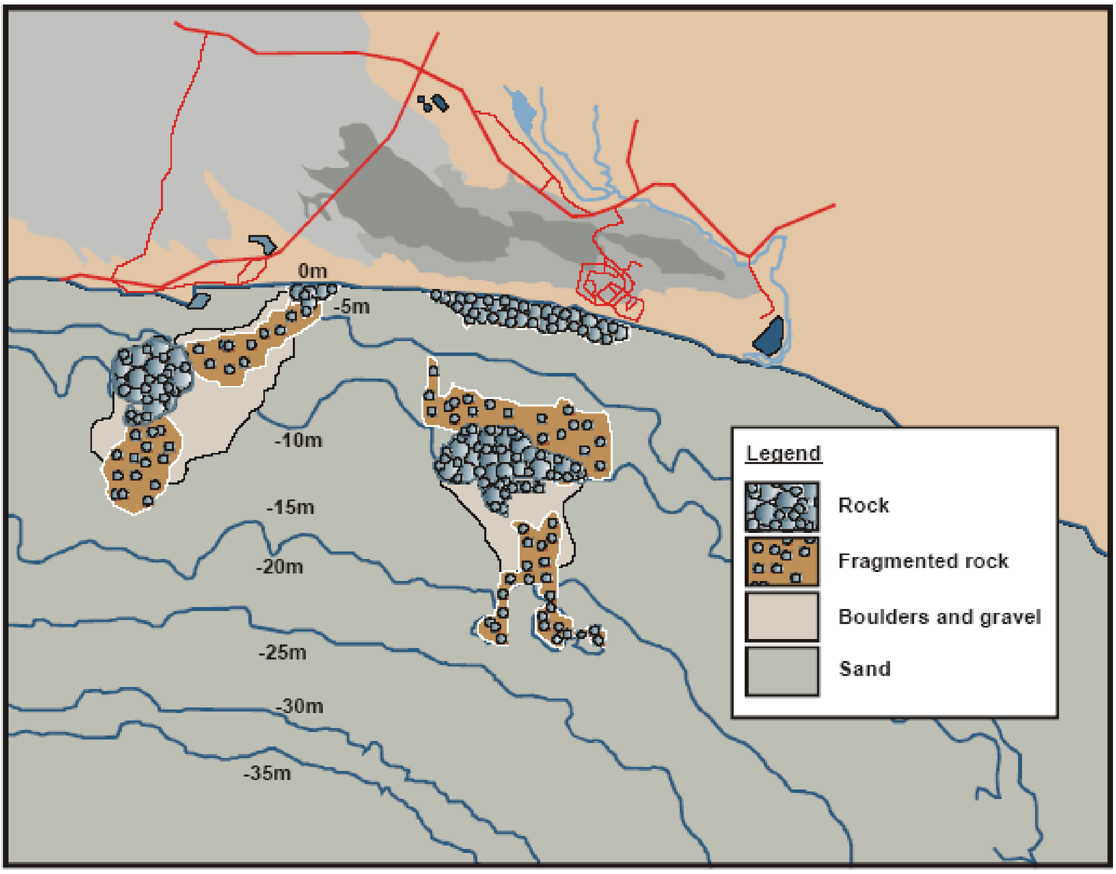



***FIGURE 6.2: Example of a side scan sonar survey providing a graphic picture of the seabed characteristics***

### *iii. Sub-seabed conditions*

An outfall may be required to be installed in a trench. The trench may have to be excavated several metres into the seabed at certain locations along its length. The method and cost of trenching will depend on the sub-seabed composition. Normally, seabed conditions beyond the surf zone are reasonably stable and consist of either sandy sediments and/or gravels, which can be excavated relatively easily, or rock, which is difficult and costly to excavate. It is possible for both extremes to be present along the length of an ocean outfall.

Seismic surveys are conducted to obtain information from beneath the sea-floor, using a sound source or transducer towed behind the survey vessel either on a surface float or below the surface. The transducer beams sound down through the seabed, and sub-bottom features reflect a fraction of the sound energy that are received on a hydrophone array that is towed behind the boat. The magnitude of the sound energy reflected depends on the interface between layers with different acoustic properties, including changes in rock type, degree of weathering or major fissures and interfaces (Figure 6.3). The received sound is then transmitted from the hydrophone to a plotter on the survey boat as the boat travels along a predetermined path.

Exploratory drilling is required to verify the results of the seismic survey. Further geotechnical investigation for the detailed design will be required:

- Soil analysis – classification, cohesive and shear strengths, angle of repose (internal friction), density properties
- Rock analysis – classification, hardness
- Seismic stability.

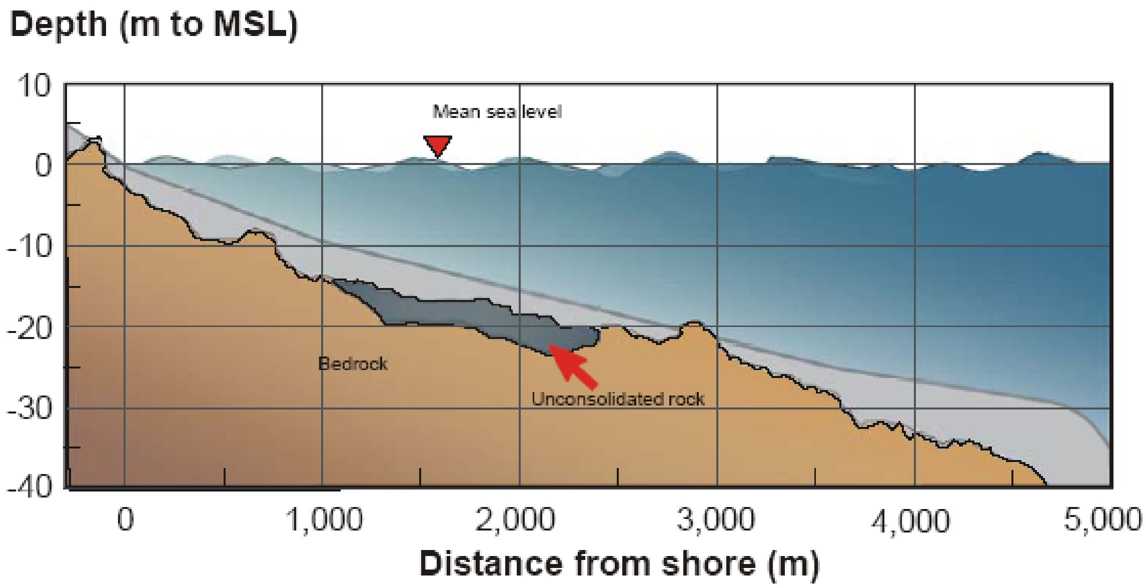



***FIGURE 6.3: Sub-bottom profile derived from a seismic trace***

### *iv. Sediment movement (erosion and sedimentation)*

In the surf zone, wave wave-induced turbulence and currents can result in large seasonal changes in the depth of sand, especially during storms. An outfall is normally installed in a trench extending across the shore and surf zone, which is then backfilled so that after completion, the outfall is not exposed. Thus the lowest depth of the sand across the beach and surf zone has to be determined, i.e. the profile resulting from storm erosion, so that the outfall can be securely installed below this level. An example of an 'envelope of variability', that is, the maximum (accretion) and minimum (erosion) levels over time is shown in Figure 6.4.

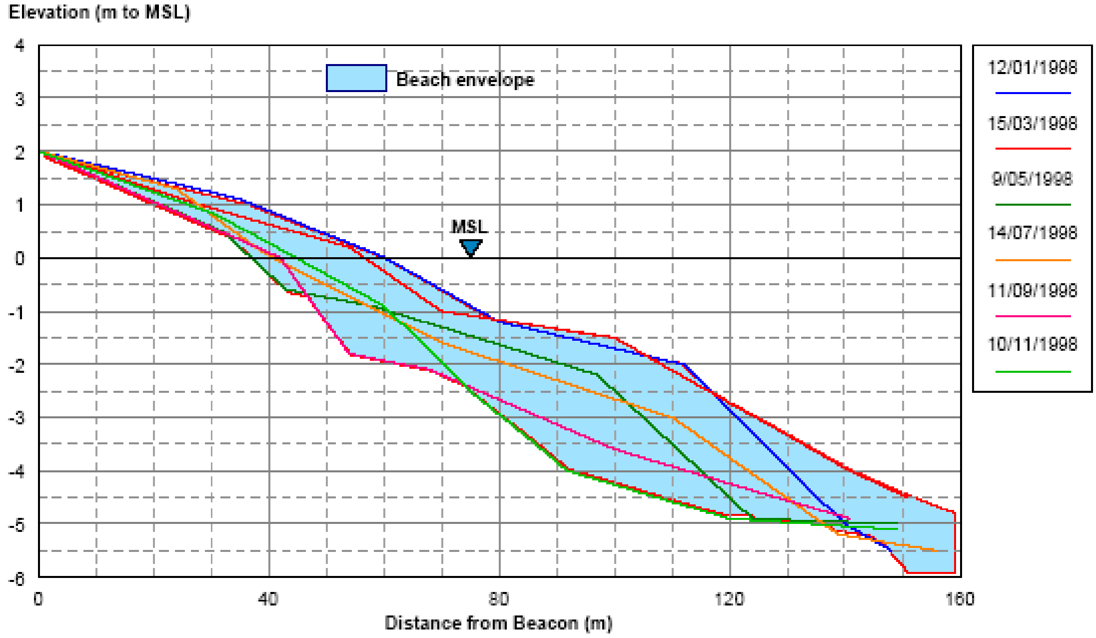



*FIGURE 6.4: Example of beach profile envelope, showing maximum and minimum profiles*

The degree (rate) of sediment transport and the probability of scouring have to be determined using historical data and supplemented with appropriate numerical modelling outputs, if required.

### *v. Waves*

Although wave data is not crucial with regard to the behaviour of the wastewater plume, it is critical for the initial deposition and redistribution of 'solid' phase particles. Wave energy is also the major consideration for the detailed (structural) design of an outfall with regard to the construction phase as well as to the forces to which exposed parts of the outfall will be subjected during its lifetime. Waves are also the crucial factor in determining the sediment dynamics in shallower water and in the shoreline geomorphology. In the case of surf zone discharges, the mixing, transport and dispersion of the wastewater plume are controlled by the breaking waves and the currents generated by waves approaching the shoreline.

Waves are defined in terms of:

| | |
|---|---|
| Wavelength ($\lambda$ in metres): | Distance between two wave crests or two wave troughs. |
| Wave height (H in metres): | Vertical measure between the bottom of a trough and the peak of the crest. |
| Wave period (T in seconds): | Time required for two crests to pass a fixed point. The period is also expressed as a frequency (Hz) = 1/T. |
| Wave celerity (c in m/s): | Where $c = (g\lambda/2\Pi)^{1/2}$ |

In the ocean, 'waves' are generated by numerous processes and can range from a period of 1 second (wind chop) to several days (lunar and solar tidal components). The type of waves that are significant for the structural design of an outfall, for the nearshore geomorphology and the hydrodynamics related to the transport and dispersion of the wastewater plume in shallow water are the wind-generated waves or swell, which are generated by the drag of the wind on the sea surface. Along the South African coastline, the wave/swell period is typically in the range of 8 to 14 seconds. The water itself does not proceed with the wave, but moves in circular orbits, clockwise in the direction of the wave (Figure 6.5). The diameter of the orbits reduces with depth until the orbital motion ceases at a depth equal to approximately half the wavelength. In water depths less than half the wavelength, the circular orbits becomes elliptical and wave action starts to act as shear forces on the seabed material or exposed structures such as pipelines. When the orbiting pattern collapses, breakers are formed. Smaller waves with long wavelengths on a gently graded beach slope will spill and lose energy during the run-up, whereas steeper (high, short-wavelength) waves will tend to plunge with a subsequent strong backwash, causing erosion.

The above description of waves refers to a regular wave train. However, the wave regime in the sea is highly irregular and it is very unlikely that two consecutive waves will have exactly the same characteristics with regard to height, length and period. Numerous waves (different frequencies), superimposed on one another, can also occur at the same time and location. Therefore, statistical measures/procedures have to be applied to a wave condition for a certain length of time in order to describe the wave conditions in terms of significant wave height and period. Typically, the statistical parameters to describe wave conditions relate to a recording period at least 20 minutes long:

| | |
|---|---|
| $H_S$ | = Significant wave height. The average height of the highest 1/3 of the waves in a recording period |
| $T_S$ | = Significant wave period. The average period of the highest 1/3 of the waves in a recording period |
| $H_{max}$ | = Maximum wave height. Maximum trough to crest height in a recording period |

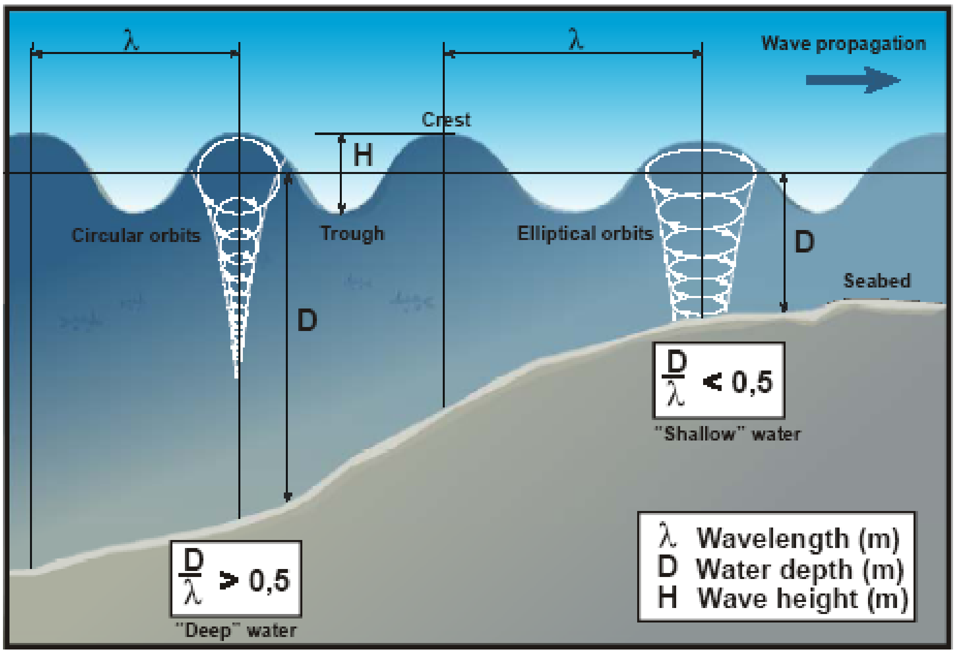



***FIGURE 6.5: Description of a wave***

Energy based parameters (Rossouw, 1984), which are the standard deviation of sea-surface elevation of all the data values obtained from the digital spectral analysis, are:

$H_{mo}$ = characteristic wave height (m)
= $4.m_o^{1/2}$ which is equivalent to $4.\sigma$

where $m_o$ = the area under the wave spectrum
$\sigma$ = square root of the variance

$T_p$ = Peak energy wave period (s)
= $1/f_p$ where $f_p$ is the peak energy frequency (Hz)

Confusion can arise when $H_S$ and $H_{Mo}$ values are ulitised. CERC (1984) describes this relationship between $H_S$ and $H_{Mo}$ with regard to the influence of water depth on the wave profile. In deep water, $H_S$ is approximately equal to $H_{Mo}$, but can be at least 30% greater in shallow water for breaking waves. A wave height H for a monochromatic wave train with the same energy as an irregular wave train with a height of $H_{Mo}$ is given as:

$H = H_{Mo}.2^{1/2}$

Wave data are typically presented as time-series plots of wave height and period, occurrences and exceedances for wave height and period and persistence of calms and storms. For the structural design of an outfall, maximum wave heights for return periods of 1, 10, 50 and 100 years must be determined with the associated wave periods and wave directions. The persistence curves are important for construction planning and scheduling, as the probability of seasonal durations of calm or storm conditions can be estimated from these curves. To determine seasonal variations, at least one full year's data from the site are needed for correlation with other available long-term data. A standard procedure for measuring wave height is the mooring of a wave measurement buoy (e.g. Waverider) at a representative location along the pipe route. The buoy samples the relative elevation of the sea surface on which it floats at 0.5-second intervals, normally for 20-minute periods each day (at six-hour intervals). The data are transmitted by the buoy to shore station receivers, where they are digitally recorded. Acoustic-Doppler-Current-Profilers (ADCP) are rapidly becoming the norm for measuring wave data as these instruments are also able to measure other parameters, such as temperature and current velocity and direction, through the water column.

Examples of wave data outputs required are provided in Figures 6.6a to 6.6e. Directional wave data can be displayed in a manner similar to the wind rose in Figure 6.11.

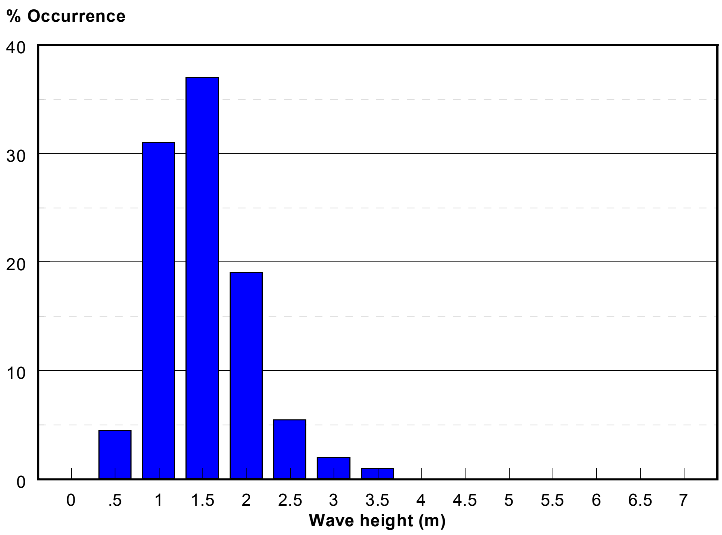



*FIGURE 6.6 a: Example: Annual wave height occurrence (%)*

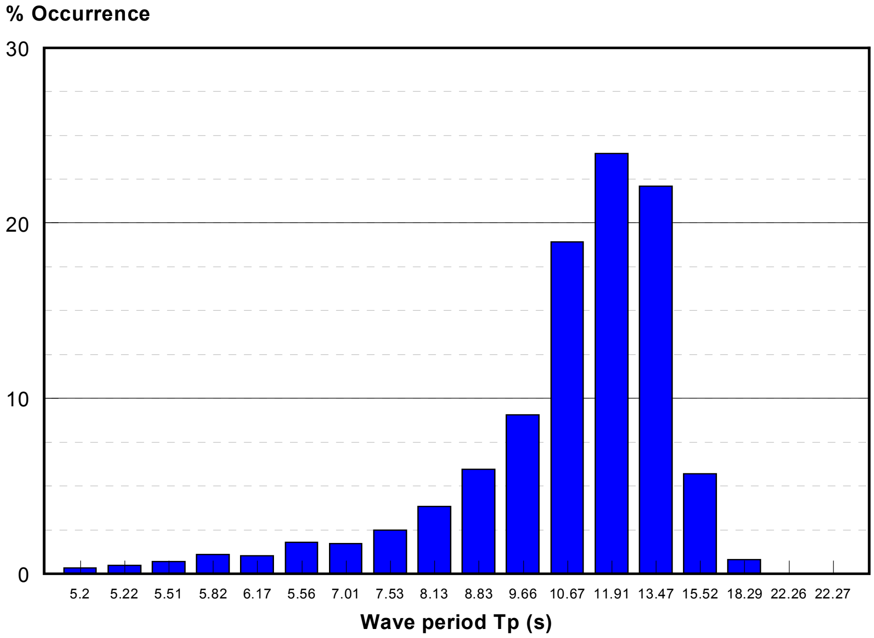



***FIGURE 6.6 b: Example: Annual wave period occurrence (%)***

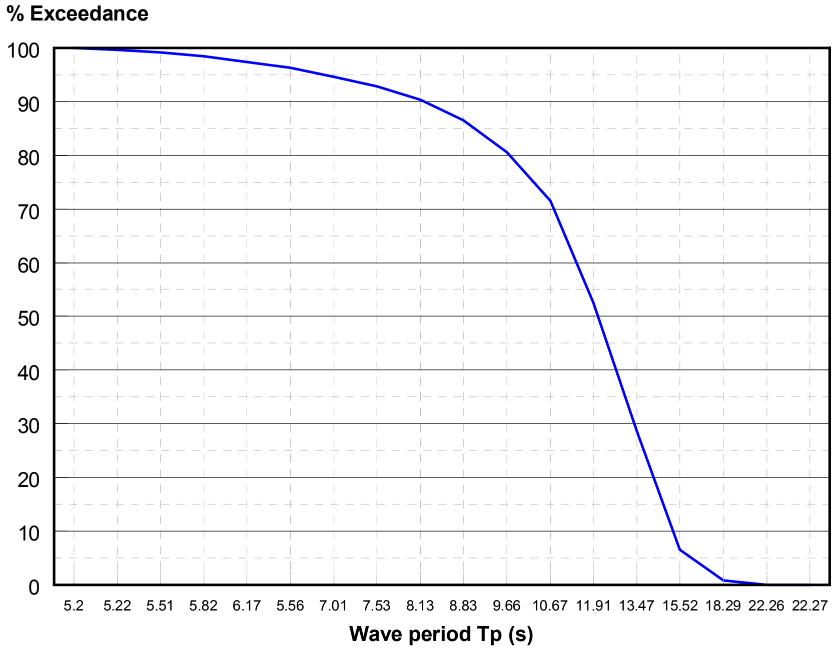



***FIGURE 6.6 c: Example: Annual wave period exceedance (%)***

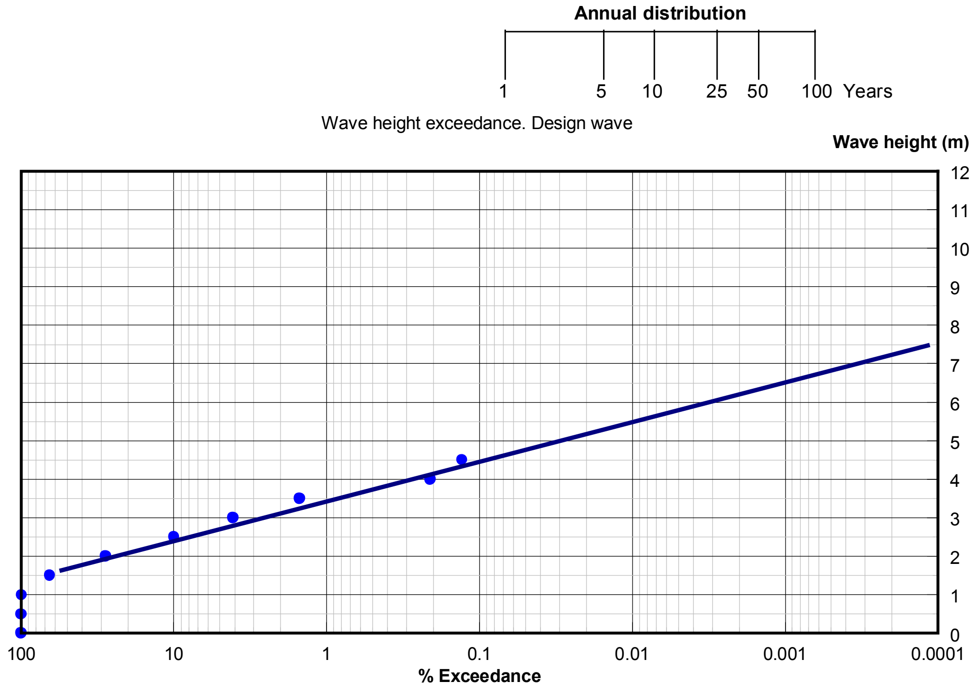



***FIGURE 6.6 d: Example: Design wave heights***

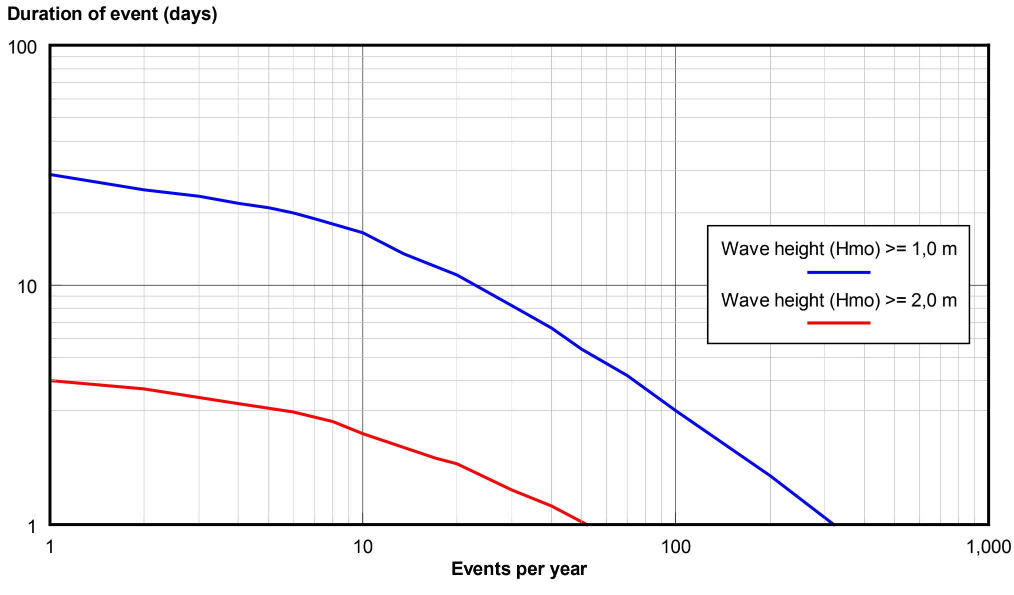



***FIGURE 6.6 e: Example: Persistence of storms***

A summary of all available wave data along the coast of South Africa up to 1984 was summarised by Rossouw (1984) and is listed in Table 6.1. The variation of wave height along the South African coastline is illustrated in Figure 6.7.

***TABLE 6.1: A summary of all available wave data along the coast of South Africa up to 1984 (Rossouw, 1984)***

| LOCATION | SIGNIFICANT WAVE HEIGHT ($H_s$ in metres) FOR DIFFERENT % EXCEEDANCES | | | WAVE PERIOD ($T_P$ in seconds) FOR 50% EXCEEDANCE |
|---|---|---|---|---|
| | 0.01% | 0.1% | 1.0% | |
| Oranjemund | 7.5 | 6.0 | 4.4 | |
| Saldanha Bay | 7.7 | 6.3 | 4.8 | |
| Koeberg | 8,3 | 6.5 | 4.8 | 12.8 |
| Slangkop | 9.3 | 7.5 | 5.6 | 12.5 |
| Gans Bay | 8.1 | 6.4 | 4.6 | |
| Mossel Bay | 6.0 | 5.0 | 3.9 | |
| Durban | 5.8 | 4.7 | 3.5 | |
| Richards Bay | 5.5 | 4.5 | 3.4 | 11.4 |

Source: Rossouw (1984)

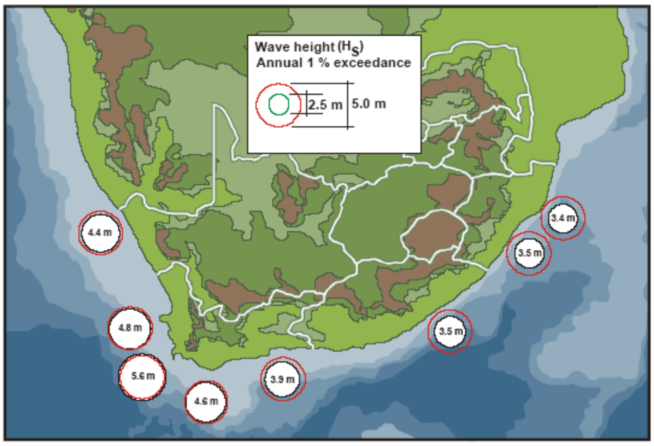



***FIGURE 6.7: Wave heights (1% exceedance) along the South African coastline***

### *vi. Wind*

Wind is an important phenomenon in that the wind-field can govern the behaviour of surface currents and the subsequent transport (speed and direction) of a buoyant wastewater plume to distant locations. Winds exert a stress on the water surface that is down through the water column by shear between the moving surface water and the lower layers. In the absence of stronger ocean currents, wind-driven currents dominate. Most of the time, the only available long-term data records at a study site are for wind. Long-term wind data records is therefore used to verify the representativeness of detailed, short-term data set measured during the study period, not only short-term wind data, but also other data influenced by wind such as currents. It is important to understand the interaction between the wind and the ocean processes, especially the nearshore circulation characteristics, in the study area.

In the offshore region, the surface wind-induced current vector is approximately 45 degrees left of the wind direction in the Southern Hemisphere, deflecting more to the left as the depth increases (at approximately 10 m depth the direction can be 90 degrees to the left of the wind direction). This phenomenon is related to the earth's rotation and is known as the Coriolis force (Neumann, 1968). In shallower water, the Coriolis effect reduces with subsequent reduction of the deflection of the current direction from the wind direction. This phenomenon is included in 3-D numerical models used for detailed investigations. For pre-assessment studies, the wind-induced surface current can be taken as 3% of the wind speed (Williams, 1985). For the transport of floatable material on the surface, a value of 7% can be used.

In the case of shallow outfalls (e.g. surf zone discharges), the diurnal land and sea breezes (resulting from the difference in temperature between the land and the sea) will result in diurnal changes of the transport (onshore/offshore) of surface waste fields (Figure 6.8). During the day the land temperatures will be higher than the sea temperatures, causing the air to rise over the land and cooler air from the sea to flow towards the land. The maximum temperature difference between the land and the sea will occur at approximately 15h00, with the sea breeze at its maximum velocity (Hydrographic Office, 1994). During the summer, the onshore wind will prevail from early morning to midnight, the season and the time of day when the South African beaches are heavily utilised for recreation. During the winter, the sea breeze will occur from noon to early evening. Typical conditions in Algoa Bay are illustrated in Figure 6.9 (Hydrographic Office, 1994).

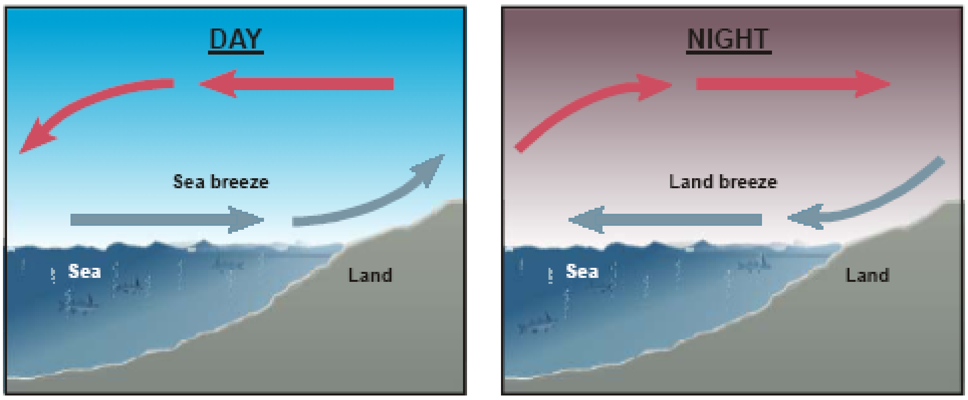



*FIGURE 6.8: Land and sea breezes (Hydrographic Office, 1994)*

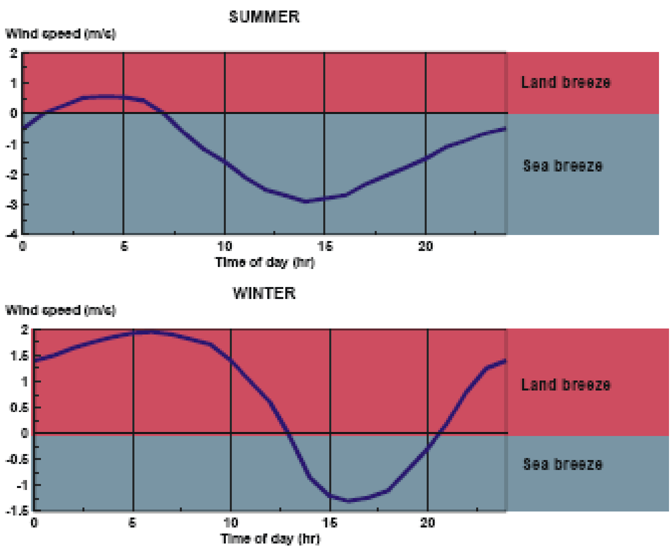



*FIGURE 6.9: Typical diurnal land- sea breeze variations (Hydrographic Office, 1994)*

Wind speed and direction are measured by automatic wind recorders connected to data loggers. Wind speed and direction at almost any predetermined sampling interval can be digitally recorded, transferred to a computer and processed. The coastal topographic features may have a local effect on the prevailing wind-fields and, in the case of a complex coastline configuration, several such recorders should be operated around a proposed outfall area to avoid biased results that may be brought about by sheltering or deflection of winds by topographic features.

Typical data outputs include annual and seasonal wind speed and directional occurrences as shown in Figure 6.10. An example of a wind-rose is shown in Figure 6.11. Exceedance data are presented in a similar manner to that for waves (refer to Figures 6.6a to 6.6e).

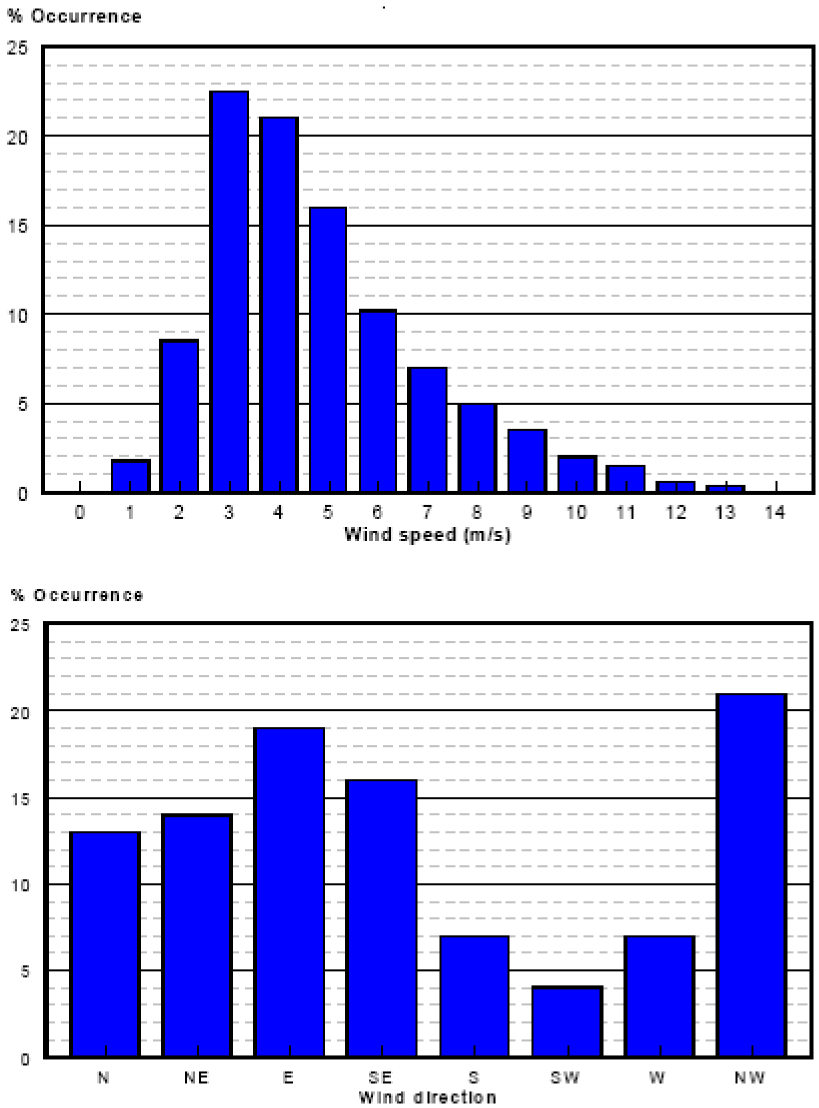



***FIGURE 6.10: Example of graphs showing speed and direction occurrence of wind***

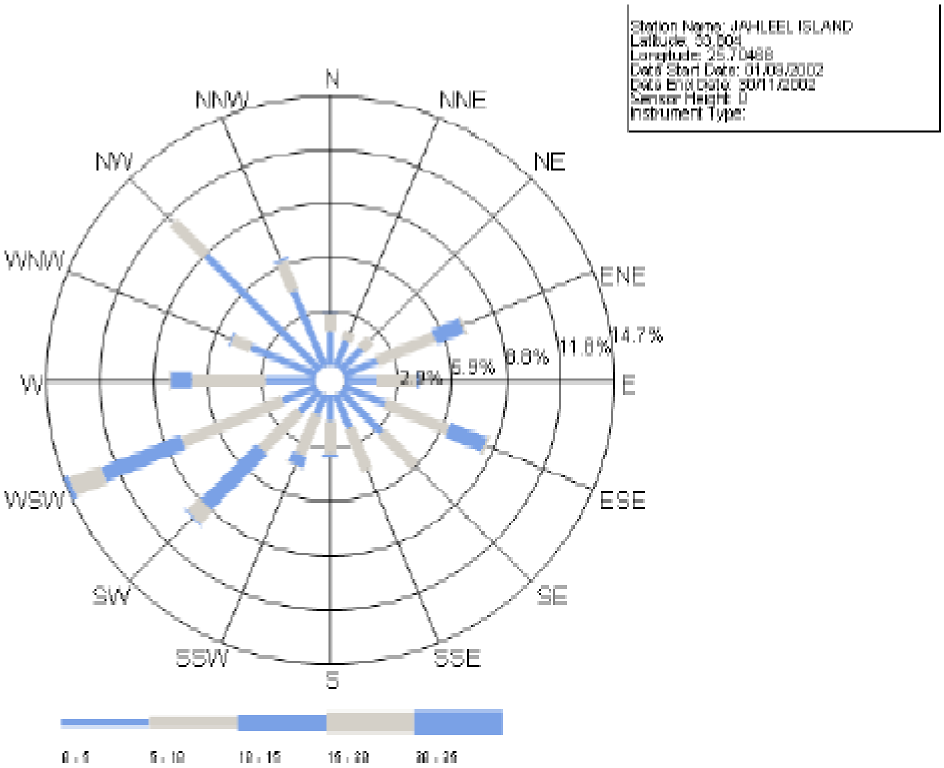



***FIGURE 6.11: Wind-rose, showing the typical wind direction and speed at a particular site***

The Hydrographic Office (1994) provides monthly wind direction occurrences for a number of coastal locations, based on more than 20 years of data. The mean annual occurrences of wind direction and speed for selected locations are presented in Table 6.2 and illustrated in Figure 6.12.

***TABLE 6.2: Mean annual occurrences of wind direction and speed for selected locations along the South African coastline***

| LOCATION | PERCENTAGE OCCURRENCE FOR WIND DIRECTIONS | | | | | | | | | MEAN SPEED (m/s) | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | N | NE | E | SE | S | SW | W | NW | Calm | 8h00 | 14h00 |
| Port Nolloth | 3 | 1 | 2 | 5 | 32 | 5 | 2 | 8 | 43 | 2.1 | 5.7 |
| Cape Columbine | 7 | 6 | 2 | 12 | 42 | 10 | 4 | 10 | 7 | 4.6 | 6.2 |
| Cape Town | 7 | 3 | 0 | 10 | 28 | 10 | 3 | 16 | 23 | 3.1 | 6.2 |
| Cape Point | 4 | 6 | 3 | 33 | 9 | 14 | 10 | 13 | 8 | 9.3 | 8.2 |
| Agulhas | 1 | 10 | 17 | 12 | 6 | 16 | 23 | 8 | 7 | 4.6 | 6.2 |
| Mossel Bay | 3 | 5 | 9 | 12 | 8 | 29 | 10 | 15 | 9 | 4.1 | 6.2 |
| St. Francis | 2 | 5 | 20 | 3 | 3 | 9 | 40 | 5 | 13 | 5.1 | 7.2 |
| Port Elizabeth | 2 | 6 | 15 | 5 | 5 | 23 | 20 | 3 | 21 | 3.6 | 7.2 |
| East London | 5 | 16 | 11 | 4 | 7 | 18 | 17 | 7 | 15 | 3.6 | 6.2 |
| Port Shepstone | 7 | 27 | 3 | 2 | 5 | 25 | 5 | 12 | 14 | 3.6 | 6.7 |
| Durban | 7 | 19 | 6 | 3 | 13 | 15 | 2 | 2 | 33 | 2.1 | 5.7 |
| St.Lucia | 9 | 22 | 4 | 10 | 11 | 14 | 6 | 17 | 7 | 5.7 | 7.2 |

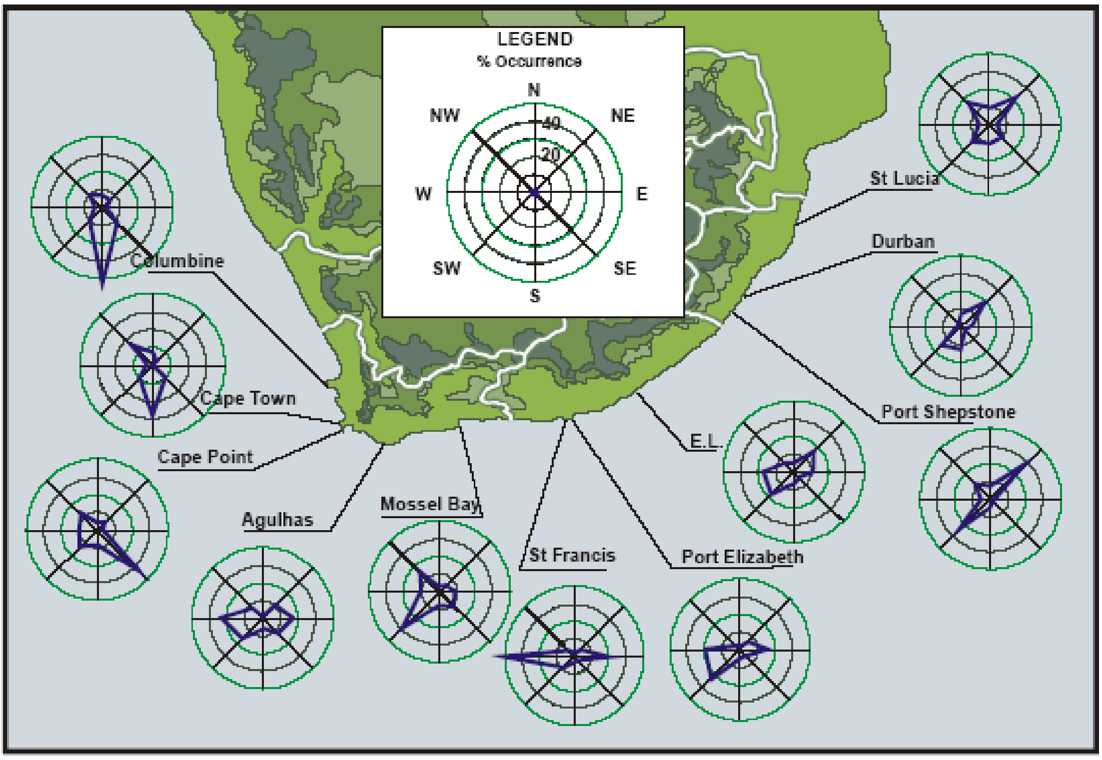



***FIGURE 6.12: The coastal wind regime (Hydrographic Office, 1994)***

## *vii. Tides*

The astronomical tides together with the nearshore bathymetry and the coastline configuration are amongst the most influential and significant factors governing the nearshore hydrodynamics and the hydrodynamics of estuaries and bays. Tidal variations (ranges) also have to be taken into account in the hydraulic design with respect to the available head (pressure or gravity) required to discharge the wastewater.

The dominant tide is semidiurnal (period of approximately 12 hours and 20 minutes). Differences between high and low water can range from 2 m during spring tide (full moon and new moon) to 0,5 m at neap tide. The lowest and highest astronomical tides predicted for South African coastal towns, based on 19 years of data, are given in Table 6.3 (SAN, 2003). These levels can be exceeded when extreme meteorological conditions coincide with the neap and spring tides.

***TABLE 6.3: Approximate spring tide ranges for the main South African coastal towns***

| *PLACE* | *LOWEST ASTRONOMICAL TIDE (LAT) (m)* | *HIGHEST ASTRONOMICAL TIDE (HAT) (m)* |
|---|---|---|
| *Port Nolloth* | *0* | *2.41* |
| *Saldanha Bay* | *0* | *2.03* |
| *Cape Town* | *0* | *2.02* |
| *Simon's Town* | *0* | *2.09* |
| *Hermanus* | *0* | *2.07* |
| *Mossel Bay* | *0* | *2.44* |
| *Knysna* | *0* | *2.21* |
| *Port Elizabeth* | *0* | *2.12* |
| *East London* | *0* | *2.08* |
| *Durban* | *0* | *2.30* |
| *Richards Bay* | *0* | *2.47* |

A typical tidal record for South African conditions is illustrated in Figure 6.13.

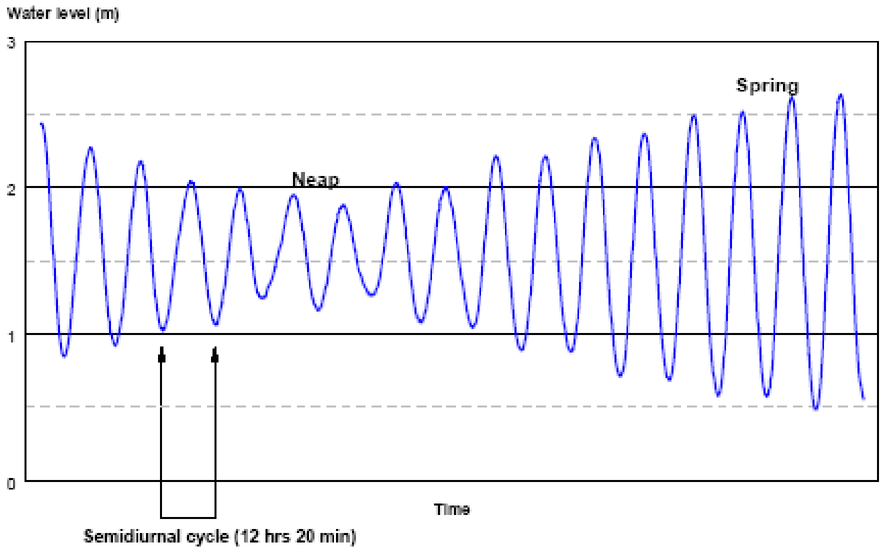



***FIGURE 6.13: Typical tidal record for South African conditions measured at Knysna***

## *viii. Coastal currents*

The South African coastline is bounded by two major circulation systems, these are the 'warm' south-bound Agulhas Current along the east and south coast and the 'cold' north moving Benguela System along the west coast (Figure 6.14). The Agulhas Current is powerful: at the surface it can reach maximum speeds of 2 m/s (Gyory *et al.*, 2000). As the Agulhas Current reaches the southern tip of Africa, it begins to turn toward the southwest. Once it reaches the Southern Ocean, the current retroflects, or turns back on itself, and flows eastward as the Agulhas Return Current.

The Benguela Current is the eastern boundary current of the South Atlantic subtropical gyre (Shannon, 1985). It begins as a northward flow off the Cape of Good Hope, skirts the western African coast equator-wards to about 24°S-30°S from where it moves further offshore. The sources of the Benguela System include Indian and South Atlantic subtropical thermocline water; saline, low-oxygen tropical Atlantic water; and cooler, fresher sub-antarctic water. Shannon (1985) gathered all available information on surface current speeds from previous studies and calculated the mean speed of the Benguela System to be 17 cm/s. Wedepohl *et al*. (2000) found that the mean speeds of the current vary from less than 11 cm/s to a maximum of 23 cm/s. Apparently the highest speeds occur in the south during summer and in the north during winter, a pattern that corresponds with seasonal wind fields. The prevailing winds are responsible for strong Ekman transport and the resulting coastal upwelling of cool, nutrient-rich water that stimulates primary productivity (Boyer *et al*. 2000; Skogen 1999).

Although the main ocean currents along the east coast of South Africa are much more dynamic than the currents along the west coast, the inshore circulation (less than 40 m water depth) is mostly tidal and wind driven. This was demonstrated by using drogues to determine surface current velocities at various locations along the South African coastline between 1980 and 1992 (CSIR 1986, CSIR 1988, CSIR 1991b, CSIR 1992). Data gathered for a minimum of one year at each location are presented in Table 6.4.

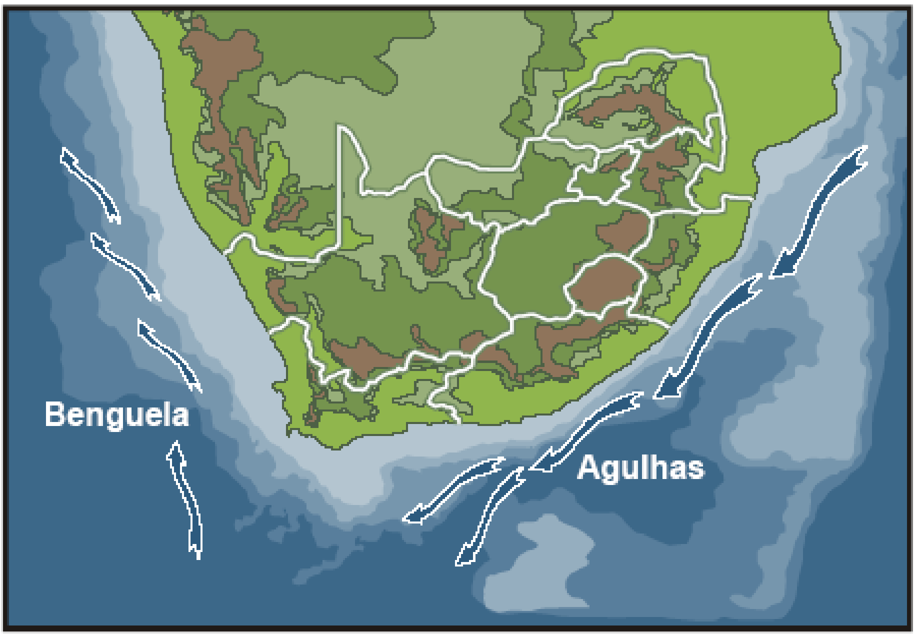



***FIGURE 6.14: South Africa's major coastal circulation systems***

***TABLE 6.4: Measured surface current velocities at various locations along the South African coastline based on drogue tracking between 1980 and 1992 (minimum 1 year)***

| LOCATION | DISTANCE FROM SHORE | AVERAGE CURRENT SPEED (cm/s) | |
|---|---|---|---|
| | | SURFACE | SUB-SURFACE (-5 m) |
| North West Bay (north of Saldanha) | 2 km<br>6 km | 20<br>30 | -<br>- |
| Hout Bay | 1 km | 16 | 11 |
| False Bay | 2 km<br>5 km | 17<br>15 | 9<br>9 |
| Mossel Bay (Dana Bay) | 1 km<br>2 km | 18<br>21 | 12<br>15 |
| East London | | | |
| Tongaat | 1,2 km<br>1,9 km | 18<br>23 | 15<br>20 |

***Offshore.*** The offshore circulation characteristics (speed and direction of currents) are the main oceanographic processes that would influence the initial dilution of a buoyant wastewater stream and its subsequent transport and dispersion to distant locations. The seasonal variation in current speeds also influences the selection of the appropriate method of construction and the structural design of the outfall.

The net (resultant) current which will transport and disperse a waste field is the result of a complex of numerous driving forces. These forces include the local wind forcing, ambient continental currents (for example the Agulhas current), surf zone long-shore and rip currents generated by waves, tidal currents and density differences. In the nearshore zone, the circulation is strongly influenced by the seabed topography and the configuration of the coastline.

Measurement of currents at sea is complicated because of the varying spatial and temporal nature of currents. The ultimate (resultant) current measured is a composite of numerous driving forces. Instantaneous measurements cannot be compared directly with the various generating forces because, for example, currents measured during a particular wind condition may be unrelated to that condition since they may still be subject to the inertia of a previous wind forced circulation. All these and other effects require that any current measurement programme be carefully designed to avoid bias.

Eulerian measurements are continuous recordings of current data collected at pre-determined time intervals by the use of moored current meters at fixed points in the study area. Eulerian data provide the basis for statistical estimates of occurrence and persistence of current speed and direction. Typically the information is measured and recorded by the instrument at 15-minute or 30-minute intervals. Moored current meters use a variety of methods for sensing speed and direction and it is important to understand the limitations and performance of each type of meter when designing a current recording programme. Current profilers, lowered to a specific depth from an anchored platform or vessel, can also be used. If the survey vessel is allowed to drift during the current measurement, accurate position fixing (using global positioning fixing (GPS) techniques) must be recorded as the drift vector must be vectorially subtracted from the measured velocity to produce the true ambient current vector.

Lagrangian measurements include spatial studies with drogues, drifters, or dye, in which the path and velocity of a particle are determined. Surface and subsurface current recordings can be made by the tracking of the movement of floats (Lagrangian) in the current field at the outfall location. Floats consist of either drogues or drift cards. Dye patch observations are also sometimes used.

Drogues are surface or subsurface floats, identified by a flag number, which drift with the current. Drogue pairs (surface and subsurface) are released at pre-determined locations around a proposed pipeline/diffuser to detect the probable path of the surface wastewater plume. A survey vessel equipped with GPS is used to track the drogues. The actual surface and subsurface current vectors for the particular day of observation can be obtained. Vectors from various drogues are then combined to produce a reliable indication of the general current pattern on that particular day. These results can also be used for the spatial calibration of a far field numerical model.

Radio drogues that transmit a signal to the shore at a predetermined frequency can also be used for larger scale spatial observations. At three shore stations, directional receivers are tuned to receive the signal and thereby provide an accurate fix of the drogue’s location.

In the design of drogues, the influence of the wind on the above-water part and the influence of the current on the nylon line connecting the float has to be minimised to ensure that current measurements are not seriously affected (Botes, 1988).

Drift cards are plastic cards, typically 10 x 15 cm in dimension, that are dumped in lots of 200 at a time at specific locations. The fate of the cards is recorded by the location and time of their deposit on the shoreline. This is used as a crude but effective way of predicting what the fate of a surface wastewater release at the same location would be. Cards of various colours can be dropped from different proposed outfall locations. The results are meaningful but cannot be used for quantitative assessments.

Dye patches provide another method of recording surface current patterns. A quantity of concentrated dye (such as Rhodamine-B dye) is released at a specific location or at several locations in the study area. The movement and spreading of the dye patch is then monitored by aerial photography or by tracing the perimeter of the patch using a survey vessel equipped with GPS.

Typical graphical output data for understanding the nearshore circulation processes for the optimisation and assessment of the potential impact of marine discharges include:

- Time series data for current velocities, directions and vectors (Figure 6.15)

- Current roses or a scatter diagram summarising the directional behaviour (refer to Figure 6.16)
- Current profiles showing the vertical variation in current velocities in the water column (Figure 6.17)
- Tracer (e.g. Rhodamine-B) data demonstrating the spatial behaviour of a wastewater plume under specific current conditions (Figure 6.18)
- Radio-buoy tracks recording the direction and speed of surface and sub-surface currents in a study area (Figure 6.19).

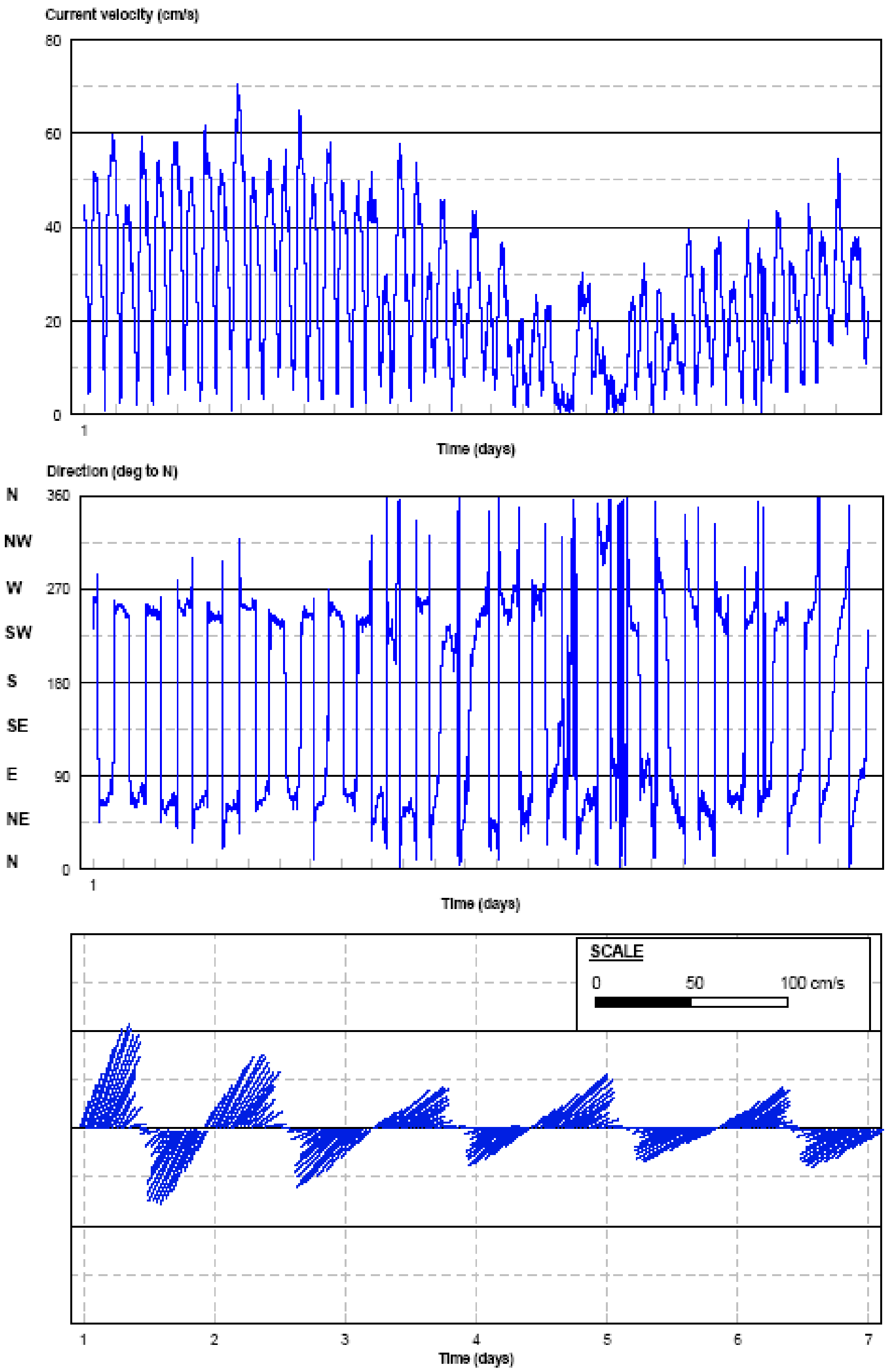



*FIGURE 6.15: Time series data for current velocities, directions and vectors*

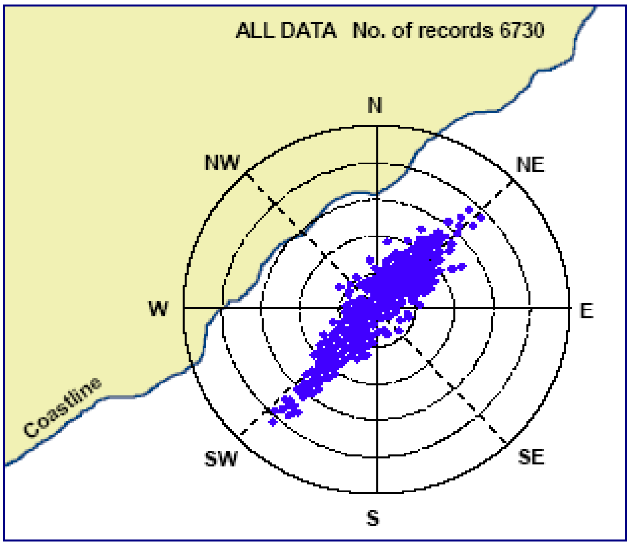



***FIGURE 6.16: Current scatter diagram***

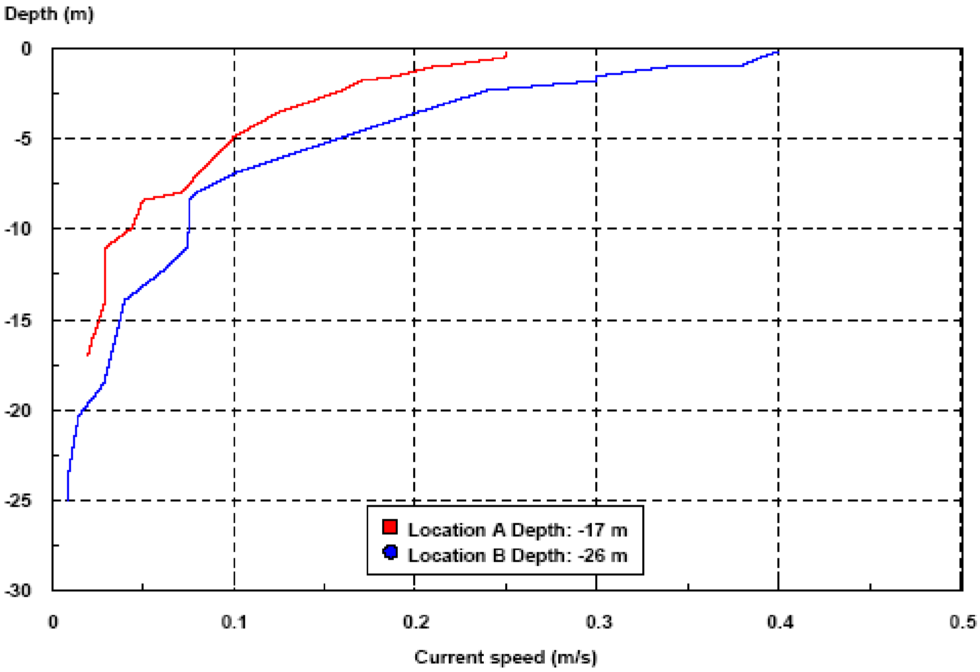



***FIGURE 6.17: Example: Current Profile***

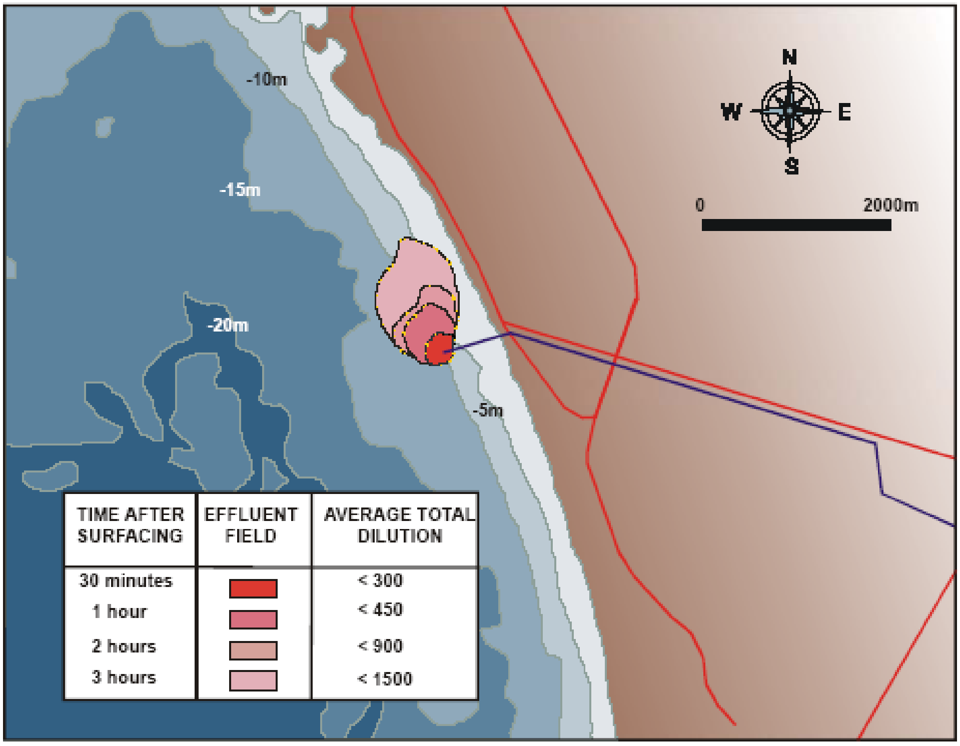



***FIGURE 6.18: Prototype tracer studies***

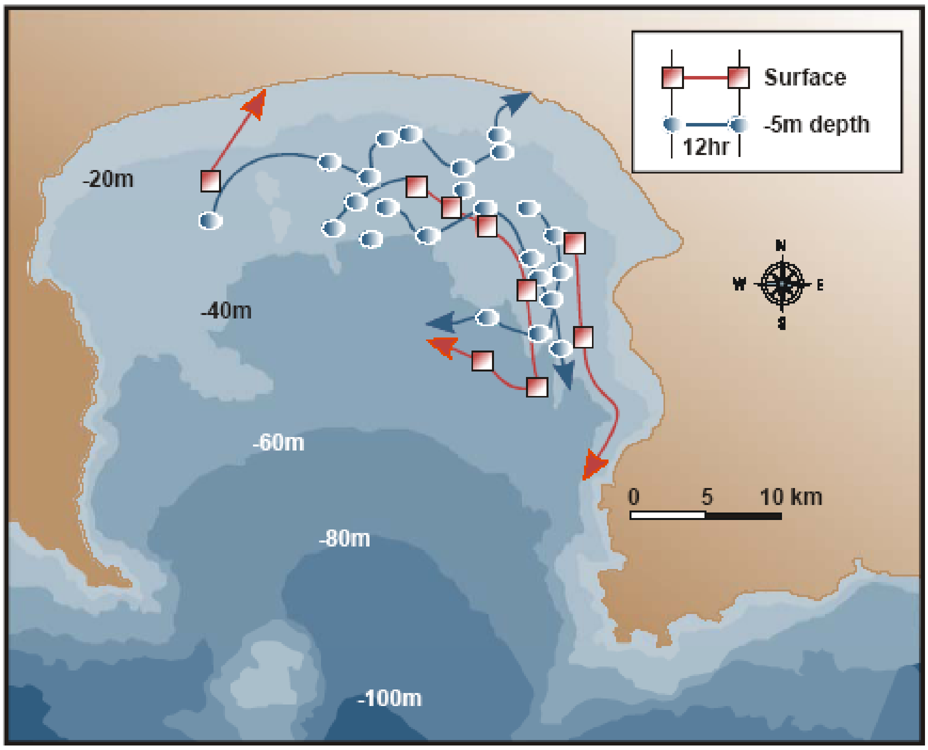



***FIGURE 6.19: Circulation patterns recorded over three days in False Bay using radio-buoy tracking (Botes, 1988)***

***Surf zone.*** Currents in the surf zone (littoral zone) are wave-dominated, and initial mixing is rapid due to the vigorous processes of which long-shore and cross-shelf transport are the most dominant. Long-shore transport is driven by the momentum flux of shoaling waves approaching the shoreline at an angle, cross-shelf transport is driven by the shoaling waves, while water is transported out of the surf zone by rip currents, which will result in the diffusion of surf zone water into the offshore waters. For wave fronts parallel to the coast, symmetrical circulation 'cells' will be formed and the surf zone width will be determined by factors such as the wave height/period and the beach slope. For example, along the Kwazulu-Natal coastline average cell widths of 600 m, with rip currents 30 to 60 m wide and offshore velocities varying between 0.3 m/s and 1 m/s, have been recorded (NTRPC, 1969). Results showed that some of the rip current water is forced back into the cells by the onshore wave transport. Observations along the Kwazulu Natal coastline indicate a typical offshore flow of approximately 400 m (NTRPC, 1969). In the case of oblique waves, the circulation 'cells' will be asymmetrical and the tendency is for the rip currents to flow to the downstream cell only. Some of the water expelled beyond the surf zone may be transported back into the surf zone by the next set of waves. It is important to note that onshore winds and the incoming tide will tend to keep water in the surf zone, whereas offshore winds and the outgoing tide will contribute to the transport of water away from the shoreline.

***Estuaries.*** There are about 250 estuaries in South Africa that fall within the definition of an estuary (see *Glossary of Terms*) (Whitfield, 1992). The water movement (hydrodynamics) and related processes in South African estuaries will depend on the status (open or closed) of the estuary mouth. About 75% of South Africa's estuaries are temporarily open/closed systems.

In permanently open estuaries, the flow in the estuary and the subsequent exchange of water between the estuary and the sea is dependent on the diurnal and semi-diurnal differences in water levels in the estuary and in the sea (due to tidal variation), the 'size' (cross-sectional area) of the estuary mouth and the volume and timing of source inflows (e.g. river inflow). Salinity distribution patterns in the estuary also affect hydrodynamic behaviour associated with the density difference between saline and freshwater. Usually, in deeper estuaries, the more dense seawater creates a salt wedge along the bottom, resulting in strong vertical stratification. In a shallower system, however, the strong tidal currents are usually sufficient to break down any vertical stratification, resulting in a well-mixed system. Each estuary has a unique salinity regime, which continuously changes according to the state of the tide and freshwater inflows. The fresh and saline water structure of a 'partially-mixed' estuary is illustrated in Figure 6.20.

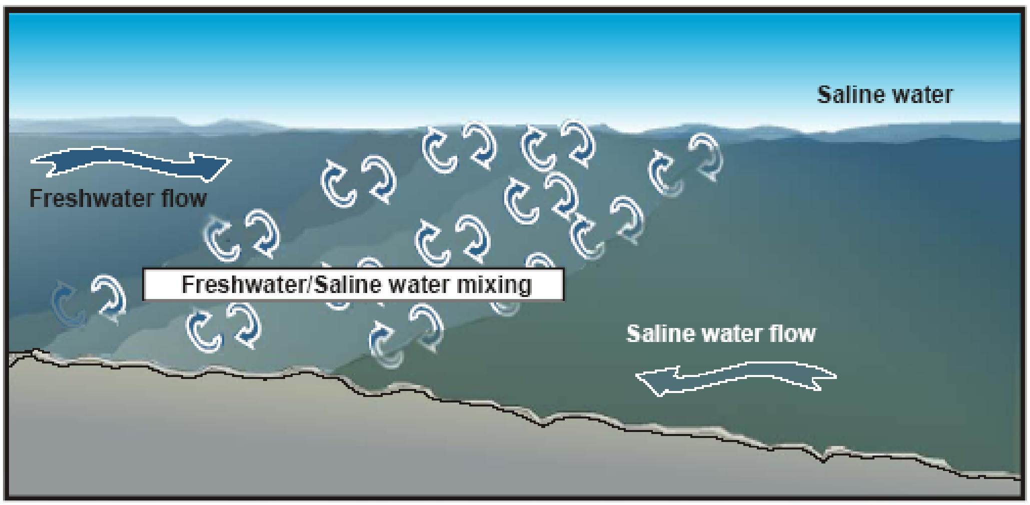



***FIGURE 6.20: Illustration of vertical mixing in an estuary***

In temporarily open/closed estuaries, in which the mouth is closed for periods when there is little or no river inflow, the water movement will depend mainly on wind stress. Elevated water levels resulting from river inflows and possible seawater input from waves overtopping the sandbar at the mouth during storms and spring tides, also play a role in the water movement within these systems. The only exchange to the sea then will be through seepage. When the mouth is open, the dynamics will be similar to those of permanently open estuaries. However, as the river inflow declines, the volume of water exchanged through the estuary mouth is reduced. Ultimately, the estuary mouth will be closed by sand transported by a combination of wave and tidal energy.

### *ix. Stratification*

Stratification is the term used to describe the phenomenon of denser sea water underlying lighter sea water thereby causing a vertical density gradient in the water column, depending on the vertical temperature gradient between warmer upper water layers and colder deeper water (thermocline) and the salinity gradient (halocline). Seawater density is a function of temperature, salinity and pressure (depth). An example of the relationship between temperature, salinity and depth is shown in Figure 6.21.

Density stratification is the major factor that influences the rising of a buoyant wastewater plume and thus determines whether the discharge from an ocean outfall remains beneath the surface as a submerged field or continues to rise to become a surface field.

The conversion formula to calculate the density as a function of temperature, salinity and depth ($\rho_{STP}$) is as follows:

$$\rho_1 = 999.842594 + 0.06793952T - 0.00909529T^2 + 0.0001001685T^3 + -0.000001120083T^4 + 0.000000006536332T^5$$

$$\rho_2 = \rho_1 + S(0.824493 - 0.0040899T + 0.000076438T^2 - 0.00000082467T^3 + 0.0000000053875T^4) + (-0.00572466 + 0.00010227T - 0.0000016546T^2) \times S^{3/2} + 0.00048314S^2$$

$$K_1 = 19652.21 + 148{,}4206T - 2.327105T^2 + 0.01360477T^3 - 0.00005155288T^4$$

$$K_2 = 3.239908 + 0.00143713T + 0.000116092T^2 - 0{,}000000577905T^3$$

$$K_3 = 0.0000850935 - 0.00000612293T + 0.000000052787T^2$$

$$K_{ST0} = K_1 + S(54.6746 - 0.603459T + 0.0109987T^2 - 0.00006167T^3) + (0.07944 + 0.016483T - 0.00053009T^2).\ S^{3/2}$$

$$A = K_2 + S(0.0022838 - 0.000010981T - 0.0000016078T^2) + 0.000191075S^{3/2}$$

$$B = K_3 + S(-0.00000099348 + 0.000000020816T + 0.00000000091697T^2)$$

$$K_{STP} = K_{ST0} + A \times D/9.81 + B \times (D/9.81)^2$$

$$\rho_{STP} = \rho_2 / [(1-D/9.81)/K_{STP}]$$

Where:

T = Temperature ($^{o}$C)
S = Salinity (ppt)
D = Water depth (m)

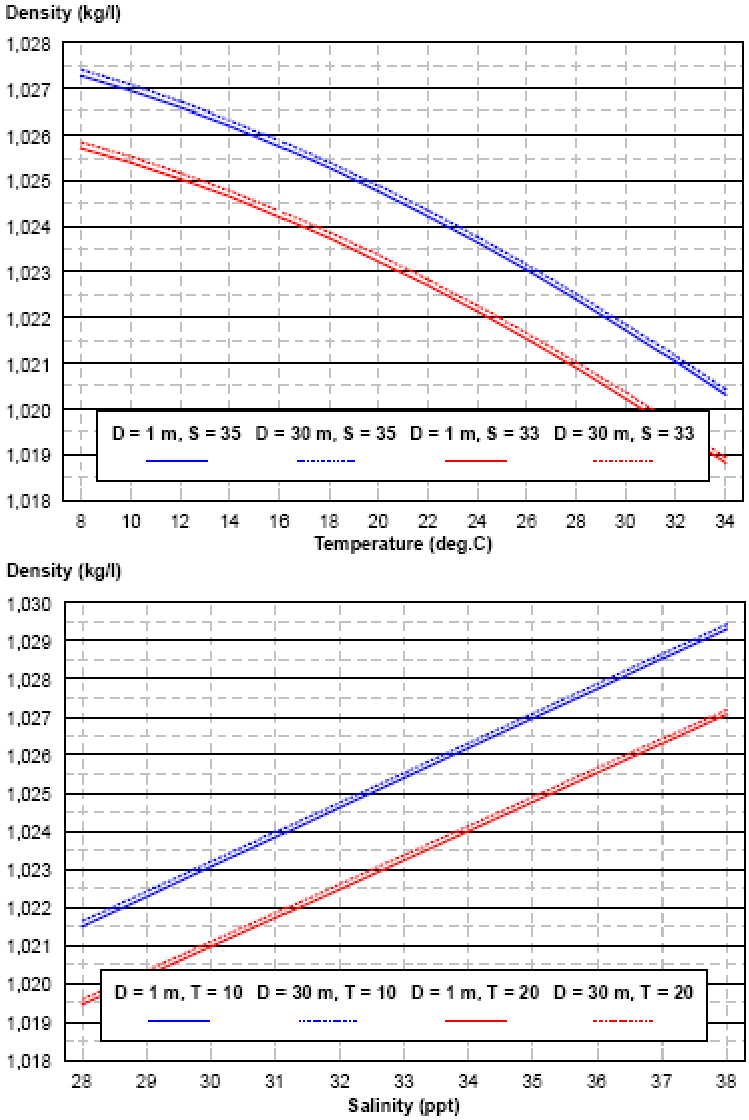



***FIGURE 6.21: Seawater density versus temperature and salinity***

Warming of the surface waters as a result of solar radiation, upwelling of colder deep ocean water (particularly on the west coast) and the movement of warmer (less dense) water towards the coast because of ambient ocean currents (an east coast phenomenon), typically cause stratification along the South African coast. An overview of the winter and summer sea surface temperatures along the South African coast is shown in Figure 6.22 (Hydrographic Office, 1994).

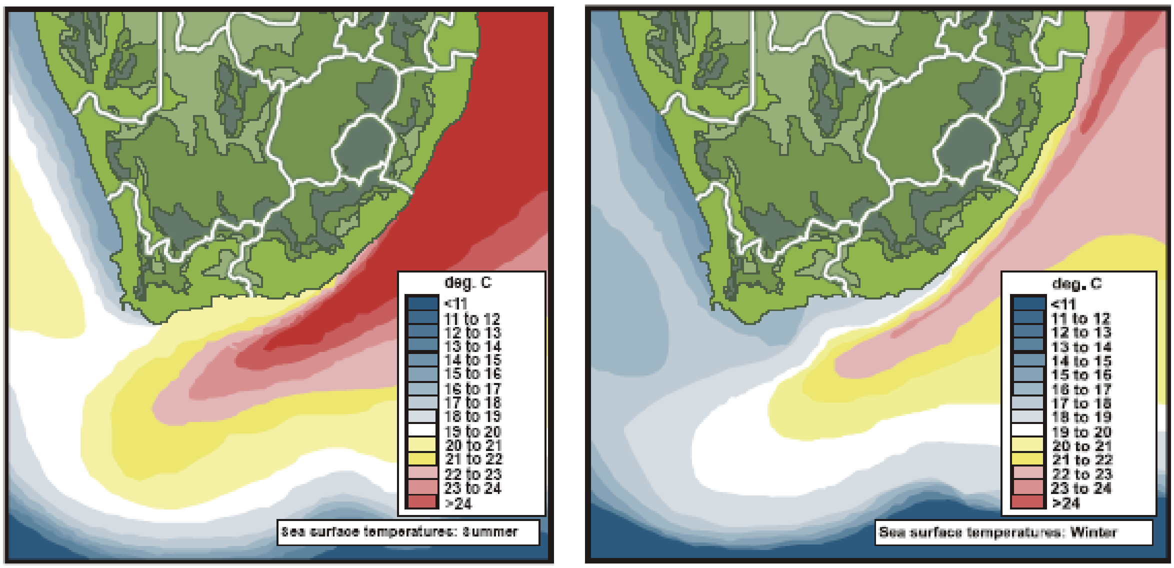



***FIGURE 6.22: Typical sea surface temperatures along the South African coast for summer and winter (Hydrographic Office, 1994)***

Between 1980 and 1992, temperature and salinity profiles were measured for a minimum of a year at various locations along the South African coastline (CSIR 1985, CSIR 1986, CSIR 1988, CSIR 1991b, CSIR 1992). These are summarised in Table 6.5.

***TABLE 6.5: Temperature (in $^{o}$C) and salinity (in ppt) profiles measured at various locations along the South African coastline based on drogue tracking between 1980 and 1992 (minimum 1 year)***

| LOCATION | DISTANCE FROM SHORE (km) | WATER DEPTH (m) | TEMPERATURE (SURFACE) | | | TEMPERATURE (BOTTOM) | | |
|---|---|---|---|---|---|---|---|---|
| | | | Maximum | Minimum | Average | Maximum | Minimum | Average |
| North West Bay | 2 | 30 | 15.5 | 9.3 | 12.3 | 11.4 | 9.4 | 10.3 |
| | 4 | 35 | 15.3 | 9.3 | 12.5 | 13.0 | 8.8 | 10.6 |
| Hout Bay | 1 | 30 | 16.6 | 10.4 | 14.0 | 15.9 | 9.7 | 12.2 |
| False Bay | 2 | 20 | 21.1 | 13.8 | 17.8 | 20.3 | 12.4 | 15.9 |
| | 5 | 30 | 21.0 | 14.5 | 17.7 | 16.1 | 12.0 | 14.3 |
| Mossel bay | 1 | 20 | 21.6 | 15.2 | 18.0 | 21.0 | 13.6 | 16.7 |
| | 2 | 40 | 21.2 | 15.7 | 18.0 | 18.2 | 11.3 | 15.6 |
| Tongaat | 1,2 | 26 | 24.8 | 18.5 | 21.17 | 24.2 | 16.4 | 20.7 |
| | 1,9 | 34 | 24.7 | 18.7 | 21.3 | 24.2 | 16.1 | 21.0 |

| LOCATION | DISTANCE FROM SHORE (km) | WATER DEPTH (m) | SALINITY (SURFACE) | | | SALINITY (BOTTOM) | | |
|---|---|---|---|---|---|---|---|---|
| | | | Maximum | Minimum | Average | Maximum | Minimum | Average |
| North West Bay | 2 | 30 | 36.25 | 34.86 | 35.28 | 35.35 | 35.03 | 35.13 |
| | 4 | 35 | 37.44 | 34.60 | 35.29 | 36.52 | 34.20 | 35.23 |
| Hout Bay | 1 | 30 | 35.45 | 33.80 | 35.06 | 35.75 | 33.60 | 35.10 |
| False Bay | 2 | 20 | 35.78 | 34.50 | 35.46 | 35.74 | 35.00 | 35.46 |
| | 5 | 30 | 35.96 | 34.50 | 35.36 | 35.67 | 35.00 | 35.32 |
| Mossel Bay | 1 | 20 | 35.40 | 35.02 | 35.20 | 35.48 | 35.50 | 35.34 |
| | 2 | 40 | 35.48 | 35.00 | 35.19 | 35.04 | 35.15 | 35.38 |
| Tongaat | 1,2 | 26 | 35.45 | 33.6 | 35.01 | 35.45 | 34.43 | 35.1 |
| | 1,9 | 34 | 35.39 | 33.57 | 34.94 | 35.72 | 34.35 | 35.08 |

In order to detect stratification in the water column, both temperature and salinity profiles should be measured since seawater density is a function of both properties. The stratification measurements should also be done on a similar grid as that used for the current measurements. It is convenient to attach the small temperature and salinity probes to the current profiler to maximise the information obtained from each measured profile.

Stratification is calculated from measured (recorded) vertical profiles of temperature and salinity. Conductivity-Temperature-Depth (CTD) profilers can be used for measurements from a survey boat. For input data to numerical models, continuous records (pre-determined time intervals) are required and can be obtained from the deployment of a thermistor string, consisting of temperature and conductivity/salinity probes, located at regular depth intervals through the water column.

Normally, stratification in the surf zone area will be insignificant because of the vigorous movement and the consequent high degree of mixing. Horizontal density differences may occur as a result of solar heating in sheltered shallow waters.

Depending on the mouth and river flow characteristics of estuaries, the horizontal and vertical distributions of salinity and temperature contribute to a complex distribution of density and the stratification varies continuously with the tidal flow, river discharge, wind shear and solar radiation. An example of the distribution of salinity and temperature measured in the Breede River Estuary on 23 August 2000 is illustrated in Figure 6.23 (CSIR, 2002).

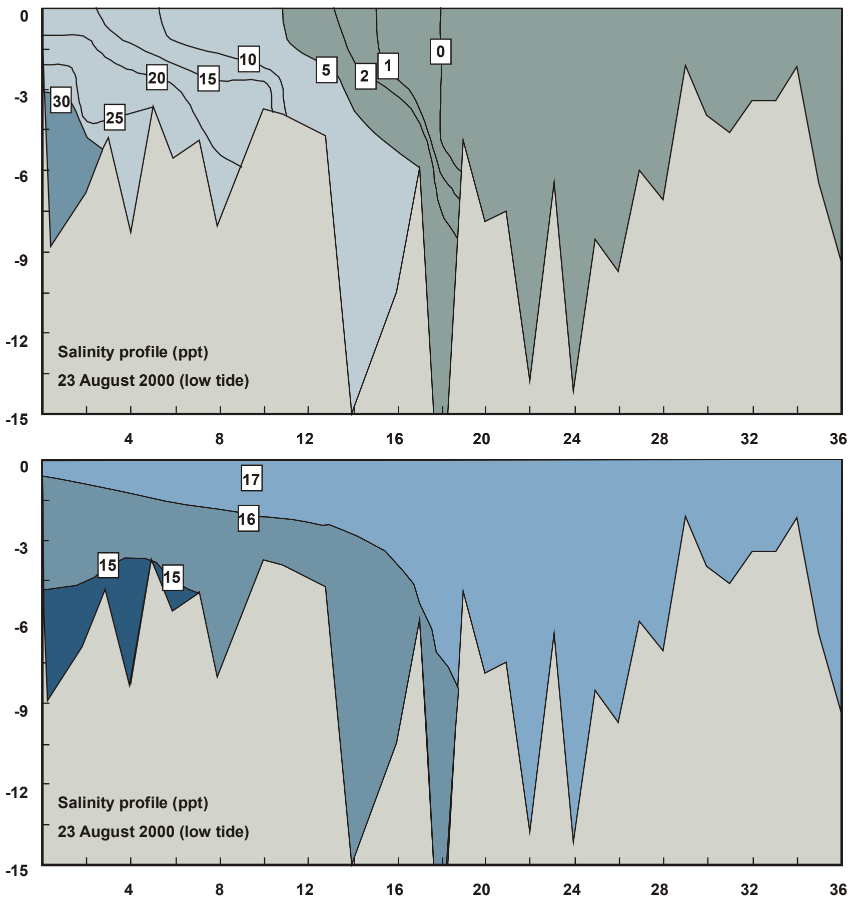



***FIGURE 6.23: Salinity and temperature distribution patterns in the Breede River Estuary on 23 August 2000 (CSIR, 2002)***

### 6.1.2 Data requirements for pre-assessment and detailed investigation

#### *i. Pre-assessment*

A pre-assessment is usually based on available data and information on the relevant physical processes, as described in Section 6.1.1. In particular, the following important data and information should be obtained:

- Observations of site-specific features, such as coastline configuration and shoreline/beach characteristics, rocky/sandy offshore areas, existing structures/obstructions, navigational routes etc.
- Existing topographical maps (1:50 000), bathymetric charts/surveys, aerial photographs
- Wind data from the nearest weather stations
- Existing wave and tidal data
- Any available data on water circulation and stratification. Where available data are insufficient, some short-term observations, using drogues or dye, can be conducted.
- Identification of potential depositional areas. These typically include areas in which current velocities are low or are protected from wave action. Sediments in these areas usually consist of fine particles.

As part of a pre-assessment, the following items need to be addressed:

- The selection of a feasible outfall site with regard to the proposed location of the head works and a feasible pipeline route (seabed slope, seabed geology, discharge depth and other physical constraints such as existing structure, wrecks, navigational routes, etc.)
- Average, maximum and minimum current velocities at the proposed outfall site for spring, mean and neap tides
- Directional occurrences and average, maximum and minimum current velocities (of particular importance is the onshore directional occurrence and the maximum velocities)
- Diurnal and seasonal variations and the spatial behaviour of the currents (average and maximum current speeds, directional occurrences)
- Wind regime (maximum and average speed, directional occurrences, seasonal and diurnal variations and persistence)
- Wave statistics (wave height and directional occurrences)
- Worst case stratification and the occurrence of density differences through the water column
- Tidal range, mean sea level, tidal currents (in the case of estuaries, the tidal prism for the different tides also needs to be determined)
- Studies involving temporarily open/closed estuaries also require an assessment of the percentage of time when the estuary mouth is likely to close.

A basic overview of statistical techniques that are typically applied in the evaluation of environmental data, where and when appropriate, is provided in Section 7.4.2. It highlights important factors that need to be taken into account when applying statistical analyses to data sets and is by no means exhaustive.

### *ii. Detailed investigations*

A detailed investigation requires intensive field measurement programmes to acquire the data and information on the relevant physical processes, as described in Section 6.1.1. In particular, the following important data and information should be obtained:

- Geophysical data. The following surveys need to be undertaken:

  - Precision bathymetric survey
  - Side scan sonar and seismic profiling
  - Geotechnical investigations (Sea bottom material and exploratory drilling)
  - Surf zone investigation and sediment movement.

- Wind data. Wind recording at the proposed site (one-year deployment of automatic weather station).
  Long-term wind data from nearby weather station need to be correlated with on-site weather station.

- Wave data. Wave recording at the proposed site (one-year deployment of wave buoy or an alternative device). Long-term wave data from nearby recording location to be correlated with on-site wave recordings.

- Current data. The acquisition of quality field data on near-shore currents is extremely costly and time consuming as it must be ensured that the measured data are representative with regard to speed, direction and persistence. It is imperative, therefore, to ensure that all existing data for a specific area have been thoroughly researched, documented and analysed before embarking on any new measurement programme.

  Continuously recording current meters, capable of taking measurements throughout the water column, moored at a number of locations would be the ideal method for describing the current field. However, due to constraints such as the cost of meters, security of meters near the surface, and logistical difficulties in operating several dozen simultaneously recording current meters, compromise is usually necessary. A limited measurement programme, although not ideal, is in most cases sufficient for the design procedures. A limited measurement programme typically consists of:

  - A few moored continuously recording current recorders along and perpendicular to the axis of the proposed diffuser
  - Spatial current profiling from a survey boat at regular intervals (weekly, monthly) at selected locations
  - Regular surface and subsurface current measurements using drogues, dye or drift cards.

  (The output of a calibrated numerical model could be used to supplement the limited current measurements, i.e. provide more extensive spatial information.)

  ***NOTE:***

  ***The measurement programme must also be arranged to reflect seasonal and other cyclical current trends adequately and have a typical duration of 12 or 18 months if previous data are not available.***

- Stratification. Spatial profiling (salinity, temperature, depth) should be undertaken at regular intervals to coincide with the current measurements. A thermistor string is to be deployed together with the continuously recording current meters.

In estuaries, longitudinal profiles of salinity and temperature variations at high and low water, both at springtide and at neap tide, are required. Simultaneously gauging of the river flow, and of water level variations near the mouth, need to be undertaken.

The following outputs are required as part of a detailed investigation:

- Geophysical investigation.

  The following are required:

  - Detailed bathymetric charts and contour maps
  - Profile of the seabed along the pipeline route
  - Map of seabed features (sand, rock, etc.)
  - Sub-bottom profile (seismic interpretation supported by diver investigations)
  - Detailed geotechnical reports to support the seismic interpretation (soil classification, cohesive and shear strength of soils, internal angle of friction, soil density characteristics, rock classification and hardness, seismic activities)
  - Sediment transport rates (together with potential scour/accretion probabilities) in the surf zone

- Nearshore circulation, wind, tides and stratification. For a detailed investigation, a 2-D or preferably a 3-D numerical model for the simulation of the composite hydrodynamic processes over the entire project area needs to be applied. This assessment also forms the basis for other process models such as a water quality model for detailed assessments of waste fields for the optimisation of the design of a marine outfall or the impact from an existing outfall(s).

  A state-of-art numerical model will include equations for the simulation of the majority of driving forces:

  - Tidal forcing
  - Surface wind shear stress
  - Seabed shear stress
  - Coriolis force (effect of earth's rotation)
  - Barotropic effects (free surface gradients)
  - Baroclinic effects (horizontal pressure gradients)
  - Water with variable density
  - Turbulence-induced mass and momentum fluxes
  - Thermocline dynamics
  - Insolation and air-sea interactions
  - Drying and flooding in shallow areas.

To assess advection-diffusion, the typical hydrodynamic differential equations for numerical applications (finite difference or finite element methods) are:

For the conservation of momentum (equation of motion) in the x- and y-planes:

$$\partial u/\partial t + u.\ \partial u/\partial x + v.\ \partial u/\partial y + g.\ \partial \eta/\ \partial x - f.v + g.u|U|/c^2 - F_x/(\rho(d+\eta) - \upsilon(\partial^2 u/\partial^2 x + \partial^2 u/\partial^2 y) = 0 \text{..... in x-direction}$$

$$\partial v/\partial t + u.\ \partial v/\partial x + v.\ \partial v/\partial y + g.\ \partial \eta/\ \partial y - f.u + g.v|U|/c^2 - F_y/(\rho(d+\eta) - \upsilon(\partial^2 v/\partial^2 x + \partial^2 v/\partial^2 y) = 0 \text{...... in x-direction}$$

For the conservation of mass (continuity of flow):

$$\partial \eta/\ \partial t + \partial[(\ d+\eta)u]/\ \partial x + \partial[(\ d+\eta)v]/\ \partial y = 0$$

The advection-diffusion equation is:

$$\partial C/\partial t - \partial/\partial x[D_x.\ \partial C/\partial x - u.C] - \partial/\partial y[D_y.\ \partial C/\partial y - v.C] - \partial/\partial z[D_z.\ \partial C/\partial z - w.C] = S$$

Where

| | |
|---|---|
| $\eta$ | = water level elevation |
| d | = water depth |
| u,v,w | = velocity in 3 planes |
| U | = total current velocity |
| $F_{x,y}$ | = external forces |
| f | = Coriolis parameter |
| g | = acceleration due to gravity |
| $\rho$ | = water density |
| $\upsilon$ | = eddy viscosity |
| c | = Chezy coefficient |
| $D_{xyz}$ | = dispersion coefficient |
| C | = concentration of constituent |
| S | = source term |

The hydrodynamic model is used to calculate (predict) the water levels and current velocities over the model grid. The model provides the hydrodynamic and dispersion data for further application of water quality models, which simulate the chemical behaviour of ambient as well as introduced (from waste streams) constituents.

The required temporal investigations (the period for which conditions are simulated), the total area to be investigated and the complexity of the bathymetry will determine the dimensions (spatial extent and the grid size) of the model.

The hydrodynamic model should be calibrated with measured Eulerian as well as Lagrangian current measurements and a stratification dataset. For the verification of the model results, a separate dataset should be used. A sensitivity test should be undertaken to determine the sensitivity of the model outputs to uncertainties in input parameters and model assumptions. Examples of spatial outputs of current velocities and temperature distribution from a 3-D model are shown in Figures 6.24a and 6.24b.

The output of the numerical model, as well as the statistical analysis of the data, is used to determine ambient current velocities for the structural design of a marine outfall.

Numerical modelling of the surf zone hydrodynamics is complex and it is not easy to calibrate and verify the model because of the continuous physical changes (beach profiles) and continuously varying shallow water flows, driven mainly by the approaching waves. Surf zone modelling provides results/outputs that tend to be qualitative rather than quantitative.

Conditions in estuaries are complex and are dependent largely on tidal variations, wind shear, and river inflow. Longitudinal and lateral variations in salinity, and hence in water density, can have a significant effect on estuarine dynamics, mixing and subsequent water quality (SEPA, 2002). Selection of an appropriate numerical model with the capability of reproducing these features (if present) is essential.

For a well-mixed 'narrow' estuary, the components of acceleration and velocity in the transverse and vertical directions are considered small enough to be neglected and a 1-D model can be used to predict the flows/water exchange in the estuary, assuming average cross-sectional flows. For wider well-mixed estuaries, a 2-D depth-averaged model can be applied. For partially mixed estuaries (vertical salinity variations), a 3-D hydrodynamic model is essential and the water quality model must be capable of simulating the complex chemical processes (Van Ballegooyen *et al.*, in press).

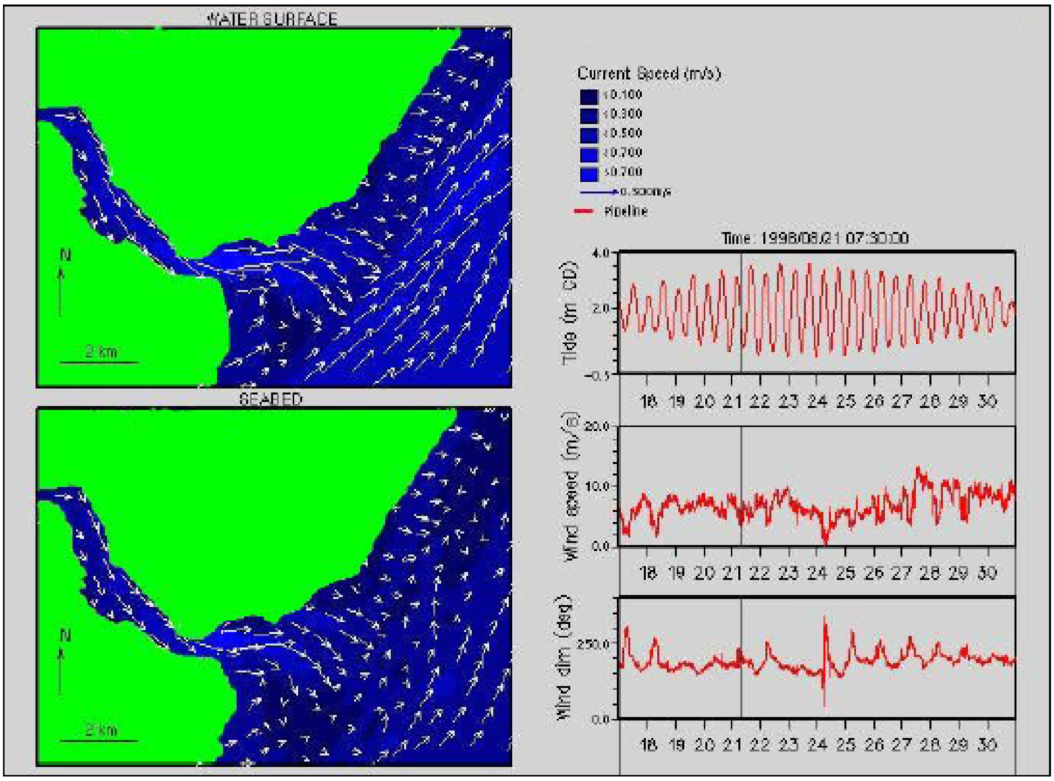



*FIGURE 6.24 a: Example: Spatial output from a 3-D numerical model, showing current velocities and direction*

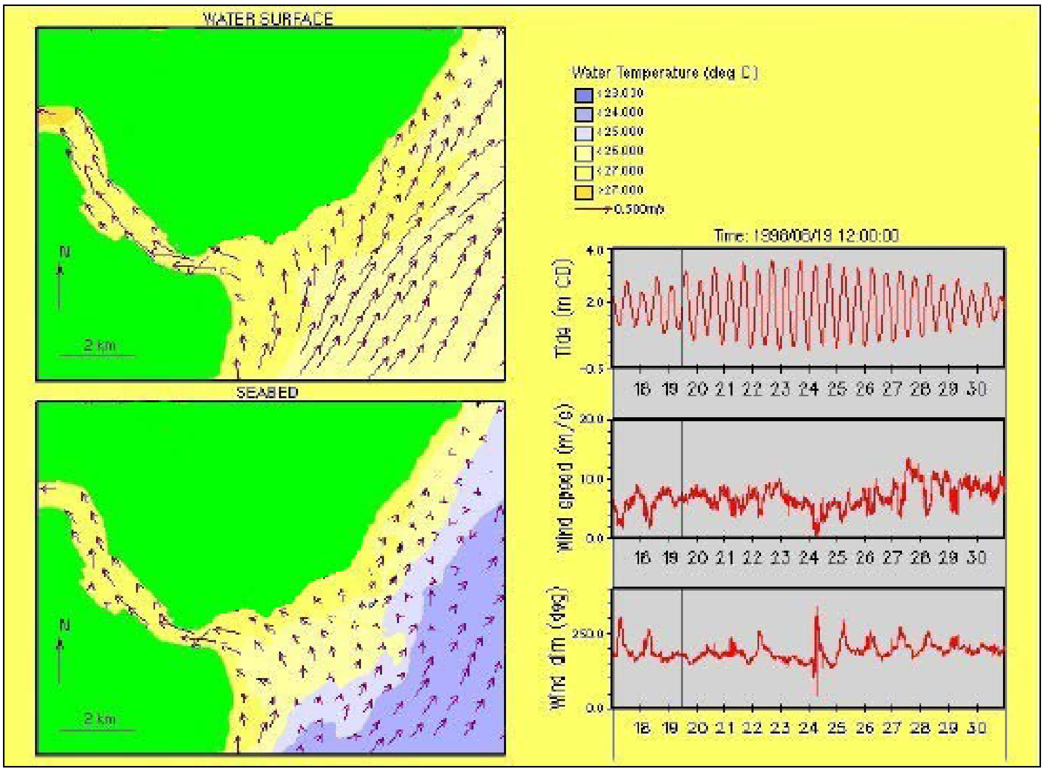



*FIGURE 6.24 b: Example: Spatial output from a 3-D numerical model, showing temperature distribution*

A theoretical schematisation for a 1-D model for estuaries, with motion in the horizontal plane, can be expressed in the following differential equations:

Along the estuary, choosing the x-axis in the upstream direction, the equation for motion at time t is:

$\partial h/\partial x \quad = 1/(g.A).\ \partial Q/\partial t - |Q|Q/(C^2.A^2.R) + 2b.Q/(g.A^2).\ \partial h/\partial t + W_x/(\rho.g.R)$

The equation for continuity of flow is:

$\partial Q/\partial x \quad = -b.\ \partial h/\partial t$

Where:

h = water level (m)
x = Distance upstream (m)
Q = Flow ($m^3/s$)
A =Cross-sectional area ($m^2$)
b = Stream width (m)
t = Time (s)
C = Chezy coefficient for friction
R = Hydraulic radius (m)
$W_x$ =Wind factor = $\tau_W$ Cos $\theta$
$\tau_W$ = Wind shear stress = $\rho_{air}.\ C_D.\ V_{10}^2$
$C_D$ = Drag coefficient = $0.5.V_{10}^{1/2}.10^{-3}$
$V_{10}$ = Wind velocity, 10 m above water surface (m/s)
$\theta$ = Angle between the wind and the channel direction
$\rho$ = Water density ($kg/m^3$)

All the differential quotients (example $\partial h/\partial x$ ) can be replaced with finite difference quotients for example:

$\partial h/\partial x \quad = (h(x + \Delta x,t) – h(x – \Delta x,t))/2\Delta x$ (a central difference approximation)

The two equations can be solved using explicit numerical methods (unknown values of h and Q at a certain time are expressed directly in terms of the known values at previous time steps) or implicitly (solving of the unknown values of h and Q over the entire model area).

For the computation of the concentration of water quality constituents (e.g. nutrients or trace metals), the change in the load (volume *x* concentration) with time can be expressed as the diffusive and advective transport in the equation below:

$\partial(V.C_C)/\partial t \quad = \partial(A.k.\ \partial C_C/\partial x)/\partial t.dx - \partial(A.u.\ C_C/\partial x)/\partial\ x.dx$

Where:

V = Volume ($m^3$)
$C_C$ = Concentration ($kg/m^3$)
k = Diffusion coefficient ($m^2s$)
u = mean velocity (m/s)

Terms to represent microbiological decay or point source loads can be added to the model as required.

A 1-D numerical model is a fairly simple technique, easy to apply, and if the estuary can be classified as 1-dimensional, provides valuable information with regard to the hydrodynamics and the subsequent mixing and transport of a point source. An example of the outputs (water levels, flows and velocities) of a calibrated 1-D model at three locations for the Swartkops River Estuary (Figure 6.25) is provided in Figure 6.26. The differences in flow and stream velocities at the three locations clearly indicate the importance of the spatial selection of an outfall location with regard to the mixing and transport of a wastewater plume.

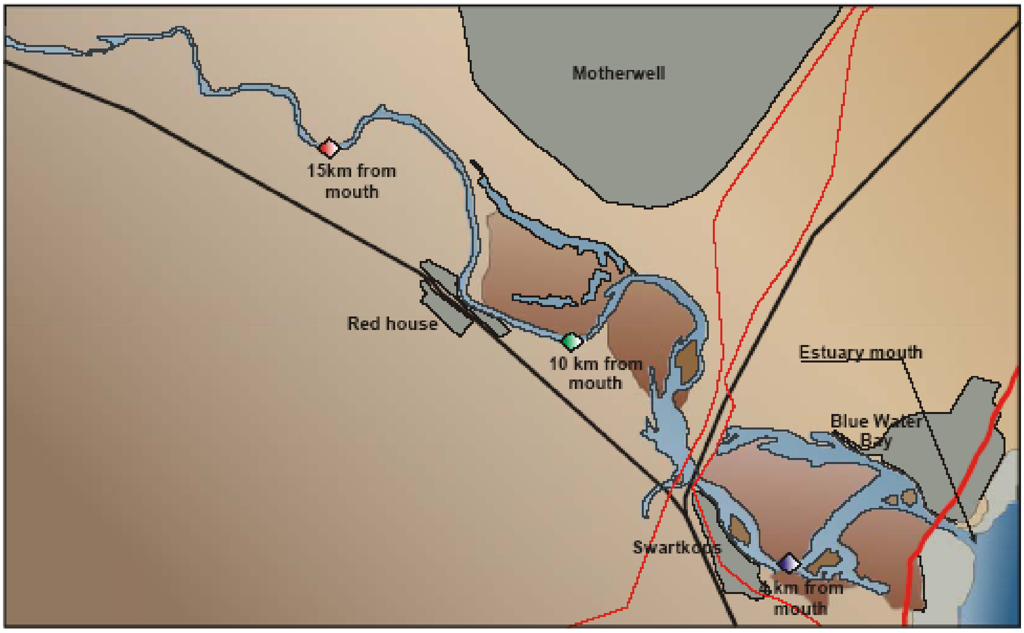



*FIGURE 6.25: A map of the Swartkops Estuary, Port Elizabeth*

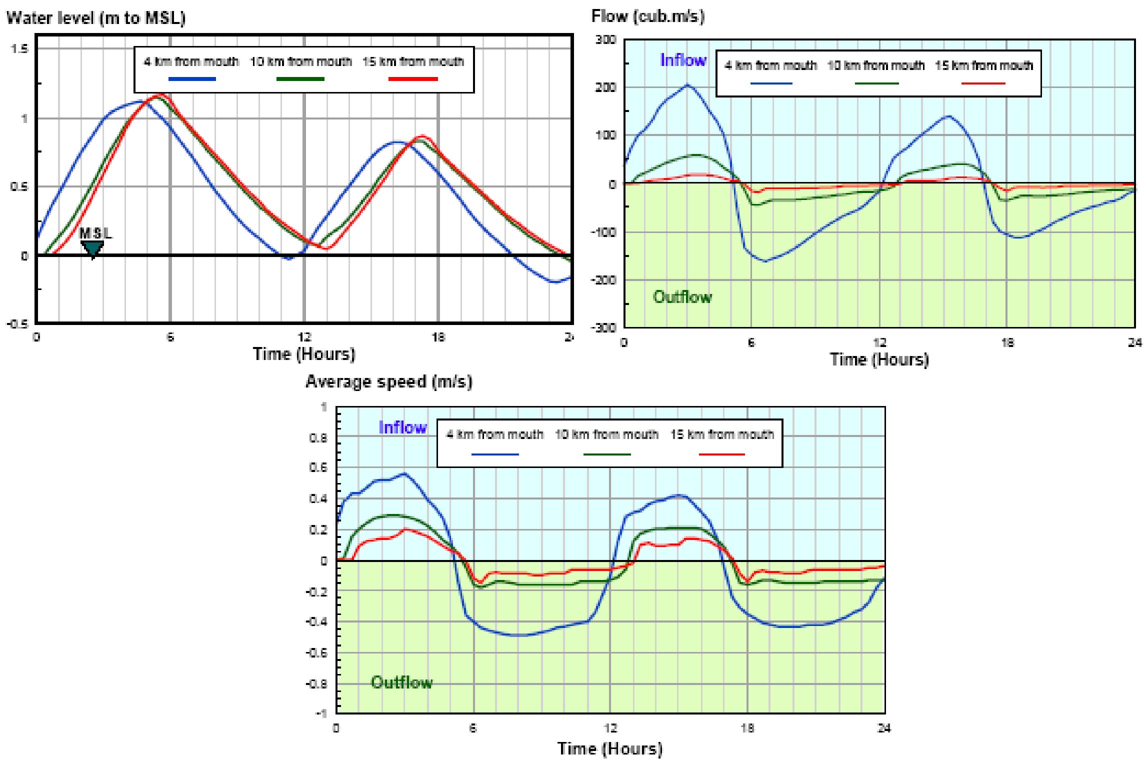



*FIGURE 6.26: Water level simulations, flow rates and average stream velocities simulated at different locations in the Swartkops River Estuary under zero river inflow*

Where a 2-D numerical model is not available, the 1-D model schematisation approach can be expanded to simulate the hydrodynamics of a more complex 2-D case by creating 1-D branches for all the streams, as illustrated for the Knysna Estuary in Figure 6.27.

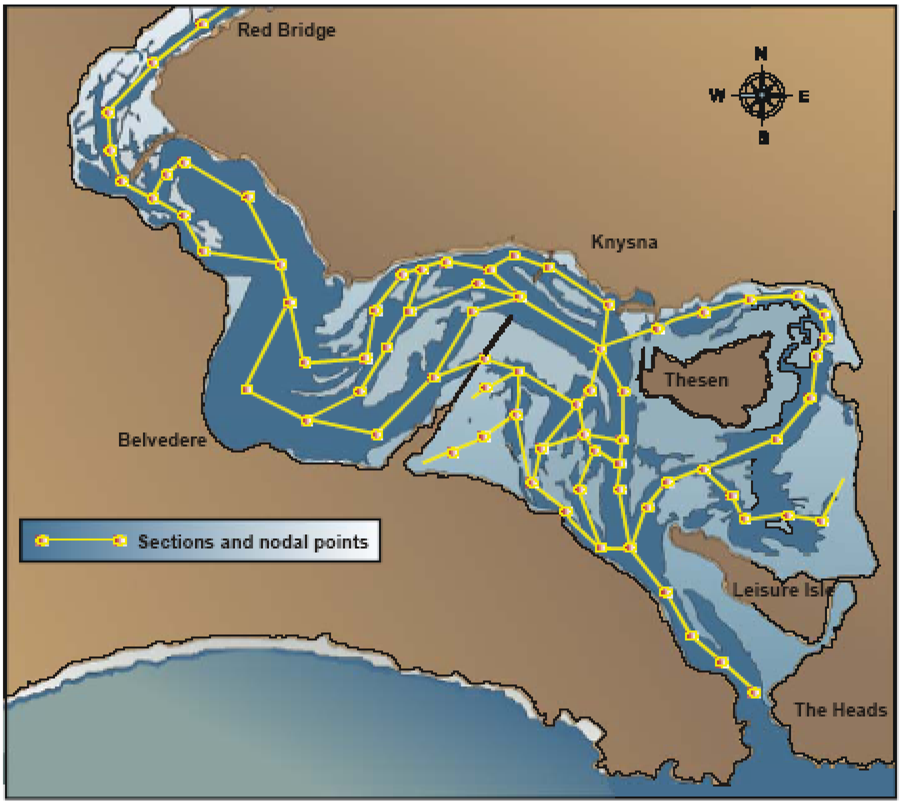
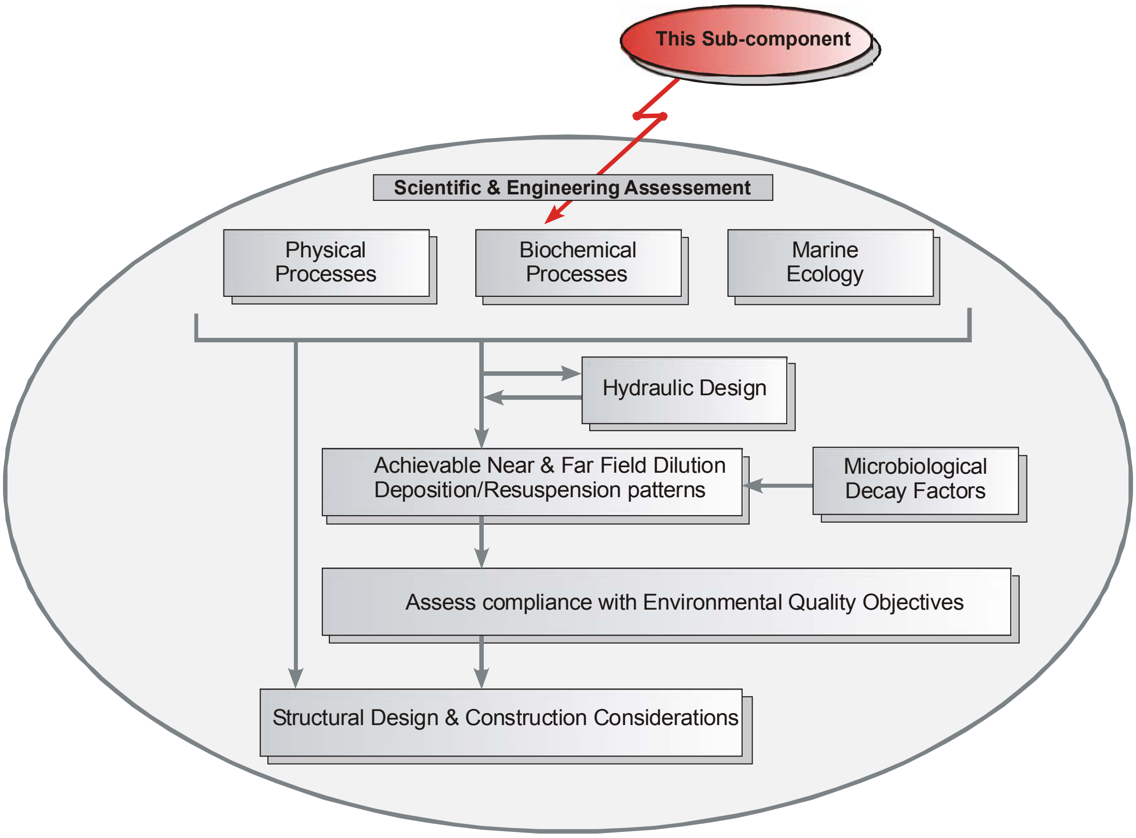



***FIGURE 6.27: 1-D schematisation for the Knysna Estuary***

## 6.2 BIOGEOCHEMICAL PROCESSES (WATER COLUMN AND SEDIMENT)




***PURPOSE:***

***The purpose of this component is to gain an understanding of those key biogeochemical characteristics and processes in the study area that may influence, or will have an influence on, the transport and fate of the wastewater plume. Any existing human interferences/activities in the study area, e.g. existing waste disposal or coastal structures, should also be taken into account.***

***Because of the site-specific nature of these types of investigations, this section is not intended to be prescriptive. Rather, it sets out the approach to follow when formulating a biogeochemical measurement programme as part of the scientific and engineering assessment process for marine disposal of land-derived wastewater. In particular, it highlights important factors to be taken into account when designing such measurement programmes, as well as the type of output that is required to provide information with sufficient confidence to allow for sound management decisions.***

***NOTE:***

**Methods for the Determination of the preliminary Ecological Reserve for Estuaries *(RSA DWAF, 2004), issued under the National Water Act, through the Directorate: Resource Directed Measures, provide guidance on the design of baseline data collection programmes. To ensure alignment with existing initiatives on estuaries, those methods (or future updates thereof) need to be consulted when designing baseline monitoring programmes for estuaries as part of this operational policy (proposed refinements to the design of baseline data collection programmes for estuaries are also provided in a WRC report entitled:* Resource Monitoring Procedures for Estuaries for Application in the Ecological Reserve Determination and Implementation Process *[Taljaard et al., 2003]).***

### 6.2.1 Overview

Biogeochemical characterisation of the marine environment requires data on the spatial and temporal variability of biogeochemical parameters, both in the water column and in the sediments, as well as an understanding of the key processes that govern such variability. It is important that data used in the characterisation reflect the present status of the receiving marine environment, i.e. any modifications to the biogeochemical characteristics and processes as a result of existing human activities need to be taken into account. This is particularly relevant when assessing the suitability of historical data sets.

Information from the physical processes study programme can be used to assist in the design of the biogeochemical data collection programme, particularly in terms of setting the critical time and space scales.

In addition to assisting in understanding the biogeochemical processes characteristic of the receiving marine environment, biogeochemical data are also used to calibrate and test the validity of model predictions (where applicable), as well as to provide a benchmark (baseline) for future monitoring programmes. It is important, therefore, that the manner in which biogeochemical data are collected is appropriate, e.g. model calibration and validation of water column parameters usually require time series data collected over a pre-determined time-scale.

### *i. Receiving marine environment*

The selection of measurement parameters to be used in the receiving environment is site-specific. A key determining factor in the selection of such parameters is the composition of the proposed wastewater discharges as well as the anticipated effects on the biogeochemical characteristics of, and processes in the receiving environment. Essential, therefore, to the design of the biogeochemical measurement programme is the preparation of a preliminary conceptual model of the key biogeochemical processes governing the 'cause-and-effect' linkages between the wastewater discharge and the receiving environment.

Biogeochemical parameters (e.g. pH, dissolved oxygen, turbidity, particulate organic carbon and nitrogen, dissolved nutrients, toxin concentrations and microbiological parameters) can be measured in the water column and/or the sediments, including interstitial waters.

Depending on the nature of the investigation, sediment data should be collected from sub-tidal and/or inter-tidal sediments. An understanding of the physico-chemical characteristics of inter-tidal sediments is particularly relevant where, for example, a wastewater discharge to the surf zone is under investigation.

Spatial scales at which data need to be collected vary. For example, time series data collected from the water column may require only one or two pre-selected locations, whereas data on spatial distribution patterns require more intensive sampling. A guiding principle is that the initial sampling should cover the near and far field scales (e.g. an entire bay), making no assumptions about the locations of, for example, short- and long-term deposition sites, in the case of sediment sampling. This typically requires a high resolution unbiased grid.

The temporal scale at which biogeochemical data need to be collected, as part of the measurement programme, largely depends on:

- The variability in the load of contaminants from waste inputs
- The variability in processes driving transport and fate of the wastewater plume in the receiving environment
- The temporal sensitivity of the ecosystem to contaminant loading, i.e. exposure time versus negative impact.

The temporal scale of sampling should at least resolve the main source of natural variability of the constituent under investigation. Scales of temporal variability are very different in the water column (minutes – days) compared with sediments (days – seasons – decades). Non-periodic events, such as storms, can also have a dramatic influence that needs to be taken into account where appropriate. Therefore, a sampling frequency that is too low relative to the underlying natural variability, will result in biased data that will make it difficult, for example, to separate an anthropogenic impact from a natural water quality anomaly.

***EXAMPLE…***

A time series plot, showing the natural variability of dissolved oxygen concentrations (mℓ/ℓ) at a depth of 10 m in Small Bay (Saldanha Bay, South Africa) is provided in Figure 6.28 (Monteiro *et al.*, 1999). It shows regular intrusion of cold, low oxygen (< 2mℓ/ℓ) coastal waters into the bay (mℓ/ℓ multiplied by density of $O_2$ = mg/ℓ).

Weekly sampling would indicate that there is an apparently random variability of high and low oxygen concentrations. Hourly sampling (automated) shows that this variability is linked to variability in upwelling, and the low oxygen concentrations are brought into the system by those upwelled waters rather than by any localised eutrophication effects. Weekly sampling would result in an apparently random variability of high and low concentrations.

The results of the monitoring show that natural variability needs to be well characterised prior to interpreting the impacts of effluents on oxygen concentrations in the receiving water body.

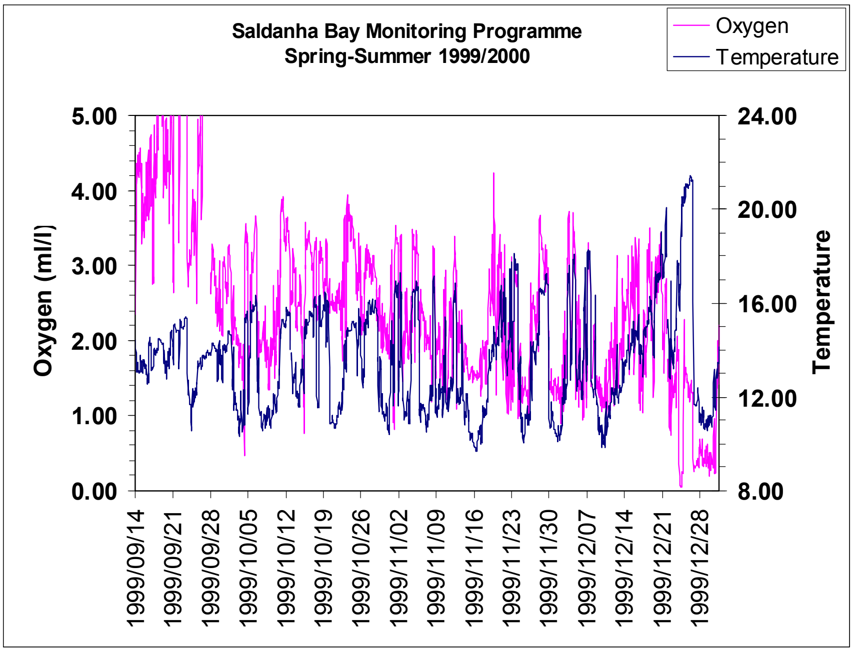



***FIGURE 6.28: Dissolved oxygen variability in the bottom water layer in Saldanha Bay, South Africa (from Monteiro et al., 1999)***

***EXAMPLES…***

I. A proposed wastewater discharge containing trace metals is considered for disposal to a marine environment receiving turbid (muddy) river inflow. For such a case, the likely fate of trace metals will be to adsorb onto the fine particles and to be deposited at some location, usually in the far field. The measurement programme, therefore, needs to be designed to enable the characterisation of the variability in river inflow and turbidity, the processes governing such variability, as well as the potential depositional sites in the study area. This will require:

- Time series data on the variability of salinity, temperature and turbidity at pre-selected position/s in the study area (e.g. intensive sampling over a period of two weeks to a month, during periods of high river inflow)
- A spatial survey of the particle size distribution, particulate organic carbon and nitrogen, and trace metal distribution patterns in sediments within the study area.

II. A proposed wastewater discharge contains relatively high dissolved inorganic nitrogen concentrations. In such, a case the processes driving nutrient distribution and primary production and the subsequent degradation of organic matter, need to be characterised. Measurement parameters need to include:

- Time series data on the variability of salinity, temperature, pH, dissolved oxygen, dissolved inorganic nitrogen and phosphate at pre-selected position/s in the study area (e.g. intensive sampling over a period of two weeks to a month, during periods of high river inflow)
- A spatial survey of the particle size distribution, particulate organic carbon and nitrogen distribution patterns in sediments within the study area.

***EXAMPLE…***

Plots showing the modelled and measured distribution of low turbulence (bed shear stress) zones in Saldanha Bay, South Africa are provided in Figure 6.29 (Monteiro, *et al.*, 1999). These plots show that long-term deposition of contaminant carrying fine particles will only occur at certain locations (blue: model and red in observations). The modelled results show that the wave climate as well as the currents governs the distribution of long-term and short-term depositional zones. The long-term depositional zones are the most vulnerable to contaminant accumulation.

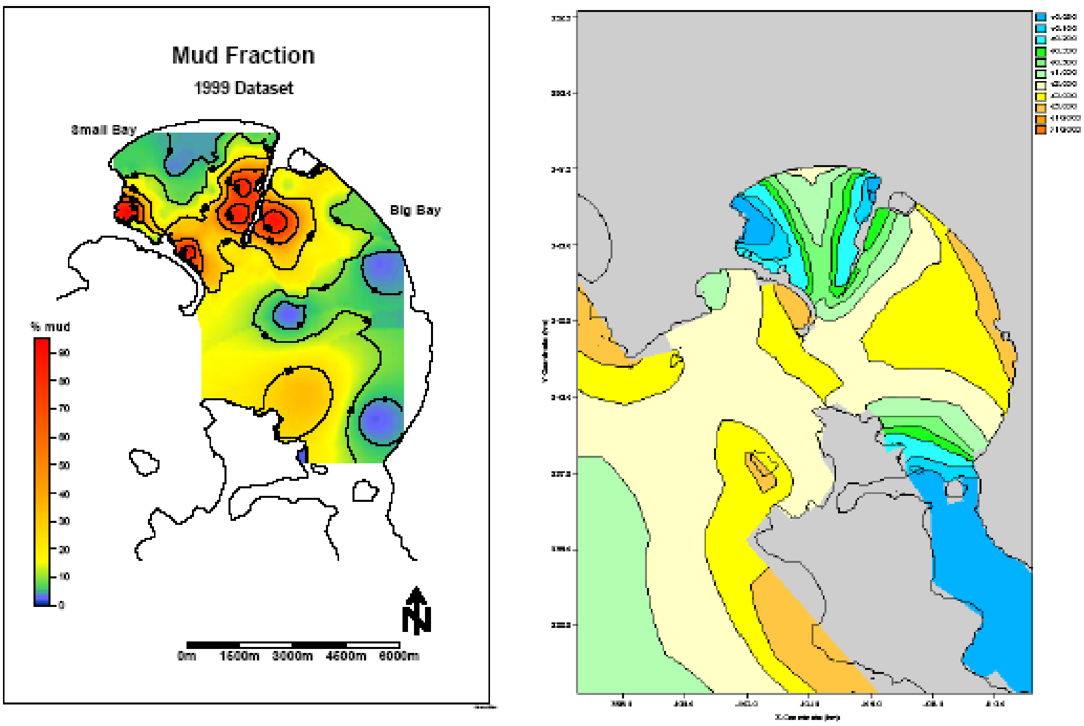



***FIGURE 6.29*** ***Plots showing the measured particle size distribution in Saldanha Bay, as well as the modelled distribution of low turbulence (bed shear stress) zones in Saldanha Bay, South Africa (Monteiro et al., 1999).***

### *ii. Behaviour of constituents*

Immediately after entering the marine environment, constituents present in a wastewater discharge can either (WHO, 1982):

- Remain in solution (i.e. remain in the 'dissolved phase')
- Adsorb onto solid phase particles
- Precipitate from the water column.

Another type of transformation is that of certain poly-aromatic hydrocarbons, in particular volatile organics (e.g. benzene, toluene, naphthlalene and xylene). On entering marine waters, such compounds do not follow the conventional 'dilution' behaviour. It is thought that these substances are actually extracted out of the aqueous phase and into the buoyant hydrophobic fraction that results in concentration as a film at the water's surface (referred to as the surface micro-layer), which subsequently evaporates to the atmosphere, rather than diluting. It will be extremely difficult to predict the transport and fate of such volatile substances in the receiving environment. Removing such compounds from the wastewater before discharging to sea best mitigates their potential risk to the marine ecosystem and other beneficial uses.

It is important, therefore, as part of the biogeochemical assessment process, to assess possible biogeochemical transformations of the wastewater (i.e. immediately after it enters the marine environment) that could have a major influence on the predicted transport and fate of the specific constituents in the wastewater plume.

***Dissolved phase***. Constituents associated with the 'dissolved' phase can either behave conservatively (i.e. their behaviour reflects only the advective and dispersive characteristics of the water body) or non-conservatively (i.e. they are rapidly transformed on entering the marine environment as a result of system variables such, as pH, salinity and temperature, being different from that in the wastewater).

Although changes in concentration of constituents behaving non-conservatively are difficult to quantify without sophisticated tools such as numerical models, the concentration after a given dilution of a constituent behaving conservatively can be calculated as follows:

$$C_{AE} \,.\, (Q_A + Q_E) = C_E.Q_E + C_A.Q_A$$

Where
$Q_A$ = Receiving water flow rate
$Q_E$ = Effluent flow rate
$C_E$ = Effluent concentration
$C_A$ = Ambient concentration in the receiving water
$C_{AE}$ = Concentration after mixing

The physical dilution (S) of the effluent can be expressed as the following dimensionless parameter:

$$S = (Q_E + Q_A)/Q_E \text{ or } Q_A = Q_E(S - 1)$$

Substituting $Q_A$ in the mass balance equation, the concentration after mixing is:

$$C_{AE} = (C_E + S.C_A - C_A)/ S$$

***Adsorption***. On entering the marine environment, toxic compounds such as trace metals and poly-aromatic hydrocarbons, poly-nuclear aromatics and pesticides, tend to adsorb onto 'solid' phase particles present either in the wastewater or in the receiving environment. 'Solid' phase particles comprise cohesive (non-biological) particles and organic particles.

Cohesive (non-biological) particles represent very fine sediment particles (< 60 µm) on which adsorption phases such as aluminium hydroxides, manganese hydroxides and iron hydroxides are common. The origin of the organic particles can be natural (e.g. phytoplankton) or introduced through anthropogenic activities (e.g. sewage disposal).

Adsorption to 'solid' phase particles is typically described by means of equilibrium partitioning, on the basis of partition coefficients, which are different for each 'solid' phase particle. Partition coefficients, in this context, are defined as (US-EPA, 1999):

For a given pH and assuming that concentration of the 'solid' phase particles is in excess with respect to $C_d$:

$K_d \sim C_p/C_d$

where

$K_d$ = Partition coefficient (in ml/g)
$C_p$ = Concentration of trace metal adsorbed onto the 'solid' phase particle at equilibrium ( in µg/g)
$C_d$ = Concentration of trace metal remaining in solution at equilibrium (in µg/mℓ)

The transport and fate of chemical constituents associated with the 'solid' phase is largely determined by the flux and sedimentation/re-suspension behaviour of solid phase particles. The sedimentation/re-suspension behaviour of solid phase particles is a sensitive indicator of the potential fate of toxic compounds in the receiving marine environment (Luger *et al.,* 1999; Monteiro, 1999).

***Precipitation***. A rise in pH and oxygen content promotes the formation of metal hydroxides, carbonates and other metal precipitates. Under such conditions, if the concentration of a trace metal is higher than the solubility of the least soluble compounds that can be formed between the metal and available anions in the receiving water, precipitation will occur.

Where appropriate, solubility products and stability constants, which describe precipitation processes and which are specific to the metal/anion complex, need to be sourced from the literature in order to quantify such transformations (Stumm & Morgan, 1970; Faust & Aly, 1984). However, most metals, with the exception of iron (Fe) and manganese (Mn) that readily precipitate their hydroxides, will usually remain in solution in seawater at concentrations, which are much higher than those occurring naturally (Solomons & Förstner, 1984; WHO, 1982).

***EXAMPLE…***

An iron (Fe)-rich, strongly acid effluent (Fe will be in the dissolved phase) will be neutralised on contact with seawater (releasing $CO_2$). The change in pH will result in the precipitation of the Fe (i.e. Fe will be in the solid phase). This modification in constituent characteristics needs to be taken into account when quantifying the transport and fate of the wastewater plume in the far field.

### 6.2.2 Data requirements for pre-assessment and detailed investigation

#### *i. Pre-assessment*

A pre-assessment is usually based on published or archived information and data, either from the study area or comparable sites. As much data and information as possible, as described in Section 6.2.1, need to be collated. For the pre-assessment, conceptual and analytical assessment techniques (rather than far field numerical modelling), are used to produce:

- An initial quantitative description of the biogeochemical characteristics of and processes in the study area
- An estimate of the ambient concentrations of relevant biogeochemical parameters
- An identification of the potential depositional areas and estimate of the extent of contamination as a result of existing waste inputs (where data are available, prepare a contour map showing the distribution of toxic compounds). The sediment grain size and particulate organic matter distribution patterns are crucial for the interpretation of results.
- A quantitative assessment of the chemical behaviour of constituents in the wastewater immediately after being discharged to the marine environment.

As part of the pre-assessment, it is also necessary to estimate required dilutions since such information is used as an input parameter in the hydraulic design of offshore marine outfalls, i.e. to provide a first indication of the <u>initial</u> dilution that needs to be achieved in the design of the outfall.

Required dilutions are defined as the dilution necessary to ensure compliance with the environmental quality objective recommended for a particular constituent, taking into account the present concentration in the receiving marine environment as well as the proposed concentration in the wastewater. The required dilution is calculated as follows:

Referring to the mass-balance equation…

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

Where
$C_E$ = Effluent concentration
$C_A$ = Ambient concentration in the receiving water
$C_G$ = Environmental quality objective (target value)
$S$ = Required dilution

***EXAMPLE…***

The dissolved ammonia concentration is about 30 mg/ℓ after preliminary treatment of sewage:

| | |
|---|---|
| Concentration of the effluent ($C_E$): | 30 mg/ℓ |
| Ambient concentration in the sea ($C_A$): | Low, assume 0.001 mg/ℓ |
| Guideline ($C_G$): | 0.600 mg/ℓ |

The required dilution (S) is:

$$S = (30 – 0.001)/(0.600 – 0.001) = 50$$

### *ii. Detailed investigation*

A detailed investigation requires an intensive data collection programme to acquire the data and information on the relevant physical processes, as described in Section 6.2.1

In a detailed assessment of the biogeochemical characteristics and processes, numerical modelling tools such as water quality models in association with hydrodynamic models need to be applied. These models are particularly useful in the interpretation of existing knowledge of the key processes and thus help to define the temporal and spatial variability of the biogeochemical characteristics of the receiving marine environment.

Outputs required for the characterisation of biogeochemical processes, as part of a detailed investigation, include:

- A contour map showing the distribution of relevant chemical constituents in the marine sediments of the study area, including details on sediment particle size distribution and particulate organic carbon and nitrogen. Expected variability, both temporally and spatially, need to be addressed.

  ***NOTE:***

  ***Geochemical ratios of trace metals can be used to determine whether the trace metals are of natural or anthropogenic origin. It is possible for conditions to arise in which the total trace metal concentration in the sediment is high (particularly in depositional areas) but completely linked to the natural structure of clay minerals, in which case the trace metals will not be bio-available. This condition would be characterised by geochemical ratios very similar to those of unpolluted sediments typical of the area. The geochemical ratio of each trace metal relative to aluminium (TM [µg/g]: Al [%]) is used, usually allowing a conservative 2-fold natural variation in the geochemical ratios. Natural geochemcial ratios are site specific for different geographical regions and need to be sourced from the literature (Monteiro & Scott, 2000).***

- Graphs showing the temporal (and, where applicable, spatial) variability of system variables (temperature, salinity, dissolved oxygen and suspended solids/turbidity), inorganic nutrients (nitrate, ammonia, reactive phosphate and reactive silicate), and organic nutrients (dissolved organic carbon, particulate organic carbon and particulate organic nitrogen) in the water column.

- Description of the expected interaction of the constituents of waste inputs with biogeochemical processes in the receiving marine environment, e.g. whether the constituents are in the 'dissolved' phase (i.e. remain in solution), will precipitate from the water column, or whether they are in the 'solid' phase (e.g. adsorbed to solid phase particles).

As part of a detailed investigation, numerical modelling should be used to refine the required dilutions that were estimated during the pre-assessment stage (refer to Section 6.2.2).

Numerical modelling techniques have proven to be powerful tools in that:

- Models provide a workable platform for incorporating the complexity of spatial and temporal variability in the marine environment

- Model assumptions and inputs provide a means of synthesising an existing understanding of the key processes

- Modelling assists in defining the most critical spatial and time scales of potential negative impacts in the receiving system

- Model outputs provide quantitative results which can be used, together with field data, to check the quality of assumptions and insights.

However, in the application of numerical modelling techniques, the following must be complied with:

- The model chosen must be appropriate to the situation in which it is utilised

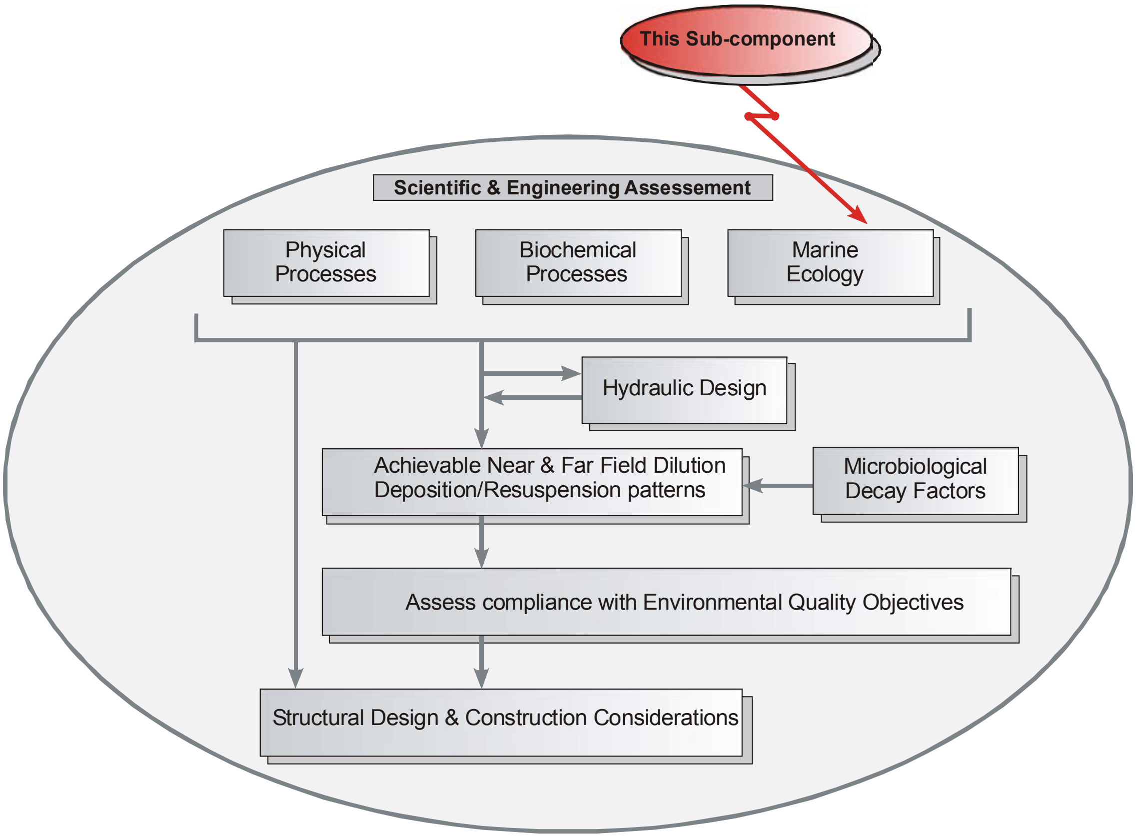
- The model must be calibrated and validated against a full field data set adequately describing the site-specific physical and biogeochemical oceanographic conditions (‘ground truthing’)

- A sensitivity analysis must be conducted to demonstrate the effect of key parameters, based on the variation in input data and controlling assumptions

- The reporting of model outputs must include a clear description of assumptions, a summary of numerical outputs, and confidence limits and sensitivity analyses.

A basic overview of statistical techniques that are typically applied in the evaluation of environmental data, where and when appropriate, is provided in Section 7.4.2. It highlights important factors that need to be taken into account when applying statistical analyses to data sets and is by no means exhaustive.

## 6.3 MARINE ECOLOGY




***PURPOSE:***

***The purpose of the ecological component, as part of the scientific and engineering assessment, is to:***

- ***Establish the biological resources within the study area***
- ***Identify biological resources that are of high conservation value***
- ***Establish which biological resources have already been lost or are stressed by anthropogenic influences, including existing waste inputs***
- ***Identify biological resources that are particularly sensitive to anthropogenic influences in the area (both existing and proposed)***
- ***Refine the ecological objectives for the study area (e.g. the description of the balanced indigenous population [US-EPA, 1994]) and related objectives pertaining to water quality***
- ***Assist in determining a suitable pipeline route so as to minimise damage to the marine ecology***
- ***Provide ecological baseline data for future monitoring programmes.***

### 6.3.1 Overview

To characterise the ecology of a particular marine environment, data on the following are required:

- Identification of habitat types, e.g. reefs, kelp beds, sandy and rocky bottoms
- Community structure within each of the habitat types
- Community composition and list of species (and abundance) associated with the different habitat types, focusing on dominant species, species of particular conservation importance and species targeted for exploitation.

The high mobility of pelagic and planktonic organisms in the water column makes representative sampling nearly impossible and particular care should be taken when interpreting data on such organisms. In addition, the distribution and abundance of marine organisms often show strong diurnal and/or seasonal variability, depending on numerous climatic, physical and biogeochemical factors. It is important, therefore, to ensure such information is collected simultaneously and is taken into account when interpreting the ecological data.

Ecological data should be adequate to perform valid statistical and community analyses as proposed below.

***NOTE:***

***For estuaries, guidelines on baseline data requirements are provided in the* Methodology for the Determination of the Preliminary Ecological Reserve for Estuaries *(RSA DWAF, 2004) issued under the National Water Act. Proposed refinements to these methods are provided in a WRC report entitled:* 'Resource Monitoring Procedures for Estuaries for Application in the Ecological Reserve Determination and Implementation Process' *(Taljaard et al., 2003).***

### 6.3.2 Data requirements for pre-assessment and detailed investigation

### *i. Pre-assessment*

A pre-assessment is based on published or archived information and data from the study area or comparable sites. As much data and information as possible, as described in Section 6.3.1, need to be collated.

As part of a pre-assessment it is important to provide the following:

- A map showing, at least conceptually, the distribution of the various habitat types and the associated biological resources (i.e. to refine the beneficial use map in terms of the distribution of marine ecosystems), and highlighting areas with:

    - Biological resources of conservation importance
    - Biological resources targeted for exploitation
    - Biological resources that have been lost, or are stressed, as a result of anthropogenic influence.

    (Site photography and video recordings have been used effectively to assist in providing information on the above.)

- Identification of the dominant species, species of particular conservation importance and species targeted for exploitation, providing best estimates of spatial and temporal variability.

- Identification of biological resources that are potentially sensitive to anthropogenic influences already present in the area and/or that may be sensitive to constituents present in the proposed wastewater discharge, and quantification of cause-and-effect relationships as best as possible (i.e. to refine the ecological quality objectives).

### *ii. Detailed investigation*

A detailed investigation requires an intensive data collection programme to acquire the data and information on the relevant ecological processes, as described in Section 6.3.1. In a detailed assessment of the ecological characteristics and processes, ecological modelling tools can be applied. These models are particularly useful in interpreting existing understanding of the key processes in order to improve quantitative predictions with respect to cause-and-effect relationships.

For a detailed investigation, the ecological component should include:

- A geo-referenced map showing the distribution of the various habitat types and their associated biological resources (i.e. to refine the beneficial use map in terms of the distribution of marine ecosystems), highlighting areas with:

  - Biological resources of conservation importance
  - Biological resources targeted for exploitation
  - Biological resources that have been lost, or are stressed, as a result of anthropogenic influence.

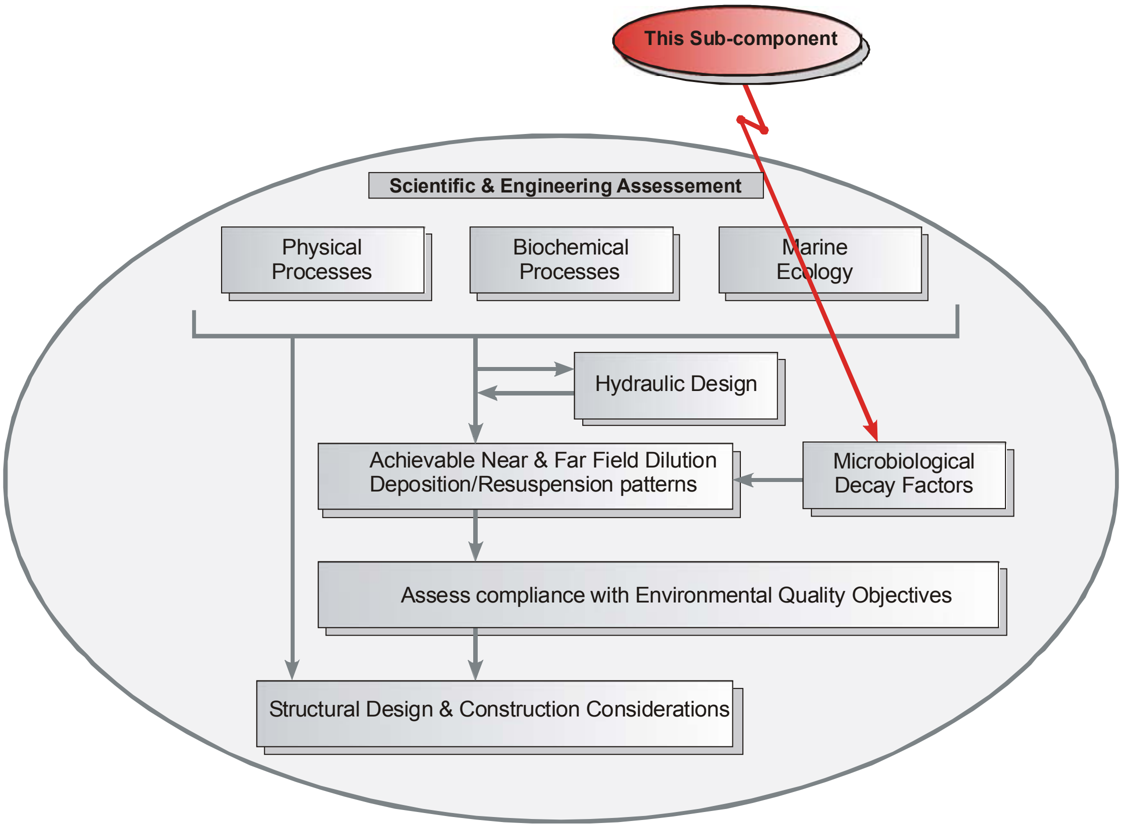
- For each of the habitat types, a listing of the key species and their abundance and community composition, as well as expected temporal and spatial variability (this may be expensive to obtain and it may therefore be more realistic to focus on selected indicator species and community structure)

- Confirmation of the presence of biological resources that are potentially sensitive to anthropogenic influences already present in the area and/or that may be sensitive to constituents present in the proposed wastewater discharge, and also provision of a quantitative assessment of cause-and-effect relationships (i.e. to refine the ecological quality objectives)

- Refinement of the ecological objectives (e.g. described in terms of a *balanced indigenous population*) for the study area (refer to Section 4.2.1 for more details) that can be used as the baseline or reference for future monitoring.

A basic overview of statistical techniques that are typically applied in the evaluation of environmental data, where and when appropriate, is provided in Section 7.4.2. It highlights important factors that need to be taken into account when applying statistical analyses to data sets and is by no means exhaustive.

## 6.4 MICROBIOLOGICAL FACTORS




***PURPOSE:***

***The purpose of this component is to determine the relevant microbiological indicator and associated decay coefficients to use in predicting microbiological die-off in the far field for a specific study area. These decay coefficients for the selected microbiological indicator, for example, depend on exposure to solar radiation (i.e. whether its day or night time) and water salinity of receiving environment.***

### 6.4.1 Overview

The types and numbers of various pathogens in sewage depend on the incidence of disease in the population of an area and the seasonal variation of infections (WHO, 1999). Numbers and types, therefore, will vary throughout the world and between seasons. A general indication of pathogenic numbers in sewage is provided in Table 6.6 (WHO, 1999).

***TABLE 6.6: A general indication of pathogens found in sewage, the associated diseases and typical counts (WHO 1999)***

| *PATHOGEN/INDICATOR* | *DISEASE/ROLE* | *COUNTS PER LITRE* |
|---|---|---|
| Bacteria | | |
| *Campylobacter spp.* | *Gastro-enteritus* | *37 000* |
| *Clostridium perfringens* | *Indicator* | $6 \times 10^5 – 8 \times 10^5$ |
| *E.coli* | *Indicator* | $10^7 – 10^8$ |
| *Salmonella spp.* | *Gastro-enteritus* | *20 – 80 000* |
| *Shigella* | *Bacillary dysentery* | *10 – 10 000* |
| Viruses | | |
| *Polioviruses* | *Indicator* | *1 800 – 5 000 000* |
| *Rotaviruses* | *Diarrhoea, vomiting* | *4 000 – 850 000* |
| Parasitic protozoa | | |
| *Cryptosporidium parvum oocysts* | *Diarrhoea* | *1 – 390* |
| *Entamoeba histolytica* | *Amoebic dysentery* | *4* |
| *Giardia lamblia cysts* | *Diarrhoea* | *125 – 200 000* |
| Helminths | | |
| *Ascaris spp.* | *Ascariasis* | *5 – 110* |
| *Ancylostoma spp.* | *Anaemia* | *6 - 190* |
| *Trichuris spp.* | *Diarrhoea* | *10 - 40* |

Methods to detect and identify infectious viruses and parasites are very expensive and do not exist for some. The use of indicator organisms to indicate the potential presence of harmful organisms has been used for a long time and the faecal indicator bacteria most commonly used today are thermotolerant coliforms, *E.coli* and Enterococci or faecal streptococci. The advantages and disadvantages of these bacteria as indicators are listed in Table 6.7 (WHO, 1999).

***TABLE 6.7: Advantages and disadvantages of different microbiological indicators***

| INDICATOR | ADVANTAGES | DISADVANTAGES |
|---|---|---|
| *Faecal streptococci/ enterococci* | *Marine and potentially freshwater human health indicator.*<br><br>*More persistent in water and sediments than coliforms.* | *May not be valid for tropical waters due to growth in soils.* |
| *Thermotolerant coliforms* | *Indicator of recent faecal contamination.* | *Possibly not suitable for tropical waters due to growth in soils and water.*<br><br>*Confounded by non-sewage sources (eg. Klebsiella spp. in pulp and paper wastewater)* |
| E. coli | *Potentially a freshwater human health indicator.*<br><br>*Indicator of recent faecal contamination.*<br><br>*Rapid identification possible if defined as b-glucuronidase producing bacteria.* | *Possibly not suitable for tropical waters due to growth in soils and water.* |

The effect of conventional sewage treatment on the removal of the major pathogen groups is illustrated in Table 6.8.

***TABLE 6.8: Effect of conventional sewage treatment on the removal of the major pathogen groups (adapted from WHO, 1999)***

| *TREATMENT* | *Enteric viruses* | *Salmonella* | *C. perfringens* | *Giardia* |
|---|---|---|---|---|
| *Raw sewage* | *100 000 – 1 000 000* | *5 000 – 80 000* | *100 000* | *9 000 - 200 000* |
| *Primary treatment* | | | | |
| *% removal* | *50 - 98,3* | *99,5 – 99,8* | *30* | *27 – 64* |
| *Counts remaining* | *1 700 – 500 000* | *160 – 3 360* | *70 000* | *7 200 – 146 000* |
| *Secondary treatment* | | | | |
| *% removal* | *53 – 99,92* | *98,65 – 99,996* | *98* | |
| *Counts remaining* | *80 – 470 000* | *3 – 1 075* | *2 000* | |
| *Tertiary treatment* | | | | |
| *% removal* | *99,983 – 99,999998* | *99,99 – 99,9999995* | *99,9* | *98,5 – 99,99995* |
| *Countsremaining* | *0 - 170* | *0 - 7* | *100* | *0 – 2 951* |

Potential risks associated with different levels of treatment and disposal location in the marine environment is indicated in Table 6.9 (WHO, 1999).

***TABLE 6.9: Potential risks associated with different levels of treatment and disposal location in the marine environment***

| *TREATMENT* | *DISCHARGE TYPE* | | |
|---|---|---|---|
| | *Surf zone or estuarine discharge* | *Marine outfall (less than 10 m water depth)* | *Marine outfall* (greater than 10 m water depth)* |
| *None* | *Very high* | *High* | *NA* |
| *Preliminary* | *Very high* | *High* | *Low* |
| *Primary* | *Very high* | *High* | *Low* |
| *Secondary* | *High* | *High* | *Low* |
| *Secondary and disinfection* | *Medium* | *Medium* | *Very low* |
| *Tertiary* | *Medium* | *Medium* | *Very low* |
| *Tertiary and disinfection* | *Very low* | *Very low* | *Very low* |

* *Assuming that the design capacity is not exceeded and that extreme climatic and oceanic conditions were considered in the design (i.e. the wastewater plume will not reach the beach).*

In 1999, the WHO (1999) concluded that the data available on $T_{90}$ values were inadequate for use in model predictions, especially in the near-shore zone. At that stage, the WHO referred to Chamberlain and Mitchell (1978), who gave a mean $T_{90}$ value of 2.2 hours for marine waters and a $T_{90}$ value of 58 hours for freshwaters. These numbers were obtained from *in situ* tests at wastewater outfalls. From these data numbers it can be assumed that the mean daytime $T_{90}$ value will be less than 2.2 hours. A summary of $T_{90}$ values provided by Gunnerson (1988) for a number of coastal areas is listed in Table 6.10

***TABLE 6.10: Summary of $T_{90}$ values for a number of coastal areas (Gunnerson, 1988)***

| *LOCATION* | *DATE* | *$T_{90}$ (hours)* |
|---|---|---|
| *Raw sewage* | | |
| *Honolulu* | *1970* | *$\leq$ 0.75* |
| *Titahi Bay, New Zealand* | *1959 – 1960* | *0.65* |
| *Rio de Janeiro* | *1963* | *1.0 – 1.2* |
| *Israel* | - | *< 1.0* |
| *Istanbul* | *1968* | *0.8 – 1.7* |
| *Genofte, Denmark* | - | *1.2* |
| *Tema, Ghana* | *1964* | *1.3* |
| *Nice, France* | - | *1.1* |
| *England* | *1965* | *0.78 – 3.5* |
| *Manila, Philippines* | *1968 – 1969* | *1.8 – 3.4* |
| *England* | *1969 – 1973* | *1.4 – 5.3* |
| *Mayaquez Bay, Puerto Rico* | - | *0.7* |
| *Montevideo, Uruguay* | - | *1.5* |
| *Santos, Brazil* | - | *0.8 – 1.7* |
| *Porlaleza, Brazil* | - | *1.1 – 1.5* |
| *Maceio, Brazil* | - | *1.2 – 1.5* |
| *Primary Treated Wastewater* | | |
| *Ventura, California* | *1966* | *1.7* |
| *Seaside, New Jersey* | *1966* | *1.8* |
| *Orange County, California* | *1954 – 1956* | *1.8 – 2.1* |
| *Santa Barbara, California* | *1967* | *2.4* |
| *Los Angeles, California* | *1954 - 1956* | *4.1* |

Typically, pathogenic organisms are modelled using faecal coliforms as the proxy with appropriate die-off responses to changes in ultraviolet light intensity linked to time of day and penetration of the water column. Night-time values for $T_{90}$ will be higher.

The variability of the $T_{90}$ value, as part of detailed investigations, is normally simulated using numerical models. For a pre-assessment, a value of 10 hours can be used.

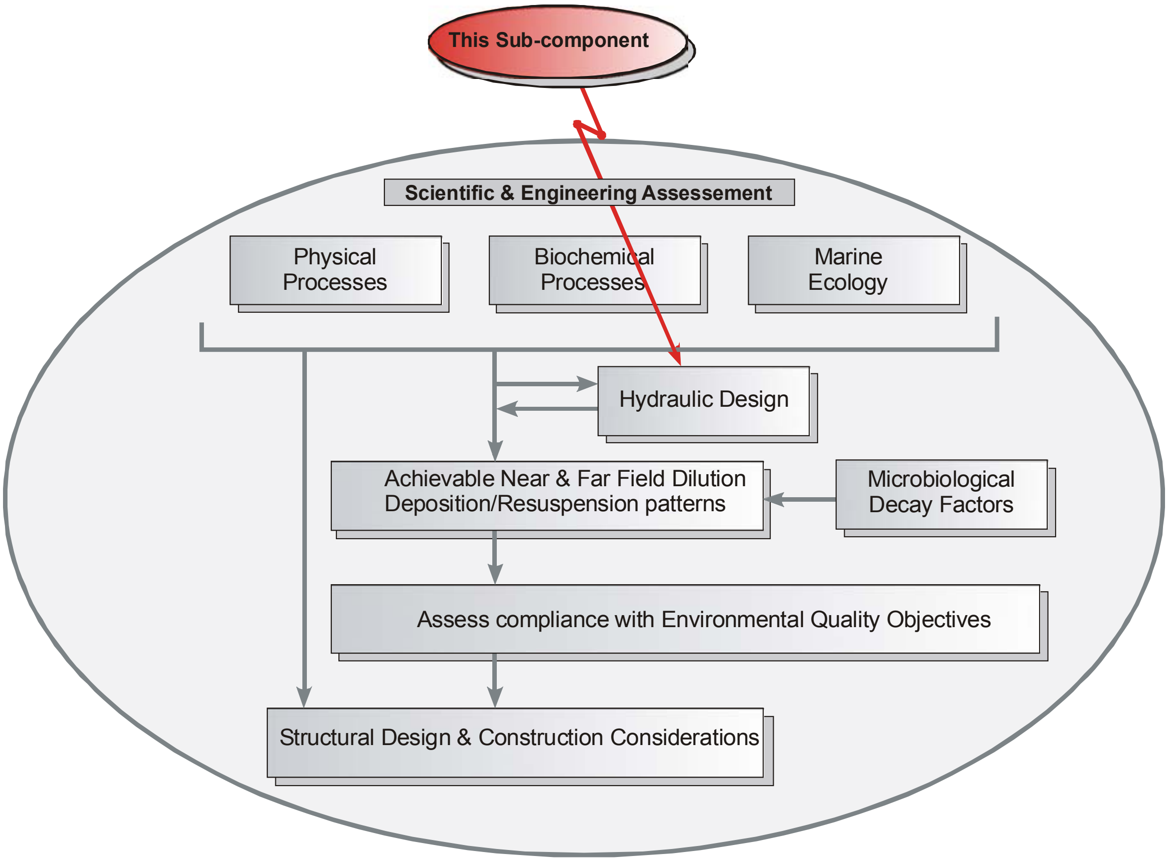
### 6.4.2 Data requirements for pre-assessment and detailed investigation

For a pre-assessment conservative die-off coefficient, for both daytime and night-time, considered to be representative of conditions at the study area, can be applied.

For a detailed investigation, the above needs to be refined, by using diurnal variations of daytime and night-time die-off coefficients in conjunction with variation in wastewater flow patterns. For large projects it may be required to measure actual die-off coefficients in the study area.

A basic overview of statistical techniques that are typically applied in the evaluation of environmental data, where and when appropriate, is provided in Section 7.4.2. It highlights important factors that need to be taken into account when applying statistical analyses to data sets and is by no means exhaustive.

## 6.5 HYDRAULIC DESIGN




***PURPOSE:***

***A sound hydraulic design for a wastewater discharge system includes the following main functional components:***

- ***The head works, to discharge the wastewater***
- ***The main outfall pipe, to convey the wastewater to the discharge location***
- ***The diffuser, to release the wastewater into the receiving environment.***

### 6.5.1 Outfall site selection

Prior to commencing with the scientific and engineering assessment process, it is important to conduct a preliminary on-site assessment of the possible location of a wastewater discharge system.

A well-designed wastewater discharge system consists of the following main components:

- Head works to discharge the wastewater
- Main outfall pipe to convey the wastewater to the discharge location
- Diffuser to release the wastewater into the receiving environment.

An illustration of the different components is provided Figure 6.30. The 'diffuser' component is specific to marine outfalls (i.e. wastewater discharges to the offshore marine environment).

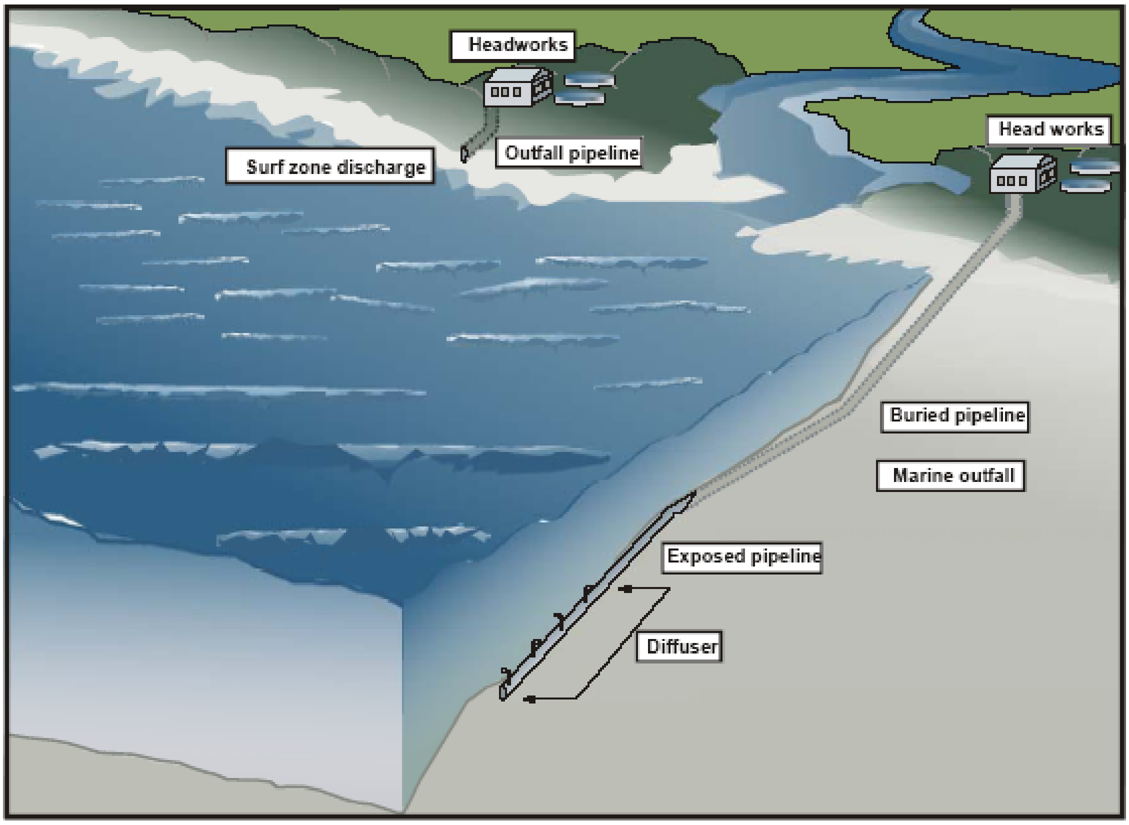



*FIGURE 6.30: An illustration of the components of a well-designed wastewater discharge system*

The location of the head works is the onshore end of the wastewater discharge system and will control the route of the outfall. Typically, the head works will be on the site where the wastewater is generated (e.g. the WWTW). For a new development, the following aspects have to be taken into account during the selection of the optimum site for the head works:

- The natural drainage of the area
- The existing or future planned reticulation system
- A suitable location for the land-based section of the system with regard to the area available, considering possible future extensions of the treatment processes as well as the existing or future development plans for the area.

In the selection of the discharge/diffuser location, the following needs to be taken into account:

- Delineated beneficial use areas along the coastline
- The bathymetry of the marine environment, keeping in mind the quality of the wastewater and the maximum required initial dilutions
- The coastline configuration and bathymetry, considering the route for the pipeline
- The typical physical processes that can be expected.

Link the onshore end (head works) to the offshore end (diffuser) via the shortest possible route for pipeline (length), considering the following basic requirements and restrictions:

- A shore crossing which will create the minimum interference to the physical characteristics of the coastline, ecology and the existing infrastructure, taking into account the most likely method of construction
- An onshore route (head works to shoreline) with regard to construction impacts on the existing and planned onshore infrastructure
- A gradually sloping seabed without, or with the minimum of, natural or manmade obstacles such as rock outcrops, underwater cables or shipwrecks.

### 6.5.2 Discharging of wastewater

Wastewater can be discharged using a gravity (potential) head, if available, or by using a pressure head provided by pumps. Where gravity flows are used, these will vary according to the diurnal flow patterns. Where pumps provide the pressure head, wastewater can be discharged intermittently at a specific flow rate from a storage tank.

***NOTE:***

***Energy (potential or gravity, kinetic and pressure) or energy losses (e.g. friction loss) per unit weight is expressed as 'head' (in meters water of specific gravity).***

There are a number of factors that need to be taken into account when planning the discharge of wastewater:

- ***Surge effects***. The most common pressure transient (or momentary) effect affecting a pumping system is the switching off of pumps or the uncontrolled loss of the power supply. Operation of valves can also cause large transient effects and, subsequently, pressure surges. Variations in discharge rates normally do not result in significant transient effects. Pressure surge analyses should be conducted for all marine outfalls that use pumps. This is to ensure that no structural damage occurs to any component of the outfall during any flow scenario as a result of possible transient effects that may occur during operations. Gravity surge, related to sudden pressure changes and subsequent rapid flow changes in a marine outfall, should also be carefully examined for both pumped systems and those using gravity flow.

- ***Air entrainment***. Air entrainment may occur during high flows in the drop shaft of an outfall, resulting in flow reduction and an increase in pressure.

- ***Saline intrusion***. Seawater intrusion into outfall diffusers can cause higher pumping heads, reduce the initial dilutions and result in sedimentation (Charlton *et al* 1987). As it is not practical to prevent saline intrusion completely without the use of non-return valves, the hydraulic design must be such that salt water can be purged from the pipeline and diffusers.

### 6.5.3 Main pipe diameter

The optimisation of the main pipe diameter, a key component in determining the cost and hydraulic performance of a marine outfall, depends on:

- Flow scenarios (present flow as well as the ultimate flow conditions)
- Available or practical head to discharge the wastewater (gravity or pumps)

In the case of small diameter outfall, scouring velocities can be maintained more easily and the capital investment will be lower. However headlosses resulting from friction will require higher pressure/gravity heads and subsequent higher running costs if a gravity head is not available. The design criteria for a first assessment are:

- Maintain a main pipe velocity of greater or equal to 0,7 m/s during low flows to prevent deposition of solids.
- Discharge maximum flows with available gravity head or by cost-effective pumping, taking into account the increase in roughness during the lifetime of the outfall as well as all losses at fittings (entrances, exits, bends, contractions, expansions, valves) in the main pipe and the diffuser.

Headloss ($h_f$) resulting from friction can be calculated using the Darcy-Weisbach equation (Shand, 1993):

$$h_f = \lambda L V^2/(2gD)$$

Where

$\lambda$ = friction factor
L = length of the pipe (m)
D = pipe diameter (m)
V = velocity in the main pipe (m/s)

The Colebrook-White formula can be applied to determine the friction coefficient ($\lambda$), as it correctly models the laws for smooth and rough pipes as well as for the transition zones (Shand, 1993). The Moody diagram for pipe friction (Shand, 1993) is given in Figure 6.31.

$$\lambda = 0.25[\log_{10}\{k_s/3.7D + 2.51/(R_e\lambda^{1/2})\}]^{-2}$$

Where

$k_s$ = roughness height (mm)
$R_e$ = Reynolds number

Typical roughness heights (i.e. the characteristic size of surface roughness) for pipe materials are (Shand, 1993):

| *PIPE MATERIAL* | *SMOOTH* | *AVERAGE* | *ROUGH* |
|---|---|---|---|
| *Plastic (PVC)* | *0.015* | *0.03* | *0.06* |
| *Coated steel* | *0.03* | *0.06* | *0.15* |
| *Cast iron or cement mortar lined* | *0.15* | *0.3* | *0.6* |
| *Spun concrete* | *0.3* | *0.6* | *1.5* |
| *Rough concrete or riveted steel* | *1.5* | *3* | *6* |

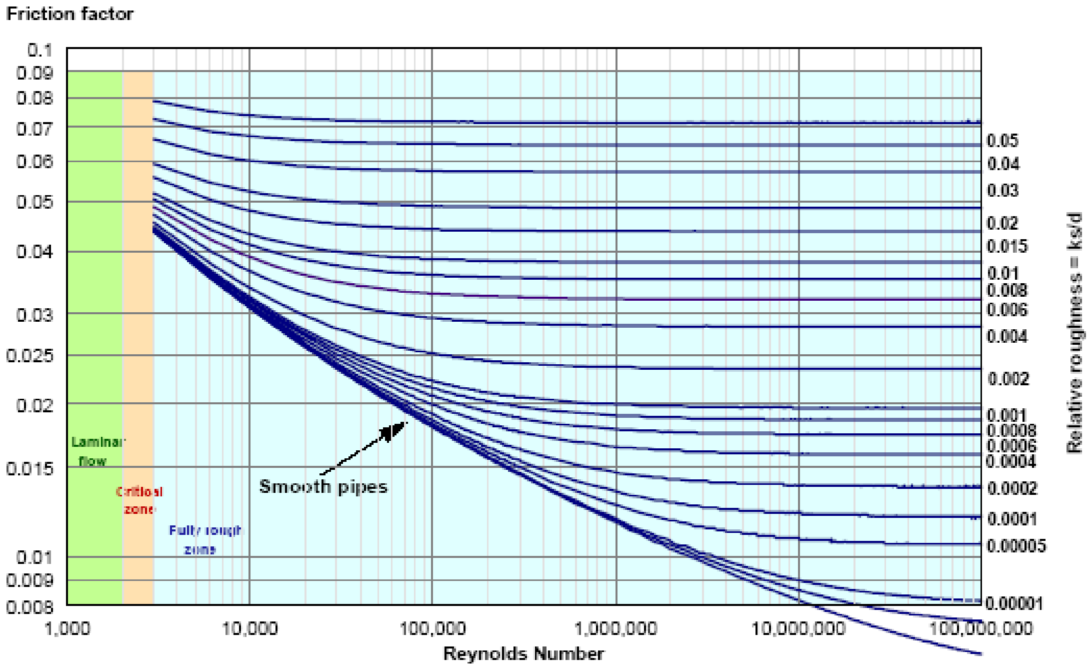



***FIGURE 6.31: Moody diagram for pipe friction (Shand, 1993)***

It is important to investigate the maximum friction losses that may occur during the lifetime of an outfall. These losses need to be determined for the maximum possible flows and for an expected pipe roughness after 20 to 30 years in operation.

Entrance, exit, bend and contraction losses ($h_e$) are defined as:

$h_{e,b,c} = K_{e,b,c}V^2/2g$

where
$K_{e,b,c}$ = Loss coefficient for entrance, exit, bend and contraction losses.

Shand (1993) summarised typical fitting loss coefficients as follows:

| PIPE DIMENSION | TYPE | $K_e$ | |
|---|---|---|---|
| *Pipe entrance* | *Projecting* | 0.80 | |
| | *Sharp cornered* | 0.50 | |
| | *Slightly rounded* | 0.25 | |
| | *Bellmouth* | 0.05 | |
| *Pipe exit* | *Projecting* | 1.0 | |
| | *Sharp cornered* | 1.0 | |
| | *Slightly rounded* | 0.5 | |
| | *Bellmouth* | 0.2 | |
| *Pipe bends* | $R_b/D$ | $K_b$ *for 45°* | $K_b$ *for 90°* |
| | 1 | 0.15 | 0.20 |
| | 2 | 0.09 | 0.13 |
| | 3 | 0.07 | 0.10 |
| | 4 | 0.06 | 0.08 |
| | 6 | 0.05 | 0.07 |
| | 8 | 0.05 | 0.07 |
| | *Where* $R_b$ *= bend radius* | | |
| *Contractions* | $D_1/D_2$ | $K_c$ | |
| | 1.1 | 0.05 | |
| | 1.2 | 0.11 | |
| | 1.4 | 0.20 | |
| | 1.6 | 0.26 | |
| | 1.8 | 0.34 | |
| | 2.0 | 0.38 | |
| | 2.5 | 0.42 | |
| | 3.0 | 0.44 | |
| | 4.0 | 0.47 | |
| | $D_1$*: Upstream diameter,* $D_2$*: downstream diameter* | | |

### 6.5.4 Diffuser design

For the optimisation of the diffuser, the following criteria must be met:

- Design flows must be discharged satisfactorily through the ports. A rule of thumb for the continuity of flow is that the total cross-sectional areas of the ports should not be less than 0.7 times the cross-sectional area of the main pipe at any point in the diffuser. A port diameter of less than 75 mm is not recommended because it will be more susceptible to blockage (from particulates in the wastewater as well as from outside).

- Maintain sufficient flow in each port to prevent the intrusion of seawater. This flow can be achieved by the gradual increasing of the sizes of the ports towards the end of the pipe. To prevent the intrusion of seawater, the port exit velocities must be such that the densimetric Froude Number for each port is greater than unity (i.e. 1).

  The densimetric Froude Number of the jet at exit is expressed as follows:

$$F_r = v_p/[g.d_p(\Delta\rho/\rho_s)]^{1/2}$$

Where

$v_p$ = port velocity (m/s)

$\Delta\rho$ = $\rho_s - \rho_e$

$\rho_s$ = seawater density (kg/m$^3$)

$\rho_e$ = density of wastewater (kg/m$^3$)

$d_p$ = port diameter (m)

- Ensure an even distribution of flows, through all the diffuser ports, because the flow is directly related to the achievable initial dilution, and the worst performing port (highest flow and lowest dilution) will be considered as representative of the performance of the diffuser. Even distribution can be achieved by the gradual increase of the port sizes.

- Maintain scouring flows within the diffuser section: this can be achieved by introducing tapers in the diffuser section together with increasing port sizes towards the seaward end of the diffuser.

- Optimum dilution will be obtained with diffusers discharging horizontally and with alternate ports directed in opposite directions.

- The distance between any two ports must be such that the plumes do not merge during the rise of the buoyant plumes. This can be achieved by ensuring that the distance between any two adjacent ports is greater than one third of the water depth.

***EXAMPLE…***

As a first assessment, outfall configuration can be obtained as follows:

For a population of 100 000 and an average flow of 250 ℓ/day per person, the total daily flow is:

$Q_d$ = 100 000 x 0.25 $m^3$/day = 25 Mℓ/day or 25 million ℓ/day

resulting in an average discharge rate of:

$Q_{ave}$ = 25 000/(3600 x 24) $m^3/s$ = 0.289 $m^3/s$

Referring to Chapter 2 of this Section, this discharge rate will result in a peak diurnal flow of approximately:

$Q_{peak}$ = 0.289 x 2 = 0.578 $m^3/s$

In order to maintain velocities (V) in the main pipe and to prevent deposition of solids in the main pipeline during average flow conditions, the diameter of the main pipe is:

$V > 0.7$ m/s as $V = Q/A$

$\therefore \quad A = Q/V$ or $\pi D^2/4 = Q/V$

$\therefore \quad D = [4QV/\pi]^{1/2}$

$\therefore \quad D \geq [(4 \times 0.289) / (0.7 \times \pi)]^{1/2} \geq 0.72$ m

where

$A$ = cross-sectional area of the main pipe = $\pi D^2/4$

Therefore, a main pipe diameter of 0,72 m will be required. For the estimation of the number (n) of ports required, apply the rule for continuity of flow:

$\Sigma$ (port cross-sectional areas) < 0.7 x cross-sectional area of the main pipe (A)

Using a port diameter ($d_p$) of 100 mm:

$n.[\pi d_p^2/4] < 0.7 \times \pi D^2/4$

$n \times d_p^2 < 0.7 \times D^2$

$n < 0.7 \times 0.72^2/0{,}1^2$

n (number of ports) < 36

Thus for a water depth of 20 m a main outfall pipe with an inside diameter of 0.72 m and a diffuser with 35 ports with a diameter of 0.1 m, spaced at 7 m intervals, will be a first estimate. The average discharge ($q_p$) per port (port flow rate) is:

$q_p$ = 289/35 = 8.25 ℓ/s and for a peak diurnal flow rate $q_p$ = 16.5 ℓ/s

The port exit velocity (vp) for average flows is:

$v_p = q_p/a_p = 0.00825/[\pi d_p^2/4] = 1.08$ m/s

Check if the Froude No > 1 for a seawater density ($\rho_s$) of 1026 kg/$m^3$ and an effluent density ($\rho_e$) of 1000 kg/$m^3$

$F_r = v_p/[g.d_p((\rho_s-\rho_e)/\rho_s)]^{1/2} = 1.08/[9.81 \text{x} 1((1026-1000)/1026)]^{1/2} = 6.85 > 1$

A detailed hydraulic analysis will have to be conducted to optimise the diffuser in order to maintain the main pipe velocity throughout the diffuser (introduce tapers) and the port diameters will have to be increased to ensure that the discharge is uniform along the diffuser.

### 6.5.5 Hydraulic analysis

The hydraulic analysis is based on the hydraulic energy balance for the complete system by comparing the specific energy between any two points in the system, taking into account all friction and fitting losses between two adjacent points. This balance ensures continuity of flow.

The flow (discharge) from a single port is given in Figure 6.32 (Rawn *et al*, 1960):

$q = C_D.a(2g.E)^{1/2}$ where $q = v_p.a$

thus

$v_p = C_D(2g.E)^{1/2}$

$E = V^2/2g + P/\rho_e - H.\ \rho_s/\rho_e$

Where

| | | |
|---|---|---|
| $q$ | = | port discharge ($m^3/s$) |
| $C_D$ | = | port discharge coefficient which is a function of the main pipe velocity and the entrance configuration (smooth, rounded edges, elbow-port, etc.). For an elbow port a value of 0.75 can be assumed for a first assessment. |
| $a$ | = | port cross-sectional area ($m^2$) |
| $E$ | = | Total energy head in the outfall line (m) |
| $v_p$ | = | port velocity (m/s) |
| $\rho_s$ | = | seawater density ($kg/m^3$) |
| $\rho_e$ | = | effluent density ($kg/m^3$) |
| $V$ | = | main pipe velocity (m/s) |
| $H$ | = | water depth (m) |
| $P$ | = | pressure in the pipe ($kg/m^2$) |

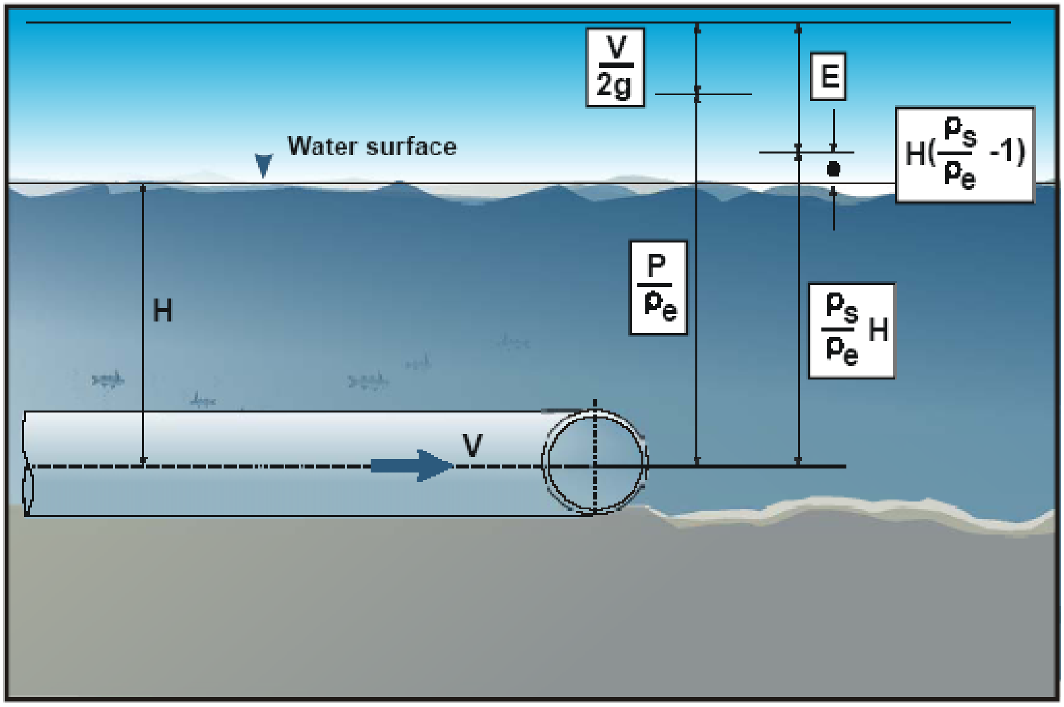



*FIGURE 6.32: Energy balance for flow from a single port*

As the flow from each port is a function of the total energy head (E), the energy increases from the offshore port inshore. This is a result of the friction loss between adjacent ports and the increase in head resulting from the increase in the slope of the seabed (Refer to the definition sketch for head losses in the main pipe in Figure 6.33 taken from Williams [1985]):

$$E = E_1 + \underset{j=1 \text{ to } n}{\Sigma[(f.s/2g.D)V_j^2]} + \underset{j=1 \text{ to } n}{[\beta(H_1 - H_j)]}$$

and for continuity of flow at each port

$$Q_j = Q_{j-1} + q_j$$

Where

| | | |
|---|---|---|
| $f$ | = | friction factor |
| $s$ | = | distance between ports (m) |
| $V_j$ | = | Velocity in the diffuser pipe between ports j and (j+1) (m/s) |
| $Q$ | = | Flow in the diffuser pipe ($m^3/s$) |
| $q$ | = | Port flow ($m^3/s$) |
| $\beta$ | = | $\rho_s/\rho_e - 1$ |

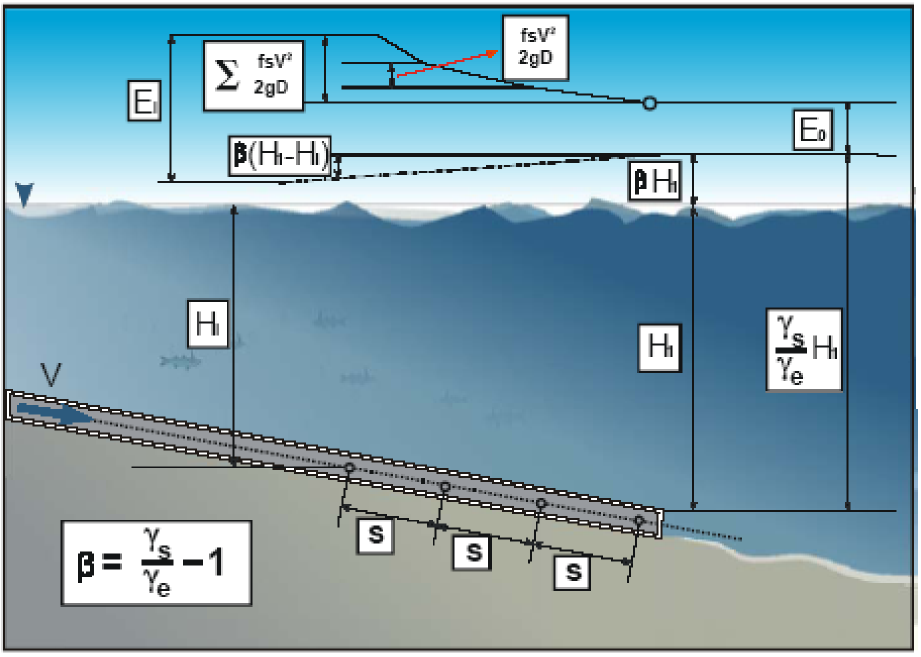



*FIGURE 6.33: Head losses in a multi-port diffuser (Williams, 1985)*

The equations presented above can be solved manually. However because hydraulic analysis is an iterative procedure, hydraulic engineers should use an existing computer programme, or write their own, to optimise a multi-port diffuser, because numerous runs may be required to comply with the requirements (ensure even port flow distribution along the diffuser, maintain main pipe velocities to prevent deposition, ensure that Froude numbers for all ports are greater than one, achieve the required initial dilutions). Typical outputs required to evaluate the functionality of a diffuser configuration are:

- Total headloss (friction and fitting losses in the main pipe as well as the diffuser)
- Minimum velocity in the main pipe at any point in the diffuser
- Minimum Froude Number.

A typical diffuser layout is shown in Figure 6.34 and the graphical outputs required to evaluate the hydraulic functionality of the outfall system in Figures 6.35a and 6.35b.

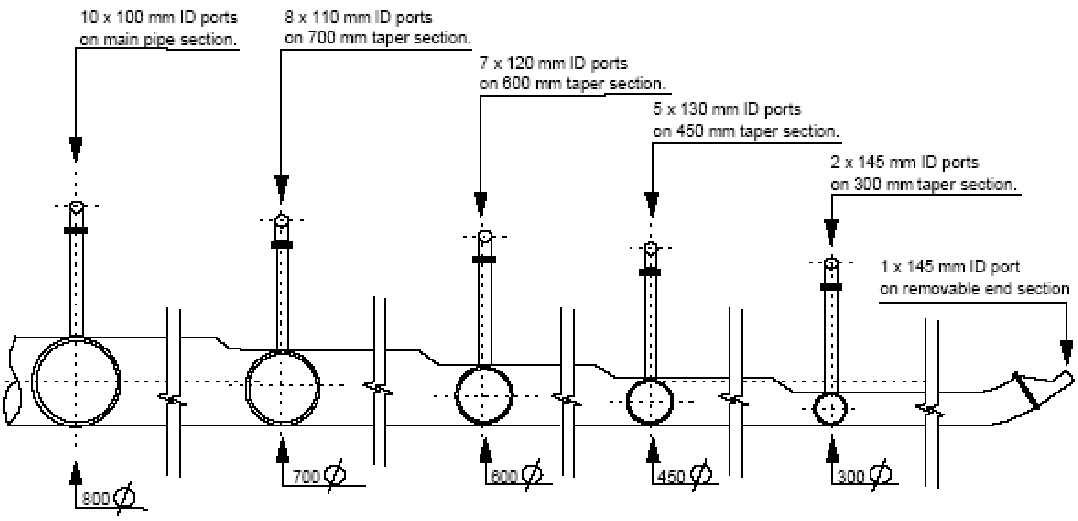



***FIGURE 6.34: Typical diffuser layout***

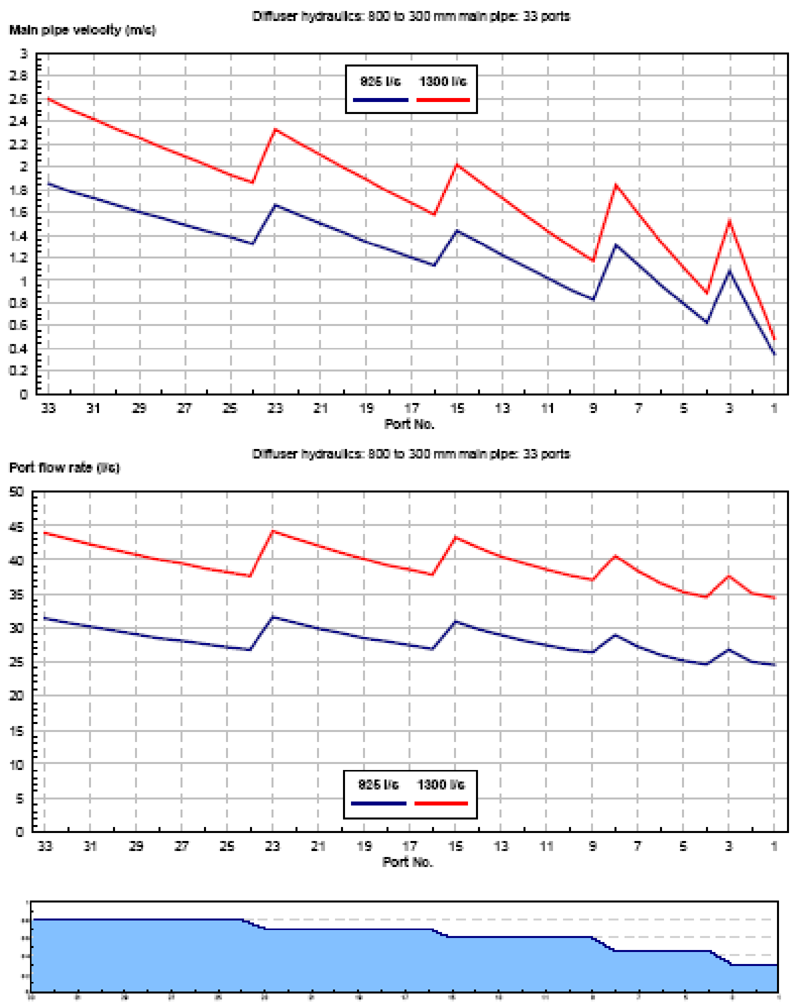



***FIGURE 6.35 a: Example of graphical output required to evaluate hydraulic functionality: Main pipe velocities in the diffuser section (top) and port flow rates (bottom)***

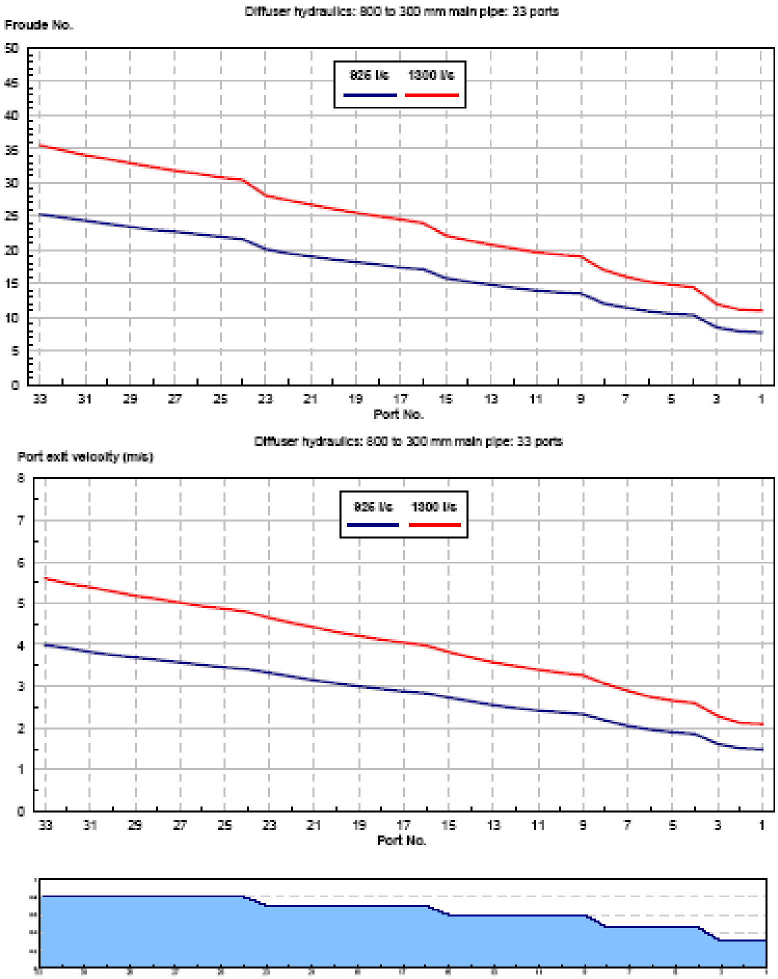



***FIGURE 6.35 b: Example of graphical output required to evaluate hydraulic functionality: Port Froude Numbers (top) and port exit velocities (bottom)***

### 6.5.6 Data requirements for pre-assessment and detailed investigation

The procedures described in Section 6.5.1 to 6.5.5 need to followed, in both the pre-assessment and detailed investigation stages, albeit to different levels of detail.

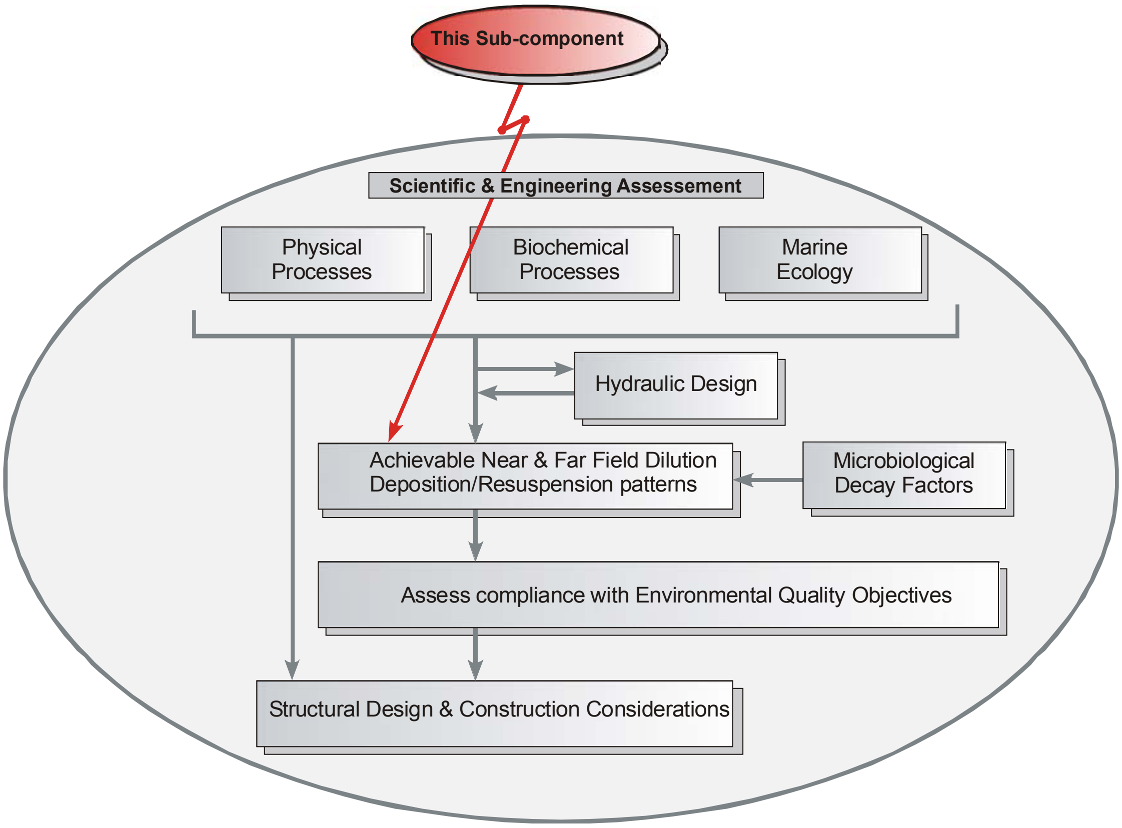
For the pre-assessment, the level of detail that would typically be required includes:

- A range of friction factors (e.g. average to maximum) for possible pipe material types
- Main friction components, e.g. taking into account only the friction in the main pipeline and the diffuser section (excluding detailed components such as bends, contractions and valves)
- Single (rather than detailed dynamic) hydraulic calculations to assess typical discharge patterns for the average to worst case wastewater flow scenarios
- Available geophysical data and information to estimate the pipeline profile and physical conditions at the discharge location.

For a detailed investigation, a much higher confidence is required and typical requirements include:

- Refinement of friction factors according to the manufacturer's specification for the selected pipe material, also taking into account detailed components such as bends, contractions and valves
- Detailed transient pressure calculations of the entire outfall system
- Detailed modelling of dynamic hydraulic processes, taking into account realistic wastewater discharge patterns
- Determining the actual pipeline profile and physical conditions at the discharge location, using detailed geophysical measurements.

## 6.6 ACHIEVABLE DILUTION




***PURPOSE:***

***The purpose of this section is to determine the achievable dilution of a wastewater plume on entering the receiving marine environment, through:***

- ***The initial dilution process ($S_i$), in which a buoyant plume rises from the diffuser or an open end pipeline to the surface of the sea***
- ***The secondary dilution ($S_e$) or subsequent dilution process (after dissipation of the energy during the initial dilution phase), in which the plume (waste field) is transported to distant locations by ocean currents.***

***The concentrations of non-conservative substances, such as microbial organisms, will be further reduced by 'decay', that is the die-off of the organisms. This reduction is discussed in the following section.***

### 6.6.1 Overview

When a buoyant wastewater plume is discharged into the sea, various physical, chemical and biological processes bring about the reduction in concentration of the constituents.

The physical dilution of a wastewater plume consists of two distinct processes:

- The initial dilution is the process ($S_i$) in which a buoyant plume rises from the diffuser or an open ended pipeline to the surface of the sea. The dilution is brought about by the entrainment of seawater during the rise of the plume. The influencing parameters are the buoyant and momentum flux of the jet, the ambient currents and the density structure of the receiving water column. Adapting the diffuser design for a certain ambient environment can optimise the dilution obtained from this process. The physical extent (i.e. height above the diffuser and distance from the diffuser) of the initial dilution process can be described as the **Initial Mixing Zone**.

  The entire concept of achievable initial dilution is based on the assumption that the receiving water is continuously moving and that 'clean' water is always available for entrainment and subsequent dilution of the wastewater plume. In estuaries and the surf zone this is not the case.

- The secondary dilution ($S_e$) or subsequent dilution (after dissipation of the energy during the initial dilution phase) occurs when the plume (waste field) is transported to distant locations by ocean currents. During the transport of the waste field, mixing occurs as a result of eddies that arise from various physical processes, also referred to as eddy diffusion. In contrast to the initial dilution process, secondary dilution cannot be influenced by the design of the outfall and is primarily dependent on the near-shore oceanographic conditions.

The concentrations of non-conservative substances, such as microbial organisms, will be further reduced by 'decay', that is, the die-off of the organisms. A predominant factor determining the rate of die-off is solar radiation. Other factors such as osmotic shock (caused by rapid salinity changes) and sedimentation can also contribute to the decay rate, although to a lesser degree.

Because of the differences in the physical processes operating in the offshore environment, the surf zone and in estuaries, different prediction techniques for secondary dilution and dispersion need to be applied in each of these environments.

The physical processes in the offshore environment are less complex than, for example those in the surf zone and estuaries. As a result, offshore processes can usually be described more accurately by applying standard numerical statistical analyses to data records representative of conditions in a particular study area. Because of the complex hydrodynamic processes in the surf zone, the prediction of the behaviour of an injected water source is less exact for the surf zone than for the offshore environment, and analytical prediction methods thus have to be based on model and prototype observations.

In the case of offshore marine outfall, the total achievable dilution of a conservative substance at a distant location will be the initial dilution ($S_i$) multiplied by the secondary dilution ($S_e$); for microbiological substances, the decay 'die-off' ($S_d$) of the organisms will further contribute to the total dilution. The process is illustrated in Figure 6.36.

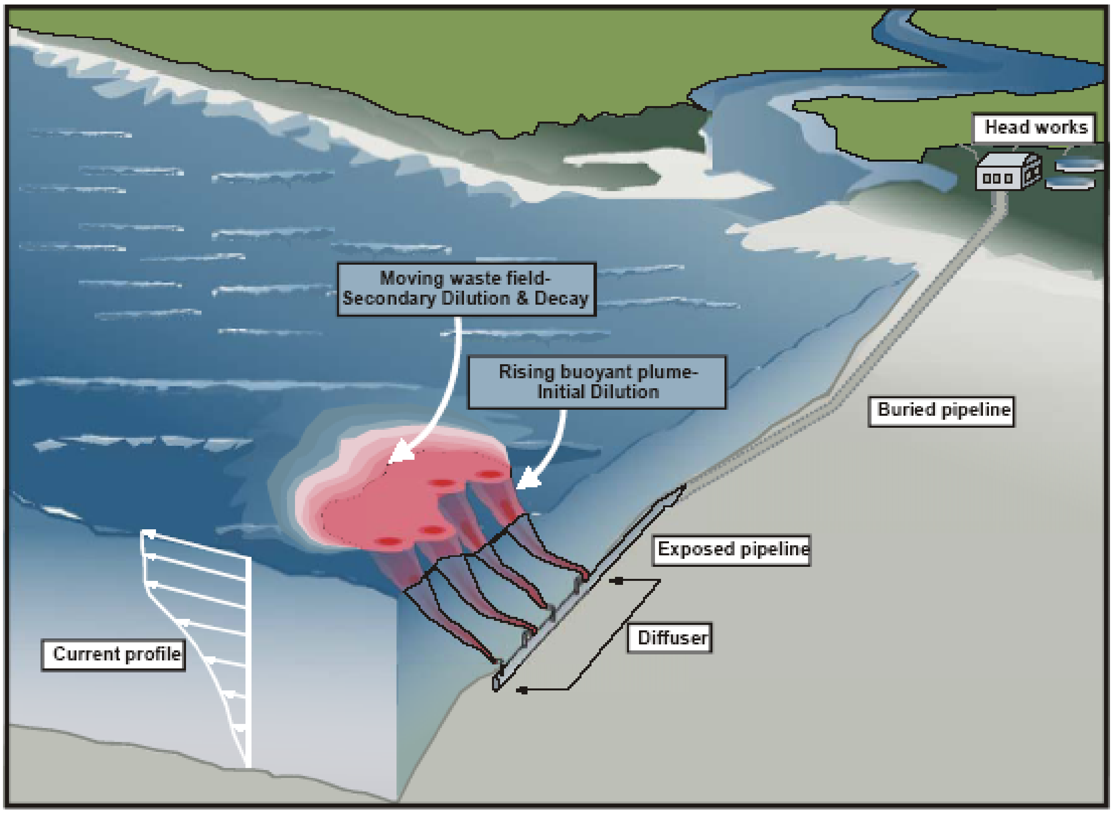



***FIGURE 6.36:*** ***Illustration of the different dilution components making up the total achievable dilution for an offshore marine outfall***

Normally the magnitude of the dilution for a deep-sea outfall in > 20 m water depth is:

- Initial dilution ($S_i$): > 200 times under calm conditions
  > 1000 under fairly moderate conditions (e.g. currents of greater than 0.2 m/s)

- Secondary dilution ($S_e$): 2 to 10 times after 1000 m

- Decay ($S_d$) will vary according to the current speed and distance (travelling time) and the value of the decay rate $T_{90}$ (see Section 6.4) - Example: $T_{90}$ value of 2 hours (daytime) and a current speed of 0.2 m/s will result in:

  Decay ($S_d$): 5 times after 1 km.

Thus for calm conditions, the total dilution can range from 400 to 2000 for conservative substances and from 2000 to 10000 for microbiological organisms. It is clear that the main contribution to the total dilution is the initial dilution process ($S_i$).

In considering discharges to the surf zone, the key factors in determining dilution are long-shore dispersion ('secondary dilution') and microbial decay (i.e. the processes resulting in 'initial dilution' are not relevant). Thus total dilutions are two to three orders of magnitude less than those that can be achieved with outfalls in the offshore environment.

The complex hydrodynamic processes affecting dilution in the surf zone are illustrated Figure 6.37.

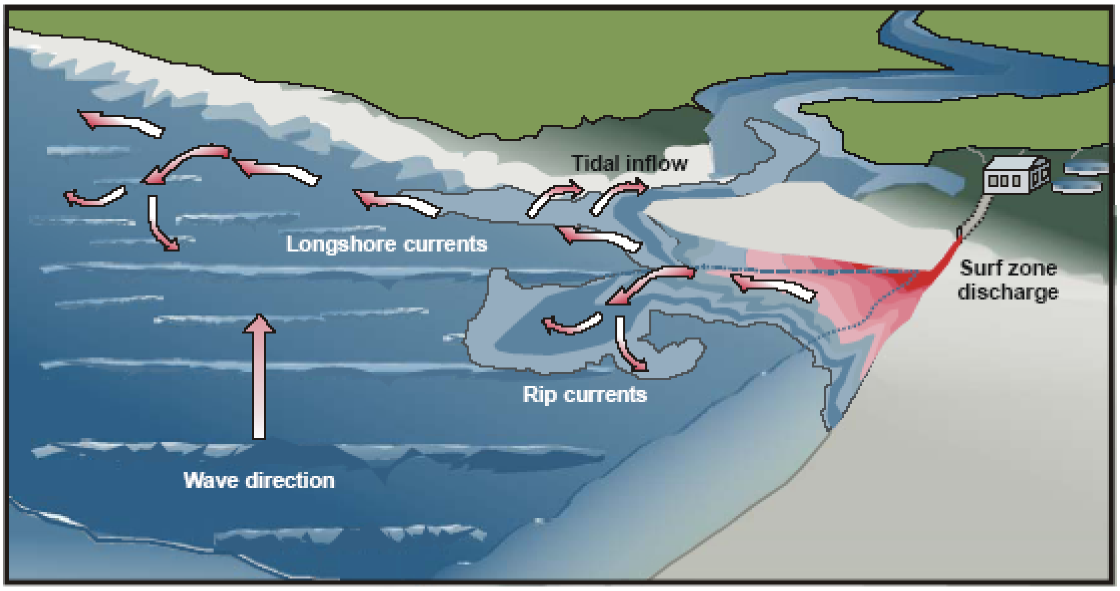



***FIGURE 6.37: Illustration of the total dilution process for a surf zone discharge***

Estuaries with permanently open mouths can be considered as semi-enclosed water bodies in which the exchange of water between the estuary and the sea is dependent on a source flow (river inflow), the diurnal and semi-diurnal tidal differences in water levels in the estuary and in the sea, and the 'size' (cross-sectional area) of the estuary mouth. The exchange of water results in dynamic conditions (currents) with a periodic velocity variation, changing direction approximately every 12 hours if the net source flow is less than the tidal flow. Theoretically, a volume of water, equal to the surface area of the estuary multiplied by the tidal height (the tidal 'prism'), will be exchanged between the sea and the estuary over a tidal cycle. Thus, if a wastewater plume is mixed uniformly with the water in an estuary, the dilution will be proportional to the tidal prism. However, there are many factors that inhibit the advection and dispersion of the wastewater stream within an estuary and prevent the wastewater from being uniformly mixed with the estuarine waters.

In the case of estuaries in which the mouth is closed, dilution (mixing) is limited to dispersion of the wastewater plume into adjacent waters in which concentrations are lower.

The different dilution processes are discussed in more detail in the following sections.

### 6.6.2 Initial dilution (only offshore)

Numerous field studies have provided a basis for the verification of theories that predict the hydraulic behaviour of an 'injected' water source into a dynamic water body. Numerical solutions were derived from analytical techniques and sophisticated models were developed for the accurate prediction of the behaviour of 'jets' (e.g. wastewater plume) released in 'deeper' waters.

The behaviour of a wastewater stream jetted into a water body with a density greater than that of the wastewater stream (generally referred to as a buoyant wastewater plume) depends on the dynamics and stratification of the receiving water body. Initial dilution is brought about by the entrainment of surrounding 'clean' seawater into the wastewater jet as it leaves the diffuser port and the further entrainment which occurs as the buoyant plume rises to the surface. Laboratory experiments and field measurements yielded a normal (bell-shaped) distribution of concentrations in a rising wastewater plume with the maximum concentrations at any point in the plume being 1.74 times the average concentration across the plume (Roberts, 1977).

The typical receiving water conditions and the subsequent behaviour of a wastewater plume are:

- *Stagnant unstratified water* (no currents and no stratification), in which the wastewater plume will rise vertically to the surface and laterally dispersed in a surface field.

- *Stagnant stratified water* (no current with a density gradient between the surface and the bottom), in which the rising of the wastewater plume can be inhibited and a submerged waste field can be formed below the surface (this is not considered to reflect a 'real' situation in the marine environment, as stagnant stratified conditions in an open marine environment rarely occur)

- *Moving, unstratified water* (currents and no stratification), in which the current component will lengthen the time and the path of the rising plume, thereby enabling it to entrain seawater and resulting in increased initial dilutions

- *Moving, stratified water* (currents and stratification), in which the current component will lengthen the time and the path of the rising plume, but the wastewater plume will become trapped below the surface because of the inhibitory effect which stratification has on the rising of the plume.

Examples of typical behaviour patterns are illustrated in Figure 6.38.

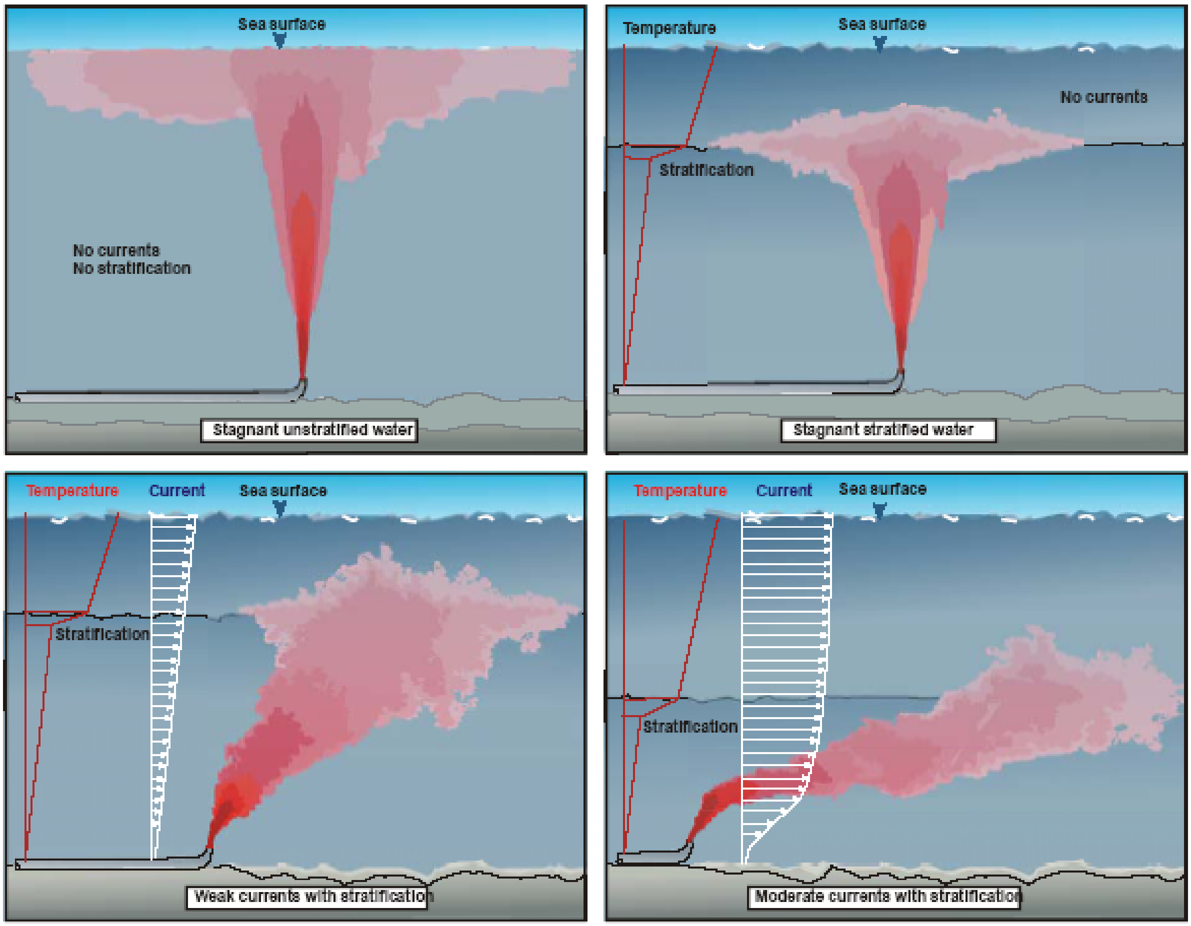



***FIGURE 6.38: Buoyant plume in (a) stagnant, unstratified water, (b) stagnant, stratified water, (c) moving (weak currents) stratified water and (d) in moving (moderate currents), stratified water***

An overview of all possible sea conditions at a certain area can be estimated or determined from available data and the worst-case initial dilution scenario (lowest dilutions) determined. However, the most exact and realistic method is to use a site-specific real-time current/stratification data record as input to a diffuser hydraulic program in order to obtain the extreme dilutions and plume rise heights together with a statistical output (occurrences/exceedances) of achievable initial dilutions.

The theories of dilutions put forward by Roberts (1977) are well-known and are widely applied in the theoretical prediction of dilutions. These theories form the basis of many analytical and numerical models. Roberts shows, by dimensionless analysis, that the minimum dilution, $S_{min}$, in stagnant uniform sea conditions, on the centre line of a buoyant plume of wastewater rising from a single round port in a diffuser on the seabed, depends on two dimensionless groupings:

$S_{min}/F_r = [y/d_p.F_r\ ,\ 1/F_r]$

Where

| | | |
|---|---|---|
| $y$ | = | height above the jet exit where $S_{min}$ is determined (m) |
| $d_p$ | = | port diameter (m) |
| $F_r$ | = | Froude number |
| | = | $v_p/[g.d_p(\Delta\rho/\rho_s)]^{1/2}$ |

and

| | | |
|---|---|---|
| $v_p$ | = | port velocity (m/s) |
| $\Delta\rho$ | = | $\rho_s - \rho_e$ |
| $\rho_s$ | = | seawater density (kg/m$^3$) |
| $\rho_e$ | = | effluent density (kg/m$^3$) |
| $d_p$ | = | port diameter (m) |

Roberts (1977) also summarised initial dilution prediction methods (Cederwall, 1967; Abraham *et al*., 1983; Fan & Brooks, 1969) and compared these with laboratory experiments (Hansen & Schroder, 1968; Cederwall, 1967; Liseth, 1970). Fan and Brooks (1969), for example developed theoretical procedures solving simultaneous differential expressions for conservation of continuity, momentum, density difference and concentration (these are solved by numerical integration). Taking all of the above into account, Roberts proposed the following equation, valid for all ratios of $y/d_p.F_r$ :

$S_{min} = 0.107.\ F_r\ [1.6 + 5(y/d_p.F_r) + (y/d_p.F_r)^2]^{5/6}$

and for $y/d_p.F_r > 20$ reduces to:

$S_{min} = 0.107.\ F_r\ (y/d_p.F_r)^{5/3}$

Average dilution ($S_{av} = 1.74S_{min}$)

***EXAMPLE…***

Dilution in an ambient current according to Roberts (1977):

The port flow ($q_p$) for peak diurnal flows: 16.5 ℓ/s
Water depth: 20 m
Seawater density: 1026 kg/m$^3$
Effluent density: 1000 kg/m$^3$
Port diameter: 0.1 m

$\Delta\rho = \rho_s - \rho_e = 1026 - 1000 = 26$ kg/m$^3$

Port exit velocity ($v_p$) = $q_p/a_p = 0.0165/[\pi d_p^2/4] = 0.0165/[\pi\ 0.1^2/4] = 2.1$ m/s

$$F_r = v_p/[g.d_p(\Delta\rho/\rho_s)]^{1/2}$$
$$= 2.1/[9.81x.1(26/1026)]^{1/2} = 13.32 < 20$$

For the dilution at the surface, 18,5 m (y) above the port (take the port exit at 1,5 m above the seabed):

$$S_{min} = 0.107 \text{ x } F_r\ [1.6 + 5(y/d_p.F_r) + (y/d_p.F_r)^2]^{5/6}$$
$$= 0.107 \text{ x}13.32\ [1.6 + 5(19.5/0.1 \text{ x } 13.32) + (19.5/0.1 \text{ x } 3.32)^2]^{5/6}$$
$$= 160$$

$$S_{av} = 278$$

For dilutions in an ambient current, Roberts (1977) used laboratory testing to provide empirically derived coefficients for relationships based mainly on dimensional analysis. The relationships relate to ‘slot plumes’ that are equivalent to a curtain of buoyant wastewater plume rising in a homogeneous sea. Such a curtain would be formed by closely spaced diffusers which would cause merging of plumes close to the seabed. He used a dimensionless Froude number $F_a$ to describe the relative strength of the ambient current:

$$F_a = u_a^3/b$$

where
$u_a$ = average ambient current (m/s)
$b$ = buoyancy flux per unit length of the diffuser
= $g\ (\Delta\rho/\rho_s).(Q/L)$
$Q$ = Total discharge (m$^3$/s)
$L$ = Total length of the diffuser (m)

Where $F_a > 0.1$ (for $F_a < 0.1$ the increased dilution is negligible), the following expression for initial dilution in a current perpendicular to a diffuser can be used:

$$S_{av} = 0.58(u_a.H)/(Q.L)$$

where
$H$ = water depth (m)

This is a conservative approach to obtain a rapid estimate of the achievable dilutions.

***EXAMPLE…***

Dilution in moving water according to Roberts (1977):

| | |
|---|---|
| The total peak diurnal discharge ($Q_{peak}$) | 0.578 $m^3/s$ |
| Water depth: | 20 m |
| Seawater density: | 1026 $kg/m^3$ |
| Effluent density: | 1000 $kg/m^3$ |

Length of the diffuser (L): 7 x 34 m = 238 m

$\Delta\rho = \rho_s - \rho_e$ = 1026 – 1000 = 26 $kg/m^3$
Buoyancy flux per unit length of the diffuser (B):
B = = g $(\Delta\rho/\rho_s).(Q/L)$ = 9.81(26/1026)(0.578 x 238) = 34.2

The current velocity that will have an effect on the achievable initial dilution when:

$F_a = u_a^3/b > 0.1$
$u_a > (0.1 \times 34.2)^{1/3}$ thus if $u_a > 1.5$ m/s

For normal inshore conditions along the South African coastline, the net current speed rarely exceeds 1.5 m/s. Thus the increase in dilution is negligible.

Wright's (1984) theory refers to individual rising plumes in an average ambient current and makes provision for a linear, stratified environment. The vertical density profile present in the ambient seawater causes the plume to entrain denser water close to the bottom so that the density of the diluted plume could equal that of the surrounding sea water at some intermediate height before reaching the sea surface. The height above the port at which the plume ceases to rise is:

$z_m = 2.3(g_1.q_p/u_a)^{1/3}.G^{-1/3}$

where

| | | |
|---|---|---|
| $q_p$ | = port discharge ($m^3/s$) | |
| G | $=g/\rho_{sb}.(\rho_{ss}-\rho_{sb})/H$ | (density gradient parameter) |
| $\rho_{sb}$ | = density of the seawater at the seabed ($kg/m^3$) | |
| $\rho_{ss}$ | = density of the seawater at the sea surface ($kg/m^3$) | |
| $g_1$ | $= g[(\rho_a-\rho_e)/\rho_a]$ | (relative density parameter) |
| $\rho_a$ | $= (\rho_{sb}+\rho_{ss})/2$ | (average density of the water column) |

After computing the rise height ($z_m$) the average dilution is determined as follows (Wright, 1984, Chu, 1979, Roberts, 1977):

$S_{av} = 0.71.u_a.z_m^2/q_p$

***EXAMPLE…***

Initial dilution for linear stratification and an ambient current:

The port flow ($q_p$) for peak diurnal flows: 16,5 ℓ/s
Water depth: 20 m
Effluent density: 1000 kg/m$^3$
Seawater density at the surface ($\rho_{ss}$) : 1025 kg/m$^3$
Seawater density at the bottom ($\rho_{sb}$) : 1026 kg/m$^3$

$G \quad =g/\rho_{sb}.(\rho_{ss}-\rho_{sb})/H = 9.81/1026\ (1/20) = 0.000478$
$\rho_a \quad = (\rho_{sb}+\rho_{ss})/2 \quad = (1026 + 1025)/2 = 1025{,}5$
$g_1 \quad = g[(\rho_a-\rho_e)/\rho_a] = 9.81[(1025.5 – 1000)/1025{,}5] = 0.2439$

For a current velocity ($u_a$) of 0,2 m/s, the rise height of the plume ($z_m$) is:

$$z_m = 2.3(g_1.q_p/u_a)^{1/3}.G^{-1/3}$$
$$= 2.3(0{,}2439 \times 0.0165/0.2)^{1/3} \times 0.000478^{-1/3}$$
$$= 8.0 \text{ m}$$

The average initial dilution at a height of 8.0 m is:

$$S_{av} = 0.71u_a.z_m^2/q_p = 0.71 \times 0.2 \times 8^2/0.0165$$
$$= 550$$

For a current velocity ($u_a$) of 0,1 m/s, the rise height of the plume ($z_m$) is:

$$z_m = 2.3(g_1.q_p/u_a)^{1/3}.G^{-1/3}$$
$$= 2.3(0.2439 \times 0.0165/0.1)^{1/3} \times 0.000478^{-1/3}$$
$$= 10.1 \text{ m}$$

The average initial dilution at a height of 10,1 m is:

$$S_{av} = 0.71u_a.z_m^2/q_p = 0.71 \times 0.1 \times (10.1)^2/0.0165$$
$$= 439$$

After an analysis of laboratory and field data, the WRc (1990) suggested an approach for horizontal round buoyant jets in stagnant water. Two regimes for round buoyant jets should be considered: (a) the buoyant-dominant condition (BDC), when the buoyancy flux of the rising plume is the controlling parameter for achievable dilutions, and (b) a current-dominant condition (CDC), in which the dilution is influenced less by buoyancy and more by ambient currents.

For discharge into a cross-flowing current a CDC exits when:

$$y < 5.B/u_a^3$$

where
$y$ = height above port exit (m)
$u_a$ = ambient current velocity (m/s)
$B$ = $g(\Delta\rho/\rho_s).q_p$ (buoyancy flux)

For a buoyant-dominant condition (BDC) the initial dilution is given as:

$$S = C_w(B^{1/3}.H^{5/3})/q_p$$

And for a current -dominant condition:

$$S = C_w(u_a.H^2)/q_p$$

$C_w$ was derived from actual measurements to provide statistical probabilities of exceedance for the minimum dilution:

| | ***Cw - VALUES*** | |
|---|---|---|
| | ***Buoyant-dominant condition (BDC)*** | ***Current-dominant condition (CDC)*** |
| *95 percentile* | *0.16* | *0.11* |
| *Median* | *0.27* | *0.27* |
| *Mean* | *0.34* | *0.32* |

***EXAMPLE…***

Initial dilution in an ambient current according to WRc (1991):

The port flow ($q_p$) for peak diurnal flows: 16,5 ℓ/s
Water depth: 20 m
Effluent density: 1000 kg/m$^3$
Seawater density: 1026 kg/m$^3$

The buoyancy flux (B) is:

B = g $(\Delta\rho/\rho_s).q_p$ = 9.81(26/1026) x 0.0165 = 0.0041018

For a current velocity ($u_a$) of 0,2 m/s:

$5.B/u_a^3$ = 2.564 which is < 20 m (water depth) thus a CDC exists and the mean initial dilution is:

$S_{mean} = C_w(u_a.H^2)/q_p$ = 0.32(0.2 x 202)/0.0165
= 1551

For a current velocity ($u_a$) of 0,1 m/s:

$5.B/u_a^3$ = 20.5 which is > 20 m (water depth) thus a BDC exists and the mean initial dilution is:

$S_{mean} = C_w(B^{1/3}.H^{5/3})/q_p = 0.34(0.0041018^{1/3} \times 20^{5/3})/0.0165$
= 486

The United States Environmental Protection Agency (US-EPA, 1985) developed methods for dilution estimates for various ambient and diffuser conditions and published standard computer programmes that are recommended for use in the evaluation of initial dilution in a standard way in the design stage. Original programmes published by Baumgartner *et al.* (1971) were updated in US-EPA (1985). One program, UOUTPLM, computes rise heights and initial dilutions for stagnant and moving water, non-uniform sea states. The computations are based on tracking a plume element as it gains mass due to ambient fluid entrainment. Horizontal momentum, energy, density, buoyancy and dilution are computed as the element rises through the water column. The computation ends when either the vertical velocity reaches zero or the water surface is reached. The input parameters include the density of the wastewater, port discharge angle, port discharge rate besides the surface, and seabed current velocities and water densities.

The UOUTPLM programme has been coupled to a multi-port diffuser hydraulic model to facilitate the optimisation of the diffuser with regard to the achievable dilutions, and applied to numerous outfall projects. The model output includes interactive visual trajectories of the plumes for all the ports of the diffuser and standard graphical outputs of the entire range of diffuser characteristics. A far field dilution prediction technique (based on the Brooks method) for conservative and non-conservative substances has also been linked to the model for an assessment of achievable dilutions for compliance with environmental quality objectives at distant locations (WAMTechnology 1996, WAMTechnology 1997, Van Ballegooyen et al., 2003, Van Ballegooyen & Botes 2003, GIBB Eastern Africa, 1997).

The procedure was further refined to provide dilutions and plume rise heights as input data to 3-dimensional and 2-dimensional far field numerical models for time series (current velocities, seawater densities and wastewater flows) in sections which correspond to the grid of the far field numerical models (CSIR 1996, CSIR 1998, HR Wallingford 1997).

### 6.6.3 Secondary dilution

#### *i. Offshore*

A standard analytical prediction method for secondary dilution for a current perpendicular to the diffuser is based on a method developed by Brooks (1960). This method defines the horizontal diffusion coefficient, a controlling parameter in the determination of secondary dilutions, as follows:

$K_o = \alpha L^n$

Where L refers to a length scale (waste field width – 10 to 10 000 m) and a value of 4/3 for n is recommended for offshore outfalls. In calmer waters the spreading of the waste field is curtailed as a result of limited eddies and n values < 1 will be more applicable. Where the lateral spreading of an effluent plume is restricted by the shoreline or estuary banks, the value for n =0.

A general $\alpha$-value (dissipation parameter) of 0.0005 $m^{3/2}/s$ is used. However, due to the diversity of the coastal processes along the 3 000 km South African coastline, the application of a 'general diffusion' coefficient has caused some concern over the last few years and, therefore, warranted investigation. Botes and Taljaard (1996) assessed all available data (data included the Lagrangian recording of currents, using pairs of surface and sub-surface drogues and tracer (i.e. Rhodamine B) measurements) measured at existing and proposed outfall sites in South Africa between 1982 and 1994, applying techniques relevant to each data type for determining the magnitude of eddy diffusivity at various locations along the South African coastline (i.e. diffusion coefficients). Botes and Taljaard (1996) found that the $\alpha$-value of 0.0005 $m^{3/2}/s$ represents the average for seven locations along the South African coastline (data from 1881 to 1994). However, the $\alpha$-value may vary from 0.0003 $m^{3/2}/s$ at the West Coast to $\pm$ 0.002 on the East Coast at Richards Bay.

Referring to Figure 6.39, it is assumed that the concentration across a wastewater plume at a distance x from the discharge location, resembles a Gaussian distribution in which the average concentration is:

$C_x = C_{max}/1.5$

The surface plume at the diffuser is taken as a steady line source of a width b and transported by a steady uniform current. The minimum secondary dilution can then be expressed as:

$S_{e(min)} = C_o/C_{max}$

For the 4/3 'law' Brooks (1960) expressed $C_{max}$ as:

$C_{max} = C_o.\ \mathrm{erf}[3/2/\{(1 + 2/3\beta.x/b)^3\text{-}1\}$

where

$\beta = 12K_o/u_a b$

$\beta = 12K_o t/xb$

$K_o$ = initial horizontal diffusion coefficient ($m^2s$)

b = initial plume width (m) if the current is perpendicular to the outfall

b = (b + 2H)Sin$\theta$

$\theta$ = angle between the wave and the diffuser

$u_a$ = current speed ($ms^{-1}$)

x = distance from the diffuser (m)

Determine a dimensionless distance (T)

$T = \beta.x/b$

$T = 12K_o x/u_a b^2$

Note: There are doubts about the accuracy when applying this theory to the direct onshore case ($\theta$ = 0: current parallel to the diffuser). Although such a condition is likely to be rare for a long marine outfall, it must be kept in mind that the method will tend to an under-prediction of dilutions.

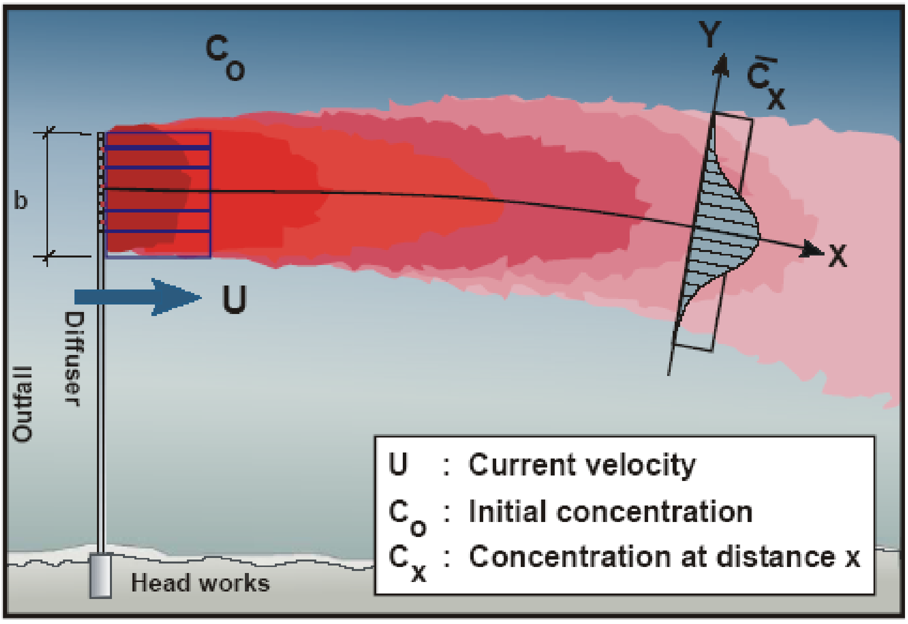



*FIGURE 6.39: An illustration of Brooks's surface plume model*

For a desktop assessment, the parameter β can be calculated and the secondary dilution obtained from the graph in Figure 6.40. The plume width at distance x can be determined from:

$$L_x = b[1 + 2/3(12K_o t/b^2)]^{3/2}$$
$$= b[\ 1 + 2T/3]^{3/2}$$

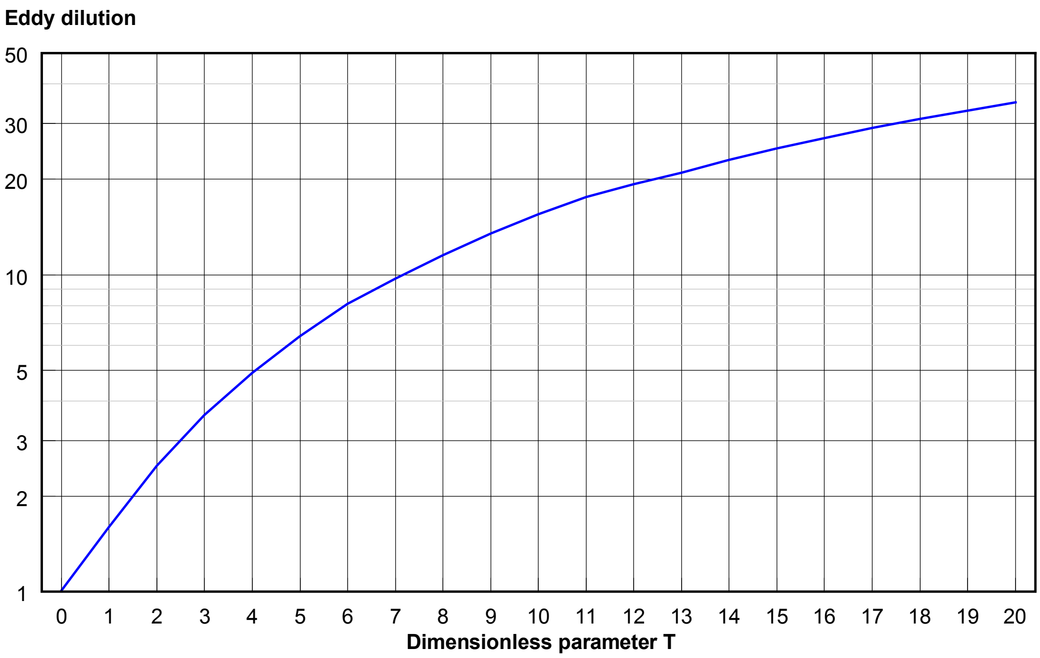



*FIGURE 6.40: Graph from which to calculate secondary dilution using the dimensional parameter* (T)

***EXAMPLE…***

| | |
|---|---|
| Diffuser length (b): | 238 m (no. of ports x port spacing) |
| $\alpha$-value: | 0,0005 $m^{3/2}/s$ |
| Current speed ($u_a$): | 0,2 $ms^{-1}$ |
| $K_o$ | $\alpha.b^{4/3}$ = 0.0005 x $238^{4/3}$ = 0.7374 |

The value of b, distance (x), time (t) and the secondary dilution from Figure 6.40 are shown below and illustrated graphically in Figure 6.41:

| Distance (x) (m) | T ($12K_ox/u_ab^2$ = 0.00078x ) | Time (h) (3600.x/$u_a$) | Se | Plume width = $b[1 + 2T/3]^{3/2}$ (m) |
|---|---|---|---|---|
| 500 | 0.39 | 0.69 | 1.1 | 256 |
| 1000 | 0.78 | 1.39 | 1.4 | 446 |
| 2000 | 1.56 | 2.77 | 2.3 | 693 |
| 3000 | 2.34 | 4.17 | 2.8 | 974 |
| 4000 | 3.12 | 5.56 | 3.6 | 1286 |
| 5000 | 3.91 | 6.94 | 4.7 | 1630 |

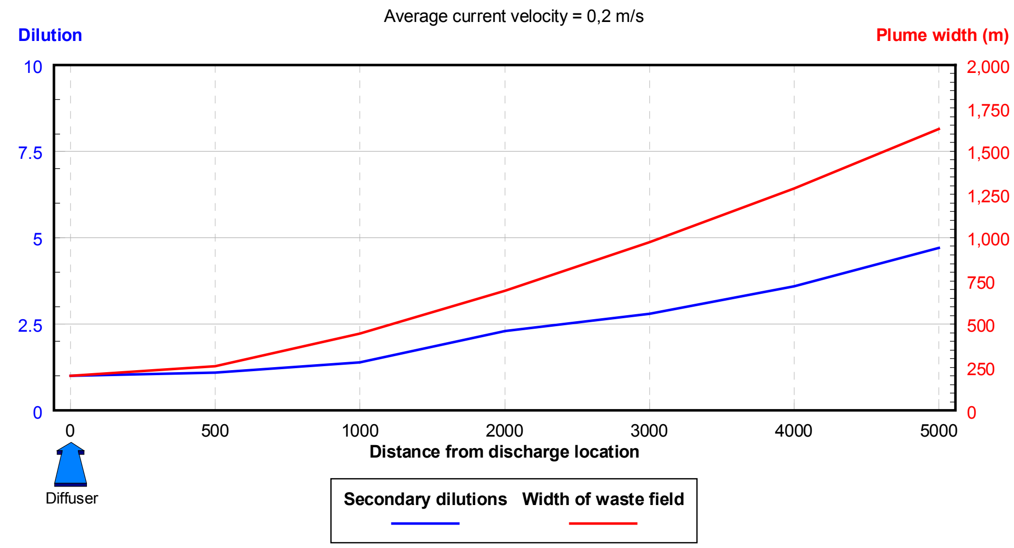



*FIGURE 6.41: Secondary dilutions and plume widths versus distance*

An illustration of a waste field along a straight coastline, using Brooks's surface plume model, is illustrated in Figure 6.42. In reality, the situation is far more complex because of the horizontal and vertical 'meandering' of nearshore currents. The more complex the physical configuration of the coastline, the more complex the circulation patterns in time and space. Therefore, the first principle when selecting an outfall site should be to search for the straightest stretch of coastline and an evenly sloping bottom topography. In Figure 6.43, the behaviour of a waste field along a rugged coastline is schematised. Currents behind promontories form complex eddies with direct onshore components and lower current velocities that will result in possible deposition of suspended solids.

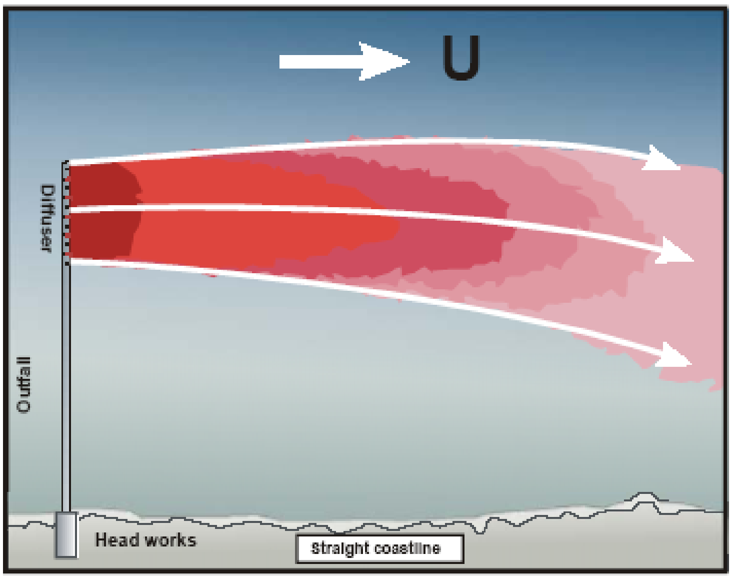



***FIGURE 6.42: Illustration of a waste field along a straight coastline using Brooks's surface plume model***

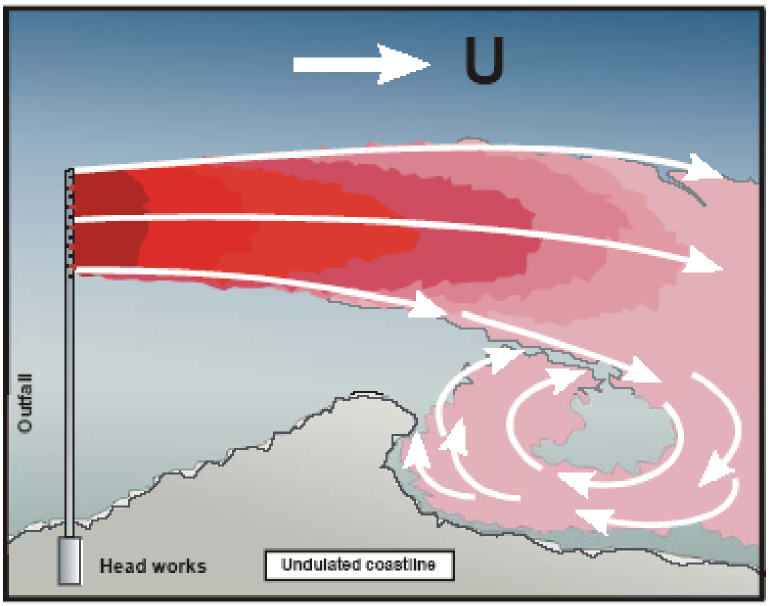



***FIGURE 6.43: Illustration of a waste field along a coastline with promontories***

Brooks's method is a highly parameterised method as evidenced by the wide range of dissipation parameters and values applicable to the South African coast. A more robust method for assessing secondary dilution is through the use of a 3-D numerical modelling, albeit a significantly greater effort and cost.

An illustration of the output of a 3-D far field numerical model on a waste field at a specific time, utilising the hydrodynamic data (illustrated in Figures 6.42 and 6.43), is in Figure 6.44.

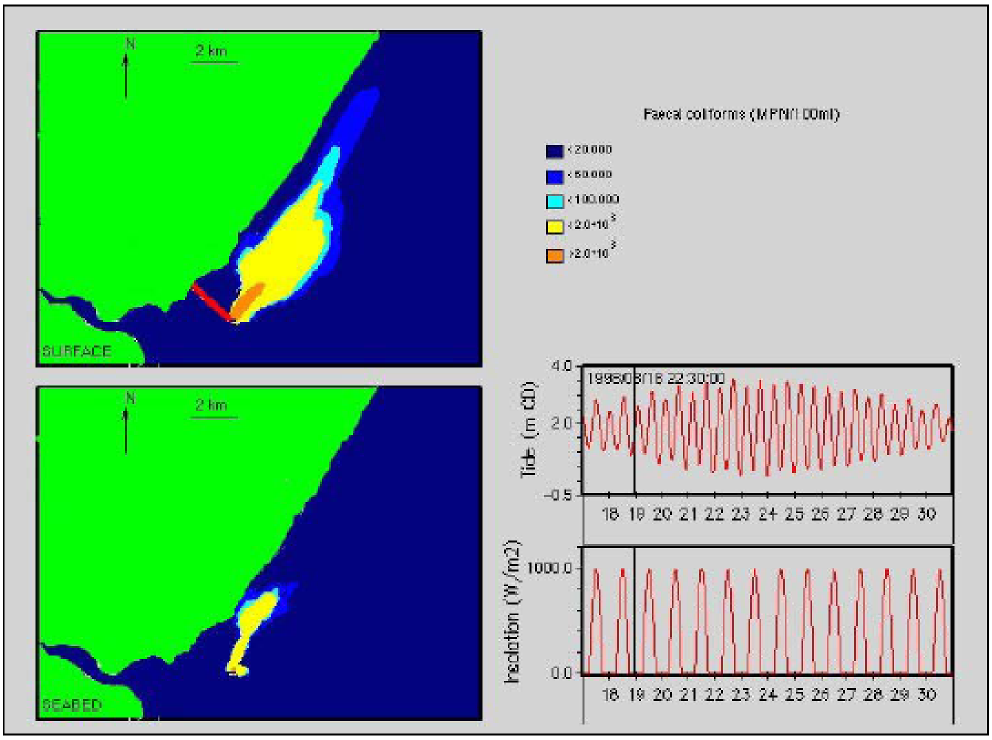



***FIGURE 6.44: 3-dimensional simulation of an outfall plume***

### *ii. Surf zone*

Currents in the surf zone (littoral zone) are wave-dominated, and initial mixing is rapid due to the vigorous processes of which the longshore and cross-shelf transport are the most dominant. Longshore transport is driven by the momentum flux of shoaling waves approaching the shoreline at an angle while cross-shelf transport is driven by the shoaling waves. Water is transported out of the surf zone by rip currents which results in the diffusion of surf zone water into the offshore waters. Some of the water expelled beyond the surf zone may be transported back into the surf zone with the next set of waves. It is important to note that onshore winds and an incoming tide will tend to keep water in the surf zone, where offshore winds and an outgoing tide will contribute to the transport of water away from the shoreline.

Normally stratification in the surf zone area will be insignificant because of vigorous processes and the subsequent high degree of mixing. Horizontal density differences may occur as a result of solar heating in sheltered shallow waters.

There are periods of meandering flow along a shore, and other times when the flow is more uniform. Some periods exhibit clear indications of rip currents, others not. It is difficult to predict the fate of a waste field in this non-linear surf zone regime and it is not easy to provide quantitative answers on the degree of mixing or transport that can be expected, even for a specific day. During field exercises in False Bay, the behaviour of a waste field changed drastically within hours (meandering of the flow, change in direction along the shore, change of rip currents, etc.) without observable changes in the weather or sea state.

The major mechanisms that contribute to the dispersion of surf zone discharges include:

- Breaking waves, which cause 'rapid' mixing normal to the shoreline within the breaker zone.
- Rip currents, which result in the longshore advection of the waste field.

Inman *et al.* (1971) describe the mixing process in the surf zone as the transport of water between 'nearshore circulation' cells, formed between adjacent rip currents, as well as the exchange of water from the surf zone to the offshore region.

According to Inman *et al*. (1971), a wastewater stream that is introduced into surf zone will be diffused by the turbulence in the breaking waves until it has a uniform distribution over the width of the breaker zone. Further dilution will then be brought about by the longshore advection related to the longshore current and the strength and spacing of the rip currents. Inman *et al*. (1971) expressed the change in concentration in terms of the concentration and a vector distance as:

$$\partial C/\partial t = \partial/\partial r\ (\upsilon\ .\ \partial C/\partial r) - \partial/\partial r(C.u) + R_s$$

where
$\partial C/\partial t$ = change in concentration in time
$\upsilon$ = kinematic coefficient of diffusion
$u$ = velocity
$R_s$ = change in concentration in time due to a source

***Initial mixing.*** The concentration ($C_o$) of a substance after initial mixing of the concentration ($C_e$ ) in the wastewater is:

$$C_o = C_e/v_x$$

where $v_x$ is the cross-shore mixing coefficient. Prototype tests yielded the following semi-empirical relation (Inman, 1971):

$$v_x = H_{brms}.X_b/T$$

where
$H_{brms}$ = rms breaker height (m)
$X_b$ = surf zone width (m)
$T$ = incident wave period (s)

The following equation was derived after further prototype dilution experiments:

$$v_x = 5.22.\ c_2.H_b.X_b/T$$

where $c_2$ is a dimensionless friction coefficient and determined as 0,1 for steeper beach slopes (1 in 10) and 0.2 for less steep slopes.

Referring to CSIR (1995), surf zone widths determined from wave data for 1990/1991 for the northern beaches at Richards Bay (South Africa) ranged between 75 m and 325 m, with an average of approximately 120 m. CSIR (1995) provided a mean breaker height of 1.55 m and mean peak wave period of 11.1 s for the area. These average values yield a cross-shore mixing coefficient of approximately 13. Measurements at two surf zone discharges (for flow rates varying from less than 0,2 $m^3$/s to 3 $m^3$/s) in False Bay indicated that dilutions rarely exceeded 10 within 100 m from the discharge location.

***Longshore dispersion***. After initial dispersion, the plume is transported by the longshore currents and separated by the rip currents where part of it is transported to the offshore region by the rip current and part of it leaks to the next circulation cell (Figures 6.45). Inman *et al.* (1971) describe the effect of the rip current as:

$R_R = Q_m/Q_{m-1}$

where
$Q_m$ = Total longshore flow in cell m
$Q_{m-1}$ = Total longshore flow in cell (m – 1)

Thus the rip current flow is:

$Q_R = Q_m - Q_{m-1}$

From field data it was found that $R_R$ is in the range 0 to 0.5, where $R_R$ approaches 0.5 when the longshore current velocity > 0,4 m/s.

The concentration in the m-th cell is:

$C_m = C_{m-1}.(R_R)$
$C_m = C_0.(R_R)^n$

An approximation in terms of the distance from the discharge location and the length of the circulation cells yielded:

$C_m = C_0.(R_R)^{y/Y}$

where
y = distance from the discharge location (m)
Y = length of the circulation cells (m)

The length of circulation cells (Y) related to the surf-zone width was extensively investigated theoretically and by assessing prototype data. Most investigations yielded a factor between 2 and 5. Analysis of aerial photos at Richards Bay over a period of 11 years yielded an average of 3.5 (range 2.3 to 5.3).

Longshore current velocities at Richards Bay (CSIR, 1995) were determined theoretically and were correlated with field measurements, using Rhodamine B. These yielded a maximum of 0.8 m/s with peak occurrences at about 0.3 to 0.4 m/s.

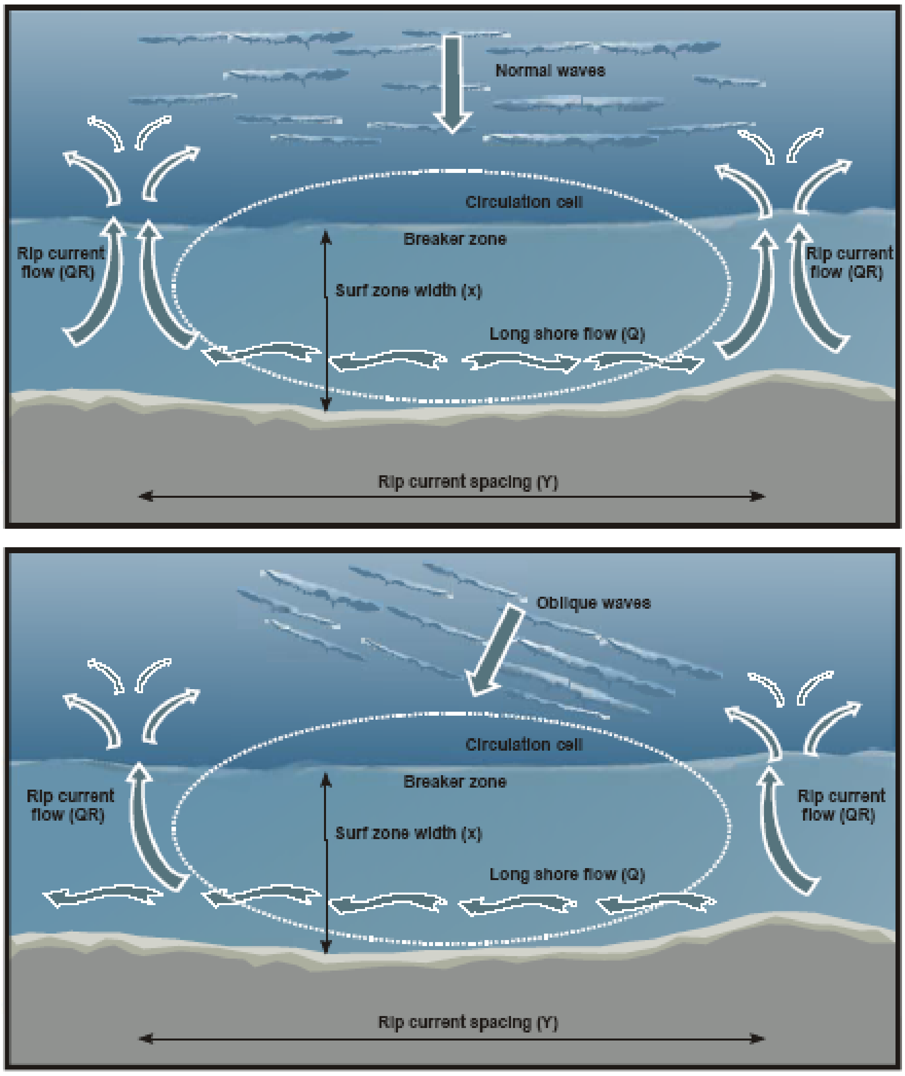



***FIGURE 6.45: Illustration of the longshore transport and dispersion processes in the surf zone under normal and oblique wave direction***

In order to obtain an idea of the magnitude of the dilutions along the beach according to the above theory, conditions along the northern beach at Richards Bay (South Africa) is used as an example (CSIR, 1995). Based 1990/1991 wave data collected in 1990/1991, surf zone widths along the beach ranged between 75 m and 325 m with an average of approximately 120 m, resulting in a predicted average length of the circulation cell of 420 m. Using a $R_R$ value of 0.5 (longshore current velocity equalled 0.4 m/s) the achievable dilutions versus distance were estimated as:

| *DISTANCE (m)* | *DILUTION* |
|---|---|
| *500* | *1* |
| *1000* | *5* |
| *1500* | *11* |

This process is further illustrated by an example of the dye dispersion patterns observed in the surf zone in False Bay (Cape Town) during a continuous release (CSIR, 1991a) (Figure 6.46).

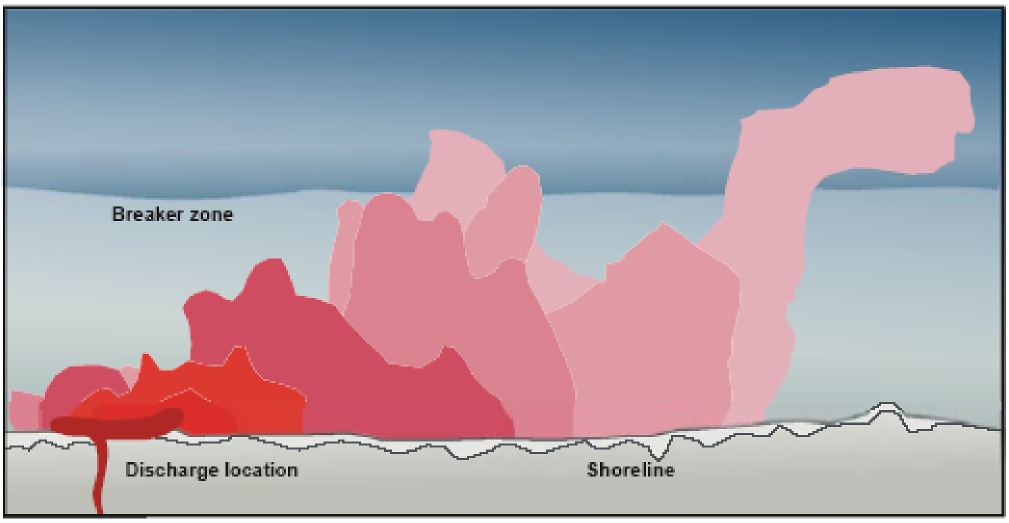



***FIGURE 6.46: Example of a dye dispersion test during a continuous release to the surf zone in False Bay (Cape Town)***

Where appropriate data are available (e.g. high resolution wave time series data), a combination of wave and hydrodynamic modelling of the nearshore can be used to predict dilutions in the surf zone in greater detail.

### *iii. Estuaries*

An open estuary can be considered as a water body with a 'pumping mechanism' due to the gravitational in- and outflow of the tide, which in theory can result in the flushing of the estuarine waters. A simplistic approach is to assume that wastewater is mixed uniformly with the estuarine waters, transported to sea during the outgoing tide and that uncontaminated water will flow to the estuary during the incoming tide. Theoretically a volume of water, equal to the area of the estuary times the tide height (the tidal 'prism'), will be exchanged between the sea and the estuary over a tidal cycle. Thus, if wastewater is mixed uniformly with the water of an estuary, there will be a 'dilution' proportional to the 'tidal prism'.

***EXAMPLE…***

The tidal prism ($V_t$ ) for an estuary 5 km long and on average 100 m wide during a tidal variation of 1 m is:

$V_t$ = 5000 x 100 x 1 = 500 000 m$^3$

For a settlement with a population of 50 000, there will be a diurnal (24 hr) effluent volume of:

$V_e$ = 50 000 x 0,25 = 12 500 m$^3$ (250 ℓ/day per capita)

The ratio $V_t/V_e$ = 40, which is theoretically the best overall 'dilution' that can be achieved. During neap tides, with smaller tidal variations, this dilution will be further reduced.

For a point source (along the bank of an estuary), numerous factors contribute to the inhibition of the spreading and mixing of the wastewater plume. The process of mixing relates to the transport of a plume with the tidal current (advection) and the mixing with the adjacent estuary water with concentrations lower than the concentrations in the plume (dispersion). In shallow water areas where the bed resistance is high, flow velocities will be reduced and the wastewater can at times be trapped, in which the strong velocity shear between fast-moving channel flows and the slow moving waters in shallow areas, in which the bed resistance is high, will enhance the mixing process.

For a surface discharge, mixing will also occur vertically, because of the velocity shear between the different vertical layers. An illustration of the complex mixing processes associated with a fresh wastewater discharge into an estuary is illustrated in Figure 6.47.

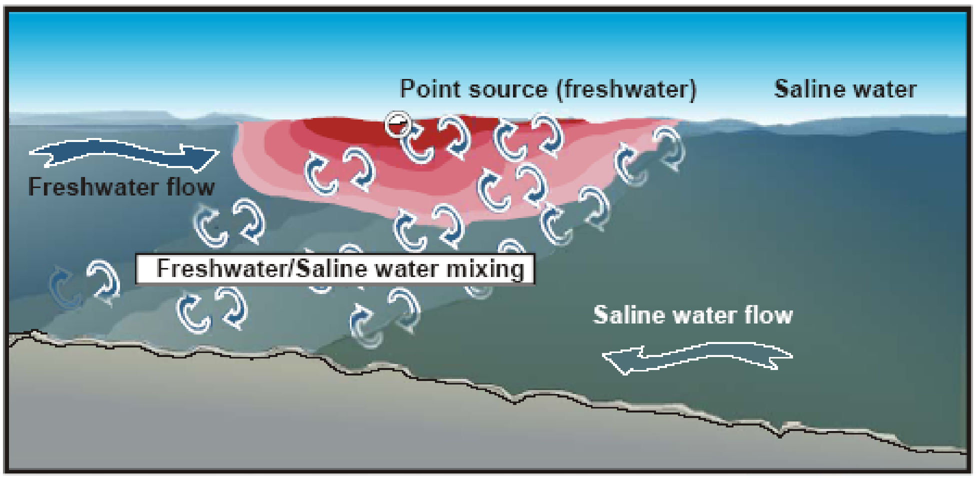



***FIGURE 6.47: Illustration of the mixing of a wastewater plume in a partially stratified estuary***

Because a discharge point is normally at the high-water mark at the estuary bank, the mixing will be site dependant, depending on the hydrodynamics at the discharge point. Tidal currents (ebb and flow) will provide some longitudinal advection dispersion. The estuary banks and cross-stream flow profile will limit lateral dispersion. The current velocity will change from a maximum to zero in six hours (as will the advection and dispersion). When changing direction, the past 6-hour plume will move with the new wastewater in the opposite direction – thus the 'mixing' will be less effective, because of mixing with the contaminated 'old' plume. Theoretically, the result will be that after a period, if the discharge is continuous, a wastewater plume will exist on both sides of the discharge location, with the maximum concentration of constituents potentially being the same as that in the wastewater. The fresh wastewater inflow will also result in reduced salinities, which in turn will increase the $T_{90}$ value for microbiological organisms, thus reducing the die-off rate and increasing the possibility of contamination with regard to microbiological organisms.

Without elaborate field measurements, the ultimate behaviour of a wastewater plume in a stratified estuary can only be predicted by a verified 3-D numerical model.

In the case of a closed estuary, dilution (mixing) is limited to dispersion of the wastewater plume into adjacent waters, in which concentrations are lower.

***EXAMPLE…***

For an estuary 5 km long and on average 100 m wide, the surface area is:

$A = 5000 \times 100 \times 1 = 500\,000\ m^3$

A settlement with a population of 50 000, there will be a diurnal (24 hours) effluent volume of:

$V_e = 50000 \times 0.25 = 12\,500\ m^3$ (250 ℓ/day per capita)

The increase in level ($\Delta h$) over 24 hours is:

$\Delta h = V_e/A = 12\,500/500\,000 = 0.025\ m$

However, the mixing processes in an estuary are complex and change continuously according to the river inflow, tidal flow and the associated density structure. Figures 6.48 and 6.49 illustrate some of the complexities in the behaviour of a wastewater plume in an estuary, using actual simulations in the Swartkops River Estuary as an example. This example does not take vertical mixing processes, which is even more complex, into consideration (Figure 6.47). Figure 6.48 illustrates the difference in average flow velocities at three locations in the estuary (4 km, 10 km and 15 km from the mouth) under zero river inflow. Figure 6.49 illustrates the difference in the extent to which a wastewater plume is transported over a 12-hour tidal cycle, after wastewater has been released at the three locations. These differences clearly indicate the importance of considering transport and dispersion characteristics in the selection of an outfall location, where these are considered in estuaries.

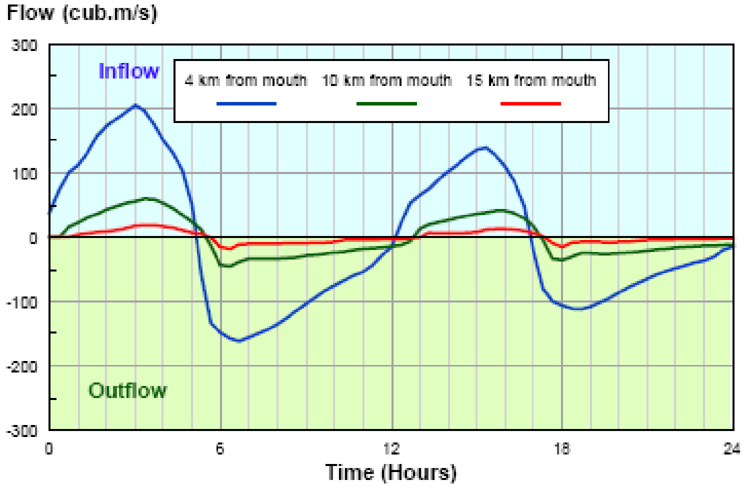



***FIGURE 6.48: Difference in average flows at three locations in the Swartkops River Estuary (river inflow is zero)***

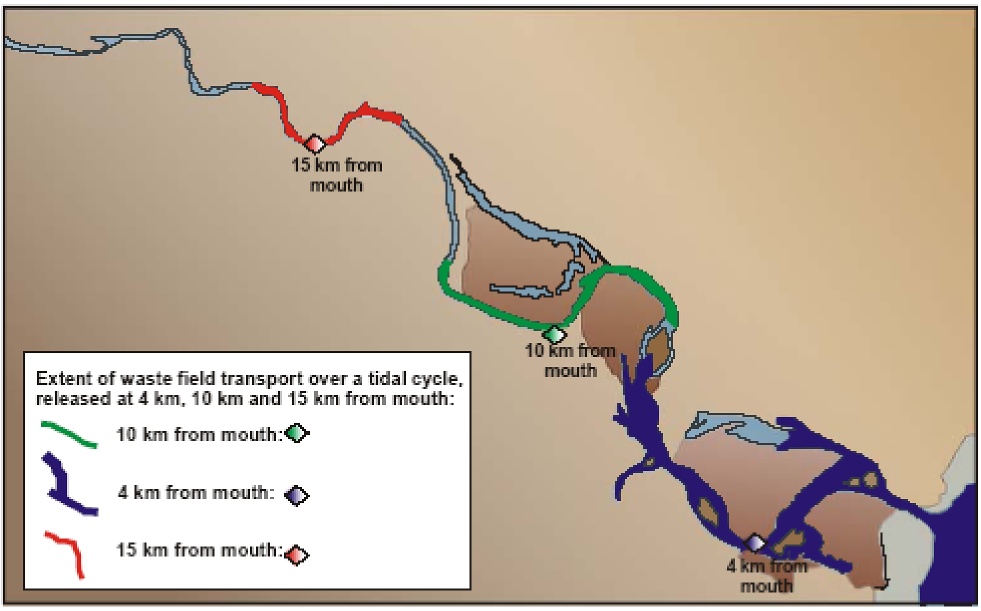



***FIGURE 6.49: A simulation showing the difference in spatial behaviour of a wastewater plume released at different locations simulated in the Swartkops River Estuary***

Apart from the dispersion and transport of a wastewater stream, discharges in estuaries can affect the physical dynamic processes of such systems, as discussed below:

- *Changes to mouth conditions*. Discharges can result in a significant increase in base flow and, as a result, can alter the pattern of mouth closure. For example, in small, temporarily open/closed estuaries, unstable conditions, resulting from almost complete draining of these systems during the open phase, prevent optimal biological production and, consequently a period of mouth closure is required for such systems to reach their optimal biological production.

***EXAMPLE…***

The Eerste River Estuary (in False Bay, South Africa) is a small estuary of approximately 10.2 ha draining a catchment of approximately 660 $km^2$ (CSIR, 2001b). Historically this was a temporally open/closed estuary, i.e. closing during summer and breaching again after the first winter rains. Now, as a result of numerous WWTW discharges, both directly to the estuary and in the catchment, elevated base flows have modified the system to a permanently open estuary.

Preliminary estimation of the correlation between river inflows and different states for the estuary is provided below:

| ***PHASE*** | ***ESTIMATED RIVER FLOW*** |
|---|---|
| *Mouth closed, but overwash of the berm by seawater occurs occasionally* | *May occur at flows less than 0.1 $m^3/s$* |
| *Mouth semi-open, where the mouth is open, but normally only outflow occurs. Some seawater intrusion takes place at spring high tides* | *May occur at flows between 0.1 $m^3/s$ and 3 $m^3/s$* |
| *Mouth open with seawater intrusion (i.e. normal estuarine function)* | *This state may occur only for brief periods after a freshette* |
| *Mouth wide open with the estuary being completely fresh* | *May occur at flows greater than 3 $m^3/s$* |

- *Changes in salinity distributions*. Base flows into the estuaries of South Africa are often relatively low, between 0 and 0.2 $m^3/s$. This results in specific distributions of salinities and temperatures, which are important for the ecological processes. Even a small disposal of waste water of 0.01 to 0.2 $m^3/s$ can have a significant effect on these distributions and in turn also on the ecological processes in those estuaries.

***EXAMPLE…***

Increase in the base flow to the Eerste River, as a result of wastewater discharges in the catchment, has resulted in a strong reduction in the intrusion of saline seawater into the system, which now occurs only for short periods. This has markedly modified the ecology of the estuary (CSIR, 2001b).

### 6.6.4 Microbiological decay

Universally, microbiological organisms, such as *E.coli* and *Enterococci*, are used as indicators of the likely presence of human pathogens and viruses of concern. Environmental quality objectives for beneficial uses where human health is an issue, such as recreation and mariculture, therefore, are typically set in terms of 'allowable counts' of these organisms.

Solar radiation is the most important factor for decay of these organisms in saline waters. The decay process is commonly described by the first order decay equation:

$C = C_o e^{-kt}$

where:

| | |
|---|---|
| $C$ | = Concentration at a distant location |
| $C_o$ | = concentration after initial dilution |
| $t$ | = travel time (h) |
| $k$ | = $2.303/T_{90}$ |

Where $T_{90}$ is the time it takes for 90% of the organisms to die

The dilution due to decay ($S_d$) can then be expressed as:

$S_d = C_o/C$
$\quad = e^{kt}$

$T_{90}$ values are different for daytime and night time and also for summer and winter conditions.

The influence of current velocities on the dilutions due to decay at distant locations from the discharge, is illustrated in Figures 6.50 and 6.51.

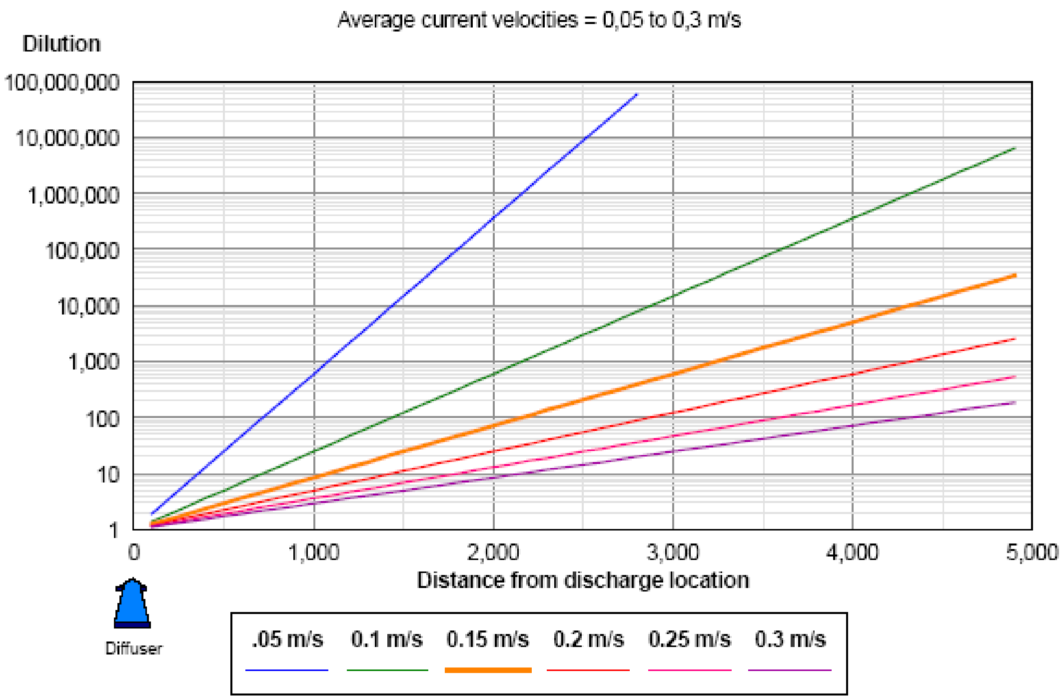



***FIGURE 6.50: Dilutions due to decay versus distance under stronger current conditions. $T_{90}$ = 2 hrs (daytime).***

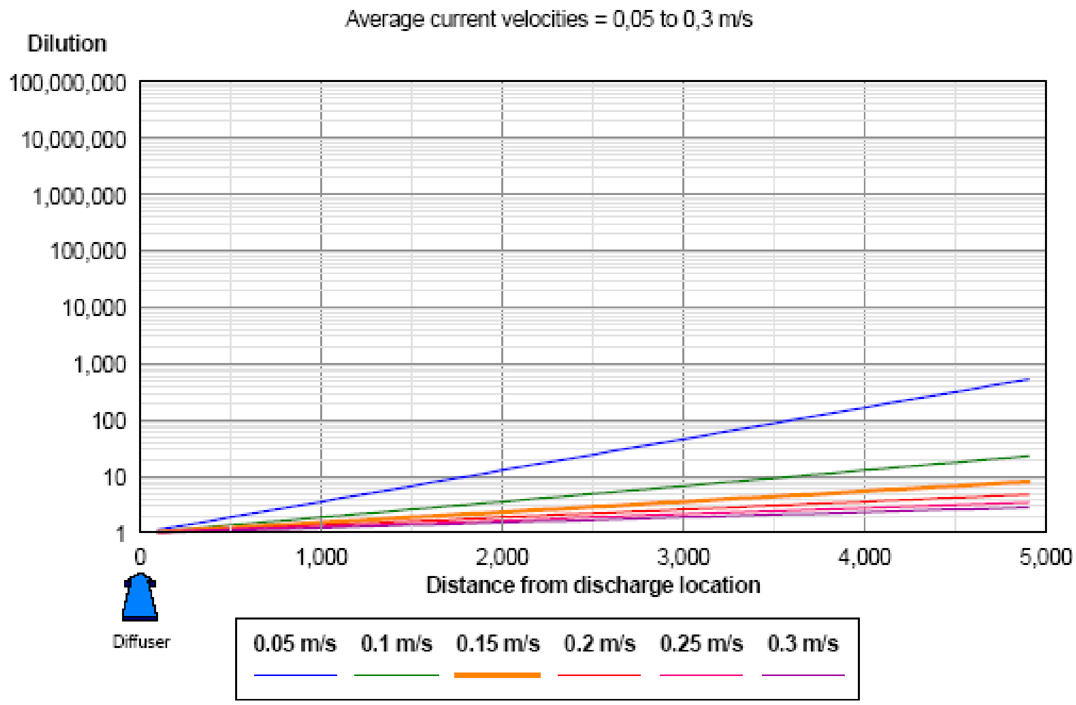



***FIGURE 6.51 Dilutions due to decay versus distance under weaker current conditions. $T_{90}$ = 2 hrs (daytime)***

***EXAMPLE…***

$T_{90}$ (daytime) 2 hrs
$T_{90}$ (night-time) 10 hrs
Current velocity ($u_a$) 0.2 m/s
Substitute t in $e^{kt}$ with $x/u_a$ as $u_a = x/t$ and k with $2.303/T_{90}$

$$S_d = e^{2.303.x/T_{90}.u_a}$$

The dilutions due to decay at distances from the outfall are:

| *DISTANCE (m)* | *$S_d$ ($e^{2303.x/T_{90}.u_a}$)* | |
|---|---|---|
| | *Daytime* | *Night-time* |
| *500* | *2.22* | *1.17* |
| *1000* | *4.95* | *1.38* |
| *2000* | *24.5* | *1.90* |
| *3000* | *121* | *2.61* |
| *4000* | *600* | *3.60* |
| *5000* | *2970* | *4.95* |

NOTE: $T_{90}$ values can vary widely, depending on the oceanographic and atmospheric conditions at a study area. The values used in the above example are considered typical values, based on measurements taken at different location across the world.

### 6.6.5 Total dilution

The total dilutions at distant locations for a wastewater discharge to the marine environment can be defined as:

$S_T$ = Initial dilution x Secondary dilution x Decay
= $S_i$ . $S_e$ . $S_d$

***EXAMPLE…***

Refer to Section 6.6.3 and the example using Wright's theory (1984): for an ambient current ($u_a$) of 0.2 m/s the achievable initial dilution for a peak flow rate is:

$S_i$ = 550

The total dilutions for conservative substances and for microbiological organisms at 500, 1000, 2000, 3000, 4000 and 5000 m distances from the outfall are:

| *DISTANCE (x in m)* | $S_e$ | $S_d$ | | $S_T$ *(conservative substances)* | $S_T$ *(Microbiological substances)* | |
|---|---|---|---|---|---|---|
| | | *Day time* | *Night time* | | *Day time* | *Night time* |
| *500* | *1,1* | *2.22* | *1.17* | *605* | *1343* | *710* |
| *1000* | *1,4* | *4.95* | *1.38* | *770* | *3811* | *1060* |
| *2000* | *2,3* | *24.5* | *1.90* | *1265* | *30992* | *2398* |
| *3000* | *2,8* | *121* | *2.61* | *1540* | *186340* | *4020* |
| *4000* | *3,6* | *600* | *3.60* | *1980* | *1188000* | *7117* |
| *5000* | *4,7* | *2970* | *4.95* | *2585* | *7677450* | *12794* |

Figure 6.52 presents a graphical illustration of the achievable dilutions versus distance. This output provides the first overview/assessment of the environmental performance of the system while optimising the outfall

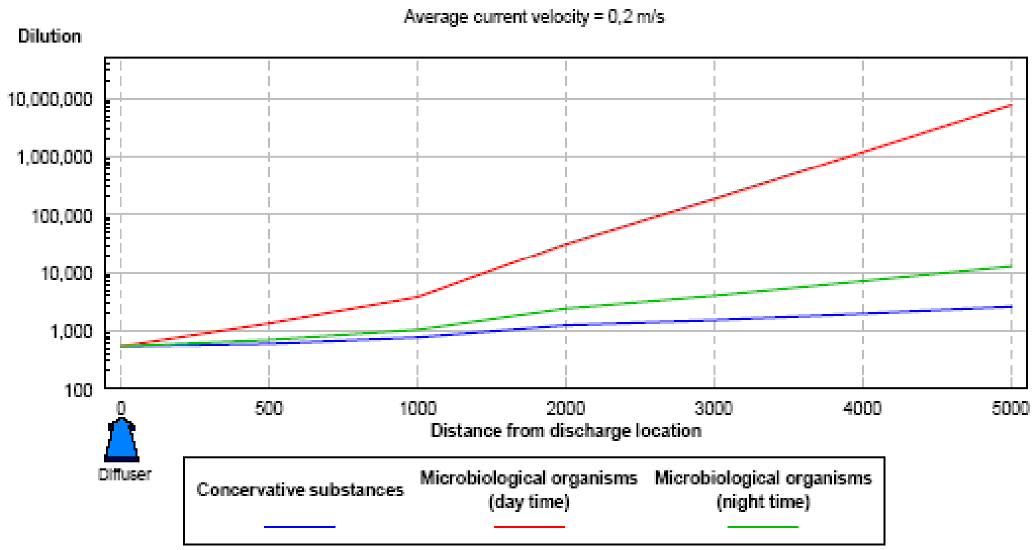



***FIGURE 6.52:*** ***A graphical illustration of the achievable total dilutions versus distance***

### 6.6.6 Data requirements for pre-assessment and detailed investigation

The procedures described in Section 6.6.2 to 6.5.5 need to be followed in both the pre-assessment and detailed investigation stages, albeit to different levels of detail.

### *i. Pre-assessment*

In computing the initial dilutions for offshore marine outfall as part of a pre-assessment, only average and worst-case scenarios related to descriptive statistical parameters of physical conditions (currents and stratification) need be used, in combination with the outputs from the hydraulic design.

For the analytical/statistical estimation of the achievable secondary dilutions for offshore, surf zone and estuarine waste water discharges as part of a pre-assessment, only average/typical and worst-case scenarios related to descriptive statistical parameters of physical conditions (currents and stratification) will be used. Typical values for day and night-time micro-biological decay rates will be used in conjunction with the analytical determination of the secondary dilutions in time and distance.

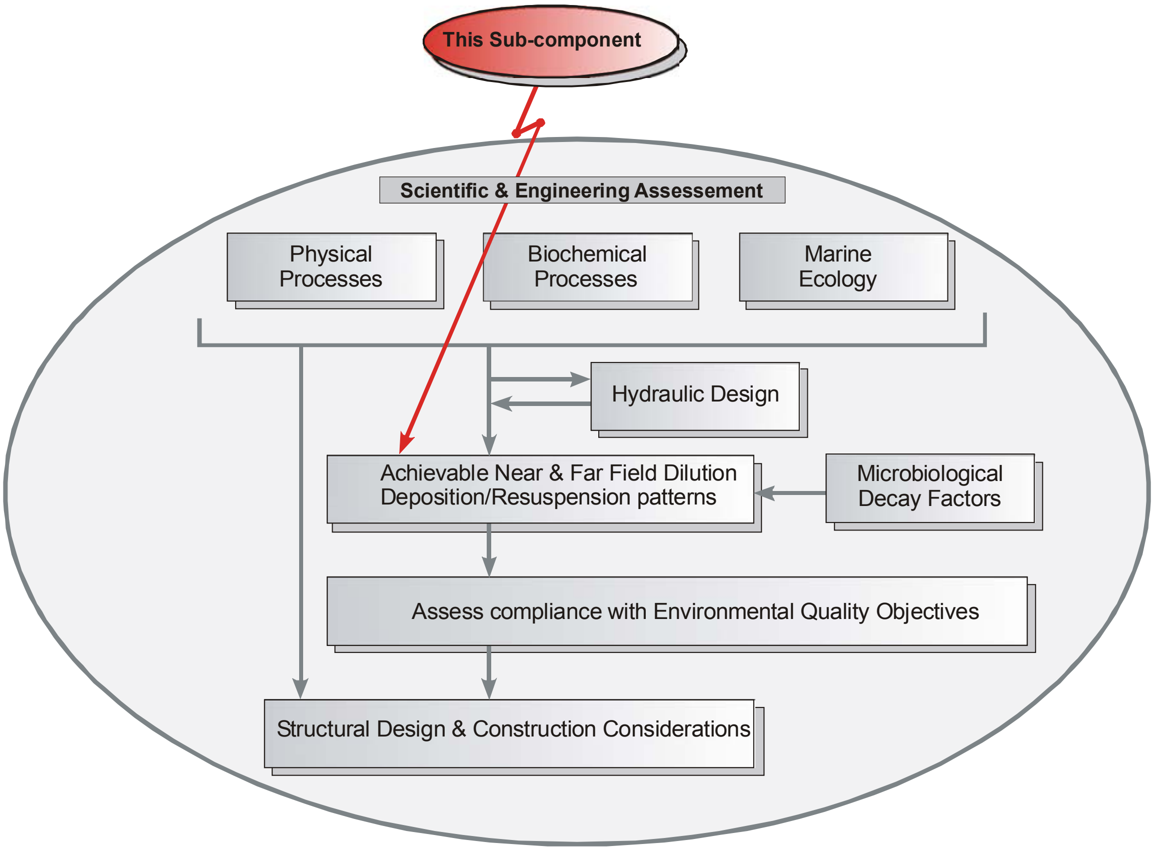
### *ii. Detailed investigation*

In the case of a detailed investigation, initial dilution needs to be modelled using measured or simulated real-time data on the physical conditions (e.g. stratification and currents) at the location of the proposed discharge in the study area, in combination with the proposed wastewater flow scenarios (including diurnal and seasonal cycles). The output needs to be presented as a time-series (i.e. dilution and plume geometry) for the determination of frequency distributions of achievable dilutions.

Depending on the volume of the wastewater discharge, the sensitivity of the receiving marine environment and the complexity of natural processes, refined analytical/statistical estimations of the achievable secondary dilutions for offshore, surf zone and estuarine waste water discharges can be conducted, using measured physical conditions such as currents and stratification. Diurnal (and seasonal) variations for day and night-time microbiological decay rates will be used in conjunction with the analytical determination of the secondary dilutions in time and distance.

Normally (especially with deep sea outfall), a 3-D numerical model (in some cases it may be justified to use a 2-D model) will be 'constructed', calibrated, verified and applied for the prediction of far field dilutions and subsequent reduction of the concentrations of the wastewater constituents. Real-time measured (or simulated) nearshore data, as well as diurnal microbiological die-off rates, will be used in conjunction with the time-series data (i.e. dilution and plume geometry) from an initial dilution model.

## 6.7 SEDIMENTATION/RE-SUSPENSION OF SOLID PHASE PARTICLES




***PURPOSE:***

***The purpose of this component is to establish the physical fate of suspended material with regard to transport, deposition and possible re-suspension after deposition. The aim is to identify possible depositional areas which could act as a sink for harmful chemical constituents (originating from wastewater inputs) when adsorbing onto suspended material.***

### 6.7.1 Overview

Sediments are generally classified according to grain size as clay, silt, sand, gravel and boulders, ranging from very fine to coarse. Suspended and deposited particles in the marine environment are not only of lithogenous origin but can also be of organic nature (breakdown of marine fauna and flora or introduced from land sources such as rivers, stormwater and wastewater streams). Temporal changes to sediments and suspended particles, such as flocculation and chemical interactions (adsorption of certain constituents onto sediment, dissolution of certain constituents in sediment particles), can occur. Sewage effluents contain organic particles, varying in density and size and mostly with low settling velocities (Gunnerson, 1988). The settling speed of the suspended particles depends on the specific gravity, size and shape of the particles as well as the specific gravity and viscosity of the receiving water.

The physical fate of the suspended material with regard to transport, deposition and possible re-suspension after deposition is primarily related to the current and wave dynamics. The effect of current velocities on sediments is illustrated in Figure 6.53 (Gunnerson, 1988).

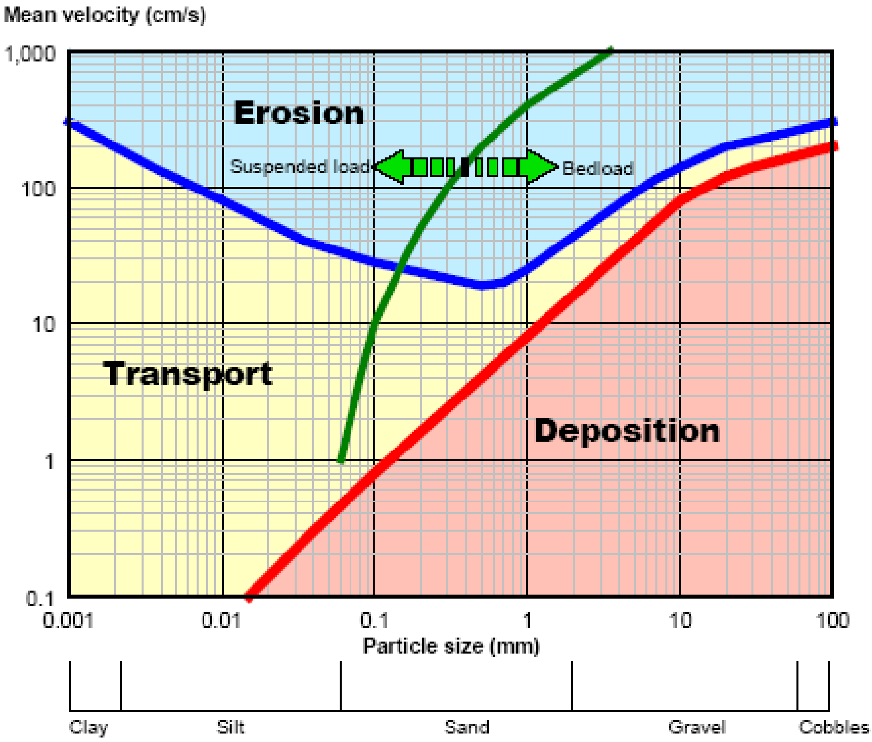



***FIGURE 6.53:*** ***The effect of current velocities on the erosion, transport and deposition of sediments (Gunnerson, 1988)***

Gunnerson (1988) also related the solids in sewage wastewater to settling velocities and velocities required for re-suspension as shown in Table 6.11. However, it should be noted that velocities are not fully representative of what occurs in terms of sedimentation and re-suspension, as it does not take into account all wave effects/turbulence. Where numerical modelling is used to predict deposition and re-suspension it is thresholds of bed shear stress that is typically the determining factor.

***TABLE 6.11: Settling velocities and velocities required for re-suspension of solids in sewage wastewater (Gunnerson, 1988)***

| *PARTICLE SETTLING VELOCITY (cm/s)* | *PERCENTAGE OF SETTLING PARTICLES WITH SETTLING VELOCITIES > THAN INDICATED* | |
|---|---|---|
| | ***RAW SEWAGE*** | ***PRIMARY TREATED WASTEWATER*** |
| *1.0* | *5* | *-* |
| *0.5* | *20* | *-* |
| *0.1* | *40* | *5* |
| *0.05* | *-* | *-* |
| *0.01* | *60* | *20* |
| *0.005* | *-* | *30* |
| *0.001* | *85* | *50* |
| ***BOTTOM CURRENT VELOCITY (cm/s) FOR RE-SUSPENSION*** | | |
| *Re-suspension unlikely* | *0 – 6* | *0 – 6* |
| *Re-suspension possible* | *6 - 30* | *6 - 20* |

Suspended particles (or 'solid' phase particles) comprise cohesive (non-biological) particles and organic particles, to be referred to as *solid phase particles*. Cohesive (non-biological) particles represent very fine sediment particles (< 60 µm) where adsorption phases such as Al(OH)x, Mn(OH)x and Fe(OH)x are common. The origin of the organic particles can be natural (e.g. algal blooms) or introduced through anthropogenic activities (e.g. sewage disposal). The transport and fate of chemical constituents associated with the 'solid' phase are therefore largely determined by the flux and sedimentation/re-suspension behaviour of these particles. The sedimentation/re-suspension behaviour of solid phase particles is therefore required in order to determine the fate of adsorbed chemical compounds in the receiving marine environment (Luger *et al*., 1999; Monteiro, 1999).

***EXAMPLE…***

A plot of the distribution of Copper in Saldanha Bay (South Africa) showing the strong correlation between predicted, observed depositional areas, is provided in Figure 6.54 (Monteiro, *et al*., 1999). The long-term depositional zones, characterised by the smaller sediment particle size fractions, are the most vulnerable to contaminant accumulation.

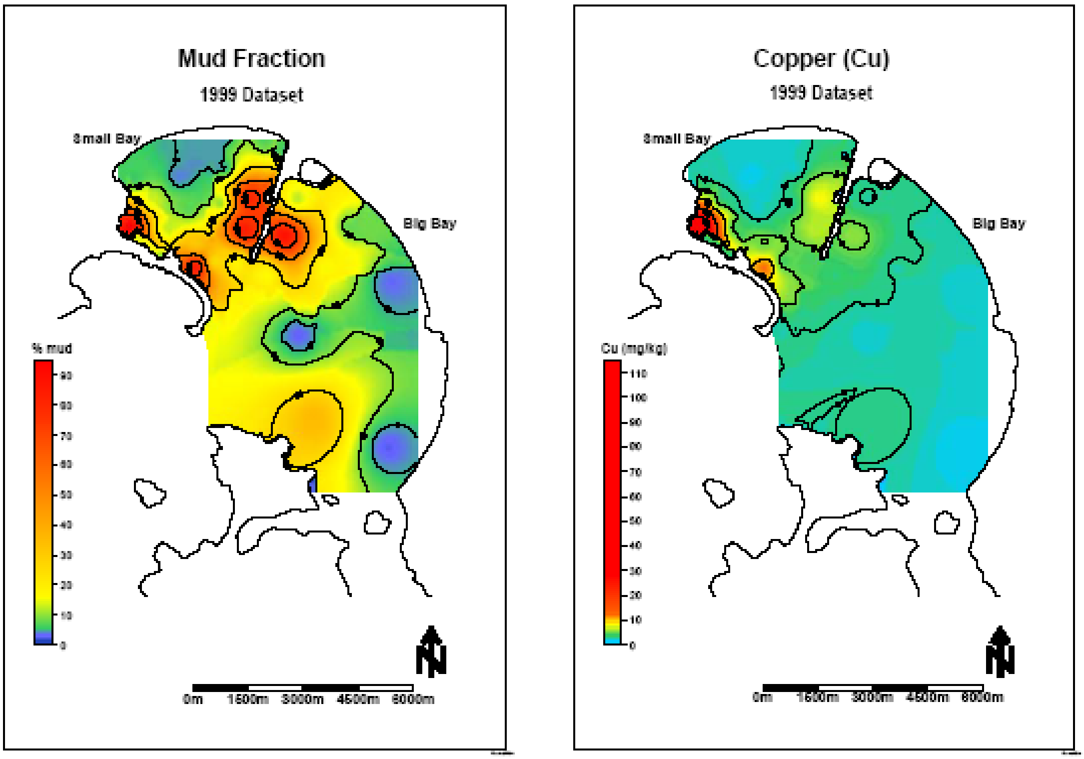



***FIGURE 6.54: Plot of the distribution of copper and particle size in Saldanha Bay (South Africa), showing the strong correlation between depositional areas and trace metals deposition***

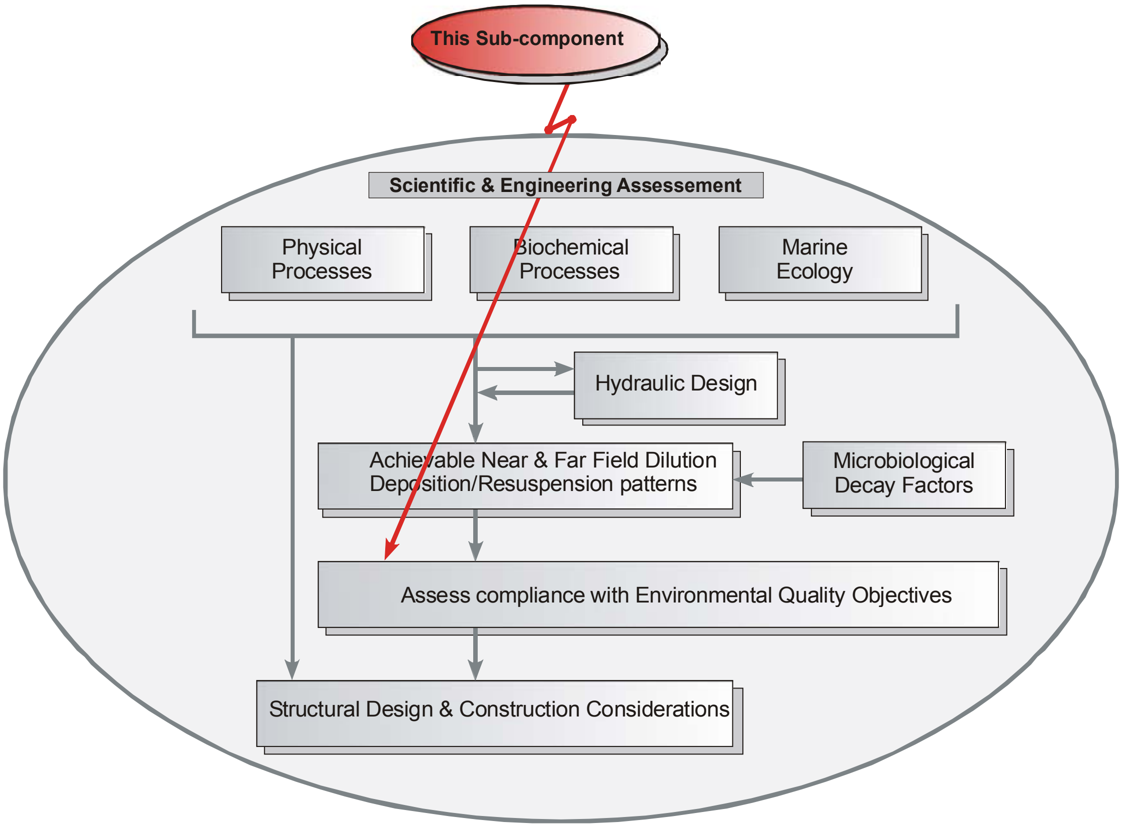
### 6.7.2 Data requirements for pre-assessment and detailed investigation

As part of a pre-assessment, the following has to be assessed, based on available information:

- The proximity of potential depositional areas in the study area, typically characterised by weak currents and low wave energy, e.g. in sheltered bay areas.
- The percentage occurrence of currents that are likely to transport a wastewater plume to depositional areas
- Concentration and loads of suspended matter in the wastewater plume after initial dilution, as well as the estimated concentration when arriving at the depositional areas
- Concentration and load of constituents in the wastewater (e.g. trace metals) that, through adsorption to organic or cohesive sediment particles, can be transported to the depositional areas
- The possibility that re-suspension will occur, based on the percentage of time within which wave and current velocities are sufficient to re-suspend deposited material.

As part of a detailed investigation, the complex physical and biogeochemical processes associated with the deposition and re-suspension of solid particles are best assessed using properly calibrated and verified numerical models.

## 6.8 COMPLIANCE WITH ENVIRONMENTAL QUALITY OBJECTIVES




***PURPOSE:***

***The purpose of this step is to:***

- ***Verify whether the environmental quality objectives will be adhered to, both in the near and far fields***
- ***Refine critical limits set for a wastewater discharge in terms of volume and/or composition.***

### 6.8.1 Overview

Compliance with environmental quality objectives, in essence, requires that the concentration of chemical constituents, after initial dilution or on reaching specific (beneficial use) areas in the receiving marine environment, comply with the environmental quality objectives specified for the study areas or for a particular use. In order to determine this compliance, the following needs to be taken into account:

- Beneficial use map of the study area and the environmental quality objectives associated with each use

- Behaviour of constituents in wastewater, taking into account interaction with the biogeochemical characteristics and processes in the receiving environment, as well as waste inputs from other activities into the defined water body (this is necessary to quantify potential synergistic effects and to assess cumulative impacts)

- Processes affecting the achievable dilution, i.e. to determine the dilution and transport of constituents in the ‘dissolved phase’, both in the near and far fields

- Processes affecting the sedimentation/re-suspension of solid phase particles, i.e. to determine the transport and fate of biogeochemical constituents associated with the ‘solid’ phase, both in the near and far fields.

Based on the above, the spatial and temporal concentrations of biogeochemical constituents in the receiving environment are predicted. These outputs are then superimposed on the beneficial use map (and associated environmental quality objectives) to establish if there is compliance with the environmental quality objectives.

### 6.8.2 Data requirements for pre-assessment and detailed investigation

#### *i. Pre-assessment*

The typical output to show compliance/non-compliance with environmental quality objectives, as part of a pre-assessment study, is as follows:

- Statistical presentations of the concentrations of the constituents in the wastewater plume after initial dilution. It is normally required that all the biogeochemical constituents in wastewater, except for microbiological indicators, comply with the environmental quality objectives within the initial mixing zone (this will be attained if the required dilution is less or equal to the initial achievable dilution). A precautionary (conservative) approach must be followed, i.e. taking into account the worst-case scenarios, such as:

    - Maximum concentrations in the wastewater and maximum discharge rates for the full range of scenarios
    - Physical conditions which will result in the minimum achievable dilutions (both for the near and far fields)
    - Using the achievable initial dilution for the worst performing port in the diffuser.

- Spatial and statistical presentations of the concentrations of constituents in the wastewater (remaining in the ‘dissolved phase’) in the waste field at the time of being transported away from the discharge location (i.e. outside the initial mixing zone).

For microbiological organisms, typical daytime and night-time decay coefficients must be used and compliance to environmental quality objectives at specific (beneficial use) areas must be determined for the full range of current velocities. For example, the variation in the distance from the discharge location at which faecal coliform counts comply with environmental quality objectives (or water quality guidelines) under a range of current velocities, irrespective of the current direction, is illustrated in Figure 6.55.

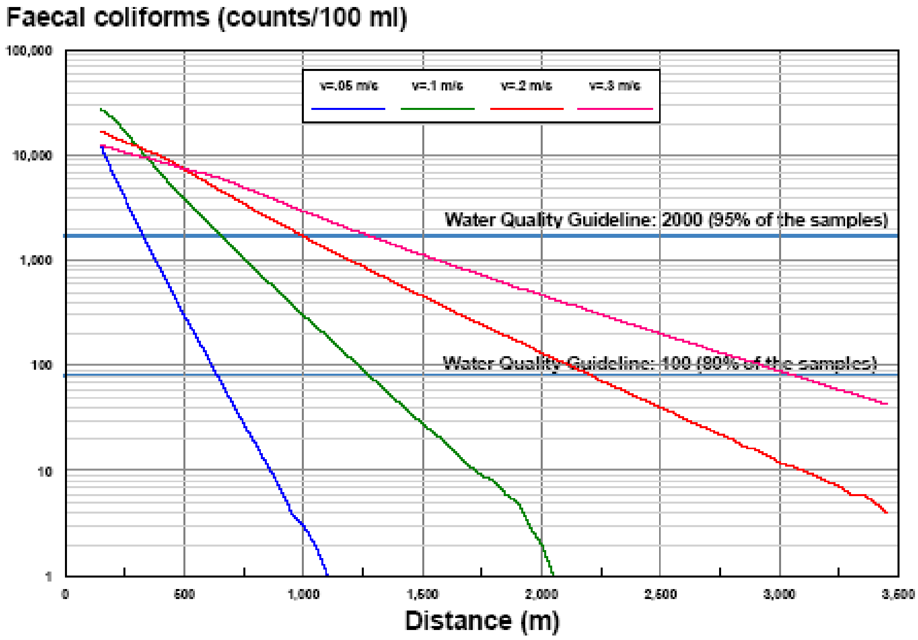



***FIGURE 6.55:*** ***Example: Variation in*** *distance from the discharge location at which faecal coliform counts comply with environmental quality objectives (or water quality guidelines) under a range of current velocities*

- Identification of the percentage of occurrence of currents, as well as average velocities on route to sheltered areas within the study area, where stagnant conditions are likely to occur. Referring to Figure 6.53, a conservative assumption is that deposition for fine material may occur in current velocities of less than 3 cm/s (considering that only preliminary treatment was applied and that a degree of flocculation will occur).

  For an ‘open’ coastline, current velocities in the upper layers of the water column rarely drop below 5 cm/s and, if this occurs it is normally for a very short time, for example, when currents change direction on account of the tide. An estimate of the percentage occurrence of current velocities less than 5 cm/s (stagnant conditions) will provide a conservative estimate of the time at which deposition can be expected.

- Spatial and statistical estimates of deposition/re-suspension of constituents in the wastewater (i.e. associated with solid phase particles) in the far field. A conservative approach to follow for a pre-assessment would be, for example, to assume that all trace metals present in the wastewater will adsorb onto solid phase particles and, thus, be transported to the depositional areas identified above.

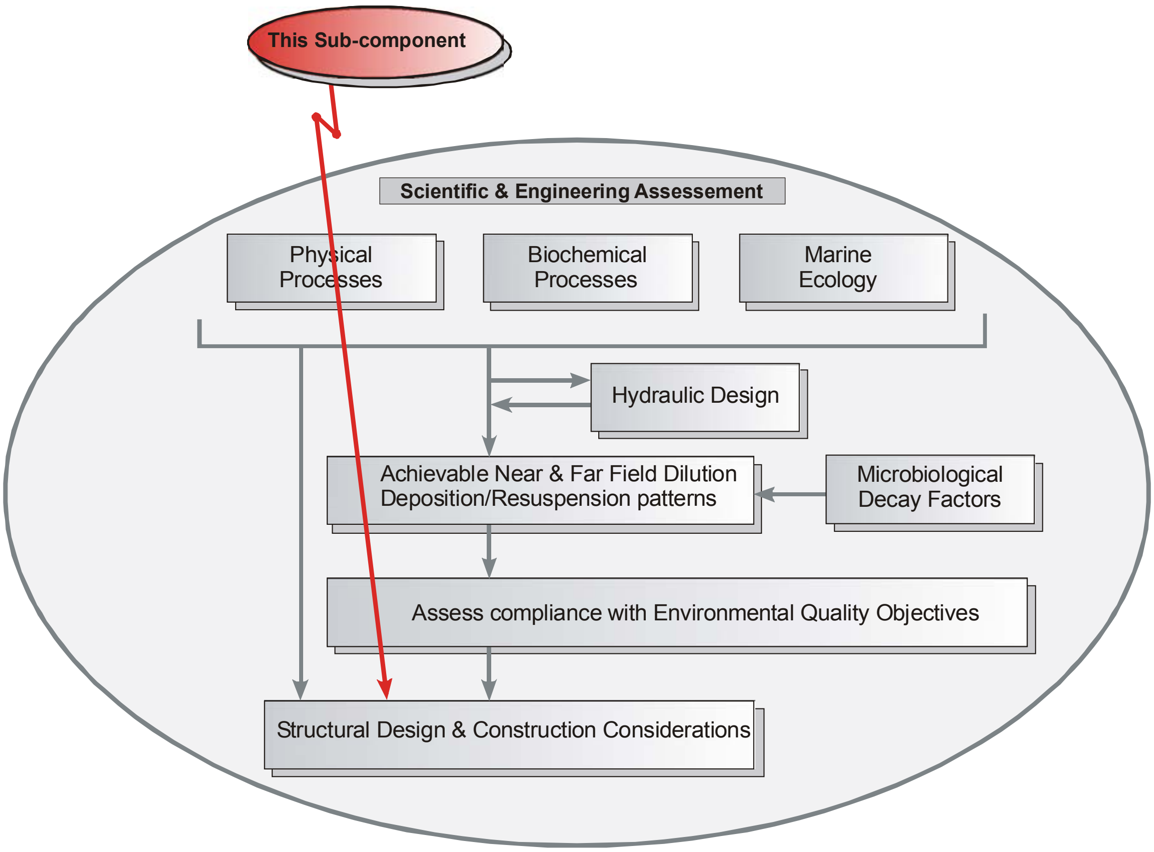
### *ii. Detailed investigation*

For a detailed investigation, a water quality model, using output from the hydrodynamic model, is used to predict more realistically the fate of constituents in the wastewater, taking into account the influence of all existing waste inputs together with the proposed discharge and variability in ambient biogeochemical conditions. In particular, the water quality model is used to predict (quantitatively and dynamically) (Luger *et al*., 1999; Monteiro, 1999) the:

- transport and fate of constituents in the wastewater that remain in the dissolved phase (e.g. Figure 6.44)
- deposition/re-suspension of constituents in the wastewater associated with the 'solid' phase (e.g. Figure 6.54)

Water quality modelling can also be applied to simulate different scenarios (e.g. variations in wastewater composition and flows) to determine the effects of these on the environmental quality objectives.

## 6.9 CONSTRUCTION CONSIDERATIONS AND STRUCTURAL DESIGN




***PURPOSE:***

***The purpose of this component is to provide specifications for the structural design and construction of a marine wastewater discharge system. Sub-components that need to be specified include:***

- ***Pipeline material***
- ***Construction methods***
- ***Detailed design of the structure***
- ***Contractual aspects***
- ***Decommissioning.***

### 6.9.1 Pipeline Material

WRc (1990) grouped materials for pipelines in three categories (main materials):

- Ferrous (Steel and cast-iron)
- Polymetric (Polyethylene, PVC, Glass-reinforced plastic)
- Cementitious (Steel reinforced concrete)

The most commonly used pipe material for ocean outfalls is steel or plastic (HDPE). Other pipeline materials are:

#### *i. Ferrous materials*

***Coated Steel***. Steel pipe has been used extensively by the oil and gas industry for underwater works because it allows a great degree of flexibility in construction techniques. Steel pipe has also been used in numerous outfalls. It is ideal for bottom-tow construction and also suitable for float-and-lower as well as the lay barge method.

Large diameter pipes (up to 3000 mm diameter) can be manufactured by rolling and welding as well as by spiral fusion welding. Both methods are flexible and allow for any diameter or wall thickness. Lengths can be specified, but usually pipes with a diameter > 500 mm are supplied in 9 m lengths.

Weld seam inspection and defect identification are of the utmost importance and detailed inspection clauses should be included in the specifications. Steel pipes for marine use are usually joined by welding, although flanged connections are sometimes used at the ends of an outfall to permit extension of the line or to add a diffuser at a later time. Welded joints are usually checked by X-ray radiography.

Steel is susceptible to corrosion in seawater. However, it has been proved that a well-coated pipeline with cathodic protection has a reasonably long life. The initial costs of steel pipe and cathodic protection may be higher than for plastic pipe. The advantages of steel pipe include its adaptability to rapid fabrication and installation, its joint tightness, inherent structural integrity, and its higher head capacity. This ability to operate at higher pressures allows the flexibility of conversion from a gravity outfall to a pumped or pressured outfall in the future.

***Cast-Iron***. Although historically a popular material, cast-iron has not been used for marine outfalls in South Africa. Its strength is comparable to steel pipe and it has corrosion-resistant properties that are superior to unprotected steel pipe (Gunnerson, 1988). However, it has poor flexibility and impact resistance and it is more expensive than other types of pipe. It is seldom used as an outfall pipe material today.

#### *ii. Polymetric materials*

***Polyethylene***. Polyethelyne pipes are categorised by the density of the constituent material, that is, low density (LDPE– 0.915 to 0.924 g/cm$^3$), medium density (MDPE– 0.925 to 0.944 g/cm$^3$) and high density (HDPE – 0.945 to 0.965 g/cm$^3$). The HDPE pipes are the most commonly used for marine outfalls. With regard to corrosion resistance (wastewater and seawater), polyethylene pipes have great advantages over ferrous and cementious materials. Polyethylene is not affected by acidic conditions, (sewage wastewater) or corrosive chloride and sulphate ions in seawater. Disadvantages are that solvents (e.g. petrol) may affect the strength of the pipe and, like all thermoplastic materials, the mechanical properties are time and temperature dependent.

Polyethylene pipes are manufactured from pre-compound granules, which contain the polymer as well as additives such as anti-oxidants, pigments, etc. Manufacturing is by extrusion, under pressure, (heated granular material) through a die to provide the required diameter (up to 1200 mm diameter) and wall thickness. The extrusion plant can be at the construction site, eliminating the transport of pipes.

Polyethylene pipes can be fusion welded into continuous pipe strings to be installed by lay-barge or the float-and-sink-method. Because the density of the pipes is less than that of seawater, the pipeline must be stabilised on the seabed by additional weights. Adding concrete collars before the pipe is towed out to sea is a possible solution.

During backfilling, care must be taken to ensure that the backfill weight is within limits with regard to possible deformation of the pipe. Low longitudinal bending stiffness makes polyethylene pipes ideal for undulating trench profiles as they can conform to the seabed profile. Continuous concrete weighting can be used to keep the bending characteristics.

***Glass-reinforced plastic (GRP)***. Although not used at present in marine outfalls in South Africa, GRP as a material for large pipe diameters recently has been used in other countries. The advantages are its strength to weight ratio, corrosive resistance and the ability to be moulded to any shape or dimension for specific requirements. Filled with water, GRP pipes have a slight negative buoyancy and can be installed by the pipe-by-pipe method, with concrete collars for additional weight and stability. A disadvantage is its susceptibility to impact damage (brittleness) during installation and backfill. Care must also be exercised when concrete collars are fitted and appropriate packing material must be used between collars and the pipeline.

***Other thermoplastic materials***. Other materials such as uPVC (unplasticised polyvinal chloride) and polypropylene have been used before in other countries but, in general, will not be considered at present for long sea outfalls.

### *iii. Cementious materials*

***Steel reinforced concrete***. Overseas, marine outfalls greater than 215 cm in diameter have been built of reinforced concrete pipe. Installation costs can be quite high. Concrete pipe is highly resistant to corrosion and to attack by seawater or marine organisms.

## 6.9.2 Construction methods

### *i. Outfall construction*

***Bottom-pull method.*** This is the most frequently used method of outfall construction due to it's suitability for exposed as well as sheltered areas. According to WRc (1990), the limiting wave height for construction is between 2 and 3 m.

Weldable steel is the most suitable material. The high bending/tensile strength of steel can withstand the installation loads. Care must be taken not to exceed the allowable stresses of the concrete protection.

The bottom-pull method can only be considered if the outfall has a straight route alignment. This method requires a relatively large construction site at the shore crossing for pipe storage, assembling and launchway. A minimum construction area of 100 m x 80 m will be required. A typical construction-launching site for the bottom-pull method is shown in Figure 6.56.

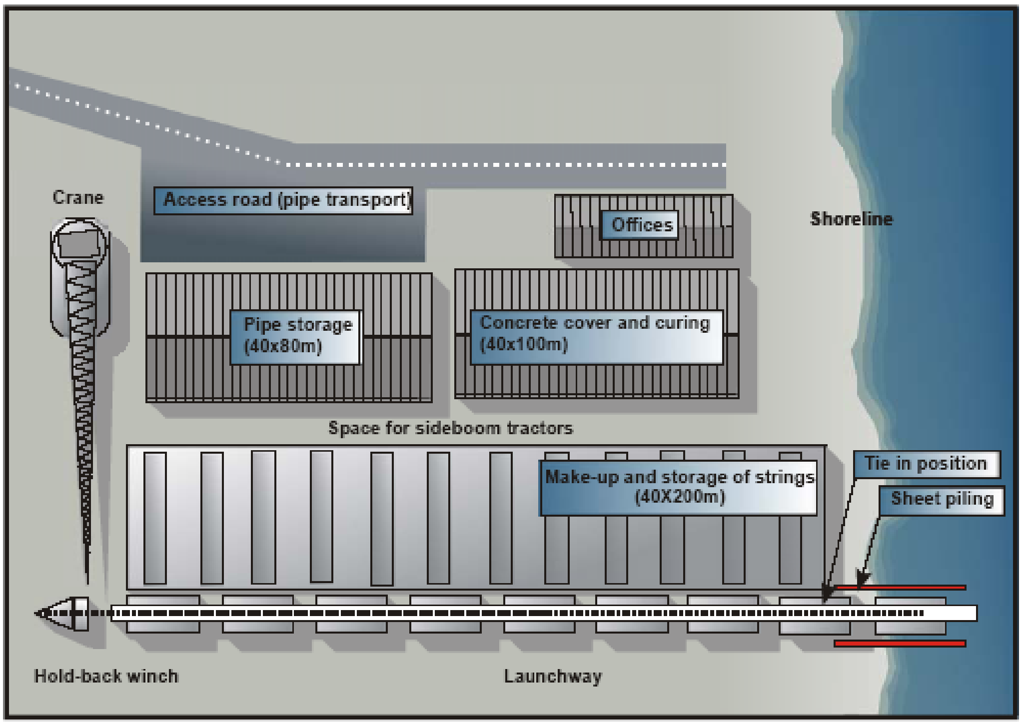



*FIGURE 6.56: A typical construction-launching site for the bottom-pull method*

Pulling sections of corrosive pre-coated steel pipe with concrete cover are assembled onshore in at least 80 m lengths. Field welds are non-destructively tested, and corrosion-resistant protective coating is applied to the field joints. Each pulling section is stored on rollers parallel to the launching ramp to be tied in as the pipe is pulled out to the sea.

The pulling barge is anchored offshore, directly in line with the launchway (route of the outfall line). Before initiating the pulling operation, one end of a cable or wire rope is connected to a pulling head welded to the leading section of pipe. Depending on the length of the pulling sections and the total pipeline length, the pulling operation can be completed within 2 weeks. If the submerged weight of the pipe is more than 50 kg/m, it may be necessary to add buoyancy to the pipe to prevent damage to the external concrete coating.

***Surface-pull (float-and-lower-method).*** Normally, this method is more suitable for outfall construction in sheltered areas. Even under moderate wave conditions, the pipe can start to oscillate and a lateral current can force the pipe off-line. The handling of a long floating pipeline requires specialised equipment and expertise.

For the construction of the pipe, a sheltered area in close proximity to the proposed route is required, where the pipeline can be left (floating or submerged) until being towed out to the site. Horizontal bending during the towing operation and vertical bending during the lowering operation require a high degree of flexibility, and plastic material such as HDPE is the most suitable for this method. Pipe lengths can be fusion welded into a continuous line or can be manufactured on site by extrusion, in which case a relatively small onshore construction site is required with minimum onshore traffic. Required weight collars are installed onto the air-filled pipeline before the tow-in operation. After flooding the pipe along the proposed route, care must be taken that no air pockets remain in the pipe.

***Pipe-by-pipe method.*** This method is suitable for shorter marine outfalls. The pipe sections are manufactured off-site (most pipeline materials can be used), transported to a barge and lowered into position to be tied up to the existing line.

Pipes can be joined by bolted flanges, or spigot and socket joints with 'O' rings can be used. Installation can take a long time, because the underwater jointing can be delayed by adverse weather. Completed sections can be protected as the installation proceeds, leaving no long unprotected sections that can be damaged during adverse sea conditions. Thus, apart from possible delays, the risks during construction are relatively small, compared to other methods.

For larger diameter pipes in deeper water, alignment with the existing line can be a problem. This can be overcome by using a pipe handling frame to provide vertical and longitudinal alignment for the tie-up of a 'new' pipe to the existing line. Alignment frames also contain a chamber that enables a diver-welder to join the sections.

***Reel-barge and lay-barge methods***. These methods require small onshore construction sites. The pipe is pulled from the sea to the high-water mark and is then lowered in the offshore direction from the barge. For the reel-barge method, only smaller diameter (300 mm) plastic pipes can be used, while large diameter pipelines of weldable steel or HDPE can be installed by the lay-barge method. These methods are suitable for long lengths of pipelines (oil and gas) and are normally not used for 'short' sea outfalls.

### *ii. Trenching*

Trench widths relate to the type of dredger that is used. Typically the width ranges from 5 m to 10 m and the side slopes will depend on the seabed material. When the seabed consists of hard material and blasting or the use of special cutter suction dredgers is required, a trial trench will facilitate the cost estimation and provide more clearly defined tender specifications. Detailed specifications are required for measures that ensure that the side slopes for the entire length of the trench remain stable until backfilling is completed and that the level and line of the trench are accurately controlled.

Normally a pipe will be buried below the lowest possible seabed profile in the surf zone area. It may also be a requirement for the offshore region that the seabed has to be restored to the original level, which will require a deep trench. The trench depth and the degree of protection will depend on:

- natural characteristics of the seabed (rock, sand, clay).
- impact of an artificial protrusion above the seabed on the natural processes and ecology
- possible threats (ship anchors, fishing gear, direct contact with a ship).

Trenching and rock protection are the most effective and secure method for protecting a marine outfall. However, in some cases (undulated rocky seabed), it is not practical or economically viable to provide a trench below the seabed along the entire route of the pipeline. The aim then will be to provide a level bed for the pipeline with protection that at some places will be above the seabed.

Land-based machinery (e.g. backhoe) on a pontoon (loading the dredged material on barges) can be used in shallow water. Two types of dredgers are normally used for outfall trenching, namely cutter suction dredgers, which can also cope with harder type material such as soft rocks (e.g. coral and 'soft' sandstone), and bucket dredgers. The dredged material is discharged to hopper barges.

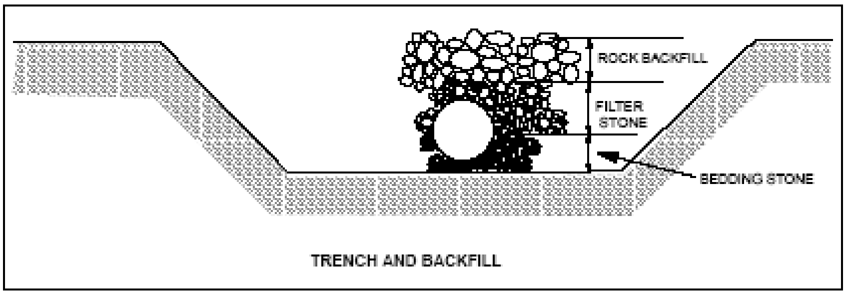
A pipe can also be jetted into the seabed, using a sledge with water jets, suction pumps, etc. This method can also be used when a trench has become filled with sediment before the installation of a pipe. Similarly, fluidisation, which involves the forcing of large quantities of water into the soil surrounding the pipeline, can be used.

### *iii. Backfilling*

Backfill material for outfalls can be placed from barges by side dumping in shallow water or by a fall pipe in deeper water for more accurate control.

Normally, dredged material (if not rock) is not suitable for backfill. When a pipe has to be protected by rock armour, a filter material must be used for the protection of the pipe against the larger rocks. If a trench is in a rocky seabed, a stable bed has to be provided for the installation of the pipe and remains stable enough to provide a permanent foundation. It is important to ensure that the 'composite' structure of pipe and rock layers is stable and that the grading of the layers is such that finer underlayers do not 'escape' through the upper layers. Specialist advice will be required to ensure that the selected material is stable under the expected extreme conditions for the site. If the trench is in hard material (rocks), tremie concrete can be used for protection instead of rock armour. For the contract specifications, it is important to specify the minimum thickness of each layer after settlement of the material.

Examples of backfill cross-sections are shown in Figure 6.57.



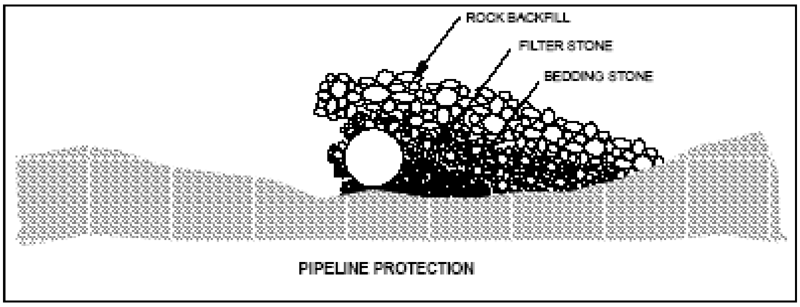




*FIGURE 6.57: Examples of backfill cross-sections*

## 6.9.3 Structural design

In general, there are three main areas to be considered in the structural design of outfall pipelines:

- **Stability –** bottom stability is a critical aspect in the design of an outfall. The pipeline must remain stable during installation (construction) and throughout its design life. Other aspects related to the ultimate stability of the pipeline that have to be investigated are trench stability, seabed stability, sediment transport and required rock armour for protection of the pipeline.

- **Stress –** stresses in the pipeline must at all times be within acceptable limits

- **Accidental damage –** the risk of accidental damage must be acceptably low.

***NOTE:***

| | |
|---|---|
| ***Under-design:*** | ***Leading to risk during operation*** |
| ***Over-design:*** | ***Leading to excessive costs.*** |

The purpose of this section is not to serve as a detailed design procedure (tool) but, with regard to the physical environment at the proposed outfall site, to provide the following:

- A checklist of the critical and important aspects to be addressed regarding the detailed structural design of an outfall

- Some basic/empirical methods for obtaining estimations of the magnitude of forces that can be expected, for a pre-assessment of suitable materials to be used, construction methods to be considered, and whether the pipeline needs to be protected or not.

Because of the specialised construction techniques for a marine outfall, the choice of material, construction technique/method and specialised equipment form a 'package' developed by recognised contractors. The preferred option should be designed in detail for the Tender Specifications and Bill of Quantities. However, due to specific contractor expertise, for major outfall projects during the past 8 years, an option has been included in the Tender for alternatives, providing that the Contractor must provide a structural design together with his bid, to be reviewed in detail by the client's engineer.

### *i. Hydrodynamic Forces*

When water flows around and over a structure, hydrodynamic forces will be exerted on the structure, resulting from pressure differences in the flow field. Buried pipelines are usually not directly subjected to such hydrodynamic forces after installation, except for the diffuser section.

***Currents (steady flow)*.** For an exposed pipeline or partially-buried pipeline, the vertical lift force (normal to the current because of low pressure on the downstream side of the pipe) and drag forces (perpendicular or normal to the pipeline) in an ambient flow (quasi-steady currents are caused by factors such as tides, continental circulation, storm surges and wind stress interacting with the wave orbital oscillations and result in net particle velocities and accelerations) can be estimated as follows WRc (1990) (Figure 6.58):

$$F_D = 0.5\,\rho.C_D D.u^2$$
$$F_L = 0.5\,\rho.C_L D.u^2$$

where

$F_D, F_L$ = Horizontal and vertical drag forces
$C_D$ = Drag coefficient (typically 0.5 to 2.0)
$C_L$ = Lift coefficient (typically 0.9 to 2.0)
$\rho$ = ambient water density (kg/m$^3$)

The drag and lift coefficients are functions of the Reynolds number and also depend on factors such as wall roughness of the exposed outfall, the velocity profile over the height of the pipe, and the roughness of the seabed.

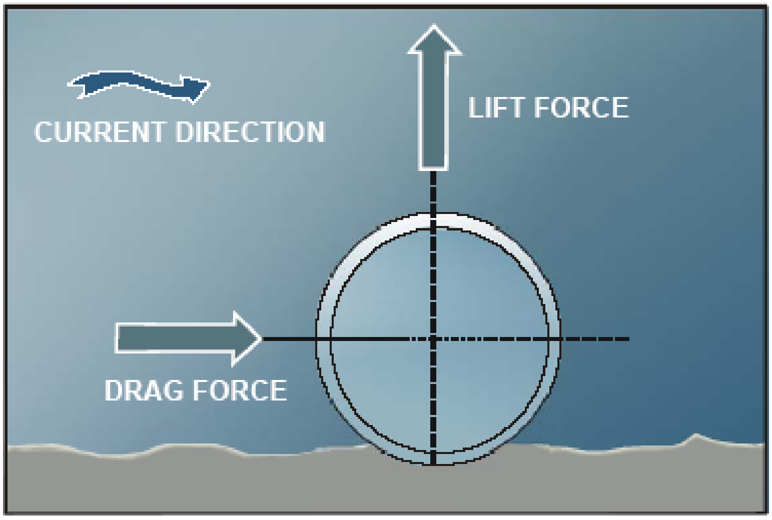



*FIGURE 6.58: Drag and lift forces on a pipeline in steady flow*

Gunnerson (1988) provided a guideline (Figure 6.59) for drag and lift coefficients for an exposed pipeline on the seabed and recommended that these be used as an upper range, as higher values result in over-design and excessive costs. The Reynolds Number ($R_e$) is defined as:

$R_e$ = $uD/\mu$

Where:
u = Current velocity (m/s)
D = External pipeline diameter (m)
$\mu$ = Kinematic viscosity of seawater ($m^2/s$)

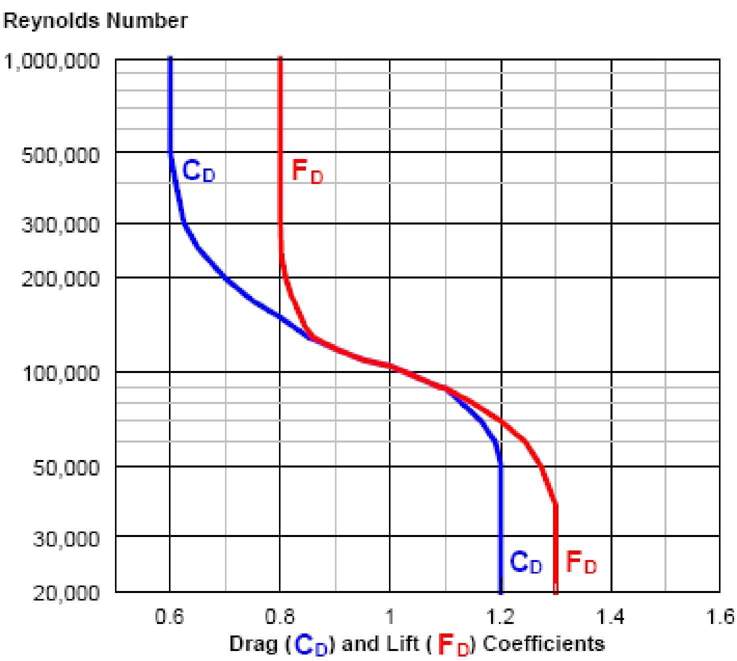



*FIGURE 6.59: Drag and lift coefficients for an exposed pipeline on the seabed (Gunnerson, 1984)*

***Waves (unsteady flow)***. Unsteady flow occurs under waves, where an outfall is subjected to an oscillatory wave-induced current (superimposed on a steady current component).

With reference to WRc (1990), most analysis methods are based on the Morison method, which is almost universally used in the submarine pipeline industry, because of its reasonable representation of prototype measured forces in unsteady flow.

The drag ($F_D$), inertia ($F_M$) and lift forces ($F_L$), according to the semi-empirical Morison equation (WRc, 1990), are:

$F_D = 0.5\ \rho . C_D D . u|u|$
$F_M = [\pi\ \rho . C_M D^2/4] du/dt$
$F_L = 0.5\ \rho . C_L D . u^2$

where
$C_D$ = Drag coefficient (typically 1.0 to 2.0)
$C_L$ = Lift coefficient (typically 1.25 to 1.5)
$C_M$ = Inertia coefficient (typically 1.65 to 3.29)
$du/dt$ = water particle acceleration normal to the pipe
$\rho$ = ambient water density (kg/m$^3$)

The choice of coefficients is controversial and careful consideration must be given to published values with regard to the approaches and assumptions followed for the determination of these coefficients. (WRc, 1990).

An alternative approach, as proposed by Grace (1978), is based on force coefficients, and the extreme horizontal and vertical forces during a wave cycle are determined.

To combine wave and current forces, the simplest approach is to vectorially add the velocity components and use the result for calculating the forces.

### *ii. Stability*

The design philosophy for outfall pipelines is to ensure complete stability under the maximum hydrodynamic forces. This stability can be achieved by complete burial of the outfall. However, in many cases, the pipeline is partially buried or fully exposed, conditions which may also occur temporarily during construction prior to backfilling. The pipeline weight or anchoring system must be adequate to resist the expected hydrodynamic forces during and after construction.

If an outfall is exposed on the seabed, it must resist lateral movement under the hydrodynamic forces induced by waves and currents. The forces on a cross-section of an unburied outfall (WRc, 1990), which resists lateral movement by its own weight ($W_S$), are:

$W_S \geq (F_D + F_M)/f + F_L$

Where
f = seabed friction factor

The seabed friction factor for a threshold of large movements on a sandy seabed according to WRc (1990), can be taken as approximately 0.7. For a partially buried pipeline, the seabed friction factor (f) will increase considerably. Due to the characteristics of clay in a dynamic environment, the determination of a constant factor is much more complicated.

Sediment transport is an important aspect regarding the stability of a pipeline. Erosion of the seabed can result in full exposure of a buried pipeline and can even cause a length of pipeline to become unsupported. The theory of sediment transport is extensive and will not be discussed here.

The biggest change in seabed profile is caused by onshore-offshore transport in the breaker zone which can vary in meters. Typically winter and summer profiles are predetermined, but major short-term variations during storm conditions may be encountered.

Long-shore sediment transport is caused by waves approaching the shoreline at an angle. If an outfall pipe is near the seabed in the surf zone, the transport of sediment will be interrupted, resulting in accretion upstream of the outfall and erosion in the lee of the pipe. It is therefore important that the depth of burial in the surf zone is below the lowest possible level of a sandy beach. The transport rate can be calculated analytically for known wave conditions. There are also numerous numerical models available which can be used to predict the longshore transport.

The best approach is to obtain information on the history of the beach/near-shore profile at the proposed outfall site. Envelopes of sufficient long-term data will provide a good estimate for the depth of burial of a pipeline.

The transport of sediment is of particular importance during the construction of a pipeline, because a trench acts as a sediment trap and, under certain conditions, can be filled in a very short time. Normally the trench will be protected by sheet piles in the surf zone area. Sufficient information on the sediment movement should be available to determine the depth and distance offshore for protecting the trench, because temporary protection is expensive. However if a trench becomes filled up during the installation of the pipeline, the consequences will be disastrous. For a bottom-pull method, the buried section can make it impossible to move the pipe.

Depending on the seabed properties, an outfall can also settle below the seabed under its own weight. Geotechnical investigations on the bearing capacity of the seabed material must be conducted. Gunnerson (1988) refers to the relation between the ultimate seabed bearing capacity (Q) and the pipe diameter (D):

$$Q \quad \alpha\ k.\ c_s D$$

where
$c_s$ = soil cohesive shear strength
$k$ = variable which is a linear function increasing to a constant when the depth is 4x pipe diameter

Attention must also be given to the possibility that a pipeline will start to float out of a trench. This floating will occur when the pipeline is lighter than the backfill material, a condition that ensues when the trench is filled naturally with soft material or where use is made of dredged material, which after handling becomes a liquid-mud with little or no shear strength.

The possibility of seabed liquefaction by wave action must also be determined.

The stability of the slopes of a trench will depend on bed material (clay or sand). In consolidated (stiff) clays, the slopes of the trench will be fairly stable. For sandy sea bottoms, the trench slopes will be between 1:3 and 1:6.

### *iii. Pipeline protection and stabilisation*

Protection of the pipeline against external forces, such as dragging anchors, can only be achieved by rock armouring. The design of rock armour as well as the filter material required between the rock armour and the pipeline/seabed, will depend on the geotechnical properties of the seabed and the hydrodynamic forces.

For stabilisation and protection against external and hydrodynamic forces above the seabed, burial in the seabed is the most economic method. It must be ensured that the depth of burial is below the extreme lowest seabed level, especially in areas where the seabed is unstable.

The pipeline can also be stabilised by increasing its weight. For steel pipes the concrete weight coat or/and the pipe wall thickness can be increased. For plastic pipes the weight of the weight collars or the number of collars can be increased. Although not general practice, pipelines can be anchored to the seabed, using piles in sandy areas or screw and rock anchors in rocky areas.

### *iv. Stress analysis*

The construction method and the pipeline material will determine the approach to follow for the stress analysis of an outfall. The total loading on an outfall will include a number of structural effects and consideration must be given to possible combined effects of superimposed stress components.

For the detailed design of long submarine pipelines, the stress analysis of the pipeline normally involves the application of a sophisticated structural numerical model. For sea outfalls, which are relatively short and rigid, analytical techniques can be applied without a loss of accuracy.

It is important to review existing outfalls with regard to the construction method, pipe materials, structural design approach and the structural behaviour and performance. These constitute the best guidelines and know-how for the optimum design of a new outfall.

### *v. Stresses arising from individual forces*

Stress may arise from the following:

- pulling forces during construction
- forces produced by pipeline curvature and spanning
- internal and external pressures
- hydrodynamic forces
- backfill forces
- thermal expansion forces.

Forces and stresses related to the construction phase,are discussed below. Compressive stresses are shown as negative and tensile stresses as positive.

***Pulling forces***. For the bottom-pull method of construction, the pull force is taken as the weight of the pipe times a longitudinal friction factor which will depend on the bed characteristics of the trench, taking into account the submerged and above ground sections. Referring to WRc (1990), the pulling force (P) is:

$$P = L_a W_a f_a + L_s W_s f$$

where

| | |
|---|---|
| $L_a$ | = Length of the pipe in air |
| $W_a$ | = Weight per unit length in air |
| $f_a$ | = friction factor of the launch way |
| $L_s$ | = Length of the submerged section of the pipe |
| $W_s$ | = Weight per unit length of the submerged pipe |
| $f$ | = seabed friction factor |

The longitudinal stress ($S_L$) induced by the pulling force is at its maximum at the seaward end, reducing to null at the landward end.

$$S_L = P/A$$

where

| | |
|---|---|
| $A$ | = cross-sectional area of the pipe, excluding all non-structural components |

***Forces resulting from curvature and spanning***. For the bottom-pull method of construction, curvature and spanning can occur as a result of an undulating seabed profile, a horizontal curve along the route, the profile of the launchway, and/or the spacing of the rollers on the launch way. Longitudinal bending will occur as the pipe axis is deflected from a straight line. For float-and-sink-methods horizontal and vertical bending will all the time during construction until the pipeline is on the seabed. Referring to WRc (1990), the longitudinal bending stress ($S_{LB}$) is:

$$S_{LB} = \pm\ 0.5ED/r$$

where

| | |
|---|---|
| $E$ | = elastic modulus of the pipe material (200 GPa for steel) |
| $D$ | = external diameter of the pipe |
| $r$ | = radius of curvature (of bend) |

To analyse the spanning lengths to which a pipeline may be subjected (between rollers during construction or unsupported sections due to scouring) the pipeline is considered as a continuous beam and the bending stress, according to WRc (1990), is given by:

$$S_{LB} = \pm\ 0.5.k\ [WL^2D]/I$$

where

| | |
|---|---|
| $k$ | = bending factor (0.08 to 0.125) |
| $W$ | = weight of the pipe per unit length |
| $L$ | = length of the span |
| $I$ | = second moment of area of the pipe = $(\pi/64)(D^4 - d_i^4)$ |
| $d_i$ | = internal diameter of the pipe |

***Internal and external pressure***. Large differences between external and internal hydrostatic pressures may result in buckling of a pipeline and the subsequent failure of the pipeline is referred to as 'collapse'. Air-filled thin-walled pipes are subjected to buckling when being installed in deep water; or for large diameter pipes with insufficient wall-thickness, buckling can occur in shallower water depths. Buckling can also occur during operation when the internal pressure falls below the external pressure, as a result of transient flow effects.

According to Gunnerson (1988), the physical properties of a steel pipe are characterised by its D/t ratio and the elastic limit is approached when the D/t ratio is approximately 250.

The compressive hoop stress ($S_H$) due to a net external pressure ($P_e$), according to WRc (1990), is:

$$S_H = - P_e D/2t$$

and the hoop tensile stress for a net internal pressure ($P_i$):

$$S_H = P_i D/2t$$

where
D = external diameter of the pipe
t = wall thickness

For a concrete coated pipe (proved compressive strength of the concrete), the compressive hoop stress ($S_H$) can be calculated as follows (WRc, 1990):

$$S_H = - 0.5.P_e D_c E/[2(E_c t_c) + E.t]$$

where

$D_c$ = overall diameter of the concrete coating
$E_c$ = Elastic modulus for concrete (22 to 29 GPa)
$T_c$ = thickness of the concrete coating

As the tensile strength of the concrete is insignificant, the concrete coating will not have an effect for a net internal pressure ($P_i$).

An external pressure will result in a circumferential membrane stress, which, when a pipe is not perfectly round, will generate additional stresses with increasing out-of-roundness. Additional out-of-roundness can be caused by the pipe weight or by the pressure from backfill loading. Refer to WRc (1990) for the calculation of increased hoop stresses resulting from the out-of-roundness of the pipe.

***External hydrodynamic loading***. During construction, a pipeline will be subjected to additional loads and stresses due to the wave and current forces. Depending on the sea and weather conditions which may occur during construction, careful consideration must be given to the construction method to be used.

Other external loadings for a well-designed outfall are generally small. For the detailed design, all possible loadings should be checked.

***Other loads***. Considering the structural design of an ocean outfall with regard to forces encountered during the installation, additional loads, such as from backfill weights or increased temperature when in operation will be relatively small. However, depending on the pipeline material, wastewater characteristics, etc. all possible additional loadings should be checked.

### *vi. Allowable stresses*

If the design is based on allowable stress, WRc (1990) proposed the calculation of the equivalent stress ($S_{eq}$), using the von Mises definition for equivalent stress, defined by Norske Veritas (1981) as:

$$S_{eq} = [S_L^2 + S_H^2 - S_L.S_H]^{1/2}$$

For steel pipes, the maximum allowable stress is equal to the yield stress, applying a factor of 0.72 (WRc, 1990), to allow for stresses which occur during the installation of the pipeline.

Longitudinal elastic buckling of a pipe under axial compression will occur when the longitudinal stress ($S_{Lc}$) is equal to (WRc, 1990):

For outfalls with composite pipeline sections (e.g. a steel pipe with internal and external coatings and

$S_{Lc}$ = 1.E.t/[D(3(1-μ2))$^{1/2}$]

where
T = pipe wall thickness
μ = Poissons ratio (0.32 for steel)
E = Elastic modulus for steel (~ 200 GPa)

Elastic buckling will occur when the hoop compressive stress ($S_{Hc}$) is:

$S_{Hc}$ = E.t$^2$/[D$^2$(1 – μ$^2$)]

concrete protection), care must be taken regarding the possible behaviour of each component. Components such as the concrete protection, may also contribute to the overall strength of the outfall, therefore the specifications for the protection are important to ensure that the characteristics will not change during the lifetime of the outfall.

### *vii. Accidental damages*

***During construction***. During construction, the risk of damage to an outfall is high as at this time when the unprotected outfall is exposed to weather/sea conditions as well as floating construction equipment.

Careful design (selection of the most suitable materials for the construction method that is appropriate for the outfall site and environmental conditions) and meticulous planning, scheduling and contingency measures are of the utmost importance. The engineers responsible for the Tender Specifications must ensure that all possible precautions with regard to the installation and protection of the pipeline are specified in detail.

Ship anchors and fishing gear. A pipeline can be damaged by dragging or dropping ship anchors. A pipeline without protection (or sunk into the seabed) is extremely vulnerable to dragging anchors.

Referring to Hoshina and Featherstone (2001), the potential anchor/fishing gear penetration depth into the seabed for different craft/equipment is shown below:

| | ***HARD MATERIAL (clay > 72 kPa and rock)*** | ***SOFT TO FIRM MATERIAL (sand, gravel, clay 18 – 72 kPa)*** | ***SOFT MATERIAL (mud, silt, clay 2 – 18 kPa)*** |
|---|---|---|---|
| *Stow net fishing anchors* | *N/A* | *2 m* | *> 2 m* |
| *Ships' anchors up to 10 000 DWT (50% of world fleet)* | *< 1.5 m* | *2.1 m* | *7.3 m* |
| *Ships' anchors up to 100 000 DWT (95% of world fleet)* | *< 2.2 m* | *2.9 m* | *9.2 m* |

It is not always feasible to demarcate the area along the route of a pipeline that is close to navigational routes near a harbour.

Protection against dragging anchors can be accomplished by an appropriate backfill. A backfill comprising 1 m of 10 kg to 60 kg rock, followed by 1 m of 60 kg to 300 kg rock armour, with appropriate slopes, will be sufficient protection against most types of anchors with a weight greater than 12 tons. This is only required in a water depth of less than 12 m water as ships with anchors greater than 12 tons will not normally navigate in shallower water.

The possible damage to a diffuser from the impact forces exerted by a dropping anchor can be eliminated by rock protection or pre-cast concrete domes.

Damage resulting from direct impact from a ship is also a possibility in shallower waters. If there is the probability of a direct hit from a ship for a specific outfall site, appropriate protection must be applied to the pipeline.

If an outfall site is within a fish trawling area, precautions must be taken, not only for the protection of the pipeline, but also against possible claims for damage to fishing gear.

### 6.9.4 Contractual aspects

The calculations in this section provide the baseline information for the actual specifications and detailed plans for cost estimating, tendering and the construction of the pipeline. Standard methods and procedures are normally used for all civil engineering contracts and typically include the following documents:

- Instructions for tenders
- Standard forms of tender, agreement and bond
- General and special conditions of contract
- Detailed design
- Specifications
- Bill of quantities
- Schedule of rates and day works
- Appendices (all other relevant information).

Because of the more hostile nature of the environment of an outfall construction area, the specialised equipment utilised and the complicated methodologies to be used, more detailed and specific information and provisions are required, compared to straightforward on-land civil construction. Many offshore contracts have had a less than satisfactory financial ending for both contractor and client, resulting from disputes over delays caused by unexpected physical and environmental conditions, lack of backup in the event of breakdown of specialised equipment, vague and divided responsibilities, etc. All of these problems could have been prevented by additional clauses in the contract documents, taking into account the unique conditions and circumstances of this type of project.

The following notes are not prescriptive. It merely highlights key points to be taken into account when detailed specifications as provided as part of the preparation of contract documents to ensure the success of this type of construction:

#### *i. General*

Due to the specialised nature of offshore engineering works, a contractual split between the onshore and offshore works is recommended. Depending on the pipe material to be used, the supply and testing during fabrication of the pipe material can also be a separate contract.

Special conditions of contract should address the following for the offshore construction:

- Environmental
- Risks.

### *ii. Specifications*

***General.*** In general, the offshore works comprise the following components:

- Manufacturing and supply of pipe and diffuser material, including protection and weighting according to the proposed method of construction
- Trenching (surf zone and offshore)
- Launching of the sea outfall, including the diffuser section
- Protection of the pipeline (filter material and rock armour protection)
- Testing of the entire system and rehabilitation according to the defined environmental guidelines.

Apart from all other standard items, the following need to be specified in detail:

- Availability of all marine surveys (bathymetry, physiography, etc.)
- Contractor's allowance for appropriate marine transport and related equipment for inspections and control by the engineer
- Exchange of information between contractor and engineer
- Temporary site facilities and transport, taking into consideration the sensitivity of the marine environment and subsequent rehabilitation after completion of the contract
- The Contractor's commitment to take all reasonable steps to minimise adverse effects on the environment. Submission of an environmental plan for prevention, as well as for mitigating options for all aspects of the construction of the pipeline
- Pollution restrictions, regulations and reporting procedures.

***Materials and workmanship***. On account of the inaccessibility to the outfall after installation, it is of the utmost importance that all material and fittings, including all aspects of welding, pipe coating and protection, shall conform to standards (if local standards are not sufficient, then to recognised international standards).

Imported backfill material, including material for the bedding of the pipeline, protection of the pipe against rock armour and the rock armour itself, shall be quarried hard stone, predominantly free of intrusions and unweathered. Rock and stone shall be clean, sound, durable, free from earth or other soft or decomposed or injurious materials and shall show no cracks or fissures caused by the initial processes of decomposition, and shall not break down in the seawater. Generally, dredged material will not be allowed to be used as a substitute for imported fill material. This substitute will be approved only after the examination of representative samples of the dredged material for each fill load.

***Marine operations***. The contractor must comply with all requirements of, and maintain liaison with, the relevant Port Authority and must ensure that all statutory requirements are observed in connection with *Notice to Mariners*.

The contractor is liable for the payment of all dock fees, anchorage charges, pilotage fees, etc. related to all materials and plant/equipment to be utilised. The contractor must provide, maintain and eventually remove temporary moorings for construction craft, according to regulations.

The contractor must provide all navigational signs (lights and marker buoys) required by the Port Authorities, or national and international maritime legislation.

The contractor will be responsible for any damage to his/her own or third party craft and to injuries to any person that may occur during the contract.

The contractor, in cooperation with the engineer, must liaise with organisations/persons which/who may have fishing interests in the area and arrange for temporary agreements during the period of construction.

All operations, both ashore and afloat, must be conducted in accordance with the existing statutory rules and regulations or such as may be issued from time to time. The contractor must list, in a schedule, all the construction plant material he/she proposes to use, complete with relevant rates. Vessels to be used as diving platforms must conform to all national and local diving regulations and provisions for emergencies in terms of standby craft. All vessels must at all times be fully equipped in accordance with the latest safety, fire-fighting and emergency equipment requirements as laid down by the relevant authorities for vessels operating on survey and diving work, or as may be applicable.

The contractor must provide survey vessels with specified equipment for echo-sounding, underwater surveys (photographic and video), as well as transport and communications for the engineer's staff, for the entire duration of the project.

The contractor must determine the location of underwater hazards (structures, cables or wrecks), take the required precautions to avoid these and liaise with the owners responsible for such items.

The contractor is responsible for removing any plant material (floating or sunken) belonging to him/her or a sub-contractor. He/she is responsible for the prevention of and mitigation measures relating to pollution from any substance which may leak from any craft (afloat or sunken). He/she is liable to all costs with regard to this and are responsible for liaison with the relevant authorities.

The contractor, in cooperation with the engineer, must comply and adhere to environmental practices with reference to international as well as national policies as far as it is practical. Special care must be taken during dredging operations during which elevated concentrations of suspended fine sediments will occur. Mitigation measures must be available, for example, sediment curtains. Monitoring must be conducted in accordance with the construction monitoring programme, which forms part of the contract specifications.

Provision also needs to be made for tide and wind recorders to be installed in close proximity to the site, if they are not already available.

All diving operations must be carried out in accordance with official regulations. Prior to underwater operations, the contractor must submit a copy of the diving rules, a general method statement (or standard operational procedures), a safety policy, etc. For all marine works, during which diving operations may be required, an easily accessible, fully maintained decompression chamber must be provided. During construction, qualified divers provided by the engineer need to carry out routine underwater inspections in accordance with the relevant regulations and the contractor's procedures. The contractor will be responsible for providing equipment and diving supervision and will control the diving operations related to inspections.

During the contract period, an appropriate closed-circuit television system with operators should be maintained, as well as equipment for still underwater photos.

The sea conditions may vary from the predicted data and estimates obtained from the field survey; allowance must be made for such variations. The contractor will not be entitled to any payment, compensation or allowance for extra expense or loss which may be due to variations, inaccuracies or omissions in the data interpretation report of the field surveys.

***Trenching and dredging operations*.** The 'ownership' of the dredged material and the dumping and handling of dredged material must be stated and defined, taking into account all relevant regulations and legislation and authorities that have to be informed.

If the average Point Load Strength Index of the seabed material exceeds 65 MPa, blasting may be required and all necessary arrangements and protocols required for blasting shall be made by the Contractor with the relevant authorities.

Equipment and the degree of accuracy for position fixing, echo sounding and line control (laser) and tide recording must be specified. A continuous fall towards the diffuser end is a basic requirement.

### *iii. Bill of Quantities*

For marine works, the following are important issues to address in the Preamble to the Bill of Quantities in order to reduce the risk of lengthy and costly disputes:

- With regard to day work rates, the rates for demurrage and charter for the marine plant must include all costs associated with the specific marine operation (personnel, fuels, consumables, etc.) and equipment. The charter rate is for productive hours only. Demurrage is compensation paid to the Contractor for loss of working time caused by naval operations or a suspension of work mutually agreed by the Consultant and Contractor for reasons of safety of staff or plant. Demurrage does not include delays resulting from weather conditions or delays not mutually agreed upon.

- Measurement of dredging of the trench must be specified (e.g. linear meters between chainages) and must be based on the profiles in the Drawings supplied with the Tender Documents. Costs for a pre-dredge survey must be included to adjust for differences between the supplied profile and the actual profile. Inclusive rates must be clearly stated (such as soundings, record keeping, removal of debris, conveying and depositing materials, providing buoys, markers and notices and removing silt deposits). For hard material (strength defined), allowance must be made as a separate item (Provisional Sum) and the measurement must be per $m^3$.

- The rates for the provision and assembly of the outfall pipeline and diffuser section must include testing of strings and diffuser and the installed outfall pipe, supply of all consumables and equipment, and must also apply to the rates for linings and coatings of the pipe.

### 6.9.5 Decommissioning

A decommission plan is required to prevent or minimise potentially negative impacts on the environment if it is decided to end the discharge practice after the lifetime of the project or for any other reason. Such planning is required as early as the construction consideration and structural design phase to ensure that decommissioning costs of different construction options are evaluated.

Decommissioning options should describe the plan, management and implementation of the decommissioning process and should, for example, address the following:

- Physical and ecological impact of removing the structure or partially dismantling the structure, including any long-term impact should the structure be abandoned (i.e. not removed)

- Technical aspects and the feasibility thereof (e.g. methods to remove the structure or parts of the structure and the re-using, recycling or disposal thereof)

- Monitoring programme after decommissioning to ensure that any remaining structure does not adversely affect the marine environment or other uses, as well as to monitor the ecological rehabilitation of any impacted area

- Procedures for removing large offshore structures are typically dangerous, lengthy and costly. Therefore, when the construction method is selected, decommissioning aspects must be taken into account. Although the initial cost for a buried pipeline is higher than for an exposed pipeline, removal of most of the offshore section will not be necessary when the structure is decommissioned. Moreover, the removal of a structure could result in a greater environmental impact than leaving it in place.

- Envisaged costs for the different options, considering labour, equipment and any other resource requirements.

Approval to proceed with the recommended decommissioning option will be reviewed and subjected to an EIA (a comparative assessment of all options and the rationale for decommissioning of each component) at such time, taking into account the actual circumstances (environment, costs as well as social aspects) at that stage (prior to decommissioning), and approval will only be given at that stage.

### 6.9.6 Specific requirements for pre-assessment and detailed investigation

Structural design and construction considerations are typically addressed as part of a detailed investigation. However, as part of a pre-assessment certain, aspects which may influence the route and location of the pipeline have to be taken into account in the assessment, such as:

- Availability of pipeline materials to construct the pipeline in a certain area
- Potential construction constraints (e.g. availability of construction area on land, offshore restrictions).