

# Marine Outfalls

## 32. Marine Outfalls

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Marine outfalls are used to discharge treated liquid waste to the environment. Not all contaminants in liquid waste can be removed by treatment. A properly designed, constructed, and operated marine outfall effectively dilutes the discharged waste which then substantially reduces the concentration of contaminants in the wastewater. In turn, this reduces the risk to biota and human users of the marine environment. An introduction to some of the main aspects of marine outfalls is provided.

Five areas are covered, commencing with the main influences associated with the decision to build a marine outfall. Included is an overview of the wastewater treatment process. Near-field numerical modeling is described and it is demonstrated how this tool can be used to assist with the design of a marine outfall. Outfall hydraulics is discussed, detailing a range of features including head losses, manifolds (or diffusers), seawater intrusion, and air entrainment. A very brief summary of the construction of a marine outfall is provided. The final area covered describes environmental monitoring that should be undertaken to confirm the putative impacts associated with a marine outfall.

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Work presented in this chapter concentrates on the discharge of wastewater to the environment through marine outfalls. Marine structures are required for intakes for drinking water (e.g., desalination plants) and water for industrial or commercial use (e.g., flushing of toilets). The focus here is on marine outfalls; marine intakes are not considered further.

The objective here is to provide practitioners with an overview of the fundamentals of marine outfalls and to

outline some initial considerations to help those new to the subject area. Understandably, the present chapter does not cover all areas in detail; the focus is on the design and monitoring aspects of marine outfalls. Information on some of the problems drawn from experience with marine outfalls is provided and reference material with additional detail is identified.

## 32.1 Terminology

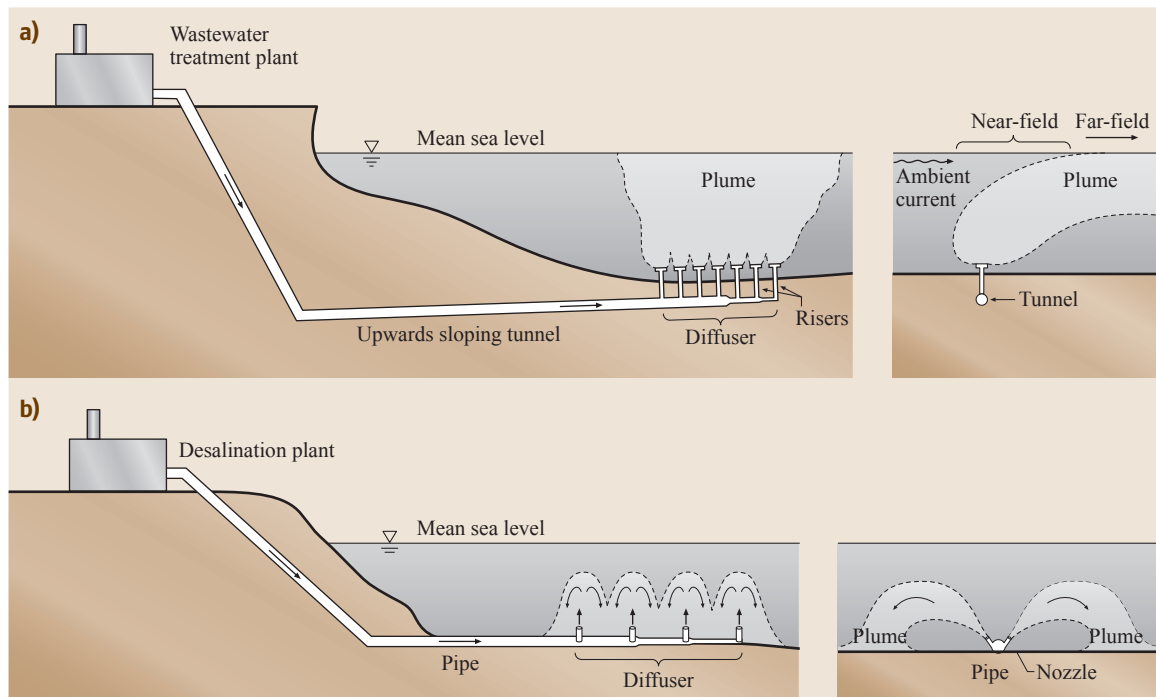
The following terminology is used throughout this chapter. Sewage is the raw input to a municipal wastewater treatment plant, the product of which is effluent. Seawater is the raw input to a desalination plant, the product of which is brine. Wastewater refers to either effluent or brine. An outfall refers to the disposal system from the treatment plant to the discharge outlets. The manifold is that part of the outfall from which there is a series of offtakes, termed risers. A diffuser comprises that section of the outfall which includes the manifold and the risers.

Two generalized marine outfalls are shown in Fig. 32.1. Figure 32.1a shows the discharge of positively buoyant effluent from a wastewater treatment plant and Fig. 32.1b shows the discharge of negatively buoyant brine from a desalination plant. Both schematics show an inclined tunnel from the wastewater treatment plant, the outfall tunnel, a diffuser comprising several risers, and the wastewater plume emanating from outlet nozzles on the top of each riser. The outfall tunnel is inclined upward to ensure any air trapped in the declined tunnel exits through the outlet nozzles (or ports) and does not remain in the tunnel. The number of risers, separation distance between risers, length

of each riser and number of outlet nozzles on each riser will depend on the specific needs for each outfall.

Some points to note are provided later:

- It is advantageous to locate the diffuser in fast flowing ambient waters. This will enhance dilution of the wastewater and rapidly transport the wastewater away from the diffuser.
- For effluent discharge, the outlet nozzles are usually horizontal. Effluent is less dense than the ambient marine waters and will rise to the sea surface or, if the stratification is sufficiently strong, become trapped below the surface. In contrast, outlet nozzles associated with a brine discharge are angled toward the surface (often, an angle of  $60^\circ$  to the horizontal is used). The density of brine is greater than that of seawater and it will fall toward the sea bed. Angling the outlet nozzles toward the surface and discharging the brine with high velocity will maximize its dilution.
- A pipeline may replace the tunnel as shown in Fig. 32.1. The pipeline is anchored to the sea bed and discharge is through outlet nozzles fixed to the pipeline. Risers are not used in these con-



**Fig. 32.1a,b** Schematics of (a) a wastewater treatment plant outfall, and (b) a desalination plant outfall, showing side views and end views

figurations. (Risers are vertical structures used to transfer the wastewater from an outfall tunnel to the outlet nozzles. They may be tens of metres in length).

- The number of outlet nozzles attached to each riser is usually restricted to eight or less. If more than eight outlet nozzles are used, the plumes from adjacent nozzles interfere with each other and reduce the effective dilution of the wastewater.
- The outfall pipeline and diffuser may be tapered to ensure the velocity of the wastewater remains suf-

ficiently high to prevent sediments from settling in the pipeline.

- Outlet nozzles may be fitted with nonreturn check valves (also called duckbill valves). These valves are closed when the wastewater flow is zero and prevent the ingress of seawater into the pipeline. One advantage of check valves is that they enhance dilution, compared with a round nozzle of the same cross-sectional area [32.1]. However, they may be fouled by biota or fishing nets, rendering them permanently open or closed.

## 32.2 Governance

There are many factors affecting the decision to build a marine outfall.

Municipal wastewater collects at the bottom of the catchment. For a coastal city, this is at the edge of the marine environment. There are large costs associated with the movement of wastewater to the top of a catchment for potable reuse, including construction of a pipe network, pumps, and energy required to operate the pumps. Furthermore, there may be high costs associated with the conversion of wastewater to potable water. The disposal of wastewater through a marine outfall may be the best overall use of resources. Despite this, the decision to proceed with a marine outfall should first examine other options and maximize the beneficial uses of recycled wastewater.

### 32.2.1 Drivers for a Marine Outfall

SPHERE is an acronym we use to describe the main factors overlying the need for government sponsored development (social, public health, environmental, regulation, economic). The first three elements of SPHERE represent the main aspects in which a marine outfall has an impact (i.e., the community values). The last two elements of SPHERE represent the constraints on the marine outfall – *regulation* tending toward high treatment and consequential high cost and *economic* tending toward low cost and consequential low treatment. Below, some of the considerations of SPHERE are described in the context of a marine outfall.

Most countries have environmental guidelines that need to be met during the design of a marine outfall. These guidelines are unique to each country and all cannot be detailed here. Suffice to say that they include meeting concentrations of contaminants which may include pathogens, nutrients, metals, and organics. These guidelines usually apply at the boundary of a mixing

zone, which needs to be clearly defined prior to construction of the marine outfall.

#### Social

What does the community expect from a marine outfall? What are the values that are important to the community? This will vary among and within different geographical regions and cultural groups. Some communities will comprise a large number of beach users. To them, the concept of a marine outfall may not be palatable unless it can be clearly demonstrated that the marine outfall poses minimal risk to their use of the marine environment.

#### Public Health

Is it safe to swim in the marine waters? What are the types and concentrations of substances that will be discharged to the marine environment? Will they be of harm to us? Much information is available to inform us about the potential harm of substances that may be discharged through a marine outfall. Most countries synthesize this information into a set of guidelines applicable to their marine environment. There is a tacit assumption that, provided the concentrations of the substances are kept below harmful levels, the health of the users of the marine environment will be maintained. This does require knowledge of the types, concentrations, and variability of the substances in the wastewater. It should be noted here that all the substances are potentially toxic given sufficiently high concentrations and the environment into which they are discharged.

#### Environmental

Will the discharge of substances through the marine outfall cause harm to marine organisms? Will the marine environment be degraded into the future? Will the beaches and marine waters be free from visible pollution – oil, grease, rags, etc.? As noted under *Public*

*Health*, most countries have environmental guidelines. Provided these guidelines are met, it is assumed that the marine environment will be protected. The guidelines are usually in the form of concentrations of substances (e.g., metals, nutrients, and bacteria) that should be met at a specific distance from the outfall (this distance defines a mixing zone). This implies that there will be a region inside the mixing zone in which the guidelines may not be met. The consequences are that the biological diversity inside the mixing zone may not be the same as that in reference areas.

### Regulation

What are the regulations that govern the discharge of any substance to the marine environment? Regulations are often in the form of licence conditions restricting the types, concentrations, and/or loads of substances that can be discharged to the marine environment. As noted above, there is a tacit assumption that keeping within these restrictions will ensure the safety of humans, and the protection of flora and fauna in the marine environment.

### Economic

Governments will invest a large amount of money for the construction of a marine outfall. Ultimately, this money is raised through taxes and governments are accountable for the wise use of the taxes they collect. Outfall dollars will be competing with funding areas as diverse as education, security, and care for the aged. What does the community value? What is the community willing to pay to protect both humans and the environment? The marine outfall is just one of many options that should be considered. Ultimately there is a balance between the level of protection offered and the cost incurred by each option. It is the responsibility of the engineer and scientist to evaluate each option and provide the government with the most effective solution.

## 32.2.2 Wastewater Treatment

Our main focus in this section is on municipal outfalls. Critical to a marine outfall is knowledge of what is being discharged, particularly the types, concentrations, and variability of contaminants in the wastewater. Discharge of contaminants from other sources including private outfalls, rivers and estuaries, atmospheric inputs, discharges from vessels, and illegal dumping are not considered. The reader is referred to *Tchobanoglous et al.* [32.2], which provides considerable detail on wastewater treatment.

Wastewater discharges from domestic, commercial, and industrial sources. Often, the wastewater systems

are not isolated from the environment and infiltration of water during storms may also occur. The composition of wastewater depends on the relative contribution of these three main sources and on the type and size of industry and/or commercial activity. Each wastewater system is unique and treatment plants are designed to deal with the quantity and quality of wastewater produced by a specific system.

Wastewater comprises particulate matter, pathogens, nutrients, organic, and inorganic material. Severe environmental damage can result if wastewater is discharged undiluted or without treatment. Therefore the main objective of sewage treatment is the elimination or reduction in concentration of these materials.

Different concentrations of substances will invoke different responses in different species. Metals may be adsorbed onto particulates that may be ingested by fish and shellfish. Organics are often adsorbed by the fatty tissues in aquatic animals. Reducing the concentrations of suspended solids, oil, and grease during the wastewater treatment process, reduces the quantity of metals and organics that may affect marine organisms.

Wastewater treatment can be broadly divided into three levels: primary, secondary and tertiary (or advanced). The levels are modular, subsequent treatments being *bolted* onto lower levels of treatment. Within each level of treatment there are multiple options that produce wastewater of similar quality. The distinction among the treatment levels themselves is blurred and will depend on how individual levels are operated and maintained. Usually, concentrations of suspended solids, biochemical oxygen demand (BOD), and indicator bacteria in the effluent are used to distinguish the levels of treatment. The type of wastewater treatment plant adopted is often based on the collective experience of the engineers and process workers within an organization.

### Primary Treatment

Primary treatment removes debris that could damage the wastewater treatment system. This is done by passing the sewage through trash racks and screens. Sewage then flows through sedimentation tanks at low velocities ensuring residence times of 2–3 h or more [32.2]. This allows sufficient time for negatively buoyant solids to settle at the bottom of the tank and positively buoyant oils and greases to rise to the surface of the tank. Chemicals can be added to the sewage to accelerate the settling process. Both the solids and oil and grease can then be easily removed. Primary treatment also helps regulate the flow of sewage to subsequent levels of treatment. Primary treatment may be used in isolation, but this usually depends on the environment into which the wastewater is discharged.

**Table 32.1** Median concentrations of substances in sewage and after various levels of treatment. The numbers are indicative only and may vary in time and between sewage treatment plants

Substance	Units	Raw sewage	Primary	Secondary	Tertiary
Faecal coliforms	cfu/100 ml	10 <sup>7</sup>	10 <sup>6</sup>	10 <sup>4</sup>	10
Suspended solids	mg/l	250	100	10	< 5
BOD	mg/l	200	100	10	< 5
Oil and grease	mg/l	50	20	< 5	< 5
Total nitrogen	mg/l	50	40	20	10
Total phosphorus	mg/l	10	7	5	3

### Secondary Treatment

Secondary treatment covers a wide range of biological processes including: activated sludge, trickling filters, rotating biological contactors, aerated lagoons, oxidizing beds, and membrane bioreactors. The basic objective of all of these processes is the removal of organic material and suspended solids. Secondary treatment may also include disinfection to reduce the concentrations of bacteria in the wastewater. A common form of secondary treatment is activated sludge in which microorganisms are mixed with the wastewater under aerobic conditions for about 4–8 h. The microorganisms metabolize the organic matter in the wastewater, ultimately producing inorganic materials.

### Tertiary Treatment

Tertiary treatment often involves the further removal of suspended materials using sand filters. High levels of nitrogen and phosphorus may remain in the wastewater after secondary treatment, which can contribute to excessive primary production and eutrophication. The basic premise for nitrogen removal is to convert nitrate to nitrogen gas, which is then discharged to the atmosphere. Biological processes and chemical precipitation are two methods used to remove phosphorus from the wastewater. Once removed, phosphate can be used as a fertilizer.

Microfiltration and reverse osmosis are specific forms of advanced wastewater treatment. The wastewater is forced through a fine membrane. The size of the membrane mesh is sufficient to allow the passage of water, but larger materials are captured and removed from the wastewater. Increasingly, micro- or milli-filtration are added to primary or secondary treatment processes. When combined with an effective outfall diffuser, the diluted wastewater may achieve licence requirements.

An indication of the median effluent concentrations of selected substances after treatment is given in Table 32.1. It is stressed that these are general values that will differ for specific wastewater treatment plants and are highly variable.

A desalination plant discharges brine in which the primary contaminant is salt. Median concentrations of

brine are about 60 psu, although it may vary between 40 and 80 psu. The median salt content in seawater is about 35 psu. Marine organisms can tolerate salt concentrations to about 39 psu [32.3], although this value varies with different organisms. Therefore, the configuration of an outfall discharging brine to the marine environment should ensure a rapid reduction in salinity to less than 39 psu.

### 32.2.3 Data Collection for Outfall Design

While the preliminary design of a marine outfall can be undertaken using minimum data, the detailed design usually requires considerable data. The main aim for these data collection programs is identification of the site (or sites) for the marine outfall. That is, for the lowest cost, identifying the level of treatment and site that best meets the environmental (and other) guidelines.

The type and volume of data required depends on the marine outfall being considered. Broadly, data include: volume and flow rates of effluent to be discharged, water quality (both in the treated effluent and in the marine waters), ocean currents, and stratification. A critical aspect of monitoring, often overlooked, is the variability of these data. Our favored approach is to use the data variability in a Monte Carlo approach, running the models for many different combinations of input values. This results in a statistical distribution of the concentrations of contaminants in the marine waters, which can be synthesized in, for example, a probability of exceedance plot.

Historical data or data collected from different projects can be used. The difference between the data needs and the historical data defines a gap that a data collection program needs to fill. Much of the data collected as part of these studies can also be used in Sect. 32.6. Some of the main data collection programs are outlined below.

The volume of effluent flow can be estimated from human population projections. This information allows assessment of when environmental guidelines are likely to be exceeded; hence when upgrades of the treatment plant are likely to be needed.

Water quality in the treated effluent will be a function of the level of treatment. Estimates can be obtained from other, similar treatment plants or from the indicative values provided in Table 32.1. Ongoing monitoring of the effluent quality after construction will help ensure maintenance of environmental standards.

Measuring water quality in the marine waters (into which the wastewater is discharged) provides background concentrations of contaminants. The background concentrations must be added to the modeled concentrations to estimate the total concentration of contaminants in the marine waters. Background concentrations may already exceed environmental guidelines, in which case, they made need to be relaxed or another outfall location sought. To obtain a representative picture of marine water quality, sampling should take place over large spatial and temporal scales and should include replication. Instruments that can be moored in the field for long periods of time are increasingly being used to obtain water quality measurements, although the accuracy of such results is less than can be achieved in the laboratory.

Current speed and direction are important for plume dilution. Moored current meters can provide detailed temporal information at a point in space (or a profile throughout the water column). However, they are expensive to deploy, maintain and retrieve, and careful consideration needs to be made in regard to the number and location of such moorings. Spatial information can be obtained by profiling currents from a vessel underway, drifters drogued at specific depths

and remotely sensed data (e.g., via satellites or airborne scanners). The number and duration of moored current meters will depend on the size of the outfall under consideration. For a moderately sized outfall, a single profiling current meter moored for 12 months and serviced monthly, provides the minimum data requirements. A roving current meter (deployed at different locations for one month at a time) may provide a compromise between the number of instruments and spatial coverage.

Density stratification of the water column largely governs the height of rise of the wastewater. (The effect is much reduced for brine discharges). In coastal marine waters, density is a function of both temperature and salinity, both of which should be measured. For shallow outfalls (outfalls in water depths less than about 10 m), stratification has little effect. However, for outfalls in deep waters, relatively small stratification may produce a submerged plume resulting in lower dilutions and a nonvisible plume (at least to a surface observer). Moored temperature/salinity strings provide a profile of density throughout the water column, although marine fouling will reduce the quality of data from salinity sensors. Such data can also be collected during the servicing of moored instruments, which may be monthly over a period of 12 months.

Other data such as surface waves and tides may also be important, particularly for shallow outfalls where the changes in water depth resulting from such processes may represent a significant proportion of the water column.

## 32.3 Predicting Near-Field Dilutions

The design of a marine outfall centers on the dilution required to meet the relevant guidelines. Occasionally, guidelines may be met after an appropriate level of sewage treatment. However, many substances will rely on the dilution with marine waters to meet these guidelines. Dilution depends on:

- Wastewater flowrate
- Depth of water into which the wastewater is discharged
- Length of the diffuser
- Outlet diameter (and whether a single or multiple outlets will be used)
- Configuration of the diffuser (e.g., whether T-section outlets or gas-burner type rosettes are used, whether nonreturn check valves are used)
- Ocean conditions (e.g., currents, stratification of the water column, tides, and ocean turbulence).

Together with cost, the above factors are used to optimize the location and configuration of the marine outfall. This is further discussed in Sect. 32.3.7.

After discharge from the marine outfall, effluent rises (whereas brine descends) due to buoyancy (Fig. 32.1). The wastewater (effluent or brine) then mixes with the ambient currents and is diluted. Two types of models are used to quantify this process: near field and far field. This separation is made because the time and space scales of the processes in each model are substantially different.

In the near field, the motion of the wastewater is dominated by its initial momentum and buoyancy; the velocities and rates of dilution are high. Up to 90% of wastewater dilution takes place within the near field at the end of which most regulations apply. The engineer can configure the outfall design to maximize dilution in the near field.



In the far field, the wastewater is passively transported by the ambient currents and the rates of dilution are much lower than in the near field. Far-field mixing is dominated by natural processes, over which the design engineer has little control.

While the use of both near field and far-field models may be necessary for the detailed design of a marine outfall, we argue that near-field modeling alone may be adequate for the initial design and emphasis in the following sections is placed on near-field modeling.

This section provides a broad introduction to near-field modeling. Wood et al. [32.4] provide considerable detail on near-field modeling and many of the problems that may be encountered in the design of a marine outfall.

### 32.3.1 Physical Models

While the focus of this section is on near-field numerical modeling, it is recognized that physical modeling can also play an important role in the design of a marine outfall. Scaled physical models of prototype marine outfalls are sometimes constructed in the laboratory and used to examine the behavior of jets and plumes in the near field. They provide good visualization of the plumes, particularly interactions between multiple plumes and include effects that are not found in most numerical models.

The fluids used in the model are typically fresh and saline water. The scales for such models are expressed as a ratio of a prototype quantity to a model quantity. Model design requires the selection of (i) the fluids which yield the reduced gravity ratio ( $g$  defined below), (ii) the length scale to ensure that the model Reynolds numbers are sufficiently high to guarantee turbulent model flows and (iii) a scaling criterion which, in this case, is the densimetric Froude number ( $Fr$ ), i. e., there is a point-to-point correspondence of  $Fr$  in prototype and model. This scaling criterion, together with the length scale, yields the velocity scale. Other scales for time, pressure and buoyancy force can then be determined. The inclusion of ambient currents in physical models is possible, but places greater demands on laboratory facilities and data acquisition systems.

Results from physical models may include information on dilutions, trajectories, the velocity field, and interactions between neighboring plumes.

### 32.3.2 Positively Buoyant Jets and Plumes

Two basic approaches to near-field numerical modeling are available: Eulerian and Lagrangian. A Lagrangian approach is followed in both Lee and Cheung [32.5] and

Tate and Middleton [32.1, 6]. Central to either approach are the conservation equations for mass, momentum, and buoyancy. In a Lagrangian framework, they are:

- Mass conservation

$$\frac{\partial (\rho V)}{\partial t} = \rho_a f_{\text{ent}} U_{\text{ent}} A$$

- Momentum conservation

$$\frac{\partial (\rho V u_i)}{\partial t} = U_i \frac{\partial (\rho V)}{\partial t} + \rho g' V$$

- Buoyancy conservation

$$\frac{\partial (g' V)}{\partial t} = -N^2 u_i V,$$

where  $\rho$  is the density of the jet/plume,  $\rho_a$  is the density of the ambient fluid,  $f_{\text{ent}}$  is an entrainment function,  $U_{\text{ent}}$  is the entrainment velocity,  $V$  is the volume of a buoyant fluid element,  $A$  is the cross-sectional area through which ambient water is entrained,  $u_i$  is the velocity of the buoyant fluid,  $U_i$  is the velocity of the ambient fluid,  $g'$  is the buoyancy modified gravity ( $= (\Delta \rho \cdot g) / \rho_{\text{ref}}$ , where  $\rho_{\text{ref}}$  is a reference density), and  $N$  is the Brunt-Väisälä frequency

$$N = \sqrt{-\left(\frac{g}{\rho_{\text{ref}}}\right) \frac{\partial \rho_a}{\partial z}}.$$

These governing equations are also applicable to negatively buoyant jets and plumes.

If the buoyant jet/plume (a) lies well away from its source (i. e., beyond the influence of the initial momentum), (b) is moving with the ambient fluid, and (c) the Boussinesq approximation is applied, then the above equations can be solved analytically to give what is known as the asymptotic results. These equations (Table 32.2) are equivalent to the *advected thermal* equations in Wood et al. [32.4] and the corresponding flow classifications are detailed in Jirka and Akar [32.7] and Jirka and Doneker [32.8].

Solutions to the asymptotic governing equations for positively buoyant plumes emerging from round (i. e., axisymmetric) outlet ports and from a slot (i. e., line source), in a flowing ambient fluid, with both linearly stratified or nonstratified marine waters, are presented in Table 32.2. It should be noted that these asymptotic solutions below should only be used at the conceptual stage of outfall design. For preliminary and detailed design, the full set of conservations equations above should be used and solved numerically.

The entrainment function has evolved from the constants used in Morton et al. [32.9] to a complex function

**Table 32.2** Solutions to the governing asymptotic equations for positively buoyant plumes when the ambient current speed is nonzero and the marine water density is linearly stratified (after [32.6]). Solutions to the asymptotic equations are applicable only at the end of the near field and do not include the outlet port diameter, angle of discharge, or the exit velocity

Axisymmetric source	Line source
$z(x) = \frac{0.98}{f_{\text{ent}}^{2/3}} \left( \frac{B_S [1 - \cos(\frac{N}{U} x)]}{UN^2} \right)^{1/3}$	$z(x) = \frac{1.00}{f_{\text{ent}}^{1/2}} \left( \frac{B_S [1 - \cos(\frac{N}{U} x)]}{UN^2} \right)^{1/2}$
$2b(z) = 2.00 f_{\text{ent}} z$	$2b(z) = 2.00 f_{\text{ent}} z$
$C(z) = \frac{C_S}{C(z)} = 3.14 f_{\text{ent}}^2 U z^2 \frac{n_{\text{ports}}}{Q}$	$S(z) = \frac{C_S}{C(z)} = 2.00 f_{\text{ent}} U z \frac{L_D}{Q}$
$z_{\text{max}} = \frac{1.24}{f_{\text{ent}}^{2/3}} \left( \frac{B_S}{UN^2} \right)^{1/3}$	$z_{\text{max}} = \frac{1.41}{f_{\text{ent}}^{1/2}} \left( \frac{B_S}{UN^2} \right)^{1/2}$
$2b(z_{\text{max}}) = 2.48 f_{\text{ent}}^{1/3} \left( \frac{B_S}{UN^2} \right)^{1/3}$	$2b(z_{\text{max}}) = 2.82 f_{\text{ent}}^{1/2} \left( \frac{B_S}{UN^2} \right)^{1/2}$
$S(z_{\text{max}}) = \frac{C_S}{C(z_{\text{max}})} = 4.84 f_{\text{ent}}^{2/3} \left( \frac{UB_S^2}{N^4} \right)^{1/3} \frac{n_{\text{ports}}}{Q}$	$S(z_{\text{max}}) = \frac{C_S}{C(z_{\text{max}})} = 2.83 f_{\text{ent}}^{1/2} \left( \frac{UB_S}{N^2} \right)^{1/2} \frac{L_D}{Q}$

$z$  is the elevation of the plume above the outlet [m],  $z_{\text{max}}$  is the maximum elevation (i. e., height of rise) of the plume [m],  $S(z) = \frac{C_S}{C(z)}$  is the average dilution at elevation  $z$ ,  $C_S$  is the concentration at the source [ $\text{kg}/\text{m}^3$ ],  $C(z)$  is average concentration at elevation  $z$  [ $\text{kg}/\text{m}^3$ ],  $x$  is distance downstream from the outfall [m],  $f_{\text{ent}}$  is the dimensionless entrainment function,  $U$  is ambient current velocity [m/s],  $N$  is the Brunt–Väisälä frequency [1/s], where  $N^2 = -\frac{g}{\rho_a} \frac{d\rho_a}{dz}$ , and  $\rho_a$  denotes a representative seawater density,  $2b(z)$  is the diameter (or thickness) of the plume [m],  $Q$  is the flow through the outfall [ $\text{m}^3/\text{s}$ ],  $L_D$  is the length of the diffuser [m],  $n_{\text{ports}}$  is the total number of outlet ports on the diffuser and,  $B_S$  is the buoyancy flux at the source,  $B_S = g \frac{\rho_a - \rho_S}{\rho_S} \frac{Q}{n_{\text{ports}}}$  for an axisymmetric source and  $B_S = g \frac{\rho_a - \rho_S}{\rho_S} \frac{Q}{L_D}$  for a line source, where  $\rho_S$  is the density of the wastewater at the source.

of the densimetric Froude number, plume geometry, the velocity of the fluid inside the plume, and the velocity of the ambient current [32.5]. Wood et al. [32.4] use a spreading function to model the entrainment of ambient fluid into the plume.

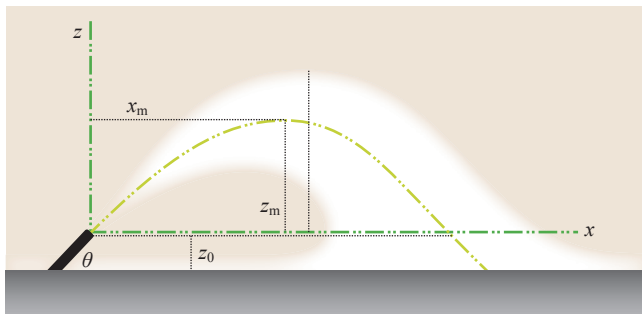
### 32.3.3 Negatively Buoyant Jets

Research conducted on negatively buoyant jets over the past several decades has sought to quantify jet behavior using a variety of analytical and experimental techniques. Results from these studies have led to the development of proportionality coefficients which relate the jet densimetric Froude number and nozzle diameter to trajectory and dilution. The particular points

of interest along the jet trajectory are the centerline peak ( $z_m$ ) and return point ( $x_r$ ) which are both defined in Fig. 32.2. A range of experimentally derived values for each of the proportionality coefficients compiled from various experimental studies are presented in Table 32.1, as reported in Lai and Lee [32.11]. Note that these coefficients are only valid for single jets discharging into quiescent ambient conditions from a nozzle orientated at  $45^\circ$  to the seabed. Coefficients for other discharge angles can be found throughout the research literature [32.12–18]. Current research on negatively buoyant jets focuses on multiport diffusers and discharge into receiving waters with ambient currents.

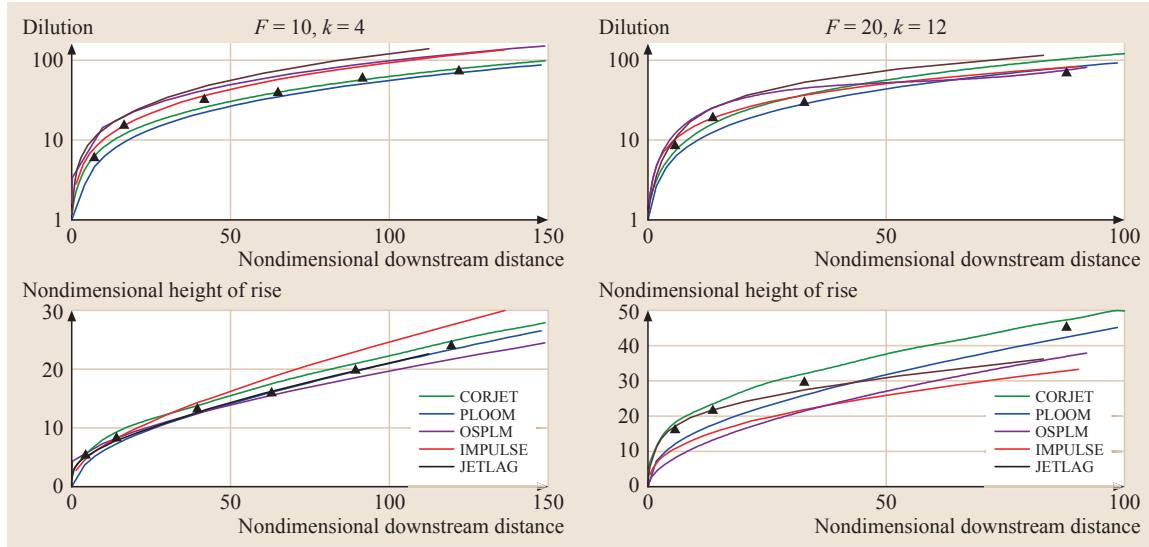
### 32.3.4 Model Validation

The information presented in Table 32.2 and Fig. 32.3 are based on asymptotic models i. e., results only at the end of the near field and should only be used at the conceptual stage of outfall design. Full numerical models detail the movement of the wastewater from the outlet nozzle to the end of the near field and include the nozzle size, the initial momentum of the wastewater and its trajectory. A limited set of results from the laboratory experiments of Fan [32.10] for a single outlet, discharging positively buoyant water into a flowing, unstratified ambient fluid are compared with several near-field models that have been used by the authors. The models are:



**Fig. 32.2** Trajectory of a negatively buoyant jet





**Fig. 32.3** Near-field model results for plume dilution and trajectory compared with laboratory data from *Fan* [32.10]

**Table 32.3** Experimentally derived coefficients for a single negatively buoyant jet discharging into quiescent ambient conditions at an angle of  $45^\circ$  to the seabed (after *Lai and Lee* [32.11])

Description	Equation	Experimentally derived coefficients
Jet terminal rise height	$z_t = \frac{C_1}{D \cdot Fr}$	$1.43 \leq C_1 \leq 1.61$
Horizontal location of return point	$x_r = \frac{C_2}{D \cdot Fr}$	$2.82 \leq C_2 \leq 3.34$
Dilution at return point	$S_r = \frac{C_3}{Fr}$	$1.09 \leq C_3 \leq 1.55$
Vertical location at jet trajectory centerline peak	$z_m = \frac{C_4}{D \cdot Fr}$	$1.07 \leq C_4 \leq 1.19$
Horizontal location at jet trajectory centerline peak	$x_m = \frac{C_5}{D \cdot Fr}$	$1.69 \leq C_5 \leq 2.09$

IMPULSE [32.19], JETLAG [32.5], CORMIX [32.7, 8], OSPLM [32.4], and PLOOM [32.1, 6]. *Fan*'s data set is used here because it is independent of the laboratory data collected by any of the above authors. The unique identifiers for *Fan*'s experiments are  $Fr$ , the densimetric Froude number

$$Fr = \frac{u_{\text{port}}}{\sqrt{g' d_{\text{port}}}},$$

where  $u_{\text{port}}$  is the velocity through the outlet port,  $d_{\text{port}}$  is the diameter of the outlet port and

$$k = \frac{u_{\text{port}}}{U}$$

and

$$g' = g \left( \frac{\rho_a - \rho_s}{\rho_s} \right).$$

Compared with the laboratory data of *Fan* [32.10], these models all produce similar results (Fig. 32.3) providing confidence in the models themselves. However,

it is recognized that different models may behave differently for different regimes and selection needs to be appropriate to the problem under investigation. For example, a particular model may provide good estimates of dilution for outfalls comprising a single point discharge but poor estimates of dilution for outfalls comprising a long diffuser with multiple risers and outlet nozzles. It is stressed that there are other near-field models available [32.20–22] that would likely provide similar results.

While laboratory studies are often used to calibrate parameters within a model, field experiments are used to validate the model predictions for a specific marine outfall. This is undertaken using outfall dilution studies. Obviously, such validation studies can only be carried out after the marine outfall has been constructed. Outfall dilution studies involve the continuous injection of a tracer into the wastewater and the measurement of its concentration downstream from the discharge point. The tracer is injected at a known rate and concentration, and the flow of wastewater is also known. Therefore, by measuring the concentration of the tracer in the marine waters, the concentration of the wastewater can be determined.

Many tracers are available, for example: rhodamine WT, fluorescein and the isotopes gold-198, technetium-99m, and tritium. Natural tracers such as salinity have also been used, but the variability in the data is usually too large to produce meaningful results. Preference is given to use a tracer that has little or no background signal; hence contact with the tracer will result in unambiguous readings. The tracer sensing device (such as a fluorometer or scintillation counter) may be towed behind a vessel and/or profiled through the water column to build a three-dimensional picture of the location and size of the plume. A critical element of the work is accurate position fixing, now usually done with differential GPS (global positioning system).

Simultaneously, the wastewater flow, ambient current speed and direction, and the density of the water column are measured. These data are used as input to the model. A direct comparison between the observations and model results can then be made. However, models are only approximations to the real world and there will be uncertainty associated with the results. Based on the results from many such experiments, predicted dilutions within a factor of two of actual dilutions are generally acceptable.

An example of the results obtained from a tracer experiment is shown in Fig. 32.4. The transect lines run parallel to the diffuser, 100 (lower panel) and 1000 m (upper panel) downstream from the outfall. Multiple transect lines are shown in each panel. In the lower panel, plumes from each of the nine risers comprising this outfall can be clearly identified. At a distance of 1000 m downstream from the outfall, plumes from the

individual risers have merged, the overall width of the plume has increased and the concentration of the tracer (or plume) has markedly reduced.

### Problems with Tracer Studies

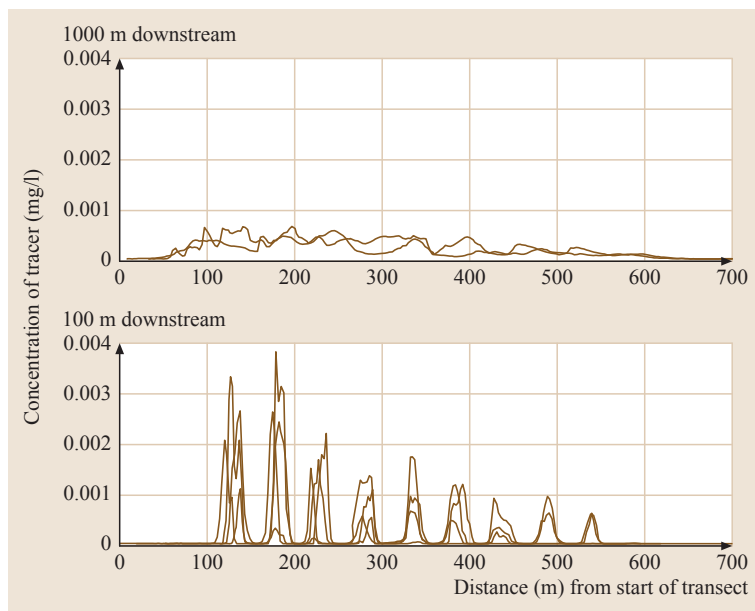
Some problems encountered by the authors in conducting tracer studies are outlined below.

Some marine outfalls may have intermittent flow, particularly early in the life of the outfall when the design flow capacity is not yet reached. With intermittent flow, the time history of the patch of wastewater is unclear. However, it may be possible to temporarily store the wastewater, to enable a continuous and steady flow over the duration of the field experiment.

Locating the plume in the field may be difficult. The tracer may not be visible when the ambient waters are stratified and the plume is trapped below the water surface. A conductivity-temperature-depth probe can be used to identify stratification in the water column and hence the likely depth at which the effluent will reside.

Isotope tracers decay with time. The half-life of technetium-99m is 6 h which is comparable with the duration of many tracer experiments. If technetium-99m is used as a tracer, the initial signal will change significantly over time and needs to be accounted for in the data analysis. Tritium, with a half-life of about 12 yr, can be used for long duration tracer experiments or when there is considerable transport time between the nuclear facility that produces the isotope and the experiment site.

When a positive contact is made with the labeled plume, it is not possible to know where this contact oc-



**Fig. 32.4** Example of tracer concentrations obtained from field studies. Concentration data were collected from 1 m below the surface, at distances of 100 and 1000 m downstream from the outfall. Note the uneven distribution of concentration along the diffuser indicating an uneven distribution of flow

curs in the wastewater plume. One solution is to take many tracer readings closely separated in space and time to identify the plume boundaries and the region of highest tracer concentration.

The fluorescence of rhodamine WT is highly temperature dependent. It loses about 3% of its fluorescence for every one degree Celsius drop in water temperature. In very cold environments, it may not be possible to detect a signal at all – as happened to the authors when first using rhodamine WT in Antarctica.

### 32.3.5 Far-Field Numerical Modeling

The emphasis in this chapter is on near-field modeling rather than far-field modeling. The reason for this is because most of the dilution of the discharged wastewater occurs in the near field, and environmental guidelines and licence conditions are usually applied at the end of the near field. However, far-field modeling is important when assessing discharges into relatively shallow waters when mixing in the near field is incomplete or when examining potential impacts at sites remote from the outfall, e.g., beach bathing waters or sensitive marine habitats or communities.

Far-field modeling usually includes hydrodynamic and water quality components. The hydrodynamic models are based on the principles of mass and momentum conservation of the marine waters; water quality models are based on mass considerations of the contaminant(s) or tracer(s) being discharged. Hydrodynamic models are usually based on a fixed mesh in space (i.e., an Eulerian formulation) and produce the depth and velocity fields as output. The water quality models require the velocity field as input; their formulation may be based on the same mesh as the hydrodynamic model. In another formulation (i.e., the Lagrangian formulation), many parcels of contaminant or tracer may be tracked as the velocity field transports and disperses them. The results from an Eulerian formulation yields the contaminant concentrations on the fixed mesh, while the Lagrangian formulation yields the number of contaminant parcels contained in each volume of fluid bounded by mesh points or nodes; these numbers can then be converted into contaminant concentrations.

The most common types of Eulerian models used are finite difference (i.e., point-wise approximations of the variables), finite elements (piecewise approximations of the variables), or finite volume (based on fluxes of mass or momentum within each mesh cell). The meshes can be regular (i.e., structured) or irregular (i.e., nonstructured); they can be 2-D (2-dimensional; i.e., depth averaged) or 3-D (three-dimensional).

In a 2-D, hydrodynamic model the mesh is in the horizontal plane. At each mesh point or node,

the unknowns consist of two velocity components and a depth. In a 3-D model, the mesh includes the 2-D horizontal plane and the mesh points or nodes in the vertical dimension. The unknowns at each 3-D mesh point or node, typically consist of two horizontal velocity components and pressure.

Usually, 2-D models require substantially less computational time and less data for calibration and running than 3-D models. In water depths exceeding about 20 m, the velocity vector at a single location in plan, may vary in magnitude and direction throughout the water column. If resolution of this variability is considered significant from the point of view of pollutant movement, a 3-D model may be preferred to a 2-D model. The horizontal spacing between mesh points or nodes will depend on the bathymetry and the presence of islands, headlands, and submarine canyons. The near-field model results need to be incorporated into the far-field model. To achieve this effectively, it may be desirable to refine the far-field mesh in the vicinity of the near field.

Far-field models run under various flow scenarios can be used in the early stages of an investigation to guide data collection programs before, during, or after commissioning of an outfall. Such model studies may be conducted using a coarse mesh for quick turnaround of results.

### 32.3.6 Data for Running the Models

A range of information is required to run the numerical models. This includes: the outfall configuration, wastewater flow, and oceanographic data (currents and stratification of the water column). Over the long term, the model results can be used to examine changes in outfall performance. Below is a summary of the information required to run the models and how that information may be obtained.

#### Outfall Configuration

The concept outlined in the following section can provide a starting point for the outfall design. In the design phase, the outfall configuration can be changed and refined until the relevant environmental guidelines are met and engineering feasibility assessed. Once the outfall has been constructed, its configuration is essentially fixed. However, some flexibility may be enabled. For example, twin pipelines may be built and only one pipeline used for present wastewater flows (the second pipeline being saved for use when wastewater flow increases with future growth in population). Similarly, a multiport diffuser may have one or more outlet ports blanked, again in anticipation of future growth.

Information needed for the outfall configuration includes:

- Water depth in which the diffuser section is located.
- Length of the diffuser section.
- Configuration of the diffuser (e.g., a single or multiport outlet).
- Diameter of each outlet port.
- Whether the outlet ports are fitted with nonreturn check valves.

#### Wastewater Flow

Wastewater flow is usually measured in the outlet pipe at the end of the treatment processes. A range of flow measuring devices are available including flows based on electromagnetic, pressure, ultrasonic, or capacitance sensors. Also important is the density of the wastewater in relation to the density of the marine waters into which the wastewater is discharged. Usually it is safe to assume that the density of the wastewater is close to that of fresh water [32.2], although large amounts of particulate material in the wastewater may alter the density of the wastewater.

For numerical modeling purposes, wastewater flow is usually assumed to be uniform throughout each outlet port. This may not be necessarily the case. Energy loss may be significant over long diffuser sections, resulting in reduced flows through outlets lying further offshore. Low flows may result in the intrusion of seawater into the diffuser and a reduction in its performance. To help establish uniform flow, diffuser sections may be tapered (Fig. 32.1) and to help prevent the intrusion of seawater, outlet ports may be fitted with nonreturn check valves.

#### Currents

Currents determine the movement and dilution of the wastewater. Often a moored Doppler profiler is used to measure the current speed and direction throughout the water column. Doppler profilers can also be ship mounted, which allows a spatial picture of the currents to be obtained. Remote sensing and shore-based radar systems can provide detailed spatial coverage of the surface currents. However, it is subsurface current data which is critical for running the near-field models.

The choice of mooring location should be as close as possible to the diffuser. However, a compromise is often made, balancing the proximity of the diffuser to the mooring, with the health of workers who service the mooring (in waters that may be contaminated with diluted wastewater) and the security of the mooring itself.

#### Stratification

Stratification is a rapid vertical change in the density of the marine waters. In coastal waters, changes in the

density are dominated by changes in temperature and salinity. The height to which a wastewater plume rises in the water column is largely governed by the strength of the stratification.

Measurements of temperature and salinity are often made using a conductivity, temperature, depth (CTD) probe. (Salinity is calculated from conductivity and temperature). The CTD probe can be lowered from a boat providing a continuous profile through the water column. CTD probes can also be moored, thereby providing a time series of density data at a fixed point. Historically, conductivity data from moored CTDs drift with time due to the gradual build-up of film on the sensors. While there have been substantial improvements in the reliability of moored conductivity sensors in recent years, the quality of the data may still be highly variable. Temperature sensors (unless heavily fouled with marine growth) do not suffer the same problem. Hence changes in stratification using data from long-term moored systems are usually estimated from temperature sensors alone.

### 32.3.7 Conceptual Design for Positively Buoyant Discharges

*Wilkinson* [32.23] described a method by which the minimum length of a *simple outfall* could be determined. In his concluding remarks, *Wilkinson* [32.23] was careful to point out that this provides a preliminary estimate only, and he provided some suggestions on ways in which the outfall configuration could be further refined. This analysis is only intended as a starting point for outfall design. Detailed analyses are site specific and must be undertaken for final design. Some site specific factors include: the bathymetry, environmental guidelines, level of wastewater treatment, and the likelihood of plumes reaching the surface or sensitive ecological areas. *Wilkinson's* [32.23] approach is modified here, by using the single set of equations (Table 32.2) and introducing construction cost as criteria for outfall design.

The following analysis is applicable to nonzero ambient currents, which is usually applicable to marine waters (e.g., currents near the Sydney, Australia deep water ocean outfalls exceed 0.05 m/s more than 90% of the time).

The total cost ( $T_c$ ) of a marine outfall can be expressed as

$$T_c = lL_p + mL_D + mn_{\text{ports}} \quad (32.1)$$

where  $l$  is cost per meter of the outfall pipeline or tunnel [\$/m],  $L_p$  = length of the outfall pipeline or tunnel [m],  $m$  is the cost per meter of the diffuser [\$/m],  $L_D$

is the length of the diffuser [m],  $n$  is the cost per outlet port [\$], and  $n_{\text{ports}}$  is the number of outlet ports.

The basic premise used in *Wilkinson* [32.23] is that the profile of the water depths as a function of distance offshore (i. e., the length of the marine outfall,  $L_p$ ) can be expressed as the power curve,  $L_p = rz^s$ , where  $r$  and  $s$  are constants that express the least-squares, best-fit shape of the across shelf bathymetry that may be obtained from navigational charts and  $z$  is the water depth.

Expressions for the length of the diffuser and the number of outlet ports can be obtained from Table 32.2 and the total cost can then be rewritten as

$$T_c = l(rz^s) + m \left( \frac{SQ}{2f_{\text{ent}}U} z^{-1} \right) + n \left( \frac{SQ}{3.14f_{\text{ent}}^2U} z^{-2} \right), \quad (32.2)$$

## 32.4 Hydraulic Analysis and Design

It is often necessary to define the physical extent of a brine or wastewater outfall system for project planning and design purposes. While the ambient sea in the vicinity of an outfall structure typically defines the downstream boundary of an outfall system, defining the upstream boundary may not necessarily be as straightforward. For the purposes of this chapter, the upstream boundary is assumed to be a free surface which exists somewhere upstream of the outfall conduit entrance. Typical locations for this boundary could be the effluent level in outfall shafts, deaeration chambers, sedimentation basins, or pumping wet wells. For configurations without any free surfaces between the treatment process and outfall conduit, the upstream boundary may be taken at some hydraulically arbitrary point. Regardless of its physical location, this boundary represents a key design interface that must be properly integrated with the treatment plant as a whole. Determining the piezometric head at the upstream boundary of an outfall system is therefore a critical hydraulic design objective.

### 32.4.1 Governing Hydraulics

Consider the gravity-driven outfall system shown in Fig. 32.5 which includes an upstream shaft to capture plant effluents, as well as rosette style outfall structures installed on the seabed. (A rosette structure typically has multiple nozzles that are arranged around its perimeter.) The piezometric head at the outfall shaft, defined by point 0 on the fluid surface, can be obtained by applying the energy equation between this point and point 1 which is located precisely at the tip of the nozzle. It does not matter which outfall structure or nozzle is used to define point 1 because this multiple riser con-

figuration is an example of parallel flow. For parallel flow, the total head loss must be equal through each parallel flow path.

where  $S$  is the dilution required to comply with licence conditions or environmental guidelines.

To minimize the total cost, the above expression is differentiated with respect to the water depth ( $z$ ), equated to zero and solved. The result gives the depth at which the minimum cost for the marine outfall is achieved. Substituting this value for depth into the equations in Table 32.2, gives the length of the diffuser and the number of outlet ports that comprise the marine outfall.

Actual costs do not need to be known. If the relative costs among  $l$ ,  $m$ , and  $n$  are known, then the total cost of the marine outfall can be expressed in terms of a normalized cost,  $T_c/l$ . Again, it is stressed that this analysis is preliminary and is only intended as a starting point for outfall design.

The energy equation applied between (0) and (1) is written in terms of total head (i. e., energy per unit weight of effluent),

$$\frac{V_0^2}{2g} + \frac{p_0}{\rho_e g} + z_0 = \frac{V_1^2}{2g} + \frac{p_1}{\rho_e g} + z_1 + \sum H_{L(0 \rightarrow 1)}, \quad (32.3)$$

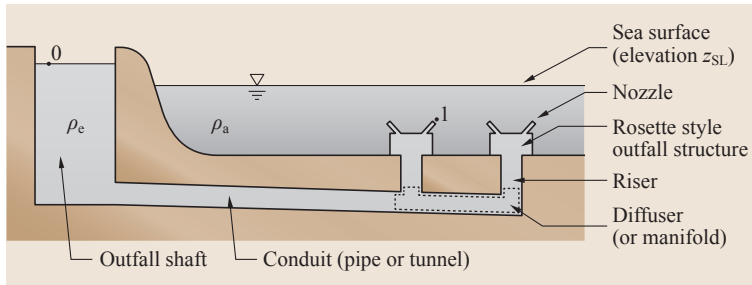
where  $V$  is the velocity [m/s],  $p$  is the pressure [Pa],  $z$  is the elevation above an arbitrary datum [m],  $z_{\text{SL}}$  is the elevation of sea level above datum [m],  $\sum \Delta H_{L(0 \rightarrow 1)}$  is the total head loss between locations (0) and (1) [m],  $\rho_e$  is the density of effluent [ $\text{kg}/\text{m}^3$ ],  $\rho_a$  is the density of ambient seawater [ $\text{kg}/\text{m}^3$ ].

Working with gauge pressures and assuming negligible effluent velocity in the outfall shaft, the first two terms on the left-hand side of (32.3) are reduced to zero. Using the assumption that the pressure at (32.1) is hydrostatic based on the density of seawater, or  $p_1 = (z_{\text{SL}} - z_1) \rho_a g$ , this relationship is substituted into (32.3) to yield an expression for effluent level in the outfall shaft,  $z_0$ .

$$z_0 = \left( \frac{V_1^2}{2g} \right) + \left[ \frac{\rho_a}{\rho_e} (z_{\text{SL}} - z_1) + z_1 \right] + \sum \Delta H_{L(0 \rightarrow 1)} \quad (32.4)$$

Equation (32.4) demonstrates that the effluent level in the outfall shaft is a combination of (i) the head required to drive effluent out of the nozzle at the specified velocity,  $V_1$ , (ii) elevation to the center of the exit port ( $z_1$ )





**Fig. 32.5** Typical outfall system with rosette style outfall structures

plus discharge depth below sea level ( $z_{SL} - z_1$ ) scaled by the ratio of fluid densities, and (iii) a summation of head losses through the system. Each of these components is briefly described in the following sections.

### Nozzle Exit Velocity

The overall diffuser configuration including total number of ports or nozzles, and nozzle diameter (or exit velocity) is typically provided as an input to the hydraulic design based on the results of near-field modeling. The nozzle configuration affects the efficiency with which effluent is diluted in the near-field region and is generally selected based on the maximum outfall flow rate. In some outfall systems, such as those at desalination plants, maximum nozzle exit velocities of the order of 10 m/s may be required to ensure the brine is adequately diluted. The corresponding velocity head would likely be the largest component of the outfall shaft water level for exit velocities of this magnitude, especially for a relatively short outfall conduit with low conduit friction loss.

### Sea Level

If seawater is discharged through the outfall system ( $\rho_c = \rho_a$ ) the second term of (32.4) simplifies to  $z_{SL}$  and the outfall shaft fluid level is equal to sea level for the no-flow case. For sewage outfalls in which  $\rho_c < \rho_a$ , the effect of the density difference is to increase the outlet shaft effluent level. Conversely, the level in the outfall shaft is decreased when brine ( $\rho_c > \rho_a$ ) is discharged. Density differences tend to be of the order of  $(|\rho_c - \rho_a| / \rho_a) \times 100 \approx 3\%$  for most sewage and desalination applications. Although this difference corresponds to a relatively small change to outfall shaft level when discharging into shallow waters, (32.4) shows that the density ratio effect is amplified for deeper outfall discharges.

In addition to changes in plant operating conditions which could increase or decrease outlet flows, changes in sea level will also cause the outfall shaft effluent level to vary. The outfall system design should therefore consider the entire range of sea levels that could occur over the project design life, accounting for tidal fluctuations

as well as storm surge. Statistical methods can be applied to sea level time series at the project location in order to determine exceedance probabilities and recurrence intervals. Using sound engineering judgment in conjunction with project requirements and/or local design standards for infrastructure design life, the results of the statistical analysis can be used to select design values for minimum and maximum sea level. To capture any seasonal trends which could include wind and barometric effects, sea level data used in the statistical analysis should include field measurements taken regularly throughout the year. It is imperative that data specific to the project location is used because sea level characteristics can vary greatly from one locale to another, regardless of the distance between them.

An allowance for sea level rise due to the effects of climate change should also be included because it could have a significant impact on maximum outfall shaft fluid level. Some statistical models estimate that the sea level will rise more than 1 m by the year 2100 (Seneviratne et al. [32.24]).

### Head Losses

The total system head loss represented by the third term of (32.4) consists of the sum of conduit friction losses and the sum of local head losses through all fittings, system components (e.g., bends and contractions), and dividing flows in the manifold/diffuser. These losses can be expressed as

$$\sum \Delta H_L = \sum \frac{V_c^2}{2g} \left( f_c \frac{L_c}{D_c} \right) + \sum \frac{V_L^2}{2g} (K_L), \quad (32.5)$$

in which the first term on the right-hand side of the equation is the Darcy–Weisbach equation for conduit friction loss and  $f_c$  = conduit Darcy friction factor [–],  $D_c$  = conduit diameter [m],  $L_c$  = conduit length [m],  $V_c$  = velocity through the conduit [m/s],  $K_L$  = local head loss coefficient at fitting or component [–], and  $V_L$  = velocity through fitting or component [m/s].

In (32.5), the term *conduit* refers to the tunnel or pipe which delivers flow to the manifold and risers. The Darcy friction factor can either be determined from manufacturers' charts for particular wall roughnesses,



computed iteratively using the implicit Colebrook–White formula given in (32.6), or approximated using the explicit Swamee–Jain equation given in (32.7). The Swamee–Jain approximation is accurate to within a few percent of the value computed using the Colebrook–White equation over the typical ranges of roughness values and fully turbulent Reynolds numbers.

$$\frac{1}{\sqrt{f_c}} = -2 \log \left( \frac{k_s}{3.7D_c} + \frac{2.51}{\text{Re}_c \cdot f_c^{1/2}} \right) \quad (32.6)$$

$$f_c \approx \frac{0.25}{\left[ \log \left( \frac{k_s}{3.7D_c} + \frac{5.74}{\text{Re}_c^{0.9}} \right) \right]^2} \quad (32.7)$$

where:  $k_s$  is the Nikuradse equivalent sand grain roughness of the conduit wall [m],  $\text{Re}_c$  is the conduit Reynolds number  $[-] = \frac{V_c D_c}{\nu}$ ,  $\nu$  is the effluent kinematic viscosity [ $\text{m}^2/\text{s}$ ].

The kinematic viscosity of water for various temperatures and salinities can be obtained using the relationships provided in *Sharqawy et al.* [32.25]. Wall roughness values for common pipe materials can be found in any hydraulics data handbook, while roughness values for segmentally lined tunnels are presented in *Pitt and Ackers* [32.26]. It is customary to make an allowance for increased wall roughness over the conduit design life to account for aging and degradation.

Local head losses arise from flow through bends, tee or wye junctions, flow or pressure control devices (e.g., valves), and expansions or contractions in cross-sectional flow area. Local head losses will also occur at conduit entrances, at submerged discharges, and any location in the system where flow separation occurs. As shown in the second term of (32.5), local head loss is expressed as a multiple of the velocity head at the particular component of interest. The local loss coefficient,  $K_L$ , depends on the component geometry and is determined experimentally. Loss coefficients for common system components can be found in any hydraulics data handbook. Several references such as *Miller* [32.27] and *Idelchik* [32.28] are devoted entirely to local head loss coefficients and include many components present in marine outfall systems.

As local head loss coefficients are always based on some reference velocity, it is important to ensure consistency between a given  $K_L$  and the velocity,  $V_L$ , in the associated velocity head term ( $V_L^2/2g$ ). For components with constant cross-sectional area such as certain bends,  $K_L$  is normally based on the average velocity through the bend. However, there is no standard reference velocity for components like nozzles or sudden expansions or contractions which have multiple cross-sectional areas; some sources may use the upstream velocity for reference, while others may use the downstream veloc-

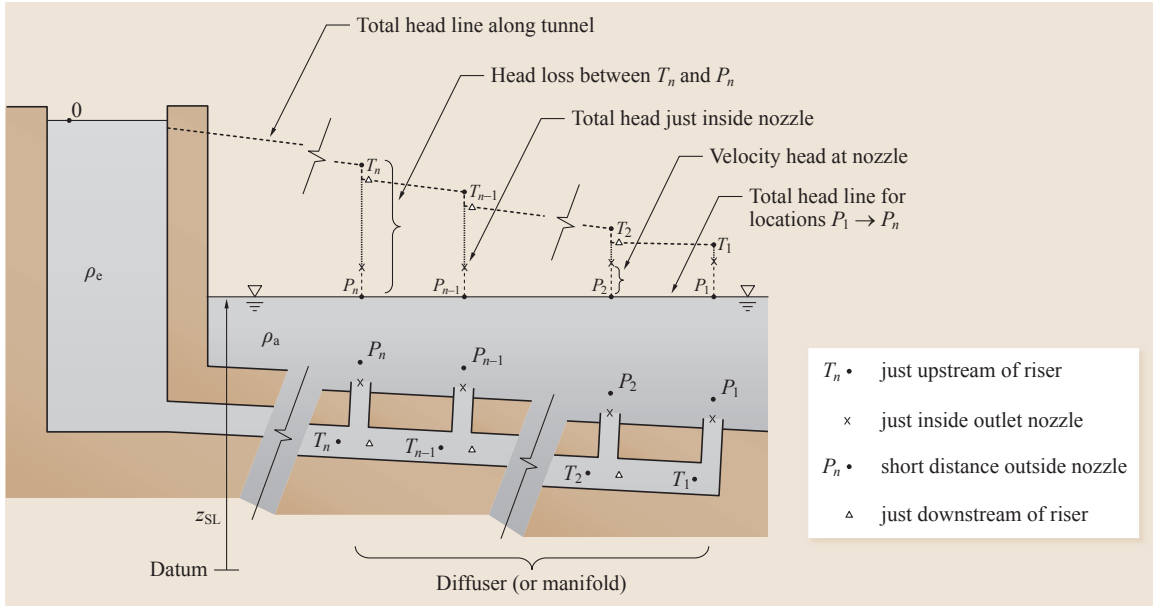
ity. Using an incorrect reference velocity could result in significantly higher or lower head losses.

For complex hydraulic systems, it is important to note that the total local head loss may not simply be the sum of individual local loss components as (32.5) suggests. Rather, head loss coefficients are typically subject to certain limitations. For example, the coefficient for a single tee-junction can only be applied to a series of tee junctions (such as in a dividing manifold) if the separation distance between successive junctions is, say, 5 to 10 times the manifold diameter. Correctly applying local head loss coefficients will help to minimize under- or over-prediction of total local head loss. For cases in which the loss coefficient limitations are not clearly defined or the system configuration cannot easily be broken down into *standard* components (such as through a rosette-style outfall structure), physical and numerical modeling can be used to confirm head losses.

### 32.4.2 Diffusers – Hydraulic Design

Outfall systems usually consist of a manifold (also referred to as a diffuser) whereby a common pipe or tunnel supplies flow to multiple risers, ports, or branches. Although a manifold is an example of parallel flow in which the head loss between the outfall shaft and each exit port is the same, it is critical to note that the flow rate out of each port will not necessarily be the same. The variation in flow rate can be attributed to (i) decreasing flow and total head along the length of the manifold, (ii) changing depth along the manifold, and (iii) head loss coefficients for tee junctions which are a function of (a) the ratio of conduit diameter to branch diameter, and (b) the ratio of local flow through the conduit to flow through the branch. Head loss curves for a range of tee-junction configurations can be found in *Miller* [32.27].

Given that effluent dilution is directly related to port exit velocity, the overall design objective should be to achieve equal exit velocities (or as close to equal as possible) at each port to ensure consistent dilution levels over the length of the manifold. Traditional analytical methods of solving for the hydraulic performance of manifolds involve making an initial guess about the flow conditions at the most downstream port, then using an iterative approach to progressively work upstream, port by port, until a final solution is reached. An alternative approach using simultaneous equations is presented in this section. The resulting set of equations can be quickly solved using a spreadsheet application with built-in equation solver. The spreadsheet can be set up to optimize port velocities by varying known quantities such as the port and manifold diameters and/or port



**Fig. 32.6** Schematic for a manifold or diffuser flow calculation

spacing. Once the final manifold configuration is selected, the same spreadsheet model can also be used to determine the resulting port velocities and outfall shaft fluid levels over a range of flow rates and/or effluent densities.

Consider the diffuser with  $n$  ports arranged along a tunnel manifold as shown in Fig. 32.6. Points  $P_1$  through  $P_n$  correspond to locations downstream of the individual port openings, while points  $T_1$  through  $T_n$  are located along the tunnel centerline, just upstream of the port with the same subscript  $i$ . Flow conditions through this system – or any similar system – can be solved using the set of simultaneous equations provided in Table 32.4. Note that the values in brackets in the 3rd and 5th columns indicate that there is an equal number of equations and unknown variables ( $8n + 1$ ). The parameters in the 6th column labeled *inputs* are assumed to be known values. The governing principles reflected in this set of equations are:

- **Continuity** – the sum of individual port flow rates ( $\sum Q_{P_i}$ ) must equal the total outfall flow rate ( $Q_T$ ). (Table 32.4, (8a))
- **Energy equation** – the total head at point  $P_n$  must be equal to the total head at point 0, less the total head loss between these two points ( $\sum \Delta H_{L(0 \rightarrow P_n)}$ ). (Table 32.4, (8j))
- **Parallel flow** – the total head loss between points  $T_{i+1}$  and

$$P_i \left( \sum \Delta H_{L(T_{i+1} \rightarrow P_i)} \right)$$

must be equal to the total head loss between points  $T_{i+1}$  and

$$P_{i+1} \left( \sum \Delta H_{L(T_{i+1} \rightarrow P_{i+1})} \right)$$

(Table 32.4, (8k)). Note that exit loss should be included in the expression for the head loss between  $T_i$  and  $P_i$  (Table 32.4, (8f)) because the velocity head has been fully dissipated by the time the discharge has reached any point  $P_i$ . The total head at all points  $P_i$  are equal.

Subscript  $P_i$  denotes individual ports, subscript  $T_i$  denotes individual tunnel manifold sections, subscript  $P_n$  denotes the most upstream port, and subscript  $T_n$  denotes the tunnel section between the outfall shaft and port  $P_n$  (Fig. 32.6).

In cases where loss coefficients  $K_{L_{T_i \rightarrow P_i}}$  in (8f) are functions of a flow ratio between the manifold and individual ports, the set of simultaneous equations in Table 32.4 may need several iterations in which adjustments are made to the loss coefficients after each iteration until the change in solution between successive iterations is negligible.

Note that the head loss coefficients in (8f) of Table 32.4 are presented in terms of the port exit velocity (i.e., the velocity at which the effluent is discharged into the sea). For cases in which the port geometry is more complex with varying diameters and multiple head losses, all loss coefficients used in (8f) of Table 32.4 must be converted such that they are based on the port exit velocity. To convert a head loss coefficient

**Table 32.4** Set of simultaneous equations for manifold flow calculations

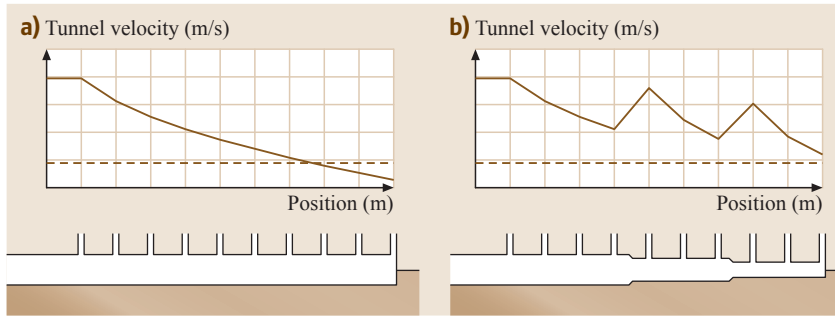
Description	Equations	No. of equations	Unknowns	No. of unknowns	Inputs	Equation
Continuity: total flow rate is equal to sum of individual port flow rates	$Q_T = \sum_{i=1}^n Q_{P_i}$	1	$Q_{P_i}$	$n$	$Q_T$	(8a)
Port exit velocity	$V_{P_i} = \frac{Q_{P_i}}{\frac{\pi}{4} D_{P_i}^2}$	$n$	$V_{P_i}$	$n$	$D_{P_i}$	(8b)
Velocity in manifold section between adjacent ports	$V_{T_i} = \frac{\sum_{j=1}^i Q_{P_j}}{\frac{\pi}{4} D_{T_i}^2}$	$n$	$V_{T_i}$	$n$	$D_{T_i}$	(8c)
Reynolds number in manifold section between adjacent ports	$Re_{T_i} = \frac{V_{T_i} D_{T_i}}{\nu}$	$n$	$Re_{T_i}$	$n$	$\nu$	(8d)
Darcy friction factor in manifold section between adjacent ports	$f_{T_i} = \frac{0.25}{\left[ \log \left( \frac{k_{s_i}}{3.7 D_{T_i}} + \frac{5.74}{Re_{T_i}^{0.9}} \right) \right]^2}$	$n$	$f_{T_i}$	$n$	$k_{s_i}$	(8e)
Head loss between manifold station $i$ and just downstream of port $i$	$\sum \Delta H_{L(T_i \rightarrow P_i)} = \frac{V_{P_i}^2}{2g} \left( \sum K_{L(T_i \rightarrow P_i)} \right)$	$n$	$\sum \Delta H_{L(T_i \rightarrow P_i)}$	$n$	$\sum K_{L(T_i \rightarrow P_i)}$	(8f)
Head loss between outfall shaft and just downstream of the $n$ -th port	$\sum \Delta H_{L(0 \rightarrow P_n)} = \frac{V_{P_n}^2}{2g} \left( f_{T_n} \frac{l_{T_n}}{D_{T_n}} + \sum K_{L(0 \rightarrow T_n)} \right) + \sum \Delta H_{L(T_n \rightarrow P_n)}$	1	$\sum \Delta H_{L(0 \rightarrow P_n)}$	1	$l_{T_n}$ $D_{T_n}$ $\sum K_{L(0 \rightarrow T_n)}$	(8g)
Head loss between manifold station $(i+1)$ and just downstream of port $i$	$\sum \Delta H_{L(T_{i+1} \rightarrow P_i)} = \frac{V_{P_i}^2}{2g} \left( f_{T_i} \frac{l_{T_i}}{D_{T_i}} + \sum K_{L(T_{i+1} \rightarrow T_i)} \right) + \sum \Delta H_{L(T_i \rightarrow P_i)}$	$n-1$	$\sum \Delta H_{L(T_{i+1} \rightarrow P_i)}$	$n-1$	$l_{T_i}$ $\sum K_{L(T_{i+1} \rightarrow T_i)}$	(8h)
Pressure just downstream of port $i$ , expressed in terms of the ambient seawater density	$p_{P_i} = (z_{SL} - z_{P_i}) \rho_a g$	$n$	$p_{P_i}$	$n$	$z_{SL}$ $z_{P_i}$ $\rho_a$	(8i)
Energy equation applied between point 0 and the most upstream port ( $n$ )	$\frac{V_0^2}{2g} + \frac{p_0}{\rho_a g} + z_0 = \frac{V_{P_n}^2}{2g} + \frac{p_{P_n}}{\rho_a g} + z_{P_n} + \sum \Delta H_{L(0 \rightarrow P_n)}$	1	$z_0$	1	$V_0$ $p_0$ $\rho_a$ $z_{P_n}$	(8j)
Parallel flow: head loss is the same between manifold station $i+1$ and just downstream of either port $i$ or port $i+1$	$\sum \Delta H_{L(T_{i+1} \rightarrow P_i)} = \sum \Delta H_{L(T_{i+1} \rightarrow P_{i+1})}$	$n-1$	-	-	-	(8k)
Total:		$8n+1$		$8n+1$		

based on some velocity into an effective loss coefficient based on another velocity, the following equation can be used,

$$\frac{V_1^2}{2g} K_{L_1} = \frac{V_2^2}{2g} K_{L'_1} \Rightarrow K_{L'_1} = \frac{V_1^2}{V_2^2} K_{L_1} \quad (32.8)$$

#### Manifold Section Diameters

The equations in Table 32.4 are kept general so that geometric parameters can be varied along the length of the manifold. In some cases, it may be necessary to progressively reduce the manifold diameter in order to maintain velocities that prevent settlement of solids



**Fig. 32.7a,b** Tunnel velocity variation along manifold – (a) constant diameter and (b) stepped diameter. The dotted line indicates a minimum self-cleansing velocity; refer to Sect. 32.3.7 for details

along the invert. (Due to the potential for excessively high head losses, it may not be possible to maintain sufficient self-cleansing velocities over the entire length of a conduit with a constant diameter.) Velocities through a typical diffuser are shown for two cases in Fig. 32.7 – constant diameter and stepped diameter. The dotted line represents a nominal minimum velocity required to prevent solids from accumulating. It should be noted that reducing the manifold section diameters is usually only feasible for applications involving piped outfalls. For tunneled solutions in which sedimentation can occur, an allowance for sedimentation build-up or a provision for periodic removal of accumulated sediments should be included in the design of the system. Refer to Sect. 32.3.7 for a discussion on sedimentation.

### Port Diameters

The number of ports as well as the port exit velocity required to ensure adequate dilution are typically determined from the results of near-field modeling. These constraints effectively set the port diameter which, in turn, establishes hydraulic performance of the system. After the port diameter has been selected, the system should be checked for excessively high head losses, unbalanced flow distribution, and seawater intrusion (for sewage outfalls only) to minimize the potential for adverse operating conditions. In practice, an iterative design process is usually required in order to strike an appropriate balance between dilution performance and favorable hydraulic conditions.

Achieving a consistent discharge velocity at all ports along a line diffuser is not necessarily a simple task due to the varying head loss coefficients along the length of the manifold. An analysis of head loss coefficients for dividing flows shows that they are a function of both the flow ratio and the area ratio, between individual ports and the corresponding conduit section [32.27]. In general, loss coefficients are greater for larger flow ratios and smaller area ratios. Assuming constant conduit and port diameters, upstream ports will therefore have relatively low loss coefficients while downstream ports will have relatively high loss coefficients.

In order to satisfy the principle of parallel flow such that head loss between the outfall shaft and each port is the same, the flow rate through each port will be different.

The flow variation among ports is not necessarily a problem from a hydraulic perspective. However, the near-field effluent dilution may become unbalanced if the flow differential among ports becomes too great. This condition may prevent near-field dilution targets from being met which could lead to adverse environmental consequences. In the case of sewage outfalls in which the effluent density is less than that of seawater, the variation in port discharges can create another undesirable condition – seawater intrusion. One way to prevent seawater intrusion from occurring is to use smaller port diameters to ensure adequate port discharge velocities. Modeling a less dense fluid flowing through an orifice into a more dense fluid shows that seawater intrusion can be prevented when the port densimetric Froude number is greater than approximately 1.6 [32.29],

$$Fr = \frac{V_{p_i}}{\sqrt{g \left( \frac{\rho_a - \rho_c}{\rho_c} \right) D_{p_i}}} > 1.6. \quad (32.9)$$

In practice however, port densimetric Froude numbers are typically kept well above this threshold value. For example, the port densimetric Froude number at the Sydney deep water ocean outfalls is of the order of 20–30.

A sensitivity analysis of port diameters using the set of simultaneous equations in Table 32.4 shows that port flow rates along the manifold do not vary significantly after port diameters are reduced below some critical value. Moreover, reducing the port diameter will increase the overall system head loss and lead to higher outfall shaft effluent levels. In some cases, the maximum possible head that can be accommodated at the outfall shaft may dictate the smallest allowable port diameters. If these diameters still do not lead to sufficiently high velocities, an alternative

configuration may be required. One option is to use variable-orifice nozzles such as the duckbill valves described in Sect. 32.4.3.

Another option is to use rosette style outfall structures. A similar analysis of flow conditions through a manifold configured with rosettes shows that port discharges can be more evenly distributed. This better flow balance occurs because head loss coefficients along the manifold are nearly uniform along its length, provided the friction losses and local losses between the conduit and risers are kept low. In this case, flow rates into each rosette will be nearly equal. Furthermore, the flow rate through each port on a given rosette structure will be the same for axisymmetric rosette designs because the total head loss coefficient will be the same regardless of which port the effluent flows through. Rosettes can have any number of nozzles, however, there tends to be an upper limit beyond which the additional nozzles will impede individual jet mixing processes. This behavior occurs because neighboring jets tend to coalesce due to reduced pressures caused by entrainment of the ambient water between them. The result is to reduce overall dilution performance. Near-field modeling can be used to determine the maximum number of nozzles for a specific rosette structure configuration.

### 32.4.3 Flow Variability

The wide range of flows that may be experienced at treatment plants is one of the greatest challenges in designing an outfall system. The preceding discussion assumes that the nozzle diameters have been sized to provide the required dilution for the maximum design flow rate, while at the same time maintaining sufficient velocity to prevent salt water intrusion at the ports and avoiding problems arising from sedimentation. In reality, treatment plants operate over a wide range of flows, and it is quite possible that the design flow might not be achieved until many years after the plant is initially constructed. In addition, plant start-up and ramp-down operations as well as any major changes to process flow requirements could create situations in which the plant will operate well below the design flow rate. The frequency and duration of any such events that result in low dilutions should be considered. If low flow scenarios are likely to result in sustained periods of insufficient dilution, the following mitigation schemes for managing low-flow scenarios can be used. Each of these options has advantages and disadvantages, and a hybrid solution may turn out to be the most suitable approach.

#### Supplemental Flows

For desalination plants, a connection between the intake and outfall systems can be incorporated in the plant

design to provide a way for seawater to be bypassed directly into the outfall system during periods of low outlet flow. The bypassed seawater effectively increases the outlet flow rate and nozzle exit velocity, thereby providing additional momentum for dilution by mixing. The bypassed seawater also acts as an initial diluting agent before effluent is discharged into the sea; as such, adequate levels of dilution may be obtained with lower nozzle exit velocities for a given brine flow. The amount of bypassed seawater required for any particular combination of outlet flow rate and salinity can be determined using results of the near-field modeling and basic conservation principles.

Although this option provides a near instantaneous level of control for managing the outfall system (when the appropriate flow control devices are included in the design), the addition of bypassed seawater will lead to increased energy use and operating costs due to additional pumping of seawater. These costs should be considered when deciding whether this option is an appropriate strategy for managing low flows. Depending on the frequency and amount of bypassing required, this solution may, in fact, be found to be the most practical or cost effective when compared with other alternatives. The incorporation of a seawater bypass system also provides a way to commission both the marine intake and outfall systems independently from the rest of the plant.

#### Staged Construction of Marine Works

Many treatment plants are designed to be initially operated at a low flow rate, then periodically upgraded until the maximum design capacity is attained. To manage the low flows associated with early stage plant operation, the marine construction works can be staged such that only some of the nozzles or rosette structures required for the ultimate flow case are initially installed. Additional nozzles, diffuser structures, or sections of a line diffuser can be installed each time the plant is upgraded; near-field modeling will inform the number of nozzles to be installed at each stage.

The main disadvantage with this solution is that additional marine construction works are required whenever the plant is upgraded. Moreover, it assumes that once the plant is upgraded to ultimate capacity, it will not operate at the lower early stage flows. This assumption may not be valid if demand can fall below the plant's maximum capacity after it is upgraded (for example, lower demand from a desalination plant due to a temporary period of increased rainfall). This solution also does not address periods of low flow that will occur even for ultimate capacity when the plant ramps up or shuts down due to operations or maintenance requirements. These periods of low flows may not be an issue if they occur infrequently and for short durations, and if



environmental licences allow for temporary periods of lower dilution. As such, plant processes and the feasibility of periodic marine construction works need to be considered should this option be pursued.

#### Nozzle Sizes and Blanks

If it is not feasible to stage the installation of marine outlet structures or sections of a line diffuser, another option for managing low flows is to install blanking plates at the nozzle openings. This solution provides greater flexibility than staged construction because each nozzle can be brought online or taken offline individually to suit the flow requirements. In addition, it allows the total flow rate to be evenly distributed since nozzles on multiple structures or at any location along a line diffuser can be blanked off. Near-field modeling is needed to confirm the number and location of nozzles to be blanked off for each long-term operating flow rate. Input from the plant operators should be sought when considering this option, as making adjustments to nozzle blanks more than two or three times per year may render this option infeasible.

A similar option is to install smaller diameter nozzles for the initial stages, then remove and replace with larger diameter nozzles each time the plant is upgraded. To minimize installation activities, each set of nozzles should be designed to have the same entrance diameter such that modification to the manifold or rosette structures is avoided. This option provides the same degree of flexibility as nozzle blanking if demand or process requirements lead to an extended period of low flow; the smaller nozzles can be re-installed at any time.

Although these options will also require marine works each time the plant is upgraded, the scope and duration of such works is likely to be considerably less than for staged installation of marine structures or the addition of more diffuser length. These options also carry some operational risk due to the fact that the plant could still be operated at any given flow rate, even with an incorrect number or size of nozzles. A nozzle configuration that is not aligned with the target operating flow could lead either to insufficient dilution (for lower flow), or excessively high head losses through the system (for higher flow). The operating procedures for these two options must ensure that plant capacity is linked to the installed nozzle configuration at any given time.

#### Variable-Orifice Nozzles

Variable-orifice nozzles, also known as *duckbill valves*, can be used instead of fixed diameter nozzles. These valves are made from a variety of flexible materials such as rubber or neoprene and are designed to close during

periods of no flow. Due to stiffness of the valve material, the valves gradually open in an elliptical shape as the flow rate increases resulting in higher velocities at lower flow rates as compared to fixed-diameter nozzles. This eliminates or reduces the need for blanking nozzles or introducing bypassed seawater into the outlet flows is reduced or eliminated. The check valve feature of duckbill valves also prevents backflow from the sea into the outfall system during periods of zero or low flow. Seawater intrusion into wastewater outfalls can therefore be prevented with this option.

As material stiffness dictates the degree to which the valves will open for any given flow rate, these valves are usually custom designed for each particular application. Hydraulic testing of the valves is recommended prior to selection and incorporation into the final design, given the specificity of duckbill valves to each outfall design. Hysteresis of the material should also be investigated to ensure that the valve will perform consistently over time. This consideration is particularly important if flows are expected to be highly variable and/or prevail for extended periods (i. e., months or longer). The local conditions in which the valves will be installed is another important aspect that should be taken into account when deciding whether duckbill valves are an appropriate solution. If high levels of marine growth are expected, the valves may not close properly if organisms attach themselves to the area near the valve opening.

### 32.4.4 Hydraulic Integration

Following a hydraulic assessment, the range of piezometric heads at the upstream boundary of an outfall system must be considered with respect to the hydraulics through the entire treatment plant. Failure to properly integrate the outfall system with other plant systems could result in adverse operating conditions such as decreased plant capacity, spillage and flooding, and inefficiencies for pumped systems. The choice between gravity-driven or pumped outfalls depends on several factors including site topography and plant elevation, management of low flow scenarios and long-term operating or maintenance costs.

#### Gravity Systems

Gravity-driven outfall systems are often preferred because they are relatively simple to operate (few electrical and mechanical components) and have virtually no long-term operating costs. Apart from periodic inspection and maintenance, the passive nature of a gravity-driven system ensures continuous reliability and minimal operator intervention. The free surface that exists at an outfall shaft is convenient when outflows from different parts of the treatment plant must be collected



and combined into a single outfall stream. In addition, a hydraulic break is usually required for certain process schemes such as removal of entrained air or settlement of suspended particles; refer to Sects. 32.4.5 and 32.4.6 for a discussion of these issues.

The range of piezometric heads in the outfall shaft should first be considered in relation to the vertical alignment of the outfall conduit and ground surface level at the shaft location. Given the complexity associated with installing an outfall conduit, the key factors in selecting its vertical alignment typically include suitability of ground conditions, constructability, cost, and construction scheduling. For example, a trenched solution which follows site topography and bathymetry is often desired as it can be the cheapest and easiest to build. However, this alignment may cause adverse hydraulic conditions such as hydraulic jumps and accumulation of air at localized high points. The potential for such conditions should be assessed before finalizing the outfall conduit alignment.

In general, the entrance to the outfall conduit should be fully submerged for all flow rates and sea levels in order to prevent operational instabilities. Otherwise, hydraulic jumps would be likely to occur at the downstream end of unpressurized steep sections or where free surface flow conditions develop along an alignment that follows a ground profile containing intermediate peaks and valleys, as illustrated in Fig. 32.8. Hydraulic jumps would likely be dynamic, unstable, and unpredictable, and could lead to air entrainment (Sect. 32.4.5 discusses the effects of air entrainment in greater detail).

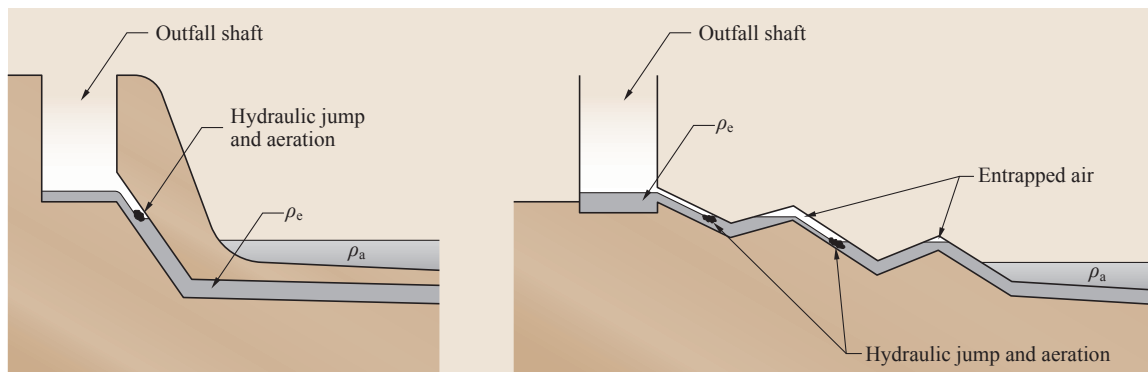
Regardless of whether the outfall shaft can be designed and constructed to accommodate the range of operating levels, these levels must also be compatible with the upstream treatment plant systems. Ideally, the plant is configured in such a way that shaft fluid levels do not influence the upstream hydraulics. At the very least, any backwater effect caused by the outfall shaft

levels should not adversely impact the treatment plant processes or reduce the plant operating capacity.

### Vertical Drops

For cases in which the treatment plant is located at a significantly higher elevation than the outfall shaft fluid levels, a means of transferring the outflows down a large vertical distance may be required. Allowing outlet flows to simply drop over a weir into the outfall shaft causes a great deal of air entrainment and results in rapid dissipation of energy which can cause operational instabilities and structural damage if not properly managed. A vortex, drop shaft system can be incorporated into the design using methods such as those outlined in *Hager* [32.30] in order to gradually dissipate energy and minimize impact on the structure, and reduce the potential for air entrainment.

As an alternative, a mini-hydroelectric scheme can be installed to recover the energy that would otherwise be *lost* by the vertical drop between the plant and the outfall shaft. Whether the recovered energy is used to power other parts of the plant or sold back to the power grid, a substantial portion of the costs associated with pumping of intake flows (e.g., at a desalination plant) may be recoverable. The potential cost savings associated with any such mini-hydro scheme should be compared with the initial cost of all necessary mechanical and electrical components, as well as their ongoing maintenance and replacement costs. The frequency and duration of flow/head combinations should also be considered when determining viability of a mini-hydro scheme. While a large amount of energy might be available for recovery during peak flow conditions, lower operating flow rates over the long term might render this option uneconomical. If a mini-hydro scheme is included, a passive system such as the vortex drop shaft may also be required as an emergency by-pass or back-up in case the turbines need to be taken offline.



**Fig. 32.8** Examples of configurations that could entrain or entrap air and cause hydraulic jumps

### Pumped Systems

If specific site conditions render a gravity-driven system unfeasible, a pumped solution will be needed to provide the driving head required for maintaining the required nozzle exit velocities. The hydraulic analysis for a pumped system is the same as for a gravity-driven system because the pumps will need to deliver flow at a head equal to the fluid level in a hypothetical outfall shaft placed at the pumping station location. Pump selection should be based not only on the required piezometric head on the discharge side of the pump (which is a function of both flow rate and sea level), but also on any flow- or time-based fluctuations of the upstream water level or pressure. Selecting a single type and size of pump to efficiently cover the entire flow and head range may not be possible. However, using a single type and size of pump is generally desired from an operations and maintenance perspective because it adds flexibility and redundancy to the system.

Pumping can also be used as a way to manage low flow scenarios or ensure constant nozzle exit velocity over the entire flow range. This option provides near instantaneous control for managing nozzle exit velocities, but the associated operating costs will also be relatively high. Nonetheless, a pumped outfall system may be required to manage low flows at a desalination plant outfall when bypassing seawater is not possible.

### 32.4.5 Air Entrainment

Managing air that becomes entrained in the treatment plant effluent stream is an important consideration for outfall system design. If air bubbles are not captured and released before effluent enters the outfall conduit, the bubbles could eventually agglomerate and form air pockets. Depending on the particular conduit configuration, air pockets can become trapped at localized high points along the outfall conduit and alter the hydraulic performance of the system. Localized head losses can be induced at these locations due to the associated flow

contractions and expansions. This condition could lead to reduced system capacity and the potential for excessively high outfall shaft water levels (in a gravity system) or inefficient pump operation (in a pumped solution). As it is typically not feasible to include air release mechanisms along an outfall conduit, trapped air pockets can remain in the system for extended periods of time.

Air pocket accumulation can also be problematic even if there are no high points along the alignment. In the case of a conduit with constant slope (including zero slope), entrained air will tend to come out of solution and accumulate along the conduit soffit. Depending on the conduit slope and diameter, and the effluent flow rate, the air pocket may move upstream or downstream. Regardless of direction, the air pocket may not start moving until it reaches some critical volume or pressure, at which point it may move suddenly and violently. This sudden release of air at either end of the conduit results in a condition called *blow-back* or *blow-out* which can cause strong vibrations and even structural damage in extreme cases. Operational instabilities in the outfall system and farther upstream may be experienced, especially in cases where the gradual accumulation of air followed by a sudden release (i.e., gulping), becomes repetitive.

It is important to recognize the processes by which air becomes entrained so that the system can be designed to minimize the adverse conditions described above:

- Flow conditions at the upstream end of the outfall conduit can be a major contributor to entrained air if the effluent is conveyed vertically through a drop shaft, over a weir, or by way of any other hydraulic structure which causes a jet to plunge through air.
- Hydraulic jumps can occur in open channels at locations where the cross-sectional flow area or depth changes abruptly. It is even possible to have hydraulic jumps occur within the outfall conduit if the upstream end is not fully submerged.
- For wastewater outfalls, gases can be generated over the entire conduit length due to biological processes in the effluent.

The two main approaches to removing entrained air are: (i) remove it upstream of the entrance to the outfall conduit, and (ii) configure the outfall conduit such that air bubbles either move upstream or downstream without forming large air pockets. A deaeration chamber or channel can be included immediately upstream of the outfall conduit entrance to capture and release air from the effluent in a controlled manner. The gen-

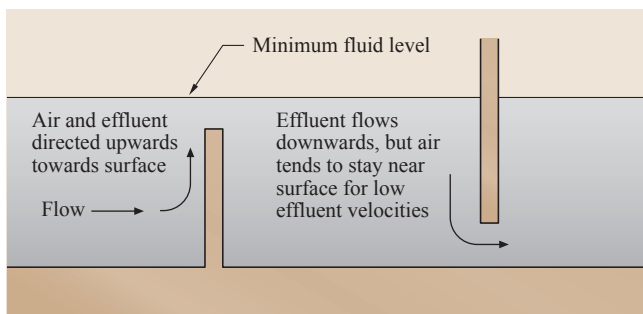


Fig. 32.9 Indicative deaeration system using baffles

eral idea behind these schemes is to provide a flow path of sufficient length such that air bubbles have enough time to rise to the surface before entering the conduit. Indicative air bubble terminal rise velocities are provided in *Falvey* [32.31] and *Lauchlan et al.* [32.32]. To minimize the deaeration chamber or channel flow path length, effluent velocities can be reduced by increasing the cross-sectional flow area. Properly configured baffle walls can also be used to direct flow upward and facilitate air removal through the fluid–air interface, or to permit only the effluent at the bottom of a chamber or channel to continue downstream toward the outfall conduit; effluent near the invert is likely to have less entrained air than that near the surface for low effluent velocities. Refer to Fig. 32.9 that shows an indicative deaeration system using a baffled configuration.

In other cases where site constraints limit the amount of space available for upstream deaeration schemes, or where a majority of entrained air is the result of biological activity occurring in the outfall conduit, an alternative approach can be taken. The conduit can be designed to allow air bubbles to travel to either the upstream or downstream ends of the system. *Falvey* [32.31] presents a series of curves which summarize a range of conditions for air bubble and air pocket movement in a closed and fully flowing conduit; refer to Fig. 32.10 in which  $Q$  is the conduit flow rate,  $D$  is the conduit diameter, and  $\theta$  is the angle of the conduit to the horizontal.

### 32.4.6 Sedimentation

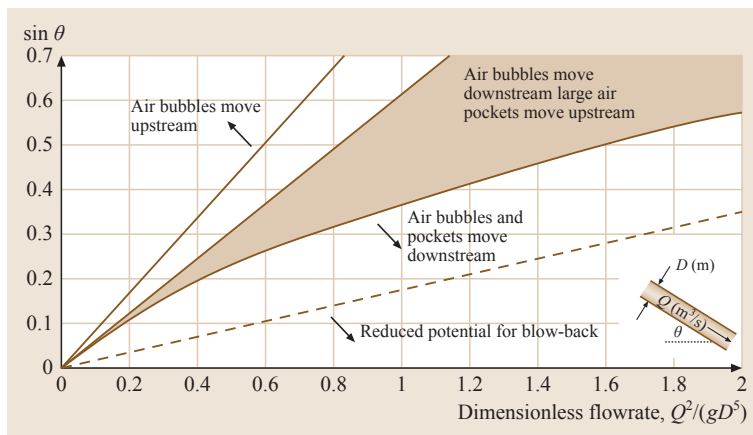
Sediment management tends to be a more relevant issue at wastewater treatment plants than at desalination plants; desalination inflow (i. e., seawater) generally has fewer solids than sewage, and its pretreatment processes generally remove a wider spectrum of material. Indeed, the filtration membranes in reverse osmosis plants are

very sensitive to fouling and require the removal of solids as well as organic compounds. Nonetheless, the solid matter removed in the desalination pretreatment processes may end up being added to the brine effluent downstream of the reverse osmosis filters if not disposed of by other means. The design of an outfall conduit for a desalination plant must therefore consider all plant processes that discharge an effluent stream into the outfall system.

The build-up of granular sediments, organic matter, and other solids along the invert of an outfall conduit will eventually lead to increased head losses and decreased hydraulic capacity. In extreme cases, the conduit section can become clogged and the function of the discharge nozzles impeded. There are two general approaches to reduce the risk of sediment build-up, each of which is described below. For some applications, a combination of these options may prove to be the most effective solution.

#### Minimize the Amount of Solids that Enter the Conduit

By providing a way for solids to settle out of the effluent stream upstream of the conduit entrance, deposition and removal becomes a matter that can be managed locally at the plant site, rather than along the entire length of the conduit. This approach using sedimentation basins or settling tanks may be the preferred solution for cases in which the outfall conduit is long and/or inaccessible for periodic inspections and cleaning. It may also be used if it is not possible to design the conduit to achieve self-cleaning velocities. The general design principle behind this approach is that the sedimentation basins/tanks reduce the effluent stream velocity to a point where the time it takes for suspended particles to settle is less than the residence time of the fluid in the basin or tank. Because particle settling time increases as the target particle size decreases, a potential drawback of this



**Fig. 32.10** Air movement in pressurized conduits (after *Falvey* [32.31, Fig. 29])

option is that a large basin or tank footprint may be required. As such, this option may only be feasible for removing larger solids. To decrease the basin/tank footprint and increase its efficiency, internal baffle walls can be installed to lengthen the flow path. However, the resulting effluent velocity should remain sufficiently low to allow sediments to settle out.

#### Design for Self-Cleansing Velocities

If upstream sedimentation control is not feasible, the outfall conduit diameter can be selected to yield velocities that are high enough to prevent deposition of solid materials. Given the wide flow variability associated with many types of treatment plants, the outfall conduit diameter should be based on the lowest-flow

scenario. If, however, the chosen diameter leads to excessively high velocities and head losses during peak flow, it may not be possible to achieve self-cleaning velocities for all flow conditions. In this case, the design objective should aim for self-cleaning velocities on a frequent basis (i. e., at least once per day) and for sustained periods of time (i. e., hours) in order to prevent build-up of sediments and organic matter over time. The research literature contains a great deal of information on the velocities and shear stresses required to move solids through the conduit and to re-suspend solids that may have already settled. Velocities to keep solids in suspension range from approximately 0.5–2.0 m/s depending on conduit diameter, particle size and specific gravity, and particle concentration [32.33].

## 32.5 Outfall Construction

An effective design is not possible without input from those engaged to construct the outfall. The design must be detailed in conjunction with how the marine outfall will be built. *Grace* [32.34, 35] and *Wood et al.* [32.4] provide good information on outfall construction approaches and techniques.

There are two fundamental types of wastewater delivery: via tunnels or pipes (either on trestles or in trenches). They can be used in combination, a tunnel for the bulk of the outfall length and a pipeline on the sea bed comprising the outlet ports. The decision is often governed by the geography of the region – tunnels may be preferred in areas with rocky coastlines or where beach access is difficult, while pipelines may be preferred in areas with easy access such as a sandy shoreline.

### 32.5.1 Construction Materials

The materials used for outfall construction include steel, reinforced concrete, and high density polyethylene. The marine environment is corrosive and steel structures must be protected by coating the steel to prevent direct contact with sea water (often concrete is used) and/or using sacrificial anodes. Ongoing inspections are required to monitor the integrity of the protective coating or the anodes and replace them as necessary.

Steel reinforcement in concrete is susceptible to corrosion by chloride salts in marine waters. To help in preventing such corrosion, low permeability concrete is used, often in conjunction with additives that inhibit corrosion. Hydrogen sulphide in wastewater is corrosive to concrete and concrete pipes are lined (usually with plastic) for protection. Regular inspections are undertaken to ensure the lining has not peeled away from

the concrete. The density of concrete is about one-third that of steel (although still about double that of seawater) and the concrete pipeline may need to be anchored to the sea floor.

High-density polyethylene is relatively light, with a density slightly less than that of seawater. Its big advantage is that it is flexible and can be relatively easy to deploy. Long sections are welded together on shore, the pipeline then towed into position and anchored to the sea floor. Purging of air from the pipe is critical to prevent the pipe from floating back to the surface.

### 32.5.2 Construction Methods

Pipelines are often manufactured in sections and assembled at an access point close to the proposed line of the outfall. When in the water, each section is buoyed to facilitate easy movement. Offshore, a vessel is anchored and pulls a section of the pipe into a trench along the line of the outfall. The next section is welded on and the pull continues. This is a popular technique for steel pipelines, which are very strong in tension. Concrete blocks or ballast rock may be used to anchor the pipe to the sea floor.

The trench is then backfilled either mechanically or by natural means. Techniques have been devised whereby trenching, pipe-laying and backfilling are all done in a single operation. The end of the pipeline terminates in a diffuser and the wastewater is discharged through outlet nozzles.

The active wave climate of the surf zone makes it the most critical region. Often, a temporary trestle and sheet piling is used to protect the pipeline through this region. Sometimes, wave action can damage the trestle and sheet piling (Fig. 32.11).

a)

b)

**Fig. 32.11a,b** Photographs from the same site, during (a) and after (b) a storm that caused extensive damage to sheet piling. The sheet piling was placed to protect the outfall pipeline (located in a trench) as it passes through the surf zone

Tunnels are expensive to construct but their big advantage over pipelines is that they avoid the surf zone where damage to a pipeline is more likely to occur. This is particularly important along high wave energy coastlines.

One of the two tunnel construction techniques is usually adopted: full face tunnel boring or drill and blast. The latter technique is more commonly used on shorter outfalls where the cost of a tunnel boring machine cannot be justified. The drill and blast technique can be dangerous, releasing gases into the tunnel or weakening the tunnel structure.

Long risers (tens of metres or more in length) are drilled from the sea floor down to the tunnel. The risers are capped with high velocity nozzles through which the wastewater is discharged. On the three Sydney deep water outfalls, between four and eight outlet nozzles are fitted in a rosette configuration to each riser.

### 32.5.3 Some Considerations

#### Environmental Impacts of Construction

During the construction of a marine outfall there will inevitably be environmental damage caused by trenching and drilling activities. Habitats may be removed,

infauna displaced and particulate material placed into suspension. Apart from the obvious physical destruction of habitats, suspended matter may reduce light in the water column (affecting photosynthesis), clog the gills of fish and, on resettling, may smother marine plants and infauna.

#### Route selection

The shortest distance may not be the optimal route for the outfall. Other influencing factors include: the presence of rocky outcrops, wave and current climate, maritime activity, fishing zones, and ecological considerations. The route of the Wollongong outfall (Australia), constructed in 2005/2006, was changed in its early design phase to avoid the habitats of the weedy seadragon (*Phyllopteryx taeniolatus*) – a local protected species.

#### Wave and Current Climate

Waves can induce drag and lift forces on pipelines. Breaking waves are most prevalent in the surf zone. They can produce very large forces on a pipeline, although their duration is short lived. Abnormally large waves can be generated from storm surges, rogue waves and tsunamis. Currents over a pipeline exert a drag force. The weight of the pipeline needs to be sufficient to prevent such forces from moving the pipeline. Grace [32.34] provides some simple calculations that estimate the force exerted on a pipeline by waves and currents. In part, this problem can be largely overcome by burying the pipeline in a trench, which is then back-filled.

#### Sediment Movement

The current and wave climate may resuspend sediments. Once in suspension, their movement and distribution can be widespread. Of particular importance is the potential for sediments to erode from beneath the pipeline, potentially placing considerable stress on the pipeline itself. Conversely, an accretion of sediments may smother the pipeline inhibiting or preventing the discharge of wastewater from the outlet ports. This may also cause sediments to enter the outfall pipeline, reducing the efficiency of the outfall.

#### Head Loss Monitoring

The hydraulic grade line for an outfall can be determined as a function of the discharge. At each discharge, upper and lower limits for the hydraulic grade line can be established and by maintaining the discharge between these limits, we can optimize the hydraulic performance of the outfall. Below the lower limit, seawater intrusion into the outfall is likely to occur and operators can increase the wastewater flow accordingly (although



increasing the flow may not always be possible). The addition of salinity sensors on individual risers can confirm whether seawater intrusion has occurred.

#### Seawater Intrusion and Purging

A rule-of-thumb for preventing seawater from entering the outfall is to keep the port densimetric Froude number (32.9) well above unity (e.g., on Sydney's deepwater ocean outfalls, the port densimetric Froude number is of the order of 20–30). Occasionally, the port densimetric Froude number may become small and seawater may enter the outfall. One method to help prevent such an intrusion of seawater is to use nonreturn check valves. However, not all outfalls are so equipped and it may be necessary to periodically purge the seawater from the outfall. This is done by backing up the wastewater in the treatment plant and releasing the wastewater

at high velocity. This also has the advantage of clearing the outfall of sediments.

#### Outfall Maintenance

Regular inspections of the marine outfall should be carried out as part of a regular maintenance program to ensure there is (a) no physical damage to any components and (b) no blockage of any outlet nozzles. This can be carried out by divers although this is accompanied by substantial health and safety concerns (e.g., diving in contaminated waters, wave action, and decompression requirements for deep diving). Our preference is to carry out inspections using remotely operated vehicles. The frequency of maintenance inspections will depend on a range of factors such as environmental conditions and construction materials. Often, annual inspections are adopted.

## 32.6 Environmental Monitoring

The fundamental objective of environmental monitoring is to quantify impacts that may arise as a result of the discharge of wastewater to the marine environment. Some of the questions that an environmental monitoring program should address include:

- Is it safe to swim?
- Is it safe to eat the seafood?
- Will the marine communities be protected into the future?
- Will the beaches be free from contamination?

Two distinct environmental monitoring programs are often implemented. The first is a pre- and postconstruction monitoring program. This is usually an intense program of short duration (perhaps five years) aimed to quantify the initial impacts of the discharges from the marine outfall. The second is a long-term (ongoing) monitoring program, which usually takes a subset of the information obtained from the pre- and postconstruction monitoring program, and uses it to define acceptable limits of change in the environmental indicators. Long-term monitoring is designed to determine whether a subsequent change lies outside these limits.

There is an assumption, often tacit, that the engineering aspects of a sewage treatment system and marine outfall are operating as designed. If there is a treatment bypass, breakdown or blockage of some of the outlet ports, the quality of the effluent (or its delivery to the marine environment) will be less than expected. Monitoring the operational performance is an important element of environmental monitoring.

### 32.6.1 Change Versus Impact

The marine environment is in a state of continual flux. The challenge of a monitoring program is to separate change that occurs via natural processes from change that is a direct result of the discharge from the marine outfall (i.e., impact). It is critical to define what constitutes an impact prior to the execution of the post-commissioning monitoring program. This places clear bounds on the interpretation of change that may be observed in the marine communities.

### 32.6.2 Pre- and Postconstruction Monitoring

Broadly, the pre- and postconstruction monitoring program compares conditions before and after the marine outfall is commissioned. It is critical that sufficient time is allowed to carry out the *before* monitoring. An *after* monitoring program can be conducted at any time after commissioning of the marine outfall. However, we only get one chance at the *before* phase of the monitoring program.

A monitoring program may be of several years' duration. Consequently, the cost of an environmental monitoring program may be large – perhaps 5% of the construction costs may be needed to properly address the main environmental issues.

#### Monitoring Philosophy

Some of the characteristics that constitute a good monitoring program are outlined below. These characteris-



tics combine to define our philosophy for environmental monitoring.

Environmental monitoring programs should be designed to establish cause-and-effect between the discharge and the environmental response. The approach we favor uses *weight-of-evidence*. This simply means applying different techniques and approaches to the same problem. If the answers that they provide are consistent with what might be expected from such discharges, then we have greater confidence in the overall result. Our weight-of-evidence is a three-pronged attack, which is detailed in the following section.

Most countries have guidelines (or licence conditions) that reflect the values that need to be protected. These values include social, public health and environmental aspects. While some values may be subjective, many can be quantified in terms of *safe* levels of contaminants that can be discharged. There is a tacit assumption, that by adhering to these concentration limits, there will be no irreversible damage done to the marine environment.

The most critical task in undertaking a monitoring program to quantify environmental impacts of a marine outfall is to ask the right questions. This requires an understanding of what is being discharged, what effects these discharges are likely to have on the marine environment and the level of change that we are willing to accept. An answer to the wrong question, no matter how accurate is that answer, will not allow impacts to be quantified.

Environmental monitoring programs must be scientifically robust and defensible. We must test a hypothesis and gather empirical evidence to support (or disprove) the hypothesis. The experiment must be repeatable, cover a range of conditions and, in theory, repeated experiments should arrive at the same conclusion. Marine outfalls may be contentious and the monitoring program may need to be defended. One way in which this can be done is via peer review of the work and publication of the results in a reputable journal.

Conditions of approval for a marine outfall are often reliant on predictions. The objectives of an environmental monitoring program should include the verification (or otherwise) of those predictions and whether the environmental values have been maintained. Further, the program should include a mechanism by which problems identified in the monitoring program can be rectified or mitigated.

### The Three-Pronged Attack

An approach to collecting weight-of-evidence for impact assessment includes three components. These are detailed below.

**Sources of Contaminants.** There are many potential sources of contaminants discharged to marine waters, including rivers, sediments (as both a source and sink), private and industrial outfalls, illegal dumping, and coastal wastewater treatment plants. An important consideration to environmental monitoring is separating the relative contribution of an outfall from contributions from the other sources. It may be possible to characterize (and isolate) the different sources via the types, concentrations, and variability of contaminants being discharged from each source.

Marine organisms may be impacted if exposed to wastewater. Toxicity testing is a technique used to determine the concentration of substance which is likely to harm to marine organisms. Whole effluent toxicity (WET) testing examines whether a toxic response is identified when an organism is subjected to the complex wastewater matrix.

If toxicity testing is carried out using a single test organism, it is then inferred that all marine organisms will exhibit the same toxic response. This is not necessarily the case and multiple organisms are recommended for toxicity testing: different organisms display different responses to different substances at different stages in their life cycle. Ideally, a range of fish, invertebrates, and algae at different stages in their life cycle should be considered for toxicity testing. Toxicity tests can be either acute or chronic and both types of tests should be included. The former determine the concentration of wastewater that is lethal to the test organism. Chronic testing examines the reduced capability (e.g., impaired development or reproductive ability) of the test organism.

**Movement of Wastewater.** Once we know what is being discharged, we determine the path that the wastewater will take after discharge and how dilute it becomes. *Emery and Thompson* [32.36] describe many techniques to measure and/or monitor the physical properties of the marine environment.

Instrumented moorings provide excellent temporal coverage although the horizontal spatial coverage is limited by the number of such systems that the monitoring program can afford to deploy. Remote sensing provides good spatial coverage of the region, although in general, the temporal scales are much longer than those associated with the movement of wastewater plumes.

Numerical modeling enables us to predict how the wastewater will react to different marine conditions and to changes in the volume or method of discharge. It is critical to calibrate and validate any numerical model to ensure confidence in the results under existing conditions and for future scenarios. Coupled with this is

a need for confidence limits to be placed on model results. Too often model results are provided without a real appreciation for the confidence that can be placed on those results.

**Quantifying Change.** There are a number of approaches to quantifying change in response to a marine outfall. One favored by the authors is the *beyond BACI* or *mBACI* approach [32.37]. BACI is an acronym for *before, after, control, impact*. Sites close to, and remote from, the outfall are monitored on several occasions both before and after commissioning of the marine outfall. Perhaps the main reason that this is a favored approach is its emphasis on statistical power and statistical error.

**Statistical Errors.** A Type I statistical error occurs when the results of our analysis incorrectly predicts that a change has occurred. We can protect against making such a mistake by specifying the level of significance. Usually this is set at 5% (equivalent to 95% confidence limits), i.e., there is a 5% chance that our analysis makes a Type I statistical error.

A Type II statistical error occurs when the results of the analysis incorrectly predicts that a change has not occurred. From an environmental point of view, this is more insidious than a Type I error because it incorrectly leads us to believe that there is no environmental problem. It is difficult to protect against a Type II error as this requires *a priori* knowledge of the variability in the system being measured. Such knowledge is acquired only after the monitoring is complete. Therefore, we need to use experience to estimate the system variability and design experiments accordingly. To protect against making a Type II statistical error, we usually design our experiments to target statistical power above 80%. However, if statistical power is too high, very small changes become statistically significant and we question whether such small changes are meaningful. It is critical to check the statistical power after the experiments have been completed.

Type III statistical errors occur when we arrive at the correct answer but have asked the wrong question.

Some of the marine components measured to detect change as a result of a new marine outfall include water quality, sediment quality and community studies (such as intertidal, plankton, pelagic, benthic, and sessile communities). Marine community studies should be tailored for specific outfalls and might include:

- Intertidal community studies for shoreline or short outfalls.
- Subtidal settlement panel studies for marine outfall in waters less than about 20 m depth (much deeper than this and the light attenuation starts to inhibit the growth of organisms on the settlement panels).
- Fish, shellfish, and planktonic communities move in, and with, marine waters. The variability among control sites may be as much as the variability between putatively impacted sites and control sites. Therefore, it may be difficult to isolate the marine outfall as the cause of change in such communities.
- Sediments have been used to assess the accumulation of contaminants and to examine infauna community variations. Our experience is that such studies have limited success. Unless the contaminant signal is very strong, it is unlikely to register in a sediment sample.
- Bioaccumulation studies are sometimes recommended as impact assessment indicators. However, caution is needed when interpreting the results of such studies because:
  - Fish move, and it is not always possible to know the region from where contaminants were accumulated. This necessitates fish home range studies, which can be expensive and, perhaps, inconclusive.
  - Species that are caught at one particular time and location may not be caught at other times or locations and pooling into higher biological levels may be required. Different species may accumulate different substances at different rates and the pooling process may mask potential impacts.
  - Moored systems comprising oysters or mussels are often used in bioaccumulation studies. However, this may involve removing the animals from their natural habitat thereby, adding stress to the organisms and confounding the results that are obtained.

### 32.6.3 Long-Term Monitoring

The pre- and postconstruction monitoring program will identify whether there has been a step change in the baseline conditions as a result of the discharge from the marine outfall. Long-term monitoring is used to identify whether further change occurs well after the marine outfall has been commissioned. It can be used to extrapolate trends and, where necessary, design and implement an appropriate mitigation strategy to prevent, or reverse, the trend. The three-pronged attack described in Sect. 32.6.2 is also applicable to long-term monitoring.

However, there is a temptation to simply implement the bulk of the pre- and postconstruction monitoring program as part of the long-term monitoring program. Apart from the expense associated in maintaining a de-

tailed monitoring program over a long period of time, environmental changes will be masked by the small time steps between consecutive sets of readings. It may be more effective to implement a low level, long-term monitoring program and revisit the detailed monitoring program, for example, over two consecutive years every 10 yr. If the long-term program indicates a potential problem, it provides the motivation for a more detailed investigation.

Where such an approach is adopted, source characterization combined with numerical modeling are used to estimate environmental impact. If the results indicate a possible impact, confirmation studies can be implemented.

### 32.6.4 Summary

This chapter provides a synopsis of the discharge of wastewater to the marine environment. Flows of effluent from municipal sewage treatment plants and brine from desalination plants are primarily considered within the context of near-field modeling and outfall hydraulics. The designers of outfalls are under increasing pressure from social, public health and environmental constraints, within a regulatory and economic framework, all of which need to be considered. This compact overview exposes the reader to fundamentals of outfall design and identifies some of the traps and problems that may arise.

### References

- 32.1 P.M. Tate, J.H. Middleton: Buoyant jets of elliptic shape: Approximation for duckbill valves, *J. Hydraul. Eng.* **130**(5), 432–440 (2004)
- 32.2 Metcalf & Eddy Inc., G. Tchobanoglous, F.L. Burton, H.D. Stensel: *Wastewater Engineering: Treatment and Reuse*, 4th edn. (McGraw-Hill, New York 2003)
- 32.3 P. Palomar, I.J. Losada: Impacts of brine discharge on the marine environment. Modelling as a predictive tool. In: *Desalination, Trends and Technologies*, ed. by M. Schorr (InTech, Rijeka 2010) pp. 279–310
- 32.4 I.R. Wood, R.G. Bell, D.L. Wilkinson: *Ocean Disposal of Waste* (World Scientific, Singapore 1993)
- 32.5 J.H. Lee, V. Cheung: Generalized Lagrangian model for buoyant jets in current, *J. Environ. Eng.* **116**(6), 1085–1106 (1990)
- 32.6 P.M. Tate, J.H. Middleton: Unification of non-dimensional solutions to asymptotic equations for plumes of different shape, *Boundary-Layer Meteorol.* **94**(2), 225–251 (2000)
- 32.7 G.H. Jirka, P.J. Akar: Hydrodynamic classification of submerged multiport-diffuser discharges, *J. Hydraul. Eng.* **117**(9), 1113–1128 (1991)
- 32.8 G.H. Jirka, R.L. Domeker: Hydrodynamic classification of submerged single-port discharges, *J. Hydraul. Eng.* **117**(9), 1095–1112 (1991)
- 32.9 B. Morton, G. Taylor, J. Turner: Turbulent gravitational convection from maintained and instantaneous sources, *Proc. R. Soc. A* **234**(1196), 1–23 (1956)
- 32.10 L.-N. Fan: *Turbulent Buoyant Jets into Stratified or Flowing Ambient Fluids*, Technical Report (CALTECH, Pasadena 1967)
- 32.11 C.C. Lai, J.H. Lee: Mixing of inclined dense jets in stationary ambient, *J. Hydroenviron. Res.* **6**(1), 9–28 (2012)
- 32.12 P.J. Roberts, A. Ferrier, G. Daviero: Mixing in inclined dense jets, *J. Hydraul. Eng.* **123**(7), 693–699 (1997)
- 32.13 A. Cipollina, A. Brucato, F. Grisafi, S. Nicosia: Bench-scale investigation of inclined dense jets, *J. Hydraul. Eng.* **131**(11), 1017–1022 (2005)
- 32.14 S. Nemlioglu, P. Roberts: Experiments on dense jets using three-dimensional laser-induced fluorescence (3DLIF), *Proc. 4th Int. Conf. MWWDD, Antalya* (2006)
- 32.15 G. Kikkert, M. Davidson, R. Nokes: Inclined negatively buoyant discharges, *J. Hydraul. Eng.* **133**(4), 545–554 (2007)
- 32.16 D. Shao, A.W.-K. Law: Mixing and boundary interactions of 30 and 45 inclined dense jets, *Environ. Fluid Mech.* **10**(5), 521–553 (2010)
- 32.17 I.G. Papakonstantis, G.C. Christodoulou, P.N. Papanicolaou: Inclined negatively buoyant jets 1: Geometrical characteristics, *J. Hydraul. Res.* **49**(1), 3–12 (2011)
- 32.18 I.G. Papakonstantis, G.C. Christodoulou, P.N. Papanicolaou: Inclined negatively buoyant jets 2: Concentration measurements, *J. Hydraul. Res.* **49**(1), 13–22 (2011)
- 32.19 V.H. Chu: L.N. Fan's data on buoyant jets in cross-flow, *J. Hydraul. Div.* **105**(4), 612–617 (1979)
- 32.20 P.J. Roberts, H.J. Salas, F.M. Reiff, M. Libhaber: *Marine Wastewater Outfalls and Treatment Systems* (IWA Publishing, London 2010)
- 32.21 B. Henderson-Sellers: *Modeling of Plume Rise and Dispersion-The University of Salford Model: USPR*, Lecture Notes in Engineering, Vol. 25 (Springer, Berlin Heidelberg 1987)
- 32.22 H.B. Fischer, E.J. List, R.C.Y. Koh, J. Imberger, N.H. Brooks: *Mixing in Inland and Coastal Waters* (Academic, New York 1979)
- 32.23 D.L. Wilkinson: Optimal design of ocean outfalls. In: *Environmental Hydraulics*, ed. by C.C. Lai, J.H. Lee (Balkema, Rotterdam 1991) pp. 275–279
- 32.24 S.I. Seneviratne, N. Nicholls, D. Easterling, C.M. Goodess, S. Kanae, J. Kossin, Y. Luo, J. Marengo, K. McInnes, M. Rahimi, M. Reichstein, A. Sorteberg, C. Vera, X. Zhang: Changes in climate extremes and their impacts on the natural physical environment. In: *Managing the Risks of Extreme Events and Disasters to Advance Climate Change Adaptation*, ed. by C.B. Field, V. Barros, T.F. Stocker, D. Qin, D.J. Dokken, K.L. Ebi, M.D. Mas-

- trandrea, K.J. Mach, G.-K. Plattner, S.K. Allen, M. Tignor, P.M. Midgley (Cambridge Univ. Press, Cambridge 2012) pp. 109–230
- 32.25 M.H. Sharqawy, J.H. Lienhard, S.M. Zubair: Thermophysical properties of seawater: A review of existing correlations and data, *Desalin. Water Treat.* **16**(1–3), 354–380 (2010)
- 32.26 J. Pitt, P. Ackers: *Hydraulic Roughness of Segmentally Lined Tunnels* (CIRIA, London 1982)
- 32.27 D.S. Miller: *Internal Flow Systems* (Miller Innovations, Bedford 2009)
- 32.28 I. Idelchik: *Handbook of Hydraulic Resistance* (Jaico, Mumbai 2003)
- 32.29 D.L. Wilkinson: Avoidance of seawater intrusion into ports of ocean outfalls, *J. Hydraul. Eng.* **114**(2), 218–228 (1988)
- 32.30 W.H. Hager: *Wastewater Hydraulics: Theory and Practice* (Springer, Berlin Heidelberg 2010)
- 32.31 H.T. Falvey: *Air–Water Flow in Hydraulic Structures*, Technical Report (Water and Power Resources Service, Denver 1980)
- 32.32 C.S. Lauchlan, R.W.P. May, R. Burrows, C. Gahan: *Air in Pipelines: A Literature Review* (HR Ltd., Wallingford 2005)
- 32.33 D. Butler, R.W.P. May, J.C. Ackers: Sediment transport in sewers part 1: Background, *Proc. ICE–Water Marit. Energy*, Vol. 118 (1996) pp. 103–112
- 32.34 R.A. Grace: *Marine Outfall Systems. Planning, Design and Construction* (Prentice–Hall, Upper Saddle River 1978)
- 32.35 R.A. Grace: *Marine Outfall Construction: Background, Techniques, and Case Studies* (ASCE, Reston 2009)
- 32.36 W.J. Emery, R.E. Thomson: *Data Analysis Methods in Physical Oceanography* (Pergamon, Oxford 1998)
- 32.37 A. Underwood: Beyond BACI: Experimental designs for detecting human environmental impacts on temporal variations in natural populations, *Mar. Freshw. Res.* **42**(4), 569–587 (1991)
- 32.38 32nocite]Please cite this reference in the text. Otherwise it must be deleted. M.S. Field, R.G. Wilhelm, J.F. Quinlan, T.J. Aley: An assessment of the potential adverse properties of fluorescent tracer dyes used for groundwater tracing, *Environ. Monit. Assess.* **38**(1), 75–96 (1995)
- 32.39 C. Leibundgut, S. Hadi: A contribution to toxicity of fluorescent tracers. In: *Tracer Hydrology*, Vol. 97, ed. by A. Kranjc (Balkema, Rotterdam 1997)

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