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Seawater Intrusion and Purging in Tunnelled Outfalls

A Case of Multiple Flow States

Wastewater outfalls tunnelled under the ocean floor terminate in a series of vertical shafts and risers to bring the flow up to the sea bottom for discharge through special jet manifolds. Since this system is hydrostatically unstable, there are special problems of seawater expulsion. This paper gives a simple analysis of the fresh-water flow rate required for purging such a system, and compares it with the much smaller discharge needed to control intrusion after purging has been accomplished. If purging is not achieved at peak discharges then seawater inflow through some risers will occur at less than peak discharges, while other risers still have outflow; in such a case, multiple flow configurations are possible. Results are presented in a parametric way to assist the designer in adjusting component sizes to achieve the desired purging and intrusion-prevention characteristics.

Introduction

Generally in hydraulic structures it is desired to have the flow uniquely determined by the imposed flows or heads. For example, sharp-crested weirs are ventilated so that the lower

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edge of the nappe may freely detach from the weir, and the head-discharge relation is unique or single-valued. An example of non-uniqueness is the liquid discharge from a tank into air through a short tube (length: diameter = 1:1) with a sharp entrance from the tank; in this case, the jet may either be a free contracted jet springing from the sharp-edged tube entrance, or it may fill the whole tube cross section.

In this paper, we shall examine some aspects of the purging hydraulics of a multiple-port ocean outfall diffuser, and see how hydraulic designers must be aware of non-unique flow situations resulting from the 2.7%-density difference between the ambient seawater and the wastewater effluent (essentially the same density as fresh water).

Problem Definition

We shall restrict our attention to tunnelled outfalls where the purging problem is more acute than for buried or surface-laid pipelines. Figs. 1 and 2 show the essential features of a tunnelled outfall system with risers and multiple ports in each riser head. Three such systems are currently under construction for Sydney, Australia, and designers are studying a similar option for discharge from the Boston metropolitan area into Massachussetts Bay. We will not present the analysis for any particular system but instead give an idealized example to illustrate the hydraulics.

After the initial decline tunnel (or vertical shaft), the tunnel under the sea floor slopes up slightly (slope S) so that any leakage during construction will drain back toward shore for safety. The risers (N = number) are connected at the invert of the tunnel to facilitate the expulsion of seawater and any accumulated sediments or settled solids. At the head of each riser a special manifold discharges through n nozzles directed horizontally in a radial pattern at angular separations of $360/n$ degrees. The total number of ports is then nN. This arrangement can give the equivalent of a line source for obtaining high dilution while keeping the number of risers reasonable [1]. The height of the riser from the top of the offtake from the tunnel to the centerline of the discharge ports (Fig. 2) is designated H.

Although the port diameters may vary slightly in order to equalize the flow, assume a representative diameter d_p such that the total discharge area is $nNd_p^2/4$. The area ratio R_1 (to-

Fig. 1. Schematic cross section of tunnelled multiport ocean outfall

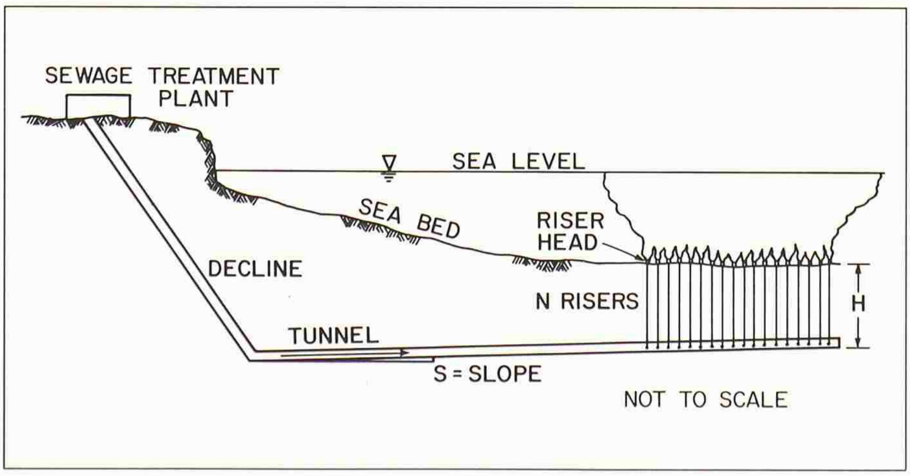
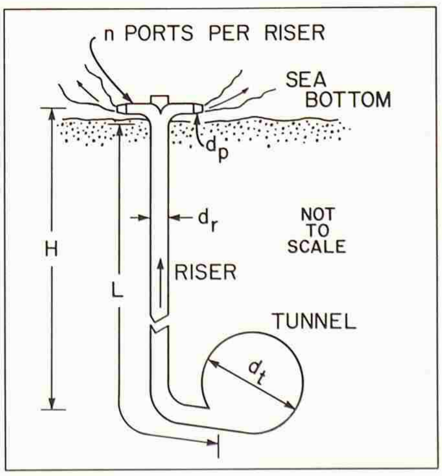


Fig. 2. Schematic cross section of a riser with bottom takeoff from the tunnel, and multiport riser head containing n ports arranged in a radial pattern



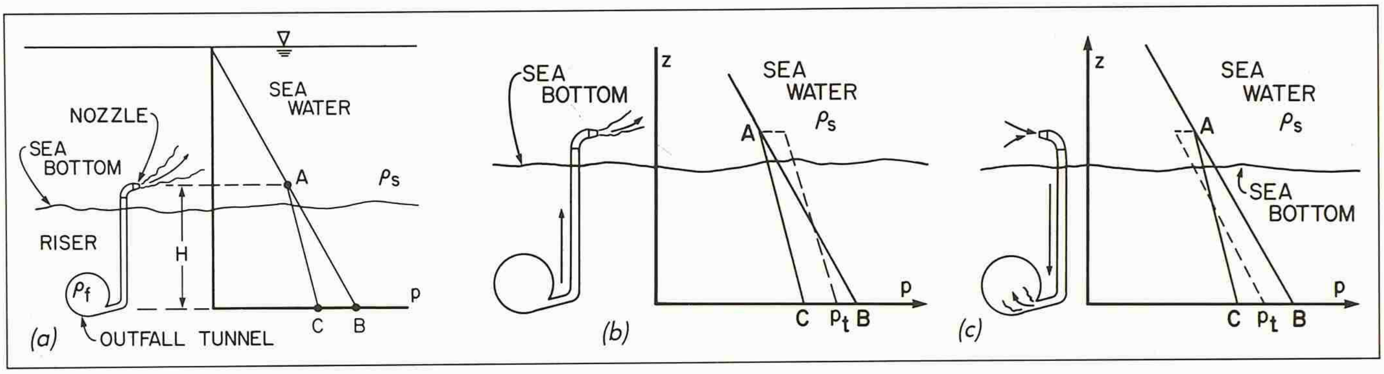


Fig. 3. Hydraulics of a single riser:
(a) Hydrostatic pressure distribution: AB for riser full of seawater; AC for riser full of effluent
(b) Pressure distribution (-----) for outflow (with pipe friction and nozzle loss) compared to hydrostatic pressures ($P_C < P_t < P_B$)
(c) Pressure distribution (-----) for inflow at same tunnel pressure

tal ports: tunnel) is nNd_p^2/d_t^2; the area ratio R_2 (ports: risers) is nd_p^2/d_r^2; and the area ratio R_3 (risers: tunnel) is $N\pi d_r^2/d_t^2$. Therefore, $R_1 = R_2 \cdot R_3$. For good manifold design all these ratios should be less than unity, for example $R_1 = 0.4$, $R_2 = 0.8$, $R_3 = 0.5$. The designer selects appropriate values based on manifold design, available head, range of discharges, costs, and purging problems, discussed here. (Other aspects of the manifold design are discussed in [2, 3, 4].)

For proper operation, the effluent flow should be capable of expelling all the seawater out of the tunnel and risers whenever the discharge is started from the condition of seawater flooding the entire system (such as the inital startup, or restart after a period of shutdown long enough for total seawater intrusion). The minimum total discharge required to purge the system (as described) is designated Q_P. After purging, the subsequent intrusion of seawater should be prevented at all operating flows. The minimum flow which will prevent intrusion is called Q_I.

Intrusion Criterion

It has been well established [4] that when a discharge port is flowing full, intrusion can be prevented by requiring the port densimetric Froude number to exceed unity, or for safety against perturbations and allowing for various geometries

$$(1) \qquad F_p = \frac{V_j}{\sqrt{g' d_p}} > 2$$

where V_j = port velocity = $q/(\pi d_p^2/4)$, q = port discharge, $g' = (\Delta\varrho/\varrho)g$, $\Delta\varrho = \varrho_a - \varrho$ π, π_a = ambient seawater density, ϱ = discharge density. By continuity we find that the required flow Q_I is:

$$(2) \qquad Q_I = 2nN(a_p)\sqrt{g' d_p}\ \text{, or}$$

$$(3) \qquad Q_I = 2R_1(\pi d_t^2/4)\sqrt{g' d_p}$$

where a_p = port area = $\pi d_p^2/4$. This criterion has been used successfully to control *intrusion*, but without much attention until the 1980's to the fact that the *purging* flow Q_P may be much larger than Q_I, especially for tunnelled outfalls. For example, some tunnelled outfalls in Great Britain [5, 6, 7] had an operating range (Q_{min} to Q_{max}) such that $Q_I < Q_{min}$, but $Q_{max} < Q_p$; since the outfalls never were fully purged, the intrusion could not be prevented by the criterion of Eq. (1).

Purging Criterion-Risers

We consider first the hydraulics of a single riser (Fig. 3). If the riser is filled with seawater (with no flow), then the hydrostatic pressure distribution would be AB. The pressure represented by B will be considered the reference pressure. If the riser is filled with fresh water (no flow), then the pressure is represented by AC. The difference in pressure $\Delta p = p_B - p_C = \Delta\varrho gH$. If the operating pressure in the tunnel at the riser is between p_C and p_B, then seawater can flow down some risers into the tunnel at the same time that fresh water is discharging from other risers [6, 8].

For bottom takeoff risers (large number N, with $d_r \ll d_t$) the criterion for starting outflow is that the tunnel pressure exceeds p_B, the salt water hydrostatic value. The excess pressure will push the riser fluid slowly up and out, and depletes any seawater wedge in the tunnel. When fresh water begins to be drawn into the riser, the mean density of the water column in the riser starts dropping, thereby *decreasing* the hydrostatic pressure and increasing the dynamic pressure leading to a progressive increase in flow rate until the normal fresh-water flow in a riser is established.

Using Bernoulli's equation for the normal riser outflow, energy in the tunnel at the riser entrance equals the sum of the entrance loss, friction loss in the riser pipe, bend loss, losses in the top manifold and the energy of the discharge (assuming bellmouth ports with no jet contraction, C_D = discharge coefficient):

$$(4) \qquad E = p_t + \varrho\frac{V_t^2}{2} = \\ = \Delta E_e + K_b\varrho\frac{V_r^2}{2} + f\frac{L}{d_r}\varrho\frac{V_r^2}{2} \\ + (1/C_D^2 - 1)\varrho\frac{V_i^2}{2} + \varrho\frac{V_i^2}{2} + p_A + \varrho gH$$

Since the entrance loss is small it may reasonably be approximated as

$$(5) \qquad \Delta E_e = \varrho\frac{V_t^2}{2} + K_e\varrho\frac{V_r^2}{2}$$

Letting $p_s = p_A + \varrho gH$ = hydrostatic pressure at tunnel level for fluid ϱ (effluent, seawater or mixed), the dynamic pressure is

(6) $p_d = p_t - p_s = \varrho \frac{V_j^2}{2}$

$\cdot [1/C_D^2 + R_2^2(K_e + K_b + fL/d_r)]$, or

(7) $p_d = \alpha \varrho \frac{V_j^2}{2}$,

where α = value of [ ] in Eq. (6).
Typical values of α, which is a system parameter, are derived as follows:

$C_D = 0.93 - 0.97$
$R_2 = 0.2 - 0.5$
$K_e = 0.1 - 0.5$
$K_b = 0.2 - 0.5$
$f = 0.015 - 0.03$
$L/d_r = 40 - 100$
$\alpha_{max} = 1/0.93^2 + 0.5^2(0.5 + 0.5 + 3)$
$= 2.16 \approx 2.2$
$\alpha_{min} = 1/0.97^2 + 0.2^2(0.1 + 0.2 + 0.6)$
$= 1.10$

Returning to Eq. (7), we see that it is a reasonable approximation to give the density in the $\varrho\, V_j^2/2$ term, a single value ϱ_o (according to the Boussinesq assumption). Finally, we get an overall discharge equation for a single riser as:

(8) $V_j = \sqrt{\frac{2(p_t - p_s)}{\alpha \varrho_o}}$

To find the purging criterion for the diffuser, we must consider a scenario. If the startup is slow, the effluent flow will establish itself in successive risers one-by-one starting with the offshore end, because of the slope of the tunnel. As Q increases an additional riser starts up whenever $p_t > p_B$, the hydrostatic seawater value. When $(N-1)$ risers have been started, the final riser will be purged when $p_t \geqq p_B$. Just before purging, there will be a slight reverse seawater inflow in the last riser which blocks the entry of fresh water to the riser until the threshold is reached. The system purging flow Q_p may be found from Eq. (8); noting that p_s = hydrostatic pressure = p_C for fresh-water discharge:

(9) $p_t - p_s = p_B - p_C = \Delta\varrho g H$

(10) $Q_P = (N-1)\, n\, a_p V_j = (N-1)\, n\, a_p$

$\cdot \sqrt{\frac{2}{\alpha} \frac{\Delta\varrho}{\varrho_o} gH}$

$= \frac{N-1}{N}(nNa_p)\sqrt{\frac{2}{\alpha} g'H}$, where $g' = (\Delta\varrho/\varrho_o)g$

Just after purging is completed in N risers, the dynamic tunnel pressure will drop back slightly to $\Delta\varrho g h\,(N-1)^2/N^2$ because the riser velocity is reduced by the factor $(N-1)/N$.

The purging flow criterion Eq. (10) may also be written like a Froude number as follows:

(11) $\frac{Q_P}{nNa_p\sqrt{g'H}} = \frac{V_j}{\sqrt{g'H}} = \frac{N-1}{N}\sqrt{\frac{2}{\alpha}}$

Note that the factor $\sqrt{2/\alpha} \approx 1.0$–$1.35$ and $(N-1)/N \sim 0.9$–0.99, so overall

(12) $\frac{V_j}{\sqrt{g'H}} \approx 0.9 - 1.3$

However, the value should be worked out for each system design, and the above values are intended only to show the magnitude.

This useful result shows the relationship of the required final jet velocity (the key variable) to the vertical height of the riser. For example, if $h = 50$ m, $N = 30$, $\alpha = 1.3$, $g' = 0.027\, g$, then

$V_j = (29/30)\sqrt{2/1.3}\,\sqrt{g'H}$
$= 1.20\,(3.64)$
$= 4.36$ m/s

If purging is desired at $Q_p = 5.0$ m³/s, then nNa_p = total port area = $5.0/4.36 = 1.15$ m²; for n = 6 ports per riser,

$a_p = \frac{1.15\ \text{m}^2}{6(30)} = .0064\ \text{m}^2$

$d_p = 9.0$ cm

We may now examine the ratio of the critical flow for intrusion Q_I to the critical flow for purging by dividing Eq. (10) by Eq. (2):

(13) $\frac{Q_P}{Q_I} = \frac{N-1}{N}\sqrt{\frac{2}{\alpha}}\sqrt{\frac{H}{d_p}} \cdot \frac{1}{2}$

For the above example

(14) $\frac{Q_P}{Q_I} = \frac{1.20}{2}\sqrt{\frac{50}{0.09}} = 14.1$

This ratio is surprisingly large, considering that for a sewerage system Q_{max}/Q_{min} rarely exceeds 10; it shows why designing to make $Q_{max} > Q_P$ is likely to be more stringent than $Q_{min} > Q_I$ for a tunnelled outfall.

The above analysis is not sufficiently detailed for final design but it can be helpful for scaling or preliminary decisions.

Munro [9] first suggested a condition in the form of Eq. (12), namely:

(15) $\frac{V_j^2}{2g} = \frac{\Delta\varrho}{\varrho} H$, or

(16) $\frac{V_j}{\sqrt{g'H}} = \sqrt{2}$

This was based on the approximation that the total dynamic head for the riser was just the velocity head of the discharge jets. Since the other losses and the factor $(N-1)/N$ reduce the required purging head it is worth taking them into account.

The previous discussion presumed a slow startup, leading to a conservative estimate of the purging flow (called the Munro condition). It may be possible to purge an outfall at a lower flow by concurrent purging of a group of risers as the last step. The procedure would be to start slowly to expel most of the seawater and establish flow in ½ to ¾ of the risers. Then a rapid rise to a higher discharge within just a few minutes could force *concurrent* startup of all the risers in the remaining group. This may be expected because the time of flow establishment in a single riser is probably of the order of a few minutes. This would probably only work if the wedge of residual seawater in the tunnel is small and confined to the diffuser section. This procedure has not yet been demonstrated. The required purging discharge would be lowered approximately in proportion to $(N-m)/(N-1)$, where m is the number started concurrently (see Eq. [10]).

Purging Criterion-Tunnel

There is a second purging criterion, related to the expulsion of water from the long sloping tunnel. When fresh water is introduced into the upward sloping tunnel, it will establish a buoyancy front, and since the channel is long, an inverted open channel flow may be established. The seawater wedge can only be driven out if the discharge is increased to the point where the tunnel must flow full with a hydraulic slope $S_e > S' = (\Delta\varrho/\varrho_o)\ S$, where S is the tunnel slope. Using the Darcy-Weisbach friction factor f, the critical flow to achieve tunnel purging is then:

$$(17)\quad Q'_P = (\pi d_t^2/4)\sqrt{\frac{8_g d_t S'}{4f}} = (\pi d_t^2/4)\sqrt{\frac{2_g' d_t S}{f}}$$

The ratio of this flow to the flow required to purge the risers is found by dividing by Eq. (10), and using the area ratio R_1 (ports to tunnel):

$$(18)\quad \frac{Q'_P}{Q_P} = \frac{N}{N-1}\,\frac{1}{R_1}\sqrt{\frac{\alpha d_t S}{Hf}}$$

This flow ratio will normally be less than unity, indicating that the seawater is driven from the tunnel (given enough time) before the purging is complete in the diffuser section. However, the following factors will *increase* the ratio of tunnel Q_P' to the riser Q_p: increase in tunnel diameter, slope, and/or riser losses (α); decrease in port areas, riser height, and/or tunnel friction factor; and reduction in Q_p by special slow/fast startup ($N-1$ replaced by $N-m$). For example, given $R_1 = 0.15$, $d_t = 4.0$ m, $H = 30$ m, $\alpha = 1.2$, $f = 0.020$, $S = 0.005$, $N = 30$, then $Q_p'\ Q_p = 1.38$. In this case, the diffuser section could purge but a residual seawater wedge would be left behind in the tunnel to be gradually removed by entrainment; in the meantime, the dead zone would be an undesirable sediment trap.

Reverse Flow in Risers

For the complete picture, we now examine the hydraulics of inflow from the sea back through the discharge ports and risers. This situation may occur (1) if: the outfall diffuser has not been completely purged; or (2) if the discharge drops below the critical value Q_I for intrusion through the ports (given by Eq. [2]), after the diffuser has previously been completely purged.

One significant difference between inflow and outflow hydraulics is that the ports will have a high energy loss for backward flow, which we describe as $K_p\ V_p^2/2g$, where V_p is the nominal port velocity not considering the severe contraction of flow at the entrance; the value of K_p may be expected to be about 3, including all the losses in the top manifold and elbow. Also at the exit from the riser into the tunnel, the full velocity head of the riser flow is assumed to be lost.

The driving force is the negative dynamic pressure in the tunnel—the amount the tunnel pressure is below the hydrostatic pressure for seawater, $p_B - p_t$. Including the intake loss, the pipe friction, bend loss, and exit loss, we obtain:

$$(19)\quad p_B - p_t = \varrho_o\frac{V_p^2}{2}\left[K_p + R_2^2(fL/d_r + K_b + 1)\right]$$

or defining β to be the sum of the coefficients in [ ]:

$$(20)\quad p_B - p_t = \beta\,\varrho_o\frac{V_p^2}{2}$$

For typical values of $K_p = 3$, $R_2 = 0.5$, $fL/d_r = 2$, $K_b = 0.3$, we obtain $\beta = 3.8$, showing the predominance of the inlet loss. For the same values, the corresponding coefficient for outflow would be about $\alpha = 1.8$ by Eqs. (6, 7). Thus the head loss for inflow is more than twice as large, or for a given driving pressure the reverse flow would be only 0.7 times as much as the normal outflow.

We can now define the possible system states when the tunnel pressure is between the hydrostatic values for fresh water and seawater ($p_C < p_t < p_B$ in Figs. 3b and 3c). If k risers have reverse flow q_i and $N-k$ have outflow q_o, and Q is the wastewater discharge, then by continuity

$$(21)\quad Q + kq_i = (N-k)q_o$$

The sum of the dynamic pressures driving the two types of flows is obtained by adding Eqs. (7) and (20) and equal to the constant difference between hydrostatic pressures for sea and fresh water (and using the Boussinesq assumption):

$$(22)\quad \alpha\,\varrho_o\frac{V_j^2}{2} + \beta\,\varrho_o\frac{v_p^2}{2} = p_B - p_C = \Delta\varrho g H$$

The velocities are respectively equal to the individual riser discharges (outflow and inflow) divided by the area of the ports na_p for each riser. It is convenient to normalize these equations by $q_{oo} = na_p V_{jo}$, where V_{jo} is the outflow port velocity corresponding to $p_t = p_B$, or no seawater inflow ($V_p = 0$ and $V_{jo} = \sqrt{z'gH/\alpha}$). The corresponding dimensionless riser flows will be designated q_o^* and q_i^* for "out" and "in" respectively. The resulting dimensionless forms of Eqs. (21) and (22) are:

$$(23)\quad Q/q_{oo} + kq_i^* = (N-k)q_o^*$$

$$(24)\quad q_o^{*2} + (\beta/\alpha)\,q_i^{*2} = 1$$

There are three unknowns, q_o^*, q_i^*, and k, but only two equations. Provided that $Q/q_{oo} < N$, there are separate solutions for all non-negative values of k satisfying the contraint $Q/q_{oo} < N-k$ or

$$(25)\quad 0 \leqq k < N - Q/q_{oo}$$

The relationship between the tunnel head, the outflow and the inflow is shown in Fig. 4. The ordinate in Fig. 4 is the dimensionless pressure difference $p^* = (p_t - P_C)/\Delta\varrho g H$, which is 0 at fresh-water hydrostatic and 1 at seawater hydrostatic pressure.

The reader is cautioned that this analysis is limited by the assumption that the outflow is at fresh-water density p_f, whereas it is likely that seawater inflows to the tunnel will be partially or possibly fully mixed with the fresh water and discharged back out. This phenomenon has been described by Wilkinson [7]. In Fig. 4, the middle curve shows the head-discharge relationship for the outflow density ½ $(p_f + p_s)$, a mixture of half fresh water and half seawater.

From Fig. 4 it is interesting to note: (a) that even a small decrease in q_0^* from 1 to 0.90 will cause an increase in inflow of q_i^* from 0 to 0.31; (b) that under the same head condition ($p^* = 0.81$), the outflow which is a mixture of 1:1 effluent and recirculated seawater would have an outflow $q^* = 0.56$. Thus, the possibility of tunnel mixing during recirculation increases still further the range of possible flow states.

The question remains: how many risers will have reverse flow? This depends on the past history of the system, and *cannot* be answered by a specification of *only* the present head or the net discharge as one might expect! If the outfall has been fully purged (the desired condition of operation), then no ris-

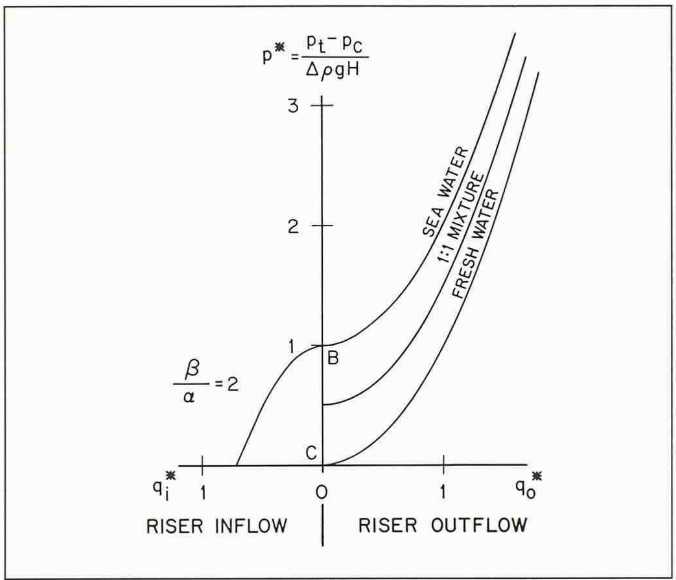


Fig. 4. Outflow and inflow as a function of tunnel pressure (for the case $\beta/\alpha = 2$). Note the possibility of either outflow of fresh water or inflow of seawater for tunnel pressures in the range $0 < p^* < 1$. (Based on Eq. (7) for outflow and Eq. (20) for inflow, with normalization)

ers will have reverse flow ($k = 0$) as long as the intrusion is prevented at each individual nozzle (Eq. [3]). If not previously purged, there will be inflow in those risers in which outflow was never established, and probably nearby risers due to tunnel mixing (not included above).

Closing Discussion

A simple hydraulics problem involving risers and discharge ports is not so simple when seawater and fresh water are involved in conditionally stable flows. Multiple states of flow are possible, with the previous flow history determining what actually happens. The analysis here has been simplified to show the role of the main outfall features.

The problem of starting a multiple riser outfall system in the ocean may be very roughly compared to starting up many fireplaces in a large cold house. When each fire is started, the fireplace reduces the room pressure due to the buoyancy in the chimney, which then starts cold air flowing *down* the chimneys not yet started. As more fireplaces are started, the downdrafts in the last chimneys become quite strong! But how can you stop this inflow long enough to get hot air into the last chimney to make it draw? (Open the front door!) Although the analogy is inexact (there would be no net flow into the house, just buoyancy flux generated inside), the comparison may help to make the confusing outfall behavior more intuitive.

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