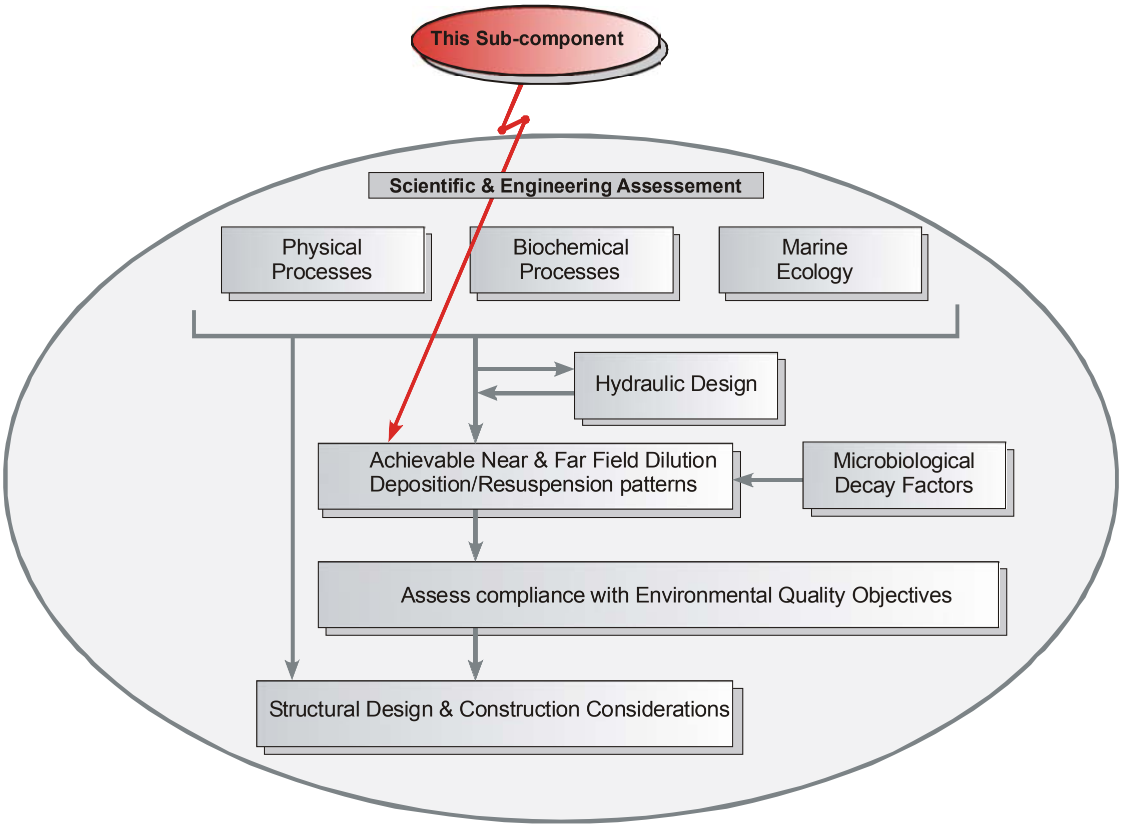
6.6 ACHIEVABLE DILUTION



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.

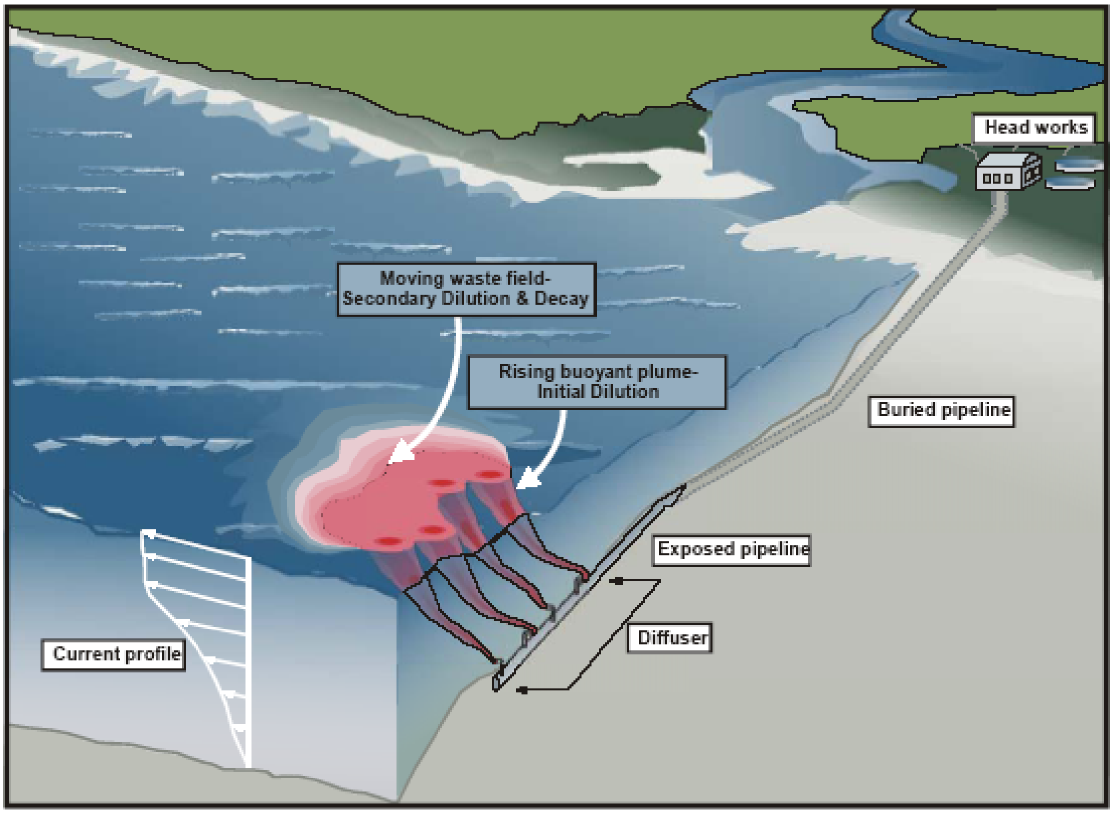


FIGURE 6.36: ***Illustration of the different dilution components making up the total achievable dilution for an offshore marine outfall***

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.

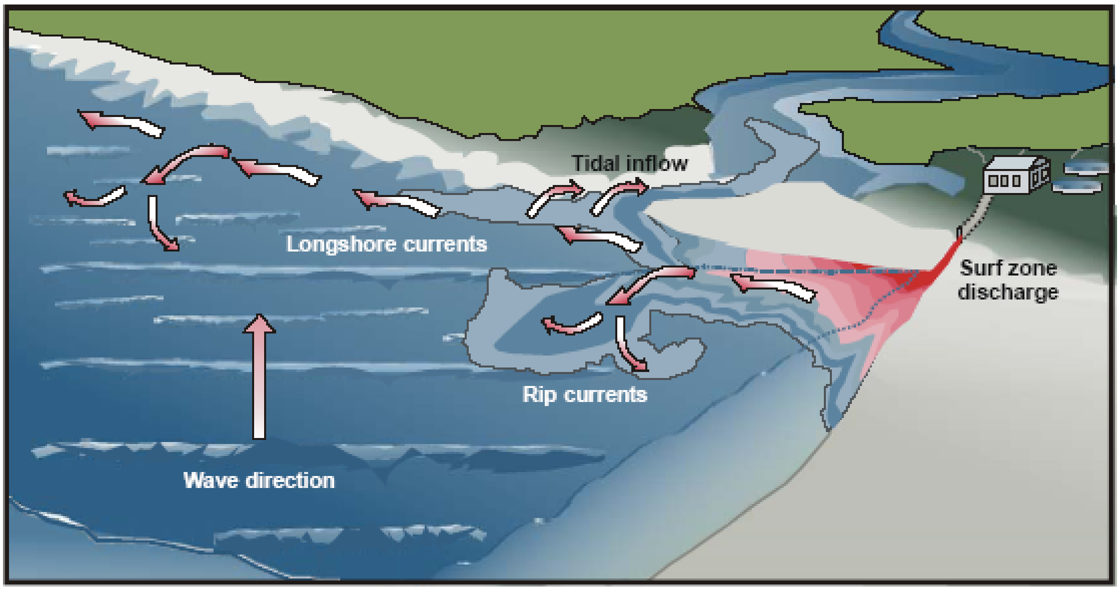


FIGURE 6.37: Illustration of the total dilution process for a surf zone discharge

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.

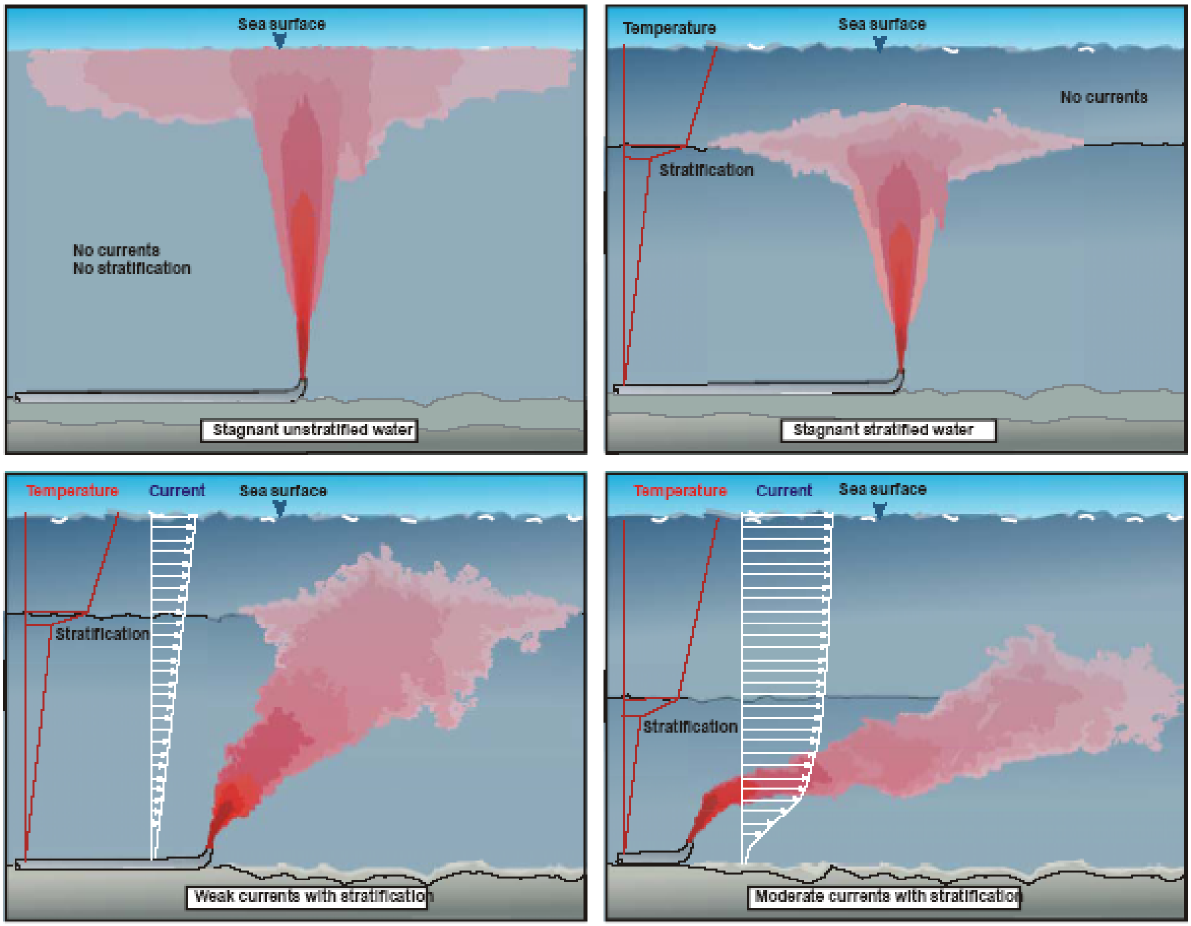


***FIGURE 6.38:** Buoyant plume in (a) stagnant, unstratified water, (b) stagnant, stratified water, (c) moving (weak currents) stratified water and (d) in moving (moderate currents), stratified water*

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)
$\Delta\rho$	=	$\rho_s - \rho_e$
ρ_s	=	seawater density (kg/m^3)
ρ_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 ($S_{av} = 1.74 S_{min}$)

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):

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

$S_{av} = 278$

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)
ρ_{sb}	= density of the seawater at the seabed (kg/m^3)	
ρ_{ss}	= density of the seawater at the sea surface (kg/m^3)	
g_1	$= g[(\rho_a-\rho_e)/\rho_a]$	(relative density parameter)
ρ_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 (ρ_{ss}) : 1025 kg/m^3
Seawater density at the bottom (ρ_{sb}) : 1026 kg/m^3

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

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:

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

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:

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

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 α-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 α-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 α-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

$\beta = 12K_o/u_ab$

$\beta = 12K_ot/xb$

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

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

b = (b + 2H)Sinθ

θ = angle between the wave and the diffuser

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

x = distance from the diffuser (m)

Determine a dimensionless distance (T)

$T = \beta.x/b$

$T = 12K_ox/u_ab^2$

Note: There are doubts about the accuracy when applying this theory to the direct onshore case (θ = 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.

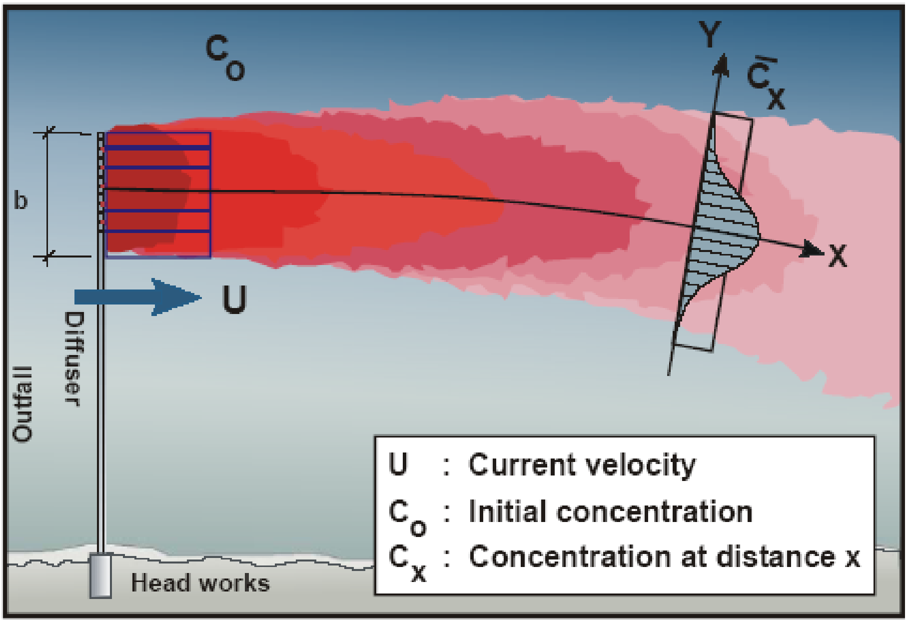


FIGURE 6.39: An illustration of Brooks's surface plume model

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:

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

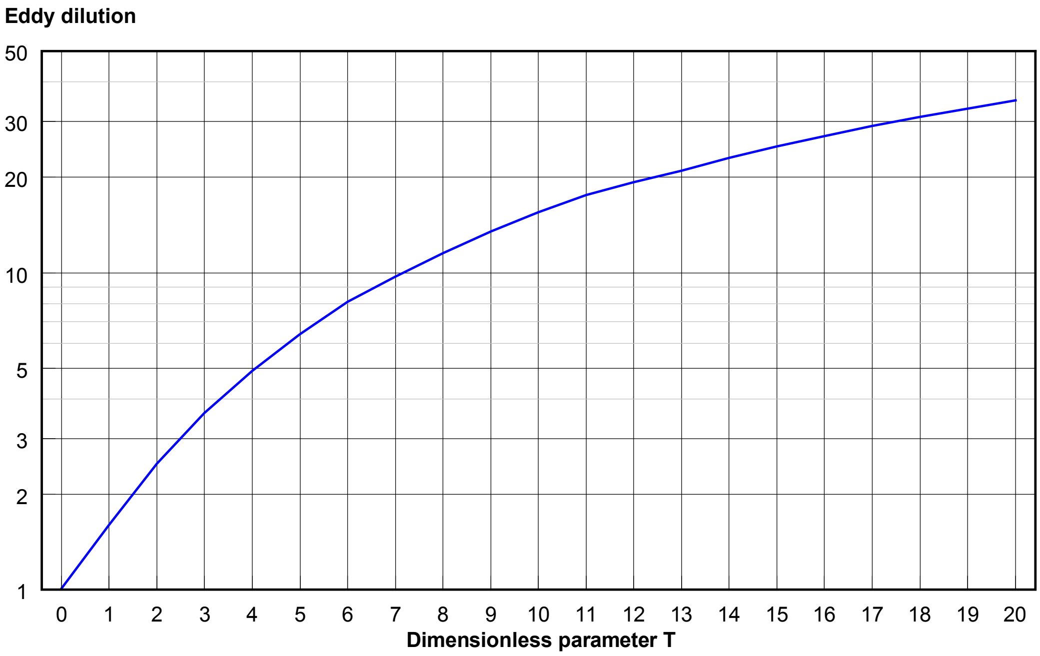


FIGURE 6.40: Graph from which to calculate secondary dilution using the dimensional parameter (T)

EXAMPLE…

Diffuser length (b):	238 m (no. of ports x port spacing)
α-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

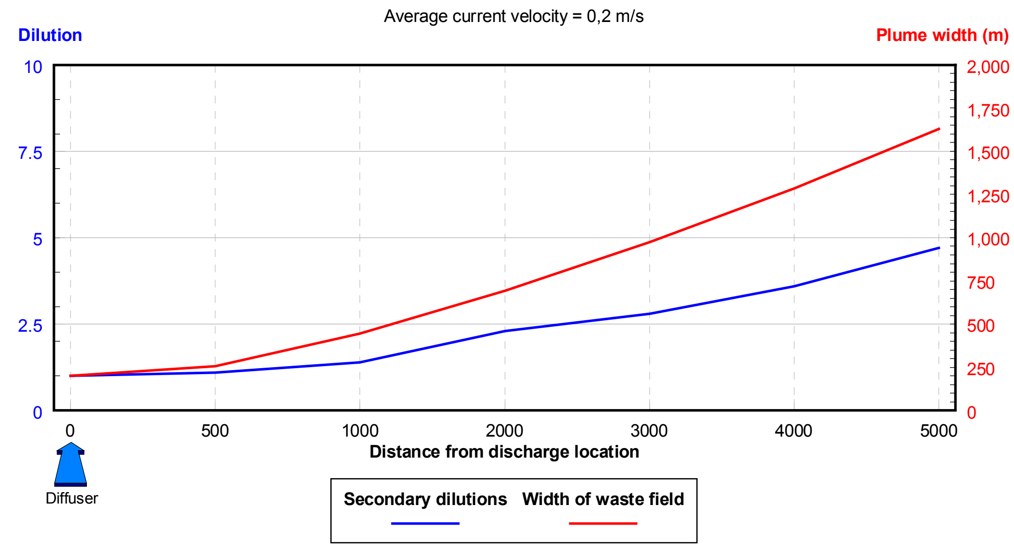


FIGURE 6.41: Secondary dilutions and plume widths versus distance

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.

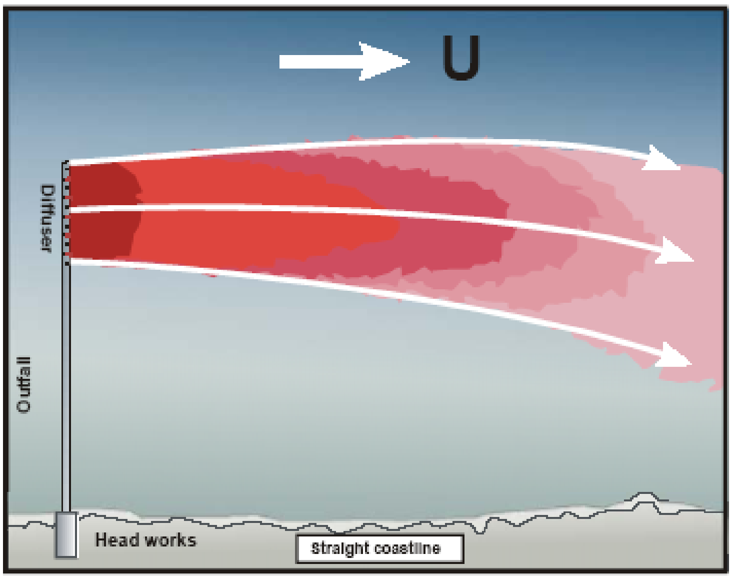


FIGURE 6.42: Illustration of a waste field along a straight coastline using Brooks's surface plume model

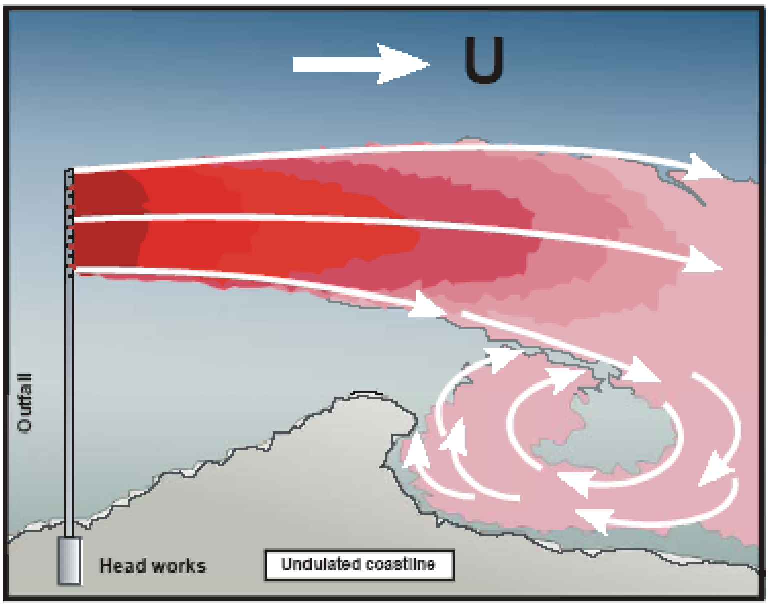


FIGURE 6.43: Illustration of a waste field along a coastline with promontories

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.

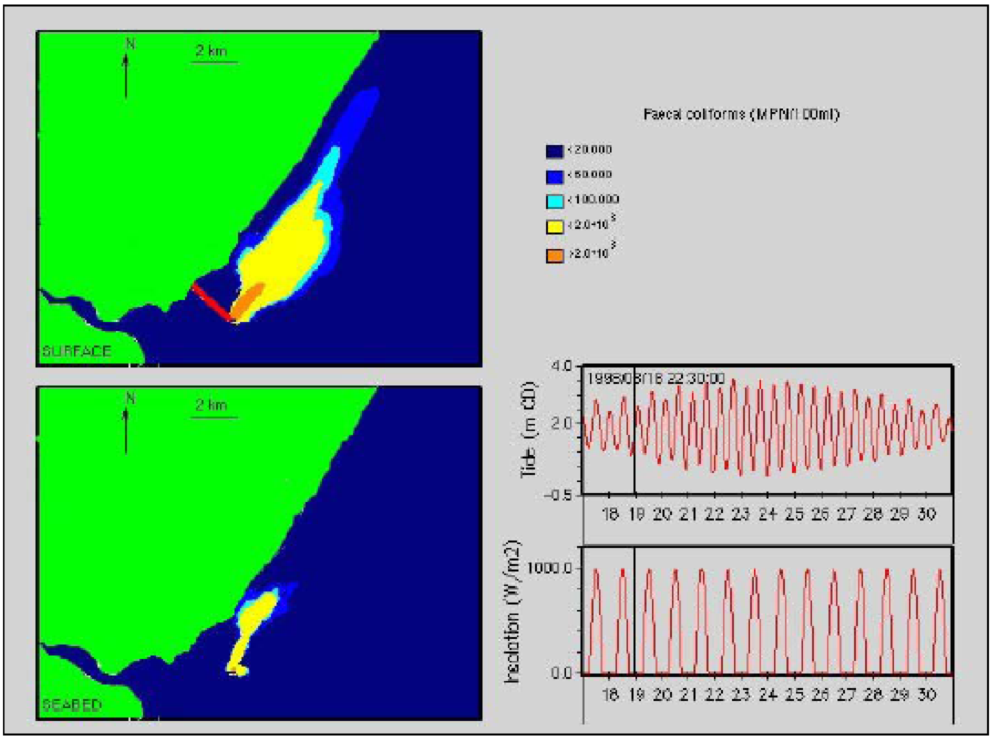


FIGURE 6.44: 3-dimensional simulation of an outfall plume

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
υ = kinematic coefficient of diffusion
u = velocity
R_s = change in concentration in time due to a source

<u>Initial mixing.</u> 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.

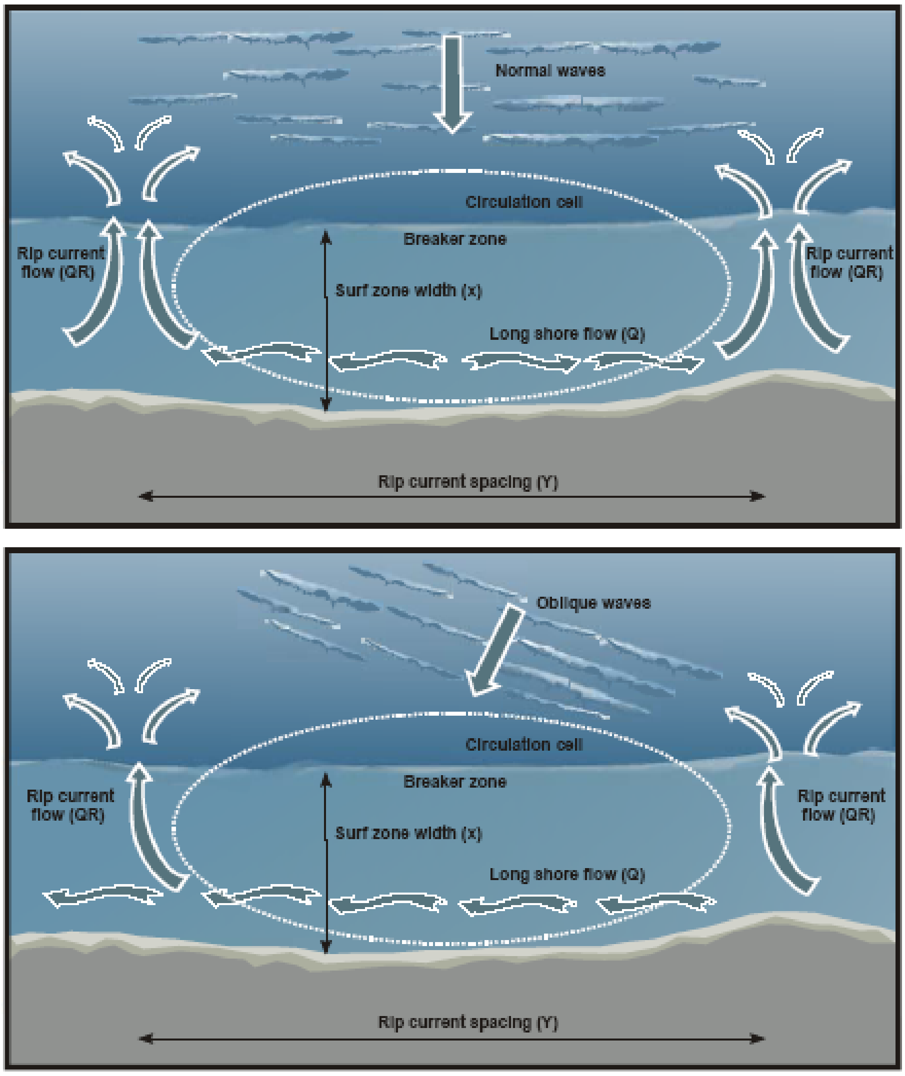


FIGURE 6.45: Illustration of the longshore transport and dispersion processes in the surf zone under normal and oblique wave direction

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).

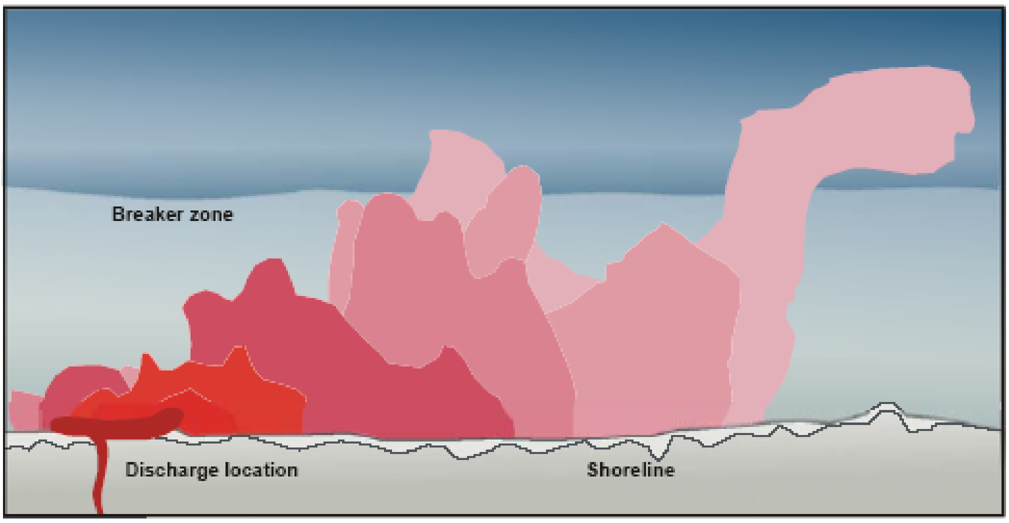


FIGURE 6.46: Example of a dye dispersion test during a continuous release to the surf zone in False Bay (Cape Town)

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.

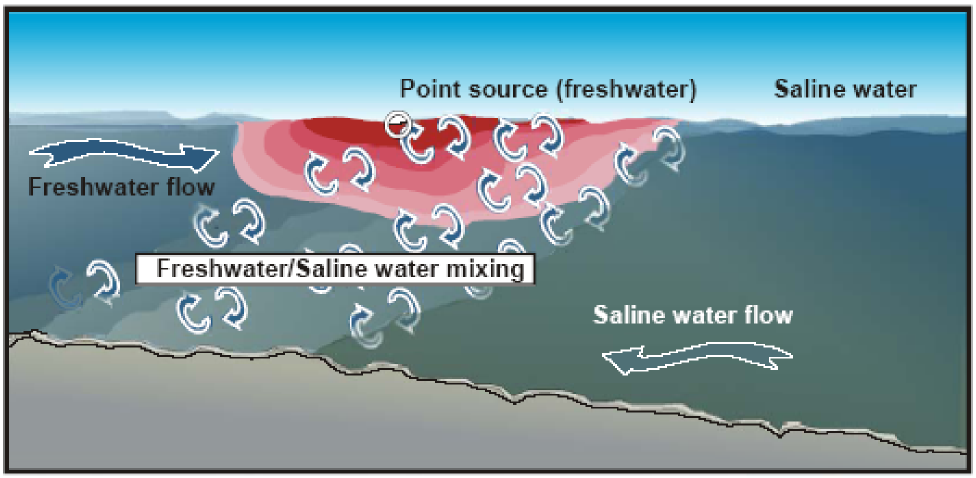


FIGURE 6.47: Illustration of the mixing of a wastewater plume in a partially stratified estuary

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 x 100 x 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 x 0.25 = 12 500 m^3 (250 ℓ/day per capita)

The increase in level (Δh) over 24 hours is:

Δh = 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.

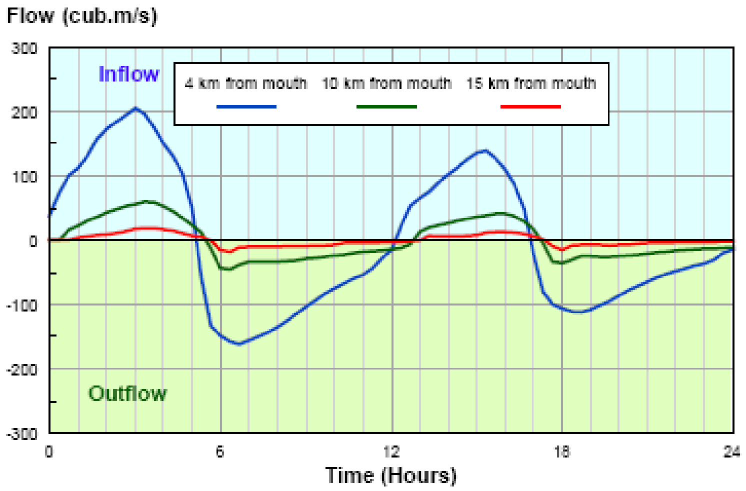


FIGURE 6.48: ***Difference in average flows at three locations in the Swartkops River Estuary (river inflow is zero)***

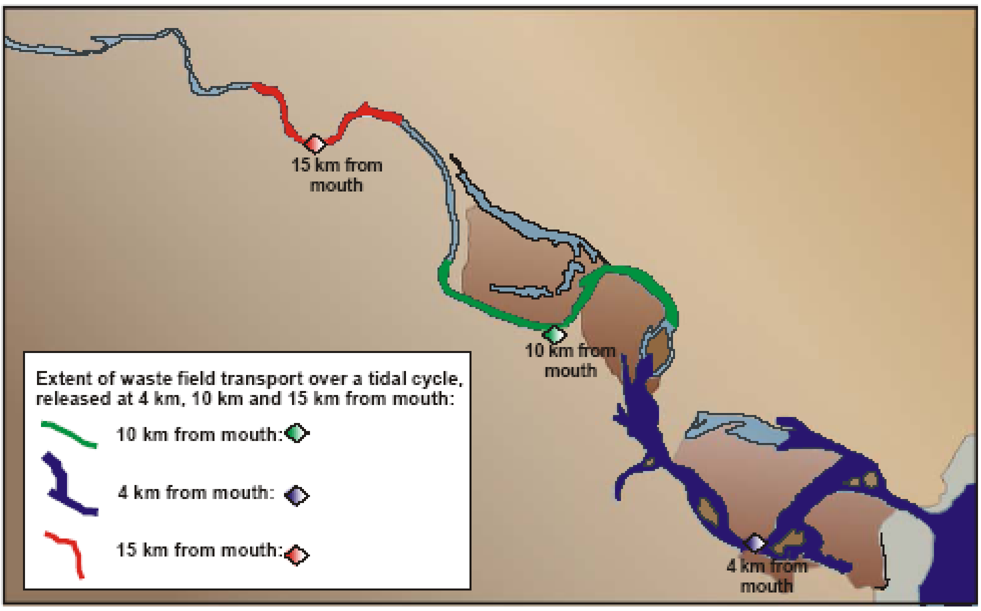


FIGURE 6.49: ***A simulation showing the difference in spatial behaviour of a wastewater plume released at different locations simulated in the Swartkops River Estuary***

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:

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

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.

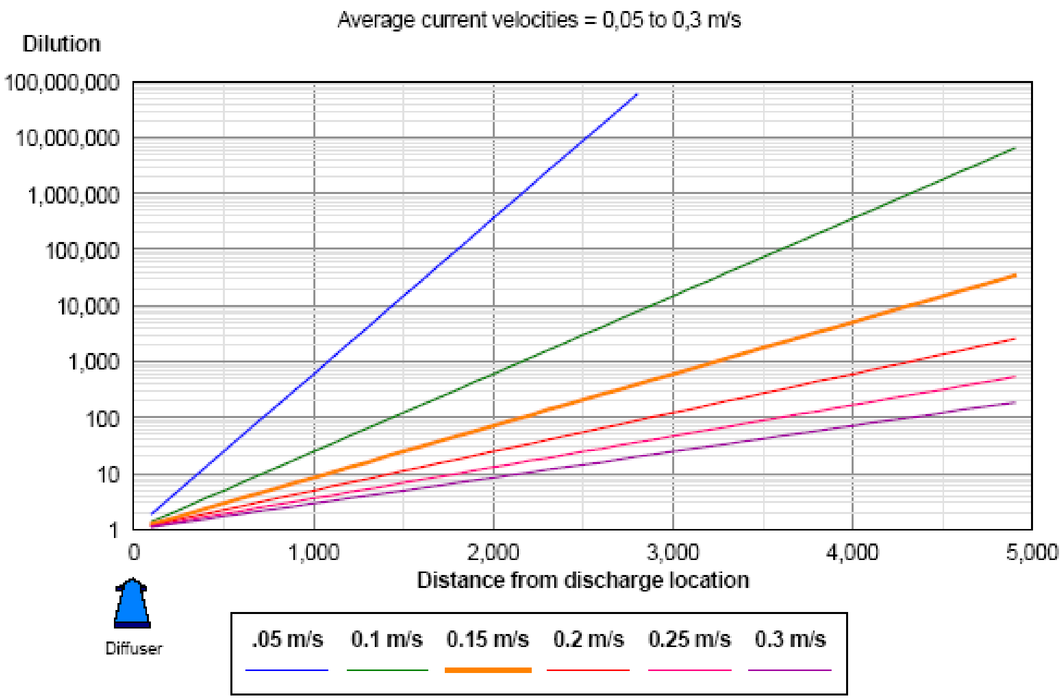


FIGURE 6.50: Dilutions due to decay versus distance under stronger current conditions. T_{90} = 2 hrs (daytime).

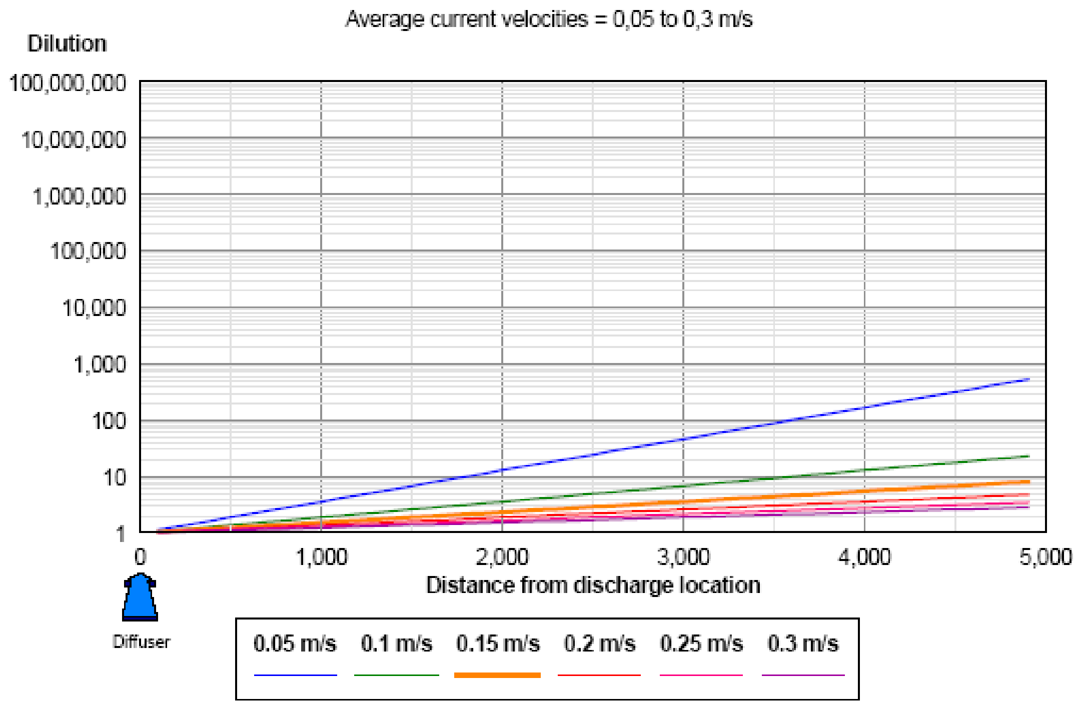


FIGURE 6.51 Dilutions due to decay versus distance under weaker current conditions. T_{90} = 2 hrs (daytime)

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

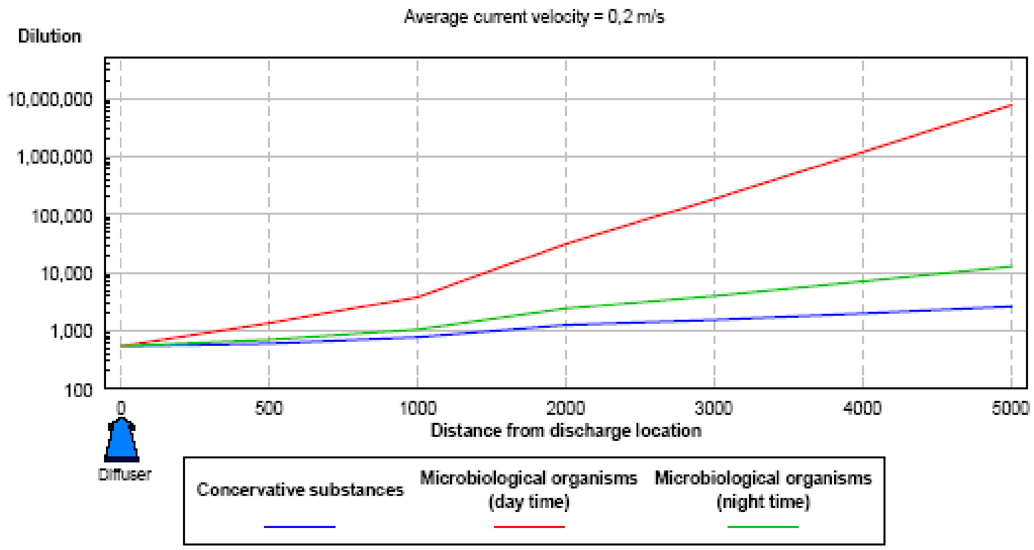


FIGURE 6.52: ***A graphical illustration of the achievable total dilutions versus distance***

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.

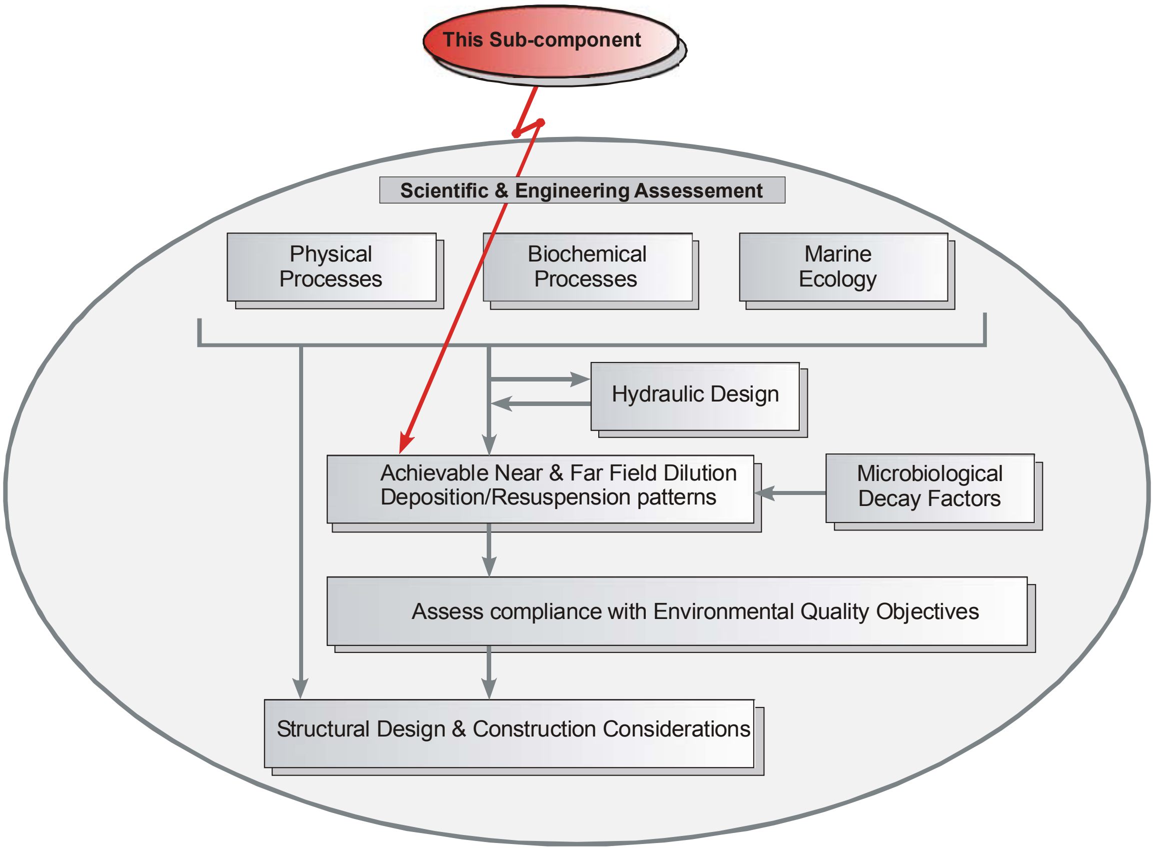
ii. Detailed investigation

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).

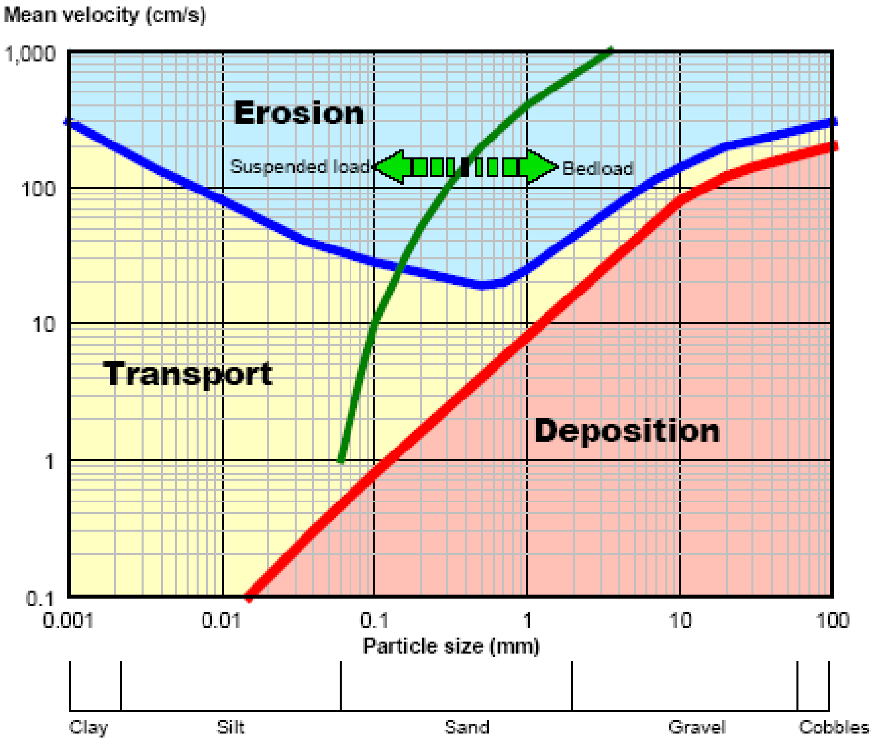


FIGURE 6.53: ***The effect of current velocities on the erosion, transport and deposition of sediments (Gunnerson, 1988)***

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.

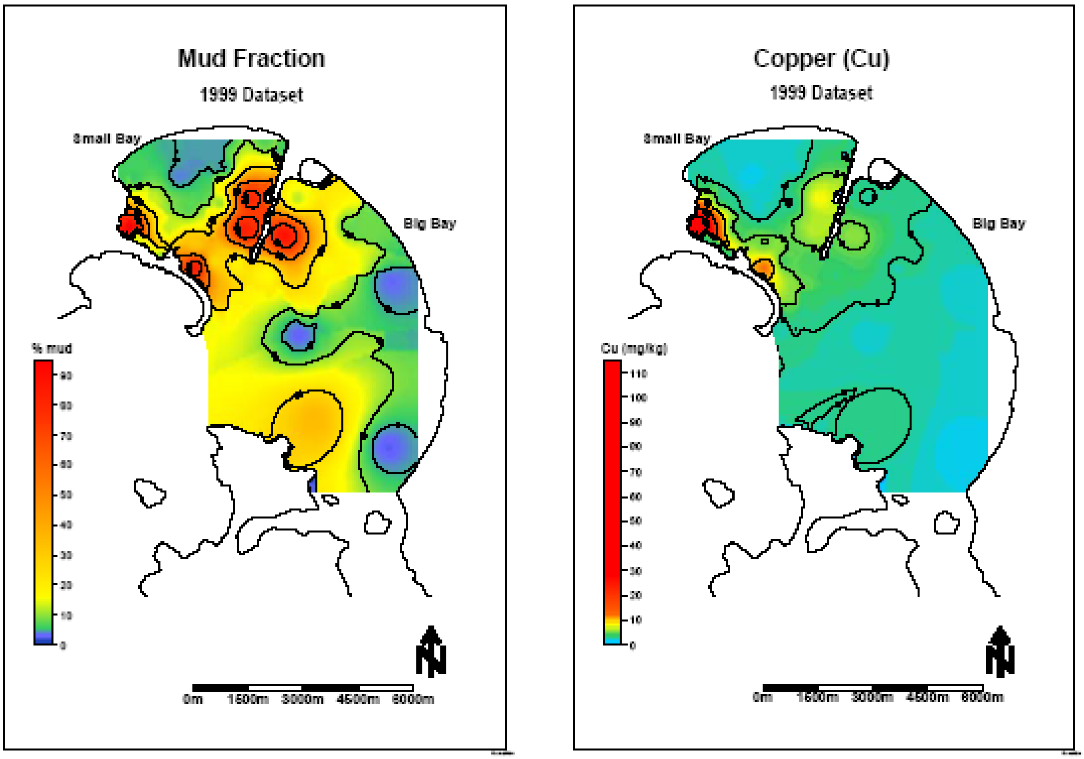


FIGURE 6.54: Plot of the distribution of copper and particle size in Saldanha Bay (South Africa), showing the strong correlation between depositional areas and trace metals deposition

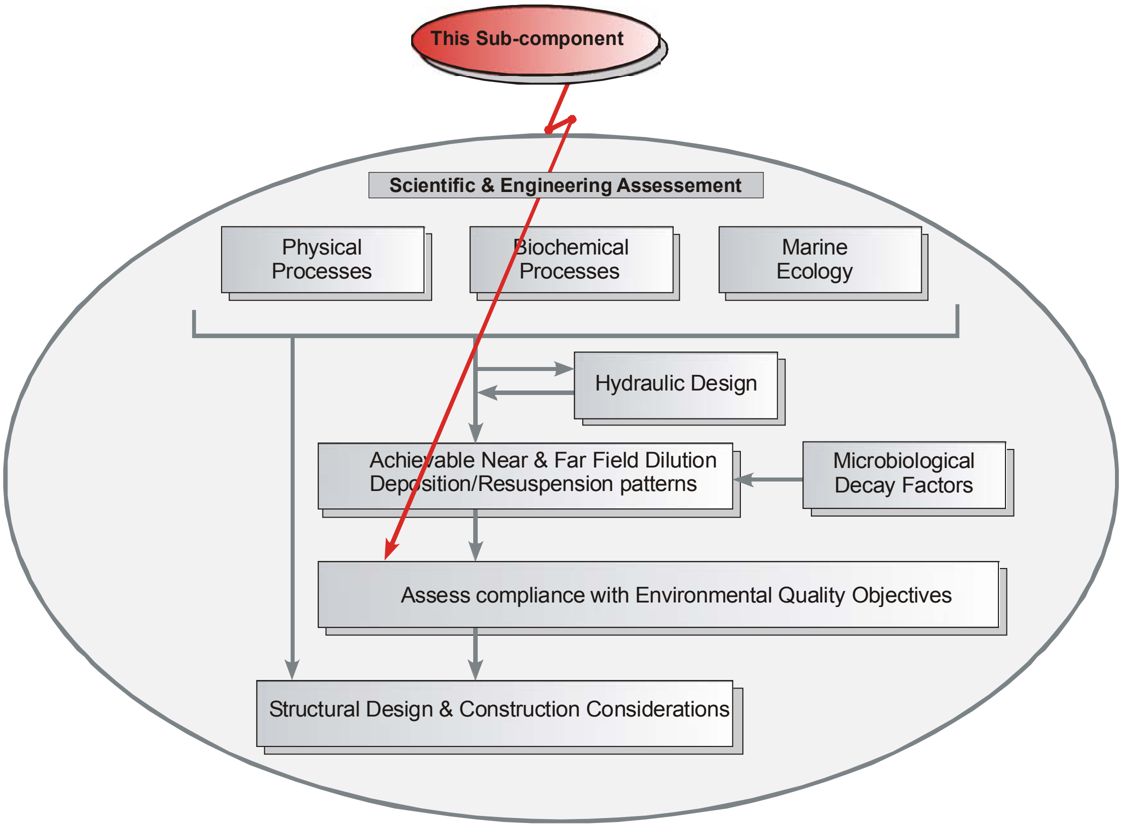
6.7.2 Data requirements for pre-assessment and detailed investigation

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.

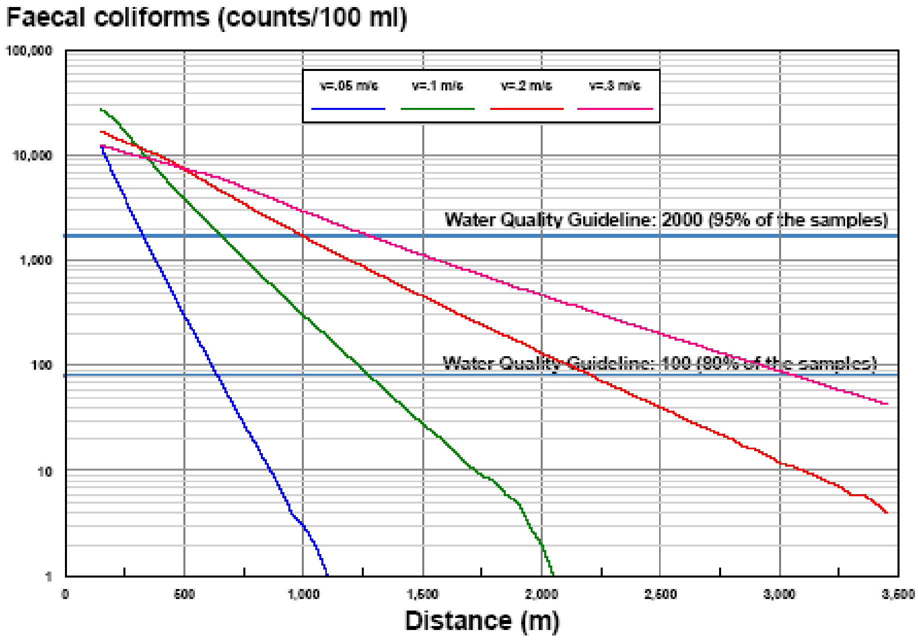


FIGURE 6.55: ***Example: Variation in*** *distance from the discharge location at which faecal coliform counts comply with environmental quality objectives (or water quality guidelines) under a range of current velocities*

- 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.

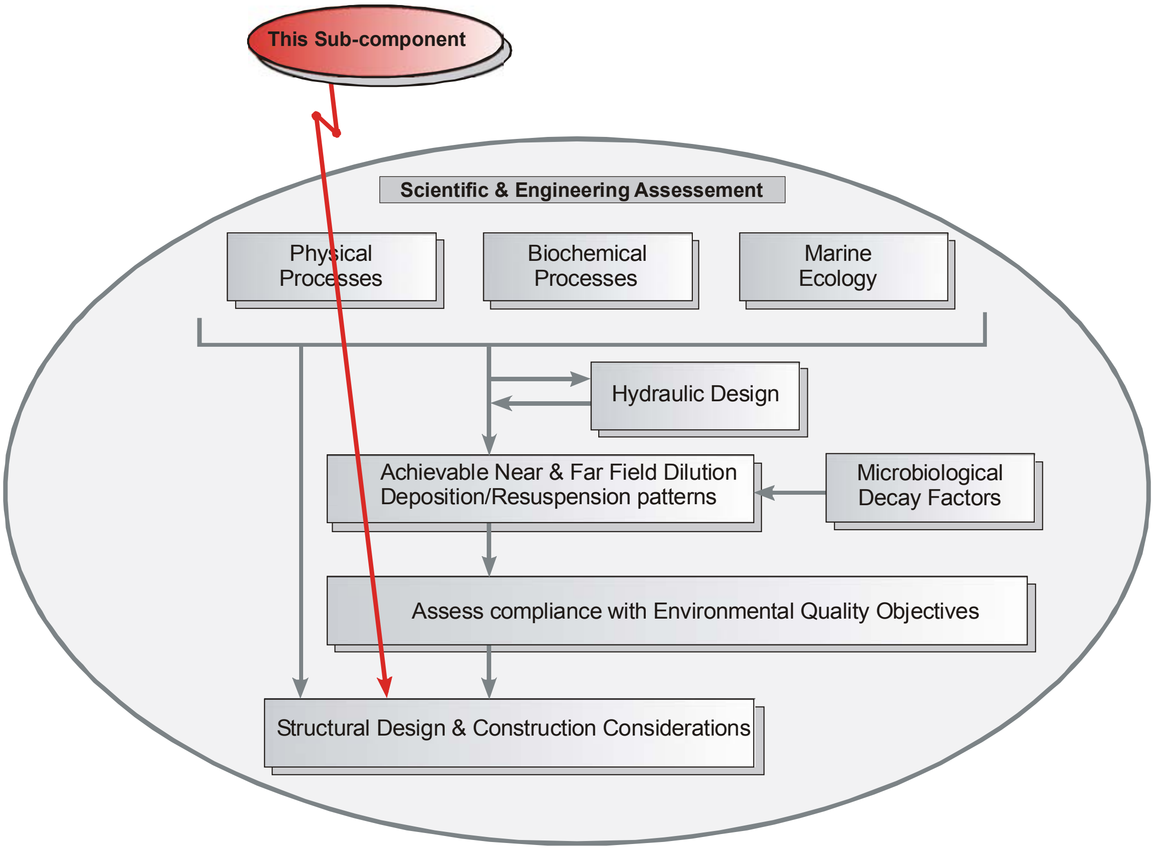
ii. Detailed investigation

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.

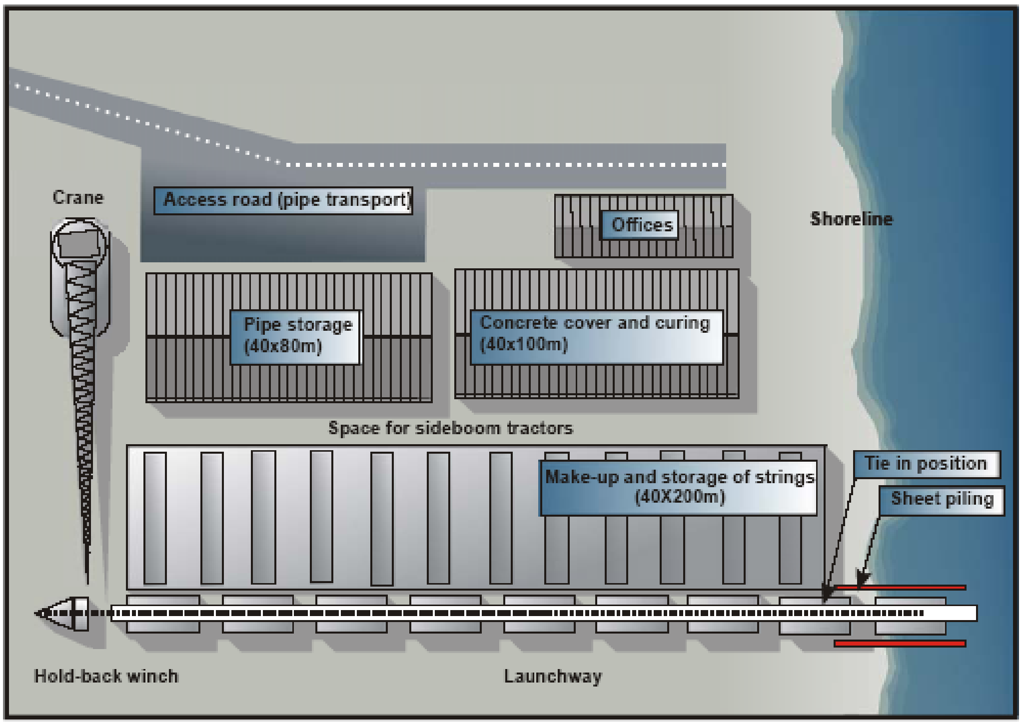


FIGURE 6.56: A typical construction-launching site for the bottom-pull method

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 <u>after settlement</u> of the material.

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

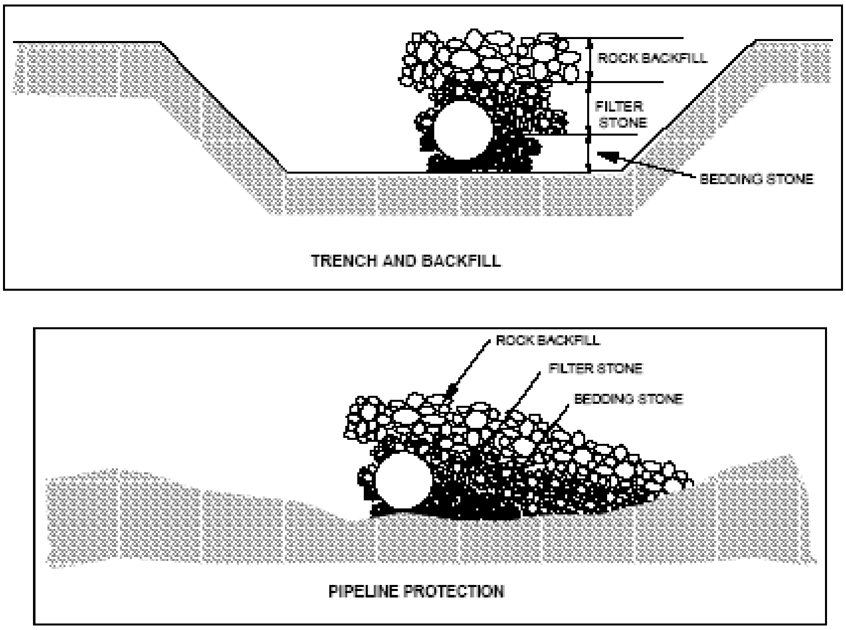


FIGURE 6.57: Examples of backfill cross-sections

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)
ρ	= 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.

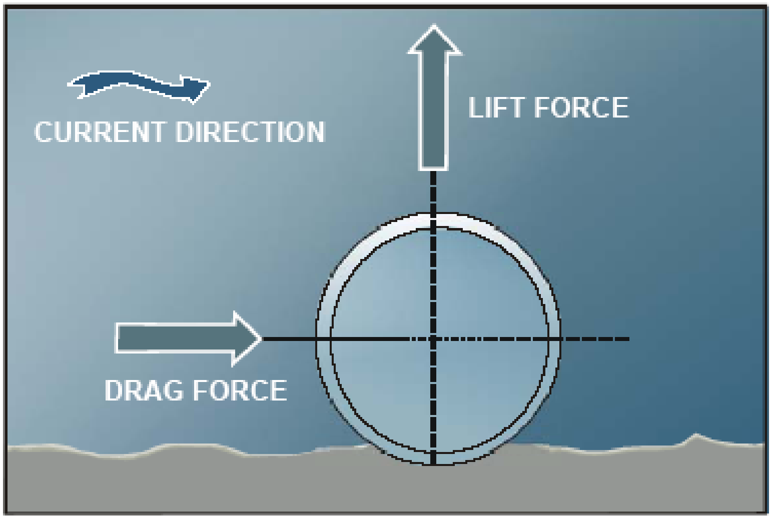


FIGURE 6.58: Drag and lift forces on a pipeline in steady flow

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/μ

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

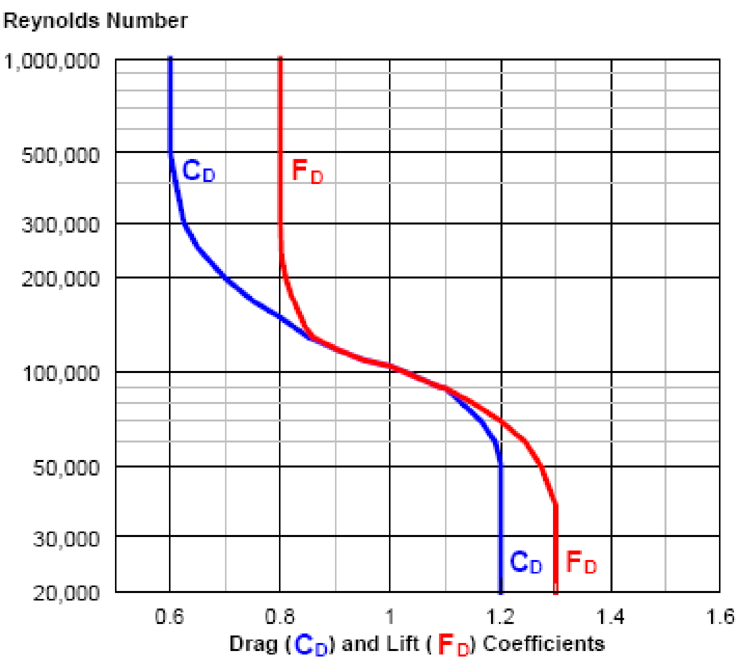


FIGURE 6.59: Drag and lift coefficients for an exposed pipeline on the seabed (Gunnerson, 1984)

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
ρ = 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).