Chapter 3 Design of Ocean Wastewater Outfall Systems

Objectives:

- find the solution for siting and initial dilution of wastewater from sewage treatment plant and heated water (thermal effluent) from coastal power plant
- analyze mixing characteristics of the multiport diffuser that minimizes detrimental effects of the discharge on the environment
- review all facets of the design of a multiport diffuser as an efficient way of maximizing initial dilution in order to meet regulatory requirement

3.1 The Design Process

3.1.1 Ocean Outfall

A. Categorization

By object

1) Watsewater outfall:

- discharge of effluent from sewage treatment plant

2) Thermal outfall:

- discharge of heated water from power plant
- discharge of cooled water from LNG plant
- discharge of brine water from desalination plant

By discharge type

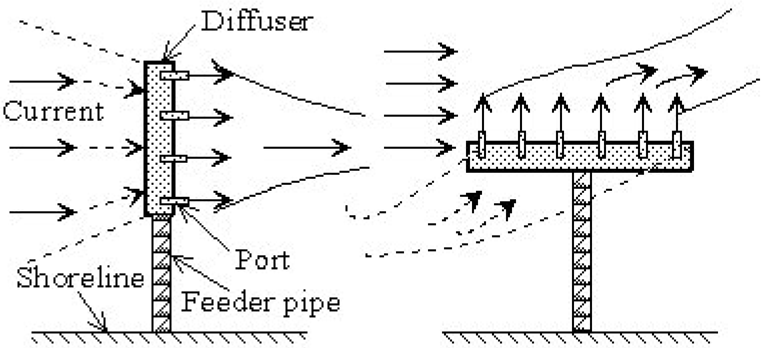
1) Surface discharge: open channel, duct

2) Submerged discharge

   - Singleport diffuser
   - Multiport diffuser:
     - Uni-directional diffuser:
       - Co-flowing diffuser
       - Tee diffuser
     - Staged diffuser
     - Alternating diffuser

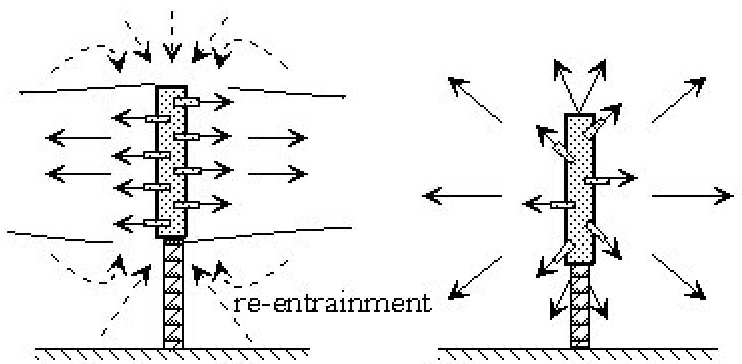
Comparison of Typical Sewage and Thermal Diffuser (after Jirka, 1982)

	Sewage Diffuser	**Thermal Diffuser**
Total Discharge (m^3/s)	8	80
Relative Density Deficit	0.025	0.0025
Total Momentum Flux (m^4/s^2)	40	400
Dilution Requirement	> 100	< 10
Ambient Depth (m)	50	10



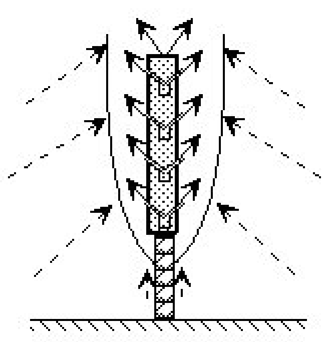
(i) Co-flowing Diffuser **(ii) Tee Diffuser**

a) Uni-dirctional Diffuser (with Crossflow)

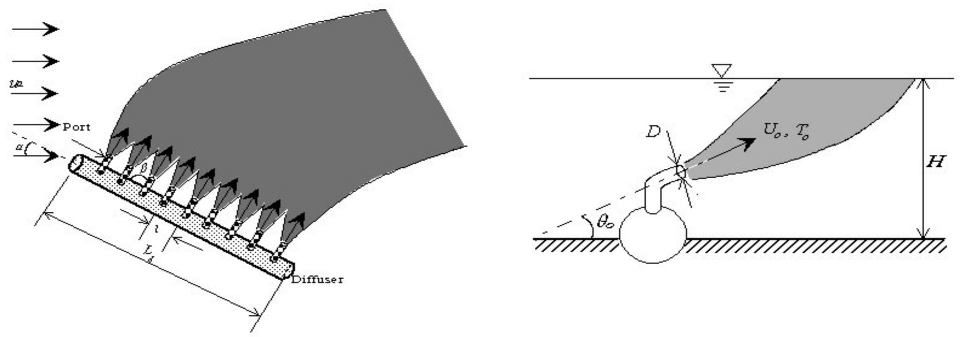


(i) Nozzles at 90° to Diffuser Line **(ii) Logarithmic Nozzle Distribution**

b) Alternating Diffuser

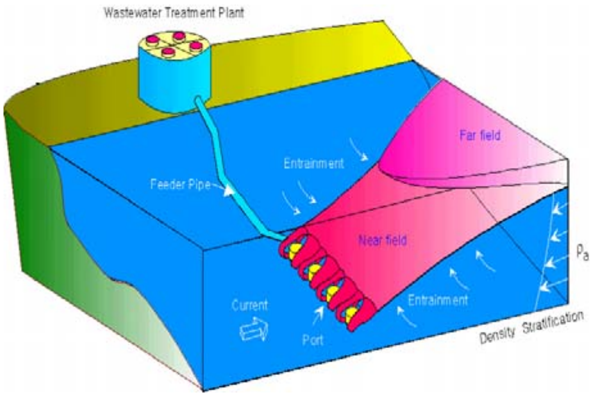


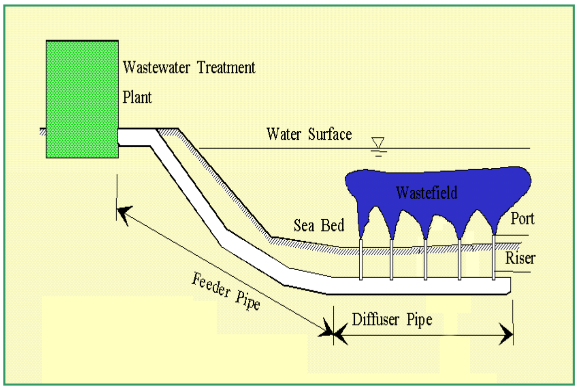
c) Staged Diffuser



a) Plan View b) Side View

Type		Diffuser alignment, α	Nozzle alignment, β	Nozzle angle, θ_0	Induced flow	Favorable conditions
Uni-directional	Co-flowing diffuser	90^0	90^0	$0\text{-}20^0$	Back	Unidirectional current
	Tee diffuser	0^0	90^0	$0\text{-}20^0$	Back, Side	Stagnant
	Oblique diffuser	$0\text{-}90^0$	90^0	$0\text{-}20^0$	Back, Side	Both
Alternating diffuser		90^0	$0\text{-}90^0$	$0\text{-}20^0$	No horizontal momentum	Bi-directional current
Staged diffuser		90^0	$0\text{-}20^0$	$0\text{-}20^0$	Offshore momentum	Bi-directional current


Wastewater Treatment Plant
Far field
Entrainment
Feeder Pipe
Near field
Current
Port
Entrainment
Density Stratification


Wastewater Treatment
Plant
Water Surface
Wastefield
Port
Sea Bed
Riser
Feeder Pipe
Diffuser Pipe

3.1.2 Design Goal

• Goal of design of an outfall diffuser system

→ accomplish rapid initial mixing of effluent with ambient water in order to minimize detrimental effects of the discharge on the environment

initial jet mixing - controlled by designer

subsequent turbulent diffusion and transport processes- controlled by nature

• Design process

Design process is basically iterative.

The final design is the culmination of many trials.

Both engineering factors and economics, aesthetics, and other social and intangible factors are considered.

• Multiport diffuser

~Submerged multiport diffusers have been found to be an efficient way of maximizing initial dilution and meeting regulatory requirements.

[Ex] Orange County Sanitation Districts

feeder pipe:	8350-m long, 3.05-m diameter
diffuser:	1829-m long
port:	7.52~10.49-cm diameter, 7.32-m spacing, 500 ports
flow:	flowrate 12.7 m^3/s, velocity 4 m/s

• Initial dilution = f (discharge, diffuser length, depth of discharge, port size and spacing, ambient current, density stratification)

→ The first step in the design process is determine the fundamental diffuser characteristics, i.e., diffuser length, depth of discharge.

→ The second design variables are port size and spacing.

When density stratification exists, dilution and submergence characteristics of the candidate design should be evaluated.

→ Selection of the site or location of the diffuser structure is important when ambient current, density stratification exist.

[Re] Order of dilution

Initial dilution: 100 (Wastewater from sewage treatment plant)

10 (Heated water (thermal effluent) from coastal power plant)

Subsequent turbulent diffusion: 5~10

Transport by currents is more important than dispersion.

• Internal system hydraulics of the outfall and diffuser

→ Distribution of flow along the manifold should be uniform.

→ Overall outfall system includes a pumping station or energy dissipator.

3.2 Mixing Phenomena

Consider a typical large ocean outfall discharging sewage effluent through a long diffuser structure as shown in Fig. 3.1

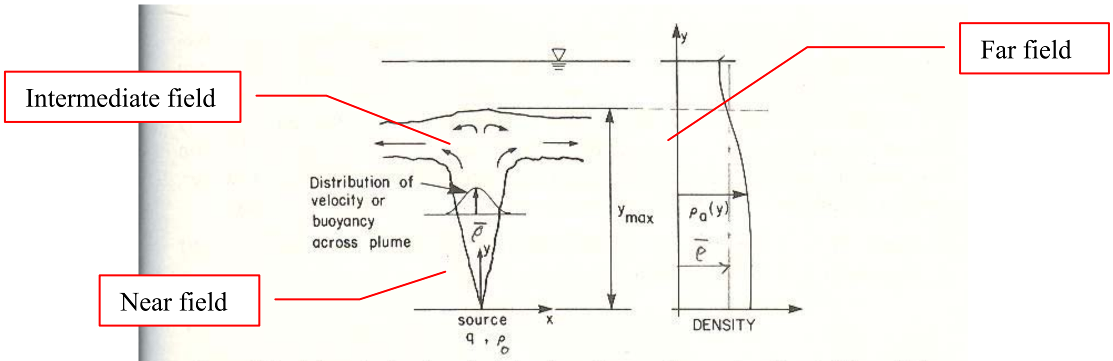


Figure 10.1 Schematic drawing of a rising plume from a submerged multiport diffuser. [After Koh and Brooks (1975).]

• Three zones of mixing and dispersion processes

→ Evaluation of diffuser design must include analysis in all three regions even though the first is under the control of the designer.

i)Near field:

- mixing is accomplished in the buoyant jet

- mixing is governed by momentum and buoyancy of discharge

ii) Intermediate field:

- zone where the sewage field is established

- mixing is governed by both momentum and buoyancy of discharge and ocean currents

iii) Far field:

- mixing and transport are accomplished by ocean currents and turbulence

- mixing and transport are relatively insensitive to the exact discharge conditions

3.2.1 Initial Mixing

3.2.1.1 Dilution in the Rising Plume

- A large ocean outfall diffuser is usually many times longer than the discharge depth.
- The ratio of length of discharge manifold to depth of discharge for a large ocean outfall diffuser is of the order of 10 or more. $(L_D / H \geq 10)$

→ The discharge may be considered two dimensional.

- Many small jets from many small ports spaced at a distance of a few meters tend to merge together to form a curtain fairly rapidly as shown in Fig. 3.1.

→ Solution for a two-dimensional buoyant plume is useful for estimating the dilution.

(i) Vertical discharge

a) Line plume for uniform, motionless ambient

$$\boxed{S_c = 0.38 g'^{1/3} \frac{d}{q^{2/3}}} \tag{3.1}$$

where S_c = centerline dilution; $g' = g\Delta\rho / \rho$; d = vertical distance above the source;

q = initial discharge per unit length

[Cf] Average dilution for plane plume (Table 2.3)

$$S_{av} = \frac{u}{q} = 0.34 \frac{B^{1/3} z}{q} = 0.34 \frac{(qg')^{1/3} z}{q} = 0.34 \frac{g'^{1/3} z}{q^{2/3}}$$

• Economic way of increasing initial dilution

Dilution from a line diffuser is improved by increasing either the discharge depth or the length of diffuser.

← Vertical distance for dilution is limited by the depth of discharge while q is inversely proportional to the length of diffuser.

→ A shorter deeper diffuser is essentiallly equivalent to a longer shallower diffuser.

Assume that outfall pipe (feeder pipe) is aligned perpendicular to shore with the diffuser parallel to shore.

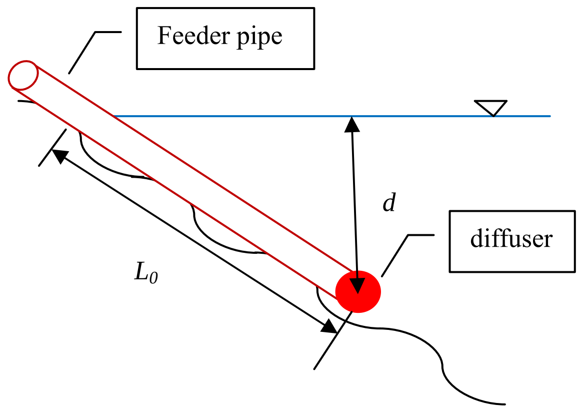
Let L_0 be the length of feeder pipe, L the length of diffuser, and α is the bottom slope.

Then, it can be deduced from Eq. (3.1) that

$$r \equiv \frac{ds / dL_0}{ds / dL} = \frac{3}{2}\alpha\frac{L}{d} \qquad (3.2)$$

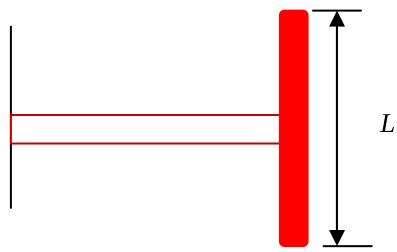
[Re] Derivation of (3.2)

$$S = a_1 \frac{d}{q^{2/3}} \qquad (1)$$



Substitute $d = L_0\alpha, \quad q = Q / L$

$$S = a_2 L_0 \alpha L^{2/3} \qquad (2)$$



Differentiate (2) wrt L_0

$$\frac{dS}{dL_0} = a_2 \alpha L^{2/3} \qquad (3)$$

Differentiate (2) wrt L

$$\frac{dS}{dL} = a_2 L_0 \alpha \frac{2}{3} L^{-1/3} = a_2 d \frac{2}{3} L^{-1/3} \qquad (4)$$

Divide (3) by (4)

$$r = \frac{dS/dL_0}{dS/dL} = \frac{3}{2}\alpha \frac{L}{d} = \frac{3}{2}\frac{\alpha L}{\alpha L_0} = \frac{3}{2}\frac{L}{L_0}$$

i) $r > 1$

It would take less pipe to achieve the same incremental improvement in dilution by increasing L_0 than L.

ii) $r \approx 1$

Critical value of L is

$$L_e \cong \frac{2}{3} L_0$$

If $L > L_e\,(r > 1)$ → It is less costly to increase L_0 to achieve the same incremental improvement in dilution.

If $L < L_e\,(r < 1)$ → It is cheaper to increase L to achieve the same incremental improvement in dilution.

b) Plume solution for linearly stratified, motionless ambient

$$\boxed{S = 0.31 g'^{1/3} \frac{y_{\max}}{q^{2/3}}} \tag{3.3}$$

where $y_{\max}$ = maximum height of rise, is given by

$$y_{\max} = 2.84 (g'q)^{1/3} \left[\frac{-g}{\rho} \frac{d\rho_a}{dy} \right]^{-1/2} \tag{3.4}$$

[Cf] For plume in linearly stratified environment (Table 2.4)

$$h_B = 2.8 \frac{B^{1/3}}{(g\varepsilon')^{1/2}}$$

[Example 3.1] Estimate the initial dilution for $q = 0.01\ \mathrm{m}^3/\mathrm{s}/\mathrm{m}$, $\Delta\rho / \rho = 0.025$, $d = 45$ m

$$S = 0.38 \times (9.81 \times 0.025)^{1/3} (45) / (0.01)^{2/3} = 230$$

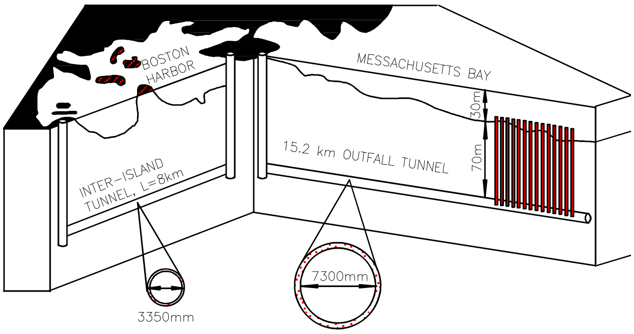
- Typical large sewage outfalls

$\Delta\rho / \rho = 0.025$, $d = 45$ m

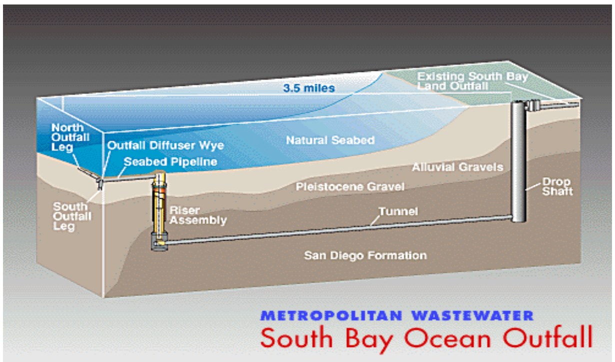
→ In order to achieve dilution of 200 the length of the diffuser should be 100 m for every cubic meter per second of waste flow, Q.

Table 3.1 Summary of Major Outfalls

Ocean Outfall		Operation Year	Feeder pipe Length (m)	Discharge Depth (m)	Diffuser Length (m)	Port Diameter (cm)	Spacing (m)	Design Flow Rate (m^3/s)
Sanitation districts of Los Angeles County(White Point No.1)		1937	1,524	34	116	21~23	5.5	
Sanitation districts of Los Angeles County(White Point No.2)		1947	2,072	47	65.8	20~23	7.3	
Sanitation districts of Los Angeles County(White Point No.3)		1956	2,400	61~64	731	16~19.1	7.3	6.57
City of Los Angeles at Hyperion		1960	8,390	59	2,410	17.1~20.7	121.9	18.42
San Diego		1963	3,510	61~64	820	20.3~22.9	121.9	10.27
Sanitation districts of Los Angeles County(White PointNo.4)		1965	2,270	50~58	1,350	5.1~9.1	15.2	9.65
Metro. Seattle(West Point)		1965	930	64~73	180	11.4~14.6	7.6	5.49
Sanitation districts of Orange County		1971	6,520	53~59	1,830	7.5~10.5	30.1	12.74
Honolulu(Sand Island)		1975	2,780	67~72	1,030	7.6~9.0	30.1	4.64
San Francisco		1990	6,400	22.9			10.97	6.22, 9.72
South Bay Ocean Outfall		1998	5,800	25	1,200			15.2
Boston Outfall		1998	15,000	32	2,000	370	20	56.0
SEFLOE II Outfalls	Hollywood		3,050					
	Broward		2,130					
	Miami-Central		2,730		39		9.8	
	Miami-North		3,350		110		0.61	
Sydney Malabar		1990	3,000	60~80			25	



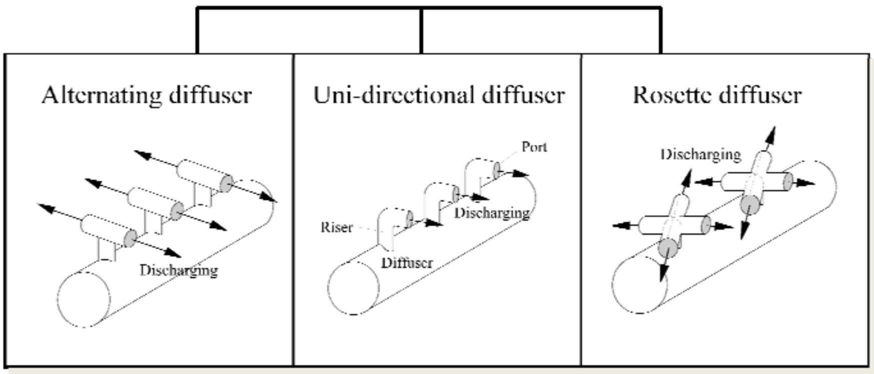
Boston Outfall



South Bay Outfall

Table 3.2 Characteristics of Korean Sewage Outfalls

Outfall		Masan/Changwon	Yongyeon	Noksan	Sokcho
Discharge Depth (m)		13.0	25.0 – 27.0	6.5	10 – 11.5
Design Flow Rate (m^3/s)		8.23	4.05	8.24	1.22
Diffuser	Diameter (*D*p) (m)	2.00	1.80	2.20	0.60
	Length (m)	210	470	45	50
Riser	Diameter (*D*) (m)	1.35	1.35	1.20	0.60
	Spacing (*l*) (m)	10.0	10.0	9.0	10.0
Port	Diameter (*d*) (m)	0.200	0.110, 0.120, 0.125	0.500	0.225, 0.250
Number of Ports in a Riser		4	4	4	4
Angles (θ_2, θ)		120°, 30 °	90°, 45 °	90°, 45 °	90°, 45 °



Sokcho Outfall

[Example 3.2]

Estimate the initial dilution for $q = 0.01$ m^3/s/m , $\Delta\rho / \rho = 0.025$, $d = 60$ m

The ocean temperature varies linearly with depth. Temperature is 20℃ on the surface and 17℃ on the bottom. The salinity is constant at 34 ‰.

[Sol] From Appendix A, density stratification is

$$\frac{-1}{\rho}\frac{d\rho_a}{dy} = \frac{(24.767 - 24.020)\times 10^{-3}}{60} m^{-1} = 1.25\times 10^{-5} m^{-1}$$

$$y_{\max} = 2.84(9.81\times 0.025\times 0.01)^{1/3}[9.81(1.25\times 10^{-5})]^{-1/2} = 35 \text{ m}$$

→ The wastefield is submerged at 25 m depth from water surface.

$$S = 0.31\frac{(9.81\times 0.025)^{1/3}(35)}{(0.01)^{2/3}} = 150$$

→ Dilution is lower than that of Example 3.1 (unstratified ambient; S = 230) because the wastefield is trapped at 35 m above the sea bottom.

(ii) Horizontal discharge

a) Plume solution for linearly stratified, motionless ambient

→ modify coefficient in Eq. (3.3) and (3.4)

$$\boxed{S = 0.36 g'^{1/3} \frac{y_{\max}}{q^{2/3}}}$$

$$y_{\max} = 2.5 (g'q)^{1/3} (g\varepsilon')^{-1/2}$$

(iii) Single port

a) Plume solution for uniform (no stratification), motionless ambient

$$\boxed{S = 0.089 g'^{1/3} \frac{y^{5/3}}{Q^{2/3}}} \tag{3.5}$$

[Cf] round plume (Table 2.3)

$$\frac{u}{Q} = S_w = 0.15 g'^{1/3} \frac{z^{5/3}}{Q^{2/3}}$$

b) Plume solution for linear stratification

$$\boxed{S = 0.071 g'^{1/3} \frac{y_{\max}^{5/3}}{Q^{2/3}}} \tag{3.6}$$

$$y_{\max} = 3.98 (g'Q)^{1/4} \left[\frac{-g}{\rho} \frac{d\rho_a}{dy} \right]^{-3/8} \tag{3.7}$$

[Example 3.3]

Estimate the initial dilution of <u>single outlet discharge</u> for $Q = 6 \text{ m}^3/\text{s}$, $\Delta\rho / \rho = 0.025$, $d = 60$ m. The ocean temperature varies linearly with depth. Temperature is 20℃ on the surface and 17℃ on the bottom.

[Sol]

$$\frac{-1}{\rho}\frac{d\rho_a}{dy} = \frac{(24.767 - 24.020)\times 10^{-3}}{60} m^{-1} = 1.25\times 10^{-5} m^{-1}$$

$$y_{\max} = 3.98(9.81\times 0.025\times 6)^{1/4}[9.81\times 1.25\times 10^{-5}]^{-3/8} = 128 \text{ m}$$

Since $y_{\max} > d$ → The waste field will surface.

→ use d instead of $y_{\max}$ in Eq. (3.6)

$$S = 0.071\times(9.81\times 0.025)^{1/3}(60)^{5/3}/(6)^{2/3} = 12$$

→ Dilution is significantly lower than that of Example 3.2 (multiport diffuser; S = 150).

→ This shows the advantage of a multiport diffuser.

→ Since $Q = 6 \text{ m}^3/\text{s}$, the diffuser length would be 600 m to have q = 0.01 m2/s.

3.2.1.2 Establishment of the Wastewater Field

Application of theories of buoyant jets and plumes to obtain dilution estimates must take into account the presence of the waste field.

→ It decreases dilution since dilution of the effluent with clean ocean water ceases when the plume reaches the bottom of the waste field.

For $y > y_b$, the mixing is with the previously discharged diluted effluent.

→ Thus, the mixing only serves the purpose of evening out the differences in concentration between the various cross-sectional portions of the rising plume.

1) Perpendicular currents: Alternating diffuser

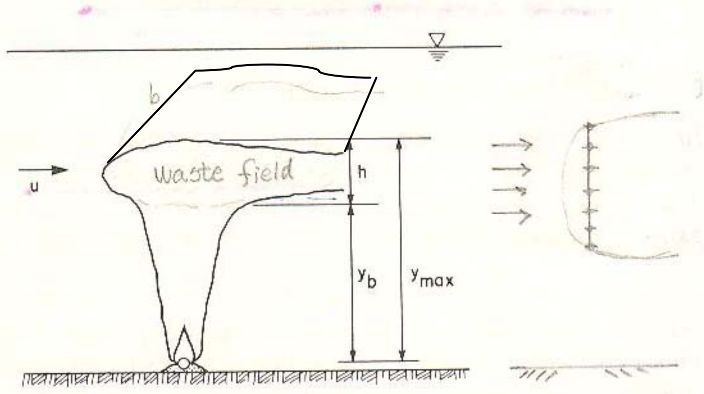


Figure 10.2 Definition sketch for establishment of the wastewater field. [After Koh and Brooks (1975).]

Approximate estimates can be made to the effect of blocking due to finite thickness of waste water field.

By continuity, and since $S_{aw} = Q / Q_0$

$$Q_0 S_{aw} = ubh = ub(y_{\max} - y_b) \tag{3.8}$$

where Q_0 = discharge from the diffuser;

S_{aw} = average dilution in waste field;

u = current speed;

b = width of the waste field;

h = thickness of the waste field;

$y_{\max}$ = y-coordinate at top of waste field;

y_b = y-coordinate at bottom of waste field.

Assuming that the average dilution at elevation y is proportional to y and S_{aw} = average dilution at $y = y_b$

$$S_{aw} / S_a = y_b / y_{\max} \tag{3.9}$$

where S_a = value calculated at the top of the plume $(y = y_{\max})$ disregarding the presence of blocking of the finite thickness of the waste field.

Combine Eq. (3.8) and (3.9)

$$Q_0 S_a = ub(y_{\max} / y_b)(y_{\max} - y_b) \tag{3.10}$$

Solving (3.10) for $y_b / y_{\max}$ gives

$$y_{\max} / y_b = (1 + Q_0 S_a / uby_{\max})^{-1} \tag{3.11}$$

Then substituting this into (3.9) results in

$$\boxed{S_{aw} = S_a (1 + Q_0 S_a / uby_{\max})^{-1}} \tag{3.12}$$

Thickness of the waste field, h

$$h = y_{\max} \frac{(Q_0 S_a / uby_{\max})}{(1 + Q_0 S_a / uby_{\max})} \tag{3.13}$$

▪Width of the waste field, b

When current is perpendicular to diffuser, $b \sim L$ (length of diffuser)

When current is parallel to the diffuser, b must be estimated differently.

2) Parallel Current

i) Uniform (unstratified) ambient

Ambient is uniform in density.

→ Waste plume rises to the surface.

Consider line source of buoyancy when current is parallel to the diffuser.

Then, significant variables are *g'q*, *L/u*, *b*.

By dimensional analysis, width of the waste field is

$$b = C(g'q)^{1/3}(L/u) \tag{3.14}$$

where $C \simeq 1.2$ (Koh, 1996; Roberts, 1977)

Substitute Eq. (3.14) into Eq. (3.12)

$$S_{aw} = S_a\left[1 + Q_0 S_a / \left\{1.2(g'q)^{1/3} L y_{\max}\right\}\right]^{-1} \tag{3.15}$$

where S_a ~ average dilution in the absence of blocking

Convert centerline dilution in Eq. (3.1) into average dilution assuming that there is no current below y_b

$$\boxed{S_a = \sqrt{2}S = 0.54 g'^{1/3} \frac{y_{\max}}{q^{2/3}}} \tag{a}$$

[Re] Table 2.2: $\dfrac{C_m}{C_{av}} = 1.4, \quad \dfrac{S_m}{S_{av}} = \dfrac{1}{1.4} \simeq \dfrac{1}{\sqrt{2}}$

Substitute (a) into Eq. (3.15)

$$\frac{S_{aw}}{S_a} = \frac{1}{1+0.54/1.2} = 0.7 \tag{3.16}$$

$$\boxed{S_{aw} = 0.7S_a = 0.38 g'^{1/3} \frac{y_{\max}}{q^{2/3}}} \tag{3.17}$$

In Eq. (3.17), dilution is not sensitive to the current speed (u) for a parallel current.

[Cf] In Eq. (3.12), dilution increases for higher perpendicular currents (Alternating diffuser).

[Example 3.4]

Evaluate the average dilution including the effect of blocking obtained in a water depth of 45 m for a 600 m diffuser discharging 6 m^3/s of sewage effluent with $\Delta\rho/\rho$ = 0.025. Use a perpendicular <u>current of 0.25 m/s</u> and a parallel current. Assume uniform ambient density.

[Sol]

$$S_a = 0.54(9.81 \times 0.025)^{1/3} \frac{(45)}{(6/600)^{2/3}} = 325$$

→ average dilution w/o effects of blocking

i)For a perpendicular current, use Eq. (3.12)

$$S_{aw} = 325 / \left\{1 + 6(325)/(0.25)(600)(45)\right\} = 255$$

ii)For a parallel current, use Eq. (3.17)

$$S_{aw} = 0.38 \times (9.81 \times 0.025)^{1/3} (45)/(0.01)^{2/3} = 230$$

Homework Assignment # 3-1

Due: 1 week from today

For various u, calculate S_{aw} for each diffuser

iii) Submerged sewage field in stratified ambient

Consider the case when the sewage field is submerged so that there is no net buoyancy in the waste field.

→ Horizontal spreading is complex because of ambient density stratification.

Then, significant variables are $g\varepsilon q_1$, L/u, b.

where $\varepsilon = -\frac{1}{\rho}\frac{d\rho}{dy}$; $q_1 = \frac{S_{aw} Q_0}{L}$

q_1 means the rate of discharge into the waste field per uint length of diffuser.

By dimensional analysis, width of the waste field is

$$b = 0.8(g\varepsilon q_1^2)^{1/4} L/u \qquad (3.18)$$

Substituting Eq. (3.18) into Eq. (3.12) gives

$$S_{aw} = S_a - A\sqrt{\frac{A^2}{4} + S_a} + \frac{A^2}{2} \qquad (3.22)$$

$$A = \frac{S_a (Q_0 / L)^{1/2}}{y_{\max} 0.8(\varepsilon g)^{1/4}} \qquad (3.23)$$

Eq. (3.3) for stratified ambient

where $S_a = \sqrt{2}\left[0.31 g'^{1/3} y_{\max} / q^{2/3}\right] = 0.44 g'^{1/3} y_{\max} / q^{2/3}$

In Eq. (3.22), dilution is independent of the current speed.

[Example 3.5]

Evaluate the average dilution and submergence characteristics including the effect of blocking by a parallel current for the following case: $d = 60$ m ; diffuser length = 600 m $Q = 6$ m^3/s; $\Delta\rho/\rho = 0.025$; $\varepsilon = -(1/\rho)d\rho/dy = 1.25\times10^{-5}$ m^{-1}.

[Sol]

Find maximum height of rise using Eq. (3.4): $y_{\max} = 2.84(g'q)^{1/3}\left[\dfrac{-g}{\rho}\dfrac{d\rho_a}{dy}\right]^{-1/2}$

$$y_{\max} = 2.84(9.81\times0.01\times0.025)^{1/3}\left[9.81\times1.25\times10-5\right]^{-1/2} = 35 < 60$$

→ The sewage field is submerged.

$$S_a = 0.44 g'^{1/3} y_{\max} / q^{2/3}$$

$$S_a = \sqrt{2}\times0.31\times(9.81\times0.025)^{1/3}\frac{(35)}{(6/600)^{2/3}} = 207$$

$$A = \frac{207(0.01)^{1/2}}{35\times0.8\left(9.81\times1.25\times10^{-5}\right)^{1/4}} = 7$$

$$S_{aw} = 207 - 7\sqrt{\frac{7^2}{4} + 207} + \frac{7^2}{2} = 128$$

→ Dilution is decreased by blocking effect.

[Example 3.2 – Extension]

If multiple jets discharged from the multiport diffuser are not merged through the depth, compute the dilution. Number of port $= 50$, Length of diffuser $= 600$ m , Total Discharge $= 600\ \text{m}^3/s$

<Sol>

$b \sim 0.1y$

Thus, merge starts $y \approx 5l_p$

set $l_p = 12$ m

$$y_{\max} = 3.98(0.12\times 9.81\times 0.025)^{1/4}(9.81\times 1.25\times 10^{-5})^{-3/8}$$

$$= 3.98\times 0.4142/0.0341 = 48.3\text{ m}$$

$$S = 0.071\frac{(9.81\times 0.025)^{1/3}(40.6)^{5/3}}{(0.12)^{2/3}} = 0.071\frac{(0.626)(480.2)}{0.243} = 87.8$$

	single port	multiport	
		w/ merging	w/o merging
S_c	12	150	87.8
$y_{\max}$	> 60 m	35 m	48 m

[Example 3.2 - Horizontal discharge]

$$g = 0.01 \text{ m}^3/\text{s/m}$$

$$\frac{\Delta\rho}{\rho} = 0.025$$

$$\varepsilon' = -\frac{1}{\rho}\frac{d\rho_a}{dy} = 1.25\times10^{-5} \text{ m}^{-1}$$

$$y_{\max} = 2.5(g'g)^{1/3}(g\varepsilon')^{-1/2}$$

$$= 2.5(9.81\times0.025\times0.01)^{1/3}(9.81\times1.25\times10^{-5})^{-1/2}$$

$$= 2.5(0.1351)/(0.0111) = 30.5 \text{ m}$$

$$S = 0.36g'^{1/2}\frac{y_{\max}}{g^{2/3}}$$

$$= 0.36(9.81\times0.025)^{1/3}(30.5)/(0.01)^{2/3}$$

$$= 0.36(0.6259)(30.5)/0.0464 = 143.1$$

For unstratified ambient, $d = 60$ m

(i) Multiport w/o merging → single set eq.

$$l_p = 12m \;\rightarrow\; n = 50$$

$$g_p = \frac{6}{50} = 0.12 \text{ m}^3/\text{s}$$

$$S = 0.089\frac{(9.81\times0.025)^{1/3}(60)^{5/3}}{(0.12)^{2/3}} = \frac{0.089(0.626)(920.8)}{0.243} = 211$$

(ii) Multiport w/ merging

$$S = 0.38 g'^{1/3} \frac{d}{g^{2/3}}$$

$$q = \frac{6}{600} = 0.01$$

$$\therefore S = 0.28(9.81 \times 0.025)^{1/3}(60)/(0.01)^{2/3}$$

$$= \frac{0.38(0.626)(60)}{0.0464} = 307.5$$

(iii) Single port

$$Q = 6 \text{ m}^3/\text{s}$$

$$S = 0.089 g'^{1/3} \frac{y^{3/5}}{Q^{2/3}} = 0.089(9.81 \times 0.025)^{1/2}(60)^{5/3}/(6)^{2/3}$$

$$= 0.089(0.626)(920.8)/(3.30) = 15.5$$

	Single port	Multiport	
		w/ merging	w/o merging
S_c	15.5	307.5	211

Eq.(3.15): $$\frac{S_{aw}}{S_a} = \frac{1}{1 + Q_0 S_a / \left\{1.2(g'q)^{1/3} L y_{\max}\right\}} \qquad \text{(B)}$$

Substitute (A) into (B)

$$\frac{S_{aw}}{S_a} = \frac{1}{1 + \dfrac{Q_0\left[0.54 g'^{1/3} y_{\max} / q^{2/3}\right]}{1.2(g'q)^{1/3} L y_{\max}}}$$

$$= \frac{1}{1 + \dfrac{(Q_0 / L) 0.54 g'^{1/3} y_{\max} / q^{2/3}}{1.2(g'q)^{1/3} y_{\max}}}$$

$$= \frac{1}{1 + \dfrac{0.54(g'q)^{1/3} y_{\max}}{1.2(g'q)^{1/3} y_{\max}}}$$

$$= \frac{1}{1 + 0.54 / 1.2} = 0.69$$

For perpendicular currents (Coflowing diffuser)

Eq.(3.12): $$S_{aw} = S_a \left(1 + \frac{Q_0 S_a}{u b y_{\max}}\right)^{-1}$$

where $S_a = 0.54 g'^{1/3} \dfrac{y_{\max}}{q^{2/3}}$

$Q_0 = Lq$

$b \approx L$

$$\therefore S_{aw} = S_a \left(1 + \frac{Lq(0.54 g'^{1/3} y_{\max} / q^{2/3})}{u L y_{\max}}\right)^{-1}$$

$$= S_a \left(1 + 0.54(g'q)^{1/3} / u\right)^{-1}$$

$$= S_a \left(\frac{u}{u + 0.54(g'q)^{1/3}}\right)$$

If $g' \approx 0.245$

$q = 0.01$

Then

$$S_{aw} = S_a \left(\frac{u}{u + 0.073} \right)$$

$\rightarrow$ $S_{aw} = S_a$

u	S_{aw} / S_a
0.1	0.578
0.2	0.732
0.3	0.804
0.4	0.846
0.5	0.872
1.0	0.932

3.2.1.3 Effect of Currents on Plume Dilution

The combination of a current and finite length diffuser makes the problem very complex.

Roberts (1977, 1979)'s experiments results for ambient water of uniform density

$$\frac{S_m\, q}{u\, d} = f(F, \theta) \tag{3.24}$$

where S_m = minimum dilution; u = ambient current speed; θ = angle of current with respect to diffuser orientation; d = depth of discharge; F = Froude number defined as

$$F = \frac{u^3}{\frac{\Delta\rho}{\rho} g\, q} = \left[\frac{u}{(g'q)^{1/3}}\right]^3 = \left[\frac{u}{b^{1/3}}\right]^3 \tag{3.25}$$

where b = buoyancy flux per unit length

Line plume solution including blocking [Eq.(3.17)] reduces to

$$\frac{S_m\, q}{u\, d} = 0.27 F^{-1/3} \tag{3.26}$$

[Re] Derivation of (3.26)

$$S_{aw} = \frac{0.38 (g'q)^{1/3} d}{q} \qquad (3.17)$$

$$\frac{S_{aw}\, q}{u\, d} = 0.38 \frac{(g'q)^{1/3}}{u}$$

$$\frac{S_m q}{ud} = \frac{0.38}{\sqrt{2}} \frac{1}{F^{1/3}} = 0.27 F^{-1/3}$$

$\sqrt{2}$ is to convert from average to minimum dilution.

The results are shown in Fig. 2.40.

→ Dilution Eq. for multiport diffuser in ambient current

(i) Parallel current (Tee diffuser) $\theta = 0^{\circ}$

a. $F = \dfrac{u^3}{b} < 0.1$; weak deflection

$$\frac{S_m q}{ud} = 0.27F^{-1/3} = 0.27\left(\frac{u^3}{g'q}\right)^{-1/3}$$

$$S_m = 0.27 g'^{1/3} d / q^{2/3}$$

b. $0.1 < F < 10$

$$S_m \cong 0.27 g'^{1/3} d / q^{2/3}$$

c. $F > 10$; strong deflection

$$\frac{S_m q}{ud} \approx 0.15 \;\rightarrow\; S_m = 0.15\frac{ud}{q}$$

(ii) Perpendicular current (Coflowing diffuser) $\theta = 90^{\circ}$

a. weak deflection, $F < 0.1$

$$S_m = 0.27 g'^{1/3} d / q^{2/3}$$

b. $0.1 < F$

$$\frac{S_m q}{ud} \approx 0.6 \;\rightarrow\; S_m = 0.6\frac{ud}{q}$$

3.2.1.4 Effects of Stratification

Density stratification is nonlinear, unsteady as shown in Fig. 3.3.

~ caused by tidal effects, internal waves, variation of currents

→ Approximation of actual density profile by an equivalent linear profile between discharge depth and maximum height of rise.

• For horizontal discharge

Eq. (3.4a): $y_{\max} = 2.5(g'q)^{1/3}\left[-(g/\rho)\dfrac{d\rho_a}{dy}\right]^{-1/2}$

The replacement of the actual density profile by an equivalent linear one between $y = 0$ and y_{max} implies

$$-\frac{g}{\rho}\frac{d\rho_a}{dy} = \frac{g}{\rho}\frac{\Delta\rho_a}{y_{\max}} \tag{3.27}$$

Substitute Eq. (3.27) into Eq. (3.4a)

$$y_{\max} = 6.25(g'q)^{2/3}\left(\frac{\rho}{g\Delta\rho_a}\right) \tag{3.28}$$

For horizontal discharge,

$$S_m = 0.36\frac{(g'q)^{1/3}y_{\max}}{q} = 0.36\frac{g'y_{\max}}{(g'q)^{2/3}} \tag{3.29}$$

Substitute Eq. (3.28) into Eq. (3.29)

$$S_m = 2.25 \frac{\Delta\rho_d}{\Delta\rho_a} \tag{3.30}$$

where $\Delta\rho_a$ = ambient density diffuser between $y=0$ and $y = y_{\max}$; $\Delta\rho_d$ = density difference between ambient and discharge

For point source of buoyancy (single port diffuser)

$$y_{\max} = 9.1(Qg')^{2/5}(g\Delta\rho_a / \rho)^{-3/5} \tag{3.31}$$

$$S = 2.8 \frac{\Delta\rho_d}{\Delta\rho_a} \tag{3.32}$$

3.3 Outfall and Diffuser Hydraulics

3.3.1 Manifold Hydraulics

• Hydraulic requirements for multiport diffuser

(i) distributing discharge uniform along the entire length of the diffuser

(ii) maintaining adequate velocities in the diffuser pipe to prevent deposition

$V_d > 0.5 \sim 0.7$ m/s

(iii) providing for means of cleaning or flushing the system

(iv) ensuring that no seawater intrusion occurs ← all ports flow full

(v) keeping the head loss reasonably small to minimizing pumping

• Typical features of multiport diffuser

(i) the diffuser pipe diameters are reduced in steps towards the far end,

→ Fig. 3.13

(ii) a flap gate is usually provided at the end which can be removed for flushing

<cf> duckbill valve

(iii) ports are relatively small and so that the total port area downstream of any section is less than the pipe area at that section.

$$n \times \left(\frac{\pi}{4} d_\rho^2 \right) < \frac{\pi}{4} D^2$$

(iv) ports have generally rounded corners. → Fig. 3.12

• A multiple port diffuser is basically a manifold.

→ Manifold flow problem is complicated by two factors.

(i) There is a density difference between seawater outside of diffuser and the sewage inside.

(ii) The friction along the pipe changes the hydraulic head inside.

- flow from a single port … Bernoulli Eq.

$$Q_p = C_D A_p \sqrt{2gE}$$

in which Q_p = port discharge; A_p = port area;

E = difference in total head across the port;

C_D = discharge coefficient accounting for various losses, contractions, and flow non uniformities.

- discharge coefficient for rounded entrance → Fig. 3.11

$$C_D = 0.975\left(1 - \frac{V_d^2}{2gE}\right)^{3/8} \qquad \backslash \qquad (3.34)$$

- discharge coefficient for sharp edged port

$$C_D = 0.63 - 0.58\left(\frac{V_d^2}{2gE}\right)$$

in which V_D = velocity in the diffuser pipe.

• Riser-nozzle assemblies

- discharge structures for Units 2 and 3 at San Onofre Nuclear Generating Station → see Fig. 3.12

(i) riser diameter is larger than the port diameter, → reduce energy loss

(ii) entrance to the riser is rounded, → reduce energy loss

(iii) elbow is used to direct the discharge in a more nearly horizontal direction,

(iv) expansion and bell mouth is used beyond the exit plane of the nozzle.

- obtain $C_D =$ using energy equation

$$C_D = \left[1 - \frac{V^2}{2gE} \right]^{1/2} \frac{1}{\sqrt{x_e + 1 / C_c^2}}$$

in which $V =$ velocity downstream of the riser; $E =$ energy loss;

$x_e =$ entrance head loss coefficient; $C_c =$ jet contraction coefficient.

3.3.2 Calculation Procedure

• Design of multiport diffuser (Rawn et al., 1961)

- definition of symbols

$D =$ diameter of pipe

$d_n =$ diameter of nth port counting from the offshore end

$a_n =$ area of n th port

$V_n =$ mean pipe velocity between n th port and (n+1)th port

$\Delta V_n =$ increment of velocity due to discharge from nth port

$= V_n - V_{n-1}$

$h_n =$ difference in pressure head between inside and outside of diffuser just upstream of n th port $= \Delta p_n / \gamma$

$E_n =$ total head at the n th port $= h_n + V_n^2 / 2g$

$C_D =$ discharge coefficient for ports

$q_n =$ discharge from the n th port

$h_{fn} =$ head loss due to friction between (n+1)th port and n th port

$f =$ Darcy friction factor

$\Delta z_n =$ change in elevation between (n+1)th port and n th port

$\Delta s / s =$ relative difference in specific gravity between discharging fluid and ambient fluid

- discharge for the first port

$$q_1 = C_D a_1 \sqrt{2gE} = C_D (\pi / 4) d_1^2 \sqrt{2gE} \tag{3.44}$$

- velocity in the pipe

$$V_1 = \Delta V_1 = \frac{q}{(\pi / 4) D^2}$$

- total head at the 2nd port

$$E_2 = E_1 + h_{f1} + (\Delta s / s)\Delta z_1$$

- find C_D by using Eq. (3.34) or (3.35) or (3.42)

- then, discharge for the 2nd port and velocity in the pipe are

$$q_2 = C_D a_2 \sqrt{2gE_2}$$

$$V_2 = V_1 + \Delta V_2 = V_1 + \frac{q_2}{(\pi / 4) D^2}$$

- continue this procedure step by step using general relations with a computer

$$C_D = f\left(\frac{V_{n-1}^2}{2g} / E_n\right) \tag{3.46}$$

$$q_n = C_D a_n \sqrt{2gE_n} \tag{3.47}$$

$$\Delta V_n = \frac{q_n}{(\pi / 4) D^2} \tag{3.48}$$

$$V_n = V_{n-1} + \Delta V_n \tag{3.49}$$

$$h_{fn} = f \frac{L_n}{D} \frac{V_n^2}{2g} \tag{3.50}$$

$$E_{n-1} = E_n + h_{fn} + (\Delta s / s)\Delta z_n \tag{3.51}$$

- stepwise calculations for small groups of ports

$$\Delta V_n = m\frac{q_n}{(\pi / 4)D^2}$$

in which m = number of ports considered in a group.

3.3.3 Selection of Port Sizes and Pipe Sizes

- Designer can change the pipe size, the port size, and port spacing.
- Size of discharge ports may be varied in order to keep discharge uniform from port to port.
- End of diffuser pipes should be bulkheaded, and bulk-heads should be removable for flushing the pipe.
- Sum of all the port areas should be less than the cross-sectional area of the outfall pipe. → best area ratio (total port area / pipe area) = 1/3 ~ 2/3
- See Fig. 3.13, Fig. 3.14, Fig. 3.15 → make a diffuser flow full

• Sand Island outfall diffuser

$$D = 2.134 \sim 1.22 \text{ m}$$

$$d_p = 8.97 \sim 7.62 \text{ cm}$$

$$n = 282$$

$$A_{Pi} = \frac{\pi}{4}(2.134)^2 = 3.6 \text{ m}^2$$

$$A_{Po} = n \times \frac{\pi}{4}(0.09)^2 = 1.8 \text{ m}^2$$

3.3.4 Diffuser Design

(1) Procedure

1. Decide on the environment variables

S_m = minimum dilution

U = current speed

Properties of the outfall:

D_0 = pipeline diameter

d_p = diameter of ports

2. Select a trial depth for the first port (landward end of the diffuser)

3. Evaluate maximum discharge from the first port(Q_1) if the required dilution is to be achieved use following eq.

$$S_m = C_1 \frac{g'^a H^b U^c}{Q^a} \rightarrow Q = \Phi_1 H^a \tag{1}$$

where $\Phi_1 = \left[\frac{C_1 g'^a U^c}{S_m} \right]^{1/d}$

$\alpha = b / d$

<Table 1> Coefficient Values for Eq.(1)

Regime	C_1	a	b	c	d	α
$H / L_H < 0.04$	0.10	1/3	5/3	0	2/3	5/2
$0.04 < H / L_H \le 1$	0.50	-1/6	13/6	3/2	7/6	13/7
$H / L_H > 1$	0.50	0	2	1	1	2

where $L_H = g'Q/U^3$

The first estimate of Q_1 should be based on the assumption that $0.04 < H/L_H \le 1$ unless $U = 0$. Having estimated Q_1 the ratio H/L_H should be checked.

4. Evaluate the pressure excess (P_1) in the diffuser required to discharge necessary flow from the first port.

$$P_1 = \frac{\rho}{2}\left[\frac{Q}{C_d A_p}\right]^2 \tag{2}$$

C_d = discharge coeff. for the port

= 0.96 for standard nozzle

* Head loss, H_L

$$H_L = H_f + \frac{P_1}{\gamma} \tag{3}$$

Where $H_f =$ friction loss between treatment plant and the first port

5. Calculated the excess pressure at the next seaward port(P_2) allowing for friction losses between the ports and for the buoyancy of the effluent

$$P_2 = P_1 - \frac{\rho f s}{2D_0}\left[\frac{Q_0 - Q_1}{A_0}\right]^2 - \rho g' s\left[\frac{dH}{dz}\right]_1 \tag{4}$$

where f = Darcy-Weisbach friction factor

s = spacing between the ports

$A_0 = \dfrac{\pi}{4} D_0$

Q_0 = total outfall discharge

$\left[\dfrac{dH}{dx}\right]_1$ = pipeline inclination at the first port

* Pressure at the (n+1)the port

$$P_{n+1} = P_n - \frac{\rho f s}{2D_0}\left[\frac{Q_0 - \sum_1^n Q_1}{A_0}\right]^2 = \rho g' s \left[\frac{dH}{dx}\right]_n \tag{5}$$

* recommended spacing $\approx 0.25H$

if plume interference is to be avoided

$\therefore\ S \approx 0.25H$

6. Discharge from the second port

$$Q_2 = C_d A_p \left(2\frac{P_2}{\rho}\right)^{1/2} \tag{6}$$

Discharge from the nth port

$$Q_n = C_d A_p \left(2\frac{P_n}{\rho}\right)^{1/2} \tag{7}$$

7. The last port $(n = N)$ is identified by the continuity equation

$$\sum_{n=1}^{N} Q_n = Q_0 + \alpha Q_n, \qquad \alpha < 1 \tag{8}$$

8. If P in Eq.(5) falls to less than 0, it means that outfall is incapable of discharging the required flow and that the first port must be located in deeper water

9. The length of the diffuser section of the outfall is given by:

$$L_D = (N-1)S \tag{9}$$

(2) Design Limitation

1. For an outfall diffuser to be effective, it must discharge effluent from all of its ports.
2. For outfalls with equally spaced ports of the same diameter, discharge the maximum at the most shoreward port and reduce with discharge seawards for the other ports.

→ Effluent dilution is minimum from the port closest to the shore, which is laid at the shallowest part.

3. Seawater intrusion begins if port Froude number (Fr) falls below 1

$$Fr = \frac{Q}{A_p\sqrt{g'D_p}}, \qquad A_p = \text{port area}$$

For $D_p = 50$ mm, seawater intrusion develops when $Q < 0.2$ L/s

For $D_p = 100$ mm, seawater intrusion develops when $Q < 1.2$ L/s

(3) Design Example

Belmont outfall, Australia

* Design discharge $Q = 2.6\ \mathrm{m}^3/\mathrm{s}$ of secondary treated effluent

* long shore current $u_a = 5$ cm/s

* minimum dilution required at the sea surface $S_m = 100$

* Darcy-Weisbach $f = 0.03$

* discharge doeff. of ports $C_p = 0.96$

Fig. 5 Outfall length as a function of diffuser depth

(4) For riser-nozzle assembly

Energy equation between the diffuser and the ambient at the location of vena contracta of the jet

$$E = \frac{V_j^2}{2g} + x_{en}\frac{V_r^2}{2g} + f_r\frac{L_r}{D_r}\frac{V_r^2}{2g} + x_l\frac{V_r^2}{2g} + x_c\frac{V_r^2}{2g}$$

x_{en} = head loss coeff. for the entrance from diffuser to the riser

x_c = head loss coeff. for elbow

V_r = velocity in the riser

L_r, D_r = length, diameter of the riser

f_r = friction factor of the riser

V_j = jet velocity

$$E = \left\{\left(x_{en} + f_r\frac{L_r}{D_r} + x_l + x_c\right)\left(\frac{D_p}{D_r}\right)^4 + \frac{1}{C_c^2}\right\}\frac{V_p^2}{2g} = X\frac{V_p^2}{2g}$$

$D_p =$ port diameter

$C_c = V_p / V_j =$ jet contraction coeff.

$V_p =$ port velocity

Experiments by McNown(1954)

For $D_r / D_d < \frac{1}{4}$, $\quad x_{en} \approx 0.406 + \left(\frac{V_d}{V_r} \right)^2 \equiv x_c + \left(\frac{V_d}{V_r} \right)^2$

$V_d =$ velocity in the diffuser

For sharp-edged entrance : 0.406

For rounded entrance : 0.1~0.2

$C_c =$ jet contraction coeff.

$= f(D_p / D_r, \alpha) \rightarrow$ Table 3.2

$\alpha = \tan^{-1}(D_r - D_p,) / 2L_c$

$L_c =$ length of contraction (Fig.3.12)

$V =$ velocity in the diffuser pipe downstream of the riser

$V = V_d - r^2 V_p$

where $r = D_p / D_d$

Combine these equations

$$C_D = \frac{-r^2 (V / \sqrt{2gE} + \left\{ X(1 - V^2 / 2gE) \right\}^{1/2} + r^4}{X + r^4}$$

For small ports cast into the walls of pipes,

$r = 0, X = x_{en} + 1 / C_c^2$

$$\therefore C_D = \left[1 - \frac{V^2}{2gE}\right]^{1/2} \frac{1}{\sqrt{x_{en} + /C_c^2}}$$

3.4 An Example Design: Sand Island Outfall in Honolulu, Hawaii

• Estimated pollutant loads

- design period: 1970 -2020 (50 years)
- design peak flow: 6.62 → 7.58 → 8.85 m^3 / s

• Water quality standards (Public Health Regulations of the Dept. of Health, State of Hawaii) → see Table 3.4

- Effluent field should be submerged.

3.4.1 Preliminary Overall Considerations

(i) Some of the water quality standards are virtually impossible to meet.

(ii) Ensuring sewage field submergence would necessitate the placement of the diffuser at a depth in excess of 90 m.

(iii) Bathymetry is such that the bottom slope becomes progressively steeper as the depth increases.

3.4.2 Design Philosophy

- In developing the diffuser design, high dilution is of overriding importance while submergence of the sewage field is given a lower priority.
- Final choice of the diffuser site was also influenced by consideration of the prevailing ocean currents, the shoreward transport probabilities.

3.4.3 Final Design

- plan and profile of final chosen design → Fig. 3.13
- diffuser: depth 72 m; length 1030 m; pipe diameter 2.13 m
- Outfall is buried to a depth of 23 m after which it gradually emerges onto the ocean floor at a depth of 26 m, beyond this point pipe is placed on the ocean bottom using rock ballast.
- Deep burial and ballast sections are necessary to protect physical integrity of the pipeline against wave attack.