4 HYDROLOGY OF COASTAL AREAS

4.1 Introduction

In the context of this subject, coastal areas comprise the lowland areas bordering estuaries and deltas, or, more in general, the lower reaches of rivers, and coastal marshlands and lagoons, which have a predominantly marine character.

There are two reasons to pay special attention to the hydrologic and water control of such coastal areas as distinct from the upper portions of a river basin: A. the special nature of the hydrologic processes and B. the economical significance of coastal areas.

A1. In the upper portions of a river the water levels and flows are governed by the precipitation upstream of the station under consideration; in a delta on the other hand the water levels and flows are governed by the conditions at the two ends: the water levels and flows at the apex (the point where the river starts branching off into distributaries or branches) and the water levels of the recipient basin (sea, ocean, inland sea or lake).

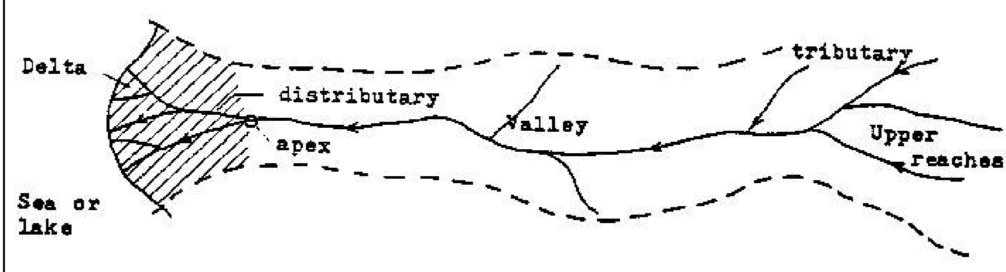

Figure 4.1: Delta

A2. In most cases the rainfall on the coastal area has little effect on the discharges and levels of the distributaries because the upstream discharge (or freshet) at the apex and the flows generated by the tides are much larger than the discharge generated by the run off from the coastal areas. The latter comes into the picture in the design of drainage systems of embanked areas.

A3. Coastal areas are characterized by small gradients (from 1.10^{-5} to 50.10^{-5}) and are exposed to extensive flooding which affects the flow processes.

A4. Coastal areas are also exposed to salt water intrusion from the sea into the open estuaries and from the ground water (section 4.4).

B The economic significance of coastal areas is demonstrated by the fact that more than 1/3 of the world population is living less than 100 km away from the coastlines and at an altitude less than 200 m above Mean Sea Level (MSL). 700 Million people are living less than 50 km away from coastlines. The largest cities of the world (Tokyo, New-York, London, Shanghai, Calcutta, etc.) are all located close to the sea. The low-lying coastal areas and especially the large deltas of Asia are important areas of agricultural production.

4.2 Astronomical tide and storm surges

4.2.1 The origin of astronomical tides

The sea level is subject to short period variations: the astronomical tides resulting from the attractive forces of the moon and the sun and the storm surges, mostly generated by strong winds.

Astronomical tides are perfectly predictable for any future real time and information can be found in "Tide Tables" for many coastal stations.

Most coastal stations show semi-diurnal tides with two high waters and two low waters during a day of 24 hours and 50 minutes. The two high waters and the two low waters show different heights with respect to MSL, the daily inequality. This is due to the declination (angle between the planes of the moon and the sun) with respect to the equator.

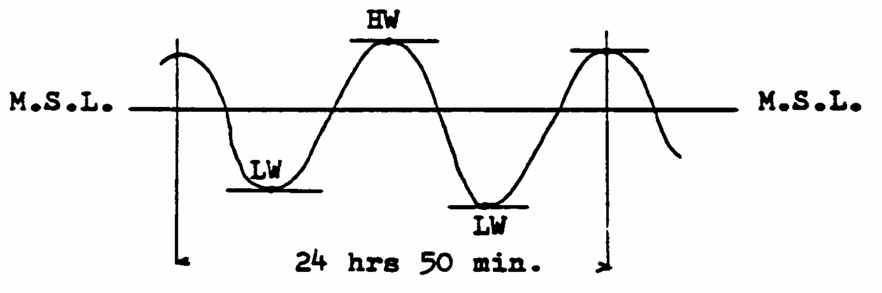

Figure 4.2

Diurnal tides (only one HW and one LW) occur in the Chinese Sea, the seas around Indonesia, the Gulf of Mexico and the Gulf of the St. Lawrence.

In some other places mixed tides occur: during a certain part of the year the tides are semi-diurnal and the other part diurnal.

The tidal range varies according to the position of the sun and the moon with respect to the earth: spring-tides at full moon and new moon and neap-tides during first and last quarter of the moon (1 or 2 days later: the age of the tide).

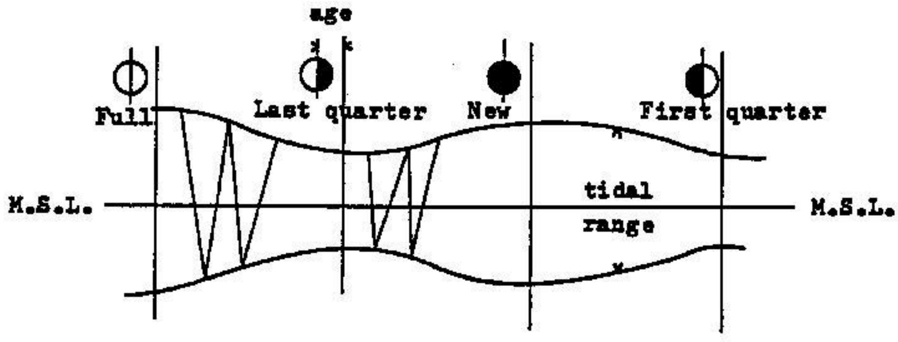

Figure 4.3

4.2.2 Storm surges

The magnitude of the tidal ranges varies considerably from one place to another; ocean tides are generally weak (a few decimetres), by resonance and reflection in seas in connection with the ocean high tidal ranges can occur with 15 m at spring-tide in the Bay of Fundy (Canada) as an extreme case.

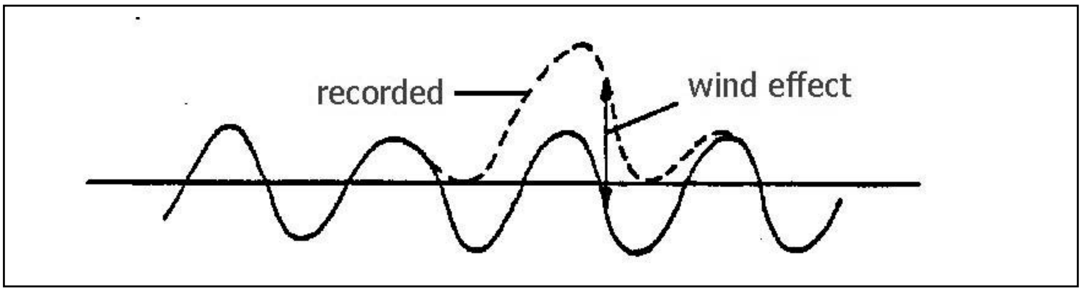

Figure 4.4: Wind effect

In many coastal stations the actually occurring sea levels deviate from the predicted astronomical tides (storm surges). This is mostly due to strong landward and seaward winds. Their effect is super-imposed on the tides. Seas and coasts where these effects are very pronounced (a few metres) are: the North Sea, the east coast of South America, the Bay of Bengal (wind effects of 6 to 9 metres), the west coast of the Pacific Ocean and the Gulf of Mexico. On many other coasts wind effects of a few decimetres may occur.

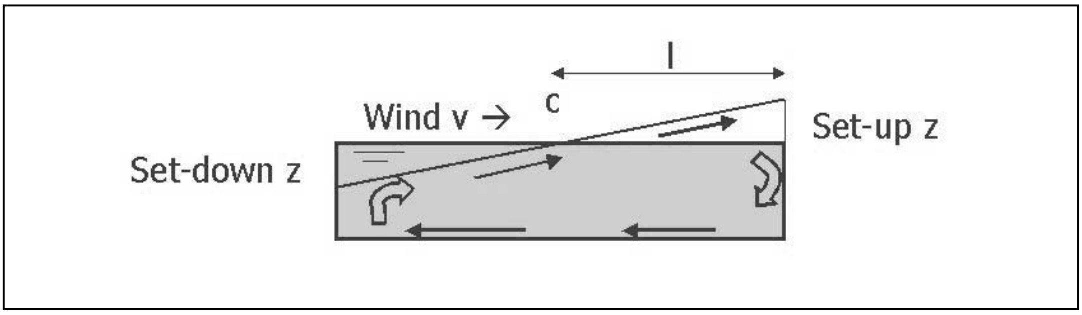

Figure 4.5: Set-up of the water level

Wind effects are generated by the 'drag' of the wind when blowing over a water surface. The friction between the wind and the water surface entrains the water and the water surface is no longer horizontal but assumes a slope.

If:	l =	distance in metres from the zero point C to the station P on the shore;
	v =	wind velocity in m/sec at 6 m height;
	d =	water depth in metres (including the set-up);
	z =	set-up of the water level
and	$z << d$	

then

$$z = 3.6 \cdot 10^{-6} \frac{v^2}{gd} \cdot l$$ *Equation 4.1*

Under these conditions the neutral line NN (no set-up) has to pass through the centre of gravity *C* of the surface of the lake. The set-up in a station *P'* is *z* (in *P*) times cos α.

If *z* is not small with respect to d the water level is no longer a straight line and the zero point no longer coincides with the centre of gravity.

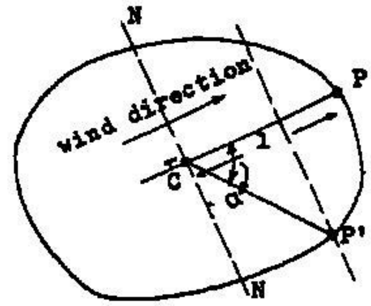

Figure 4.6: Neutral line

The set-up may be considerable: e.g. *v* = 50 m/sec, *l* = 600 kilometres and *d* = 100 m, *z* = 5.4 m. This may cause flood disasters in low-lying coastal areas: 1953 in The Netherlands, 1959 in Japan (Nagoya), and in Bangladesh (in 1970 more than 300 000 people drowned and in 1991 over 130 000 casualties).

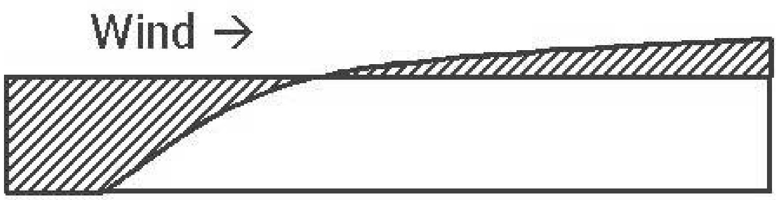

Figure 4.7

The formula for z refers to equilibrium conditions i.e. conditions when the wind has blown a sufficiently long time to displace the water in the basin. This may take say 2 days (North Sea).

Storm surges are generated by depressions moving in the atmosphere. In this respect distinction is made between:

- extratropical depressions of the temperate zones. These depressions cover large areas, move relatively slowly and generally affect the water levels during 1 - 3 days. Maximum hourly wind velocities (50 -100 years return period) are of the order of 25 - 30 m/sec;
- tropical depressions which according to the region are indicated as: (*v*: 40 - 50 m/sec 1 day or less);
  - cyclones (Bay of Bengal);
  - typhoons (Pacific coast of Japan, Taiwan, etc.);
  - hurricanes (Gulf of Mexico).

Abnormally high sea levels can also result from remote effects:

- differences in barometric pressure;
- tsunamis (long wave generated by submarine earthquakes, landslides and volcanic eruptions). These waves travel through the oceans and may hit coastal regions.

In monsoon climates with different wind directions during the different seasons the mean sea levels during each of these seasons may be affected by the prevailing wind during that season and differ a few decimetres from the mean sea level during other seasons.

4.3 Propagation of astronomical tides and storm surges into estuaries

4.3.1 Astronomical tides

Tides and storm surges propagate in open river estuaries and lower river reaches. To understand the hydrology of such area it is necessary to have at least a general understanding of the hydraulic processes involved.

The central problem "tidal hydraulics" (shallow water tidal waves) can be formulated as follows: given the variations of the sea level, the upland discharge and the geometry of the channels, to compute the water levels and discharges in the estuaries. Although considerable progress has been made in recent decades in solving this problem by the use of mathematical models, it is still necessary for the calibration and verification of the models to carry out observations in the field of water levels and discharges. Although it is not necessary for the understanding of the hydrologic conditions of coastal areas to know the methods of tidal hydraulics, it is only with these models that predictions can be made of changes in the flow regime caused by changes in the system (diversions, damming off of branches, deepening, etc.).

If the sea level would be constant the water levels would only be governed by the upland discharge and the flow in the estuary would be permanent in the entire lower reach. It can be noted that in the coastal strip the effect of the upland discharge on the water level is small.

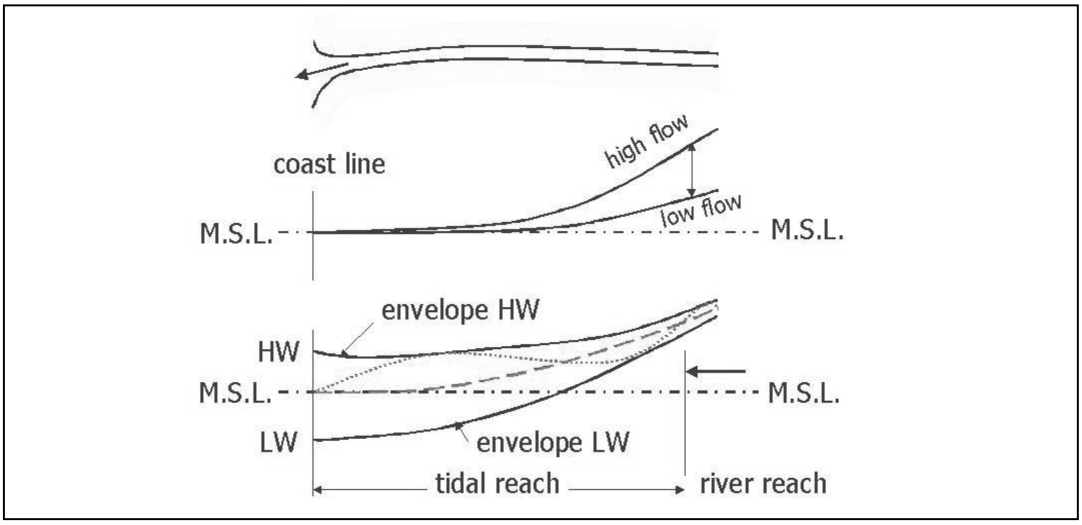

Figure 4.8

The situation is much more complicated in case of tidal variations at the sea boundary. In the tidal reach the water levels in a station also vary between two levels (for a given discharge) and the figure represents the two envelope curves of the high and low water levels in the estuary.

Since the propagation of the tide in the estuary takes time, the momentary situation may be as shown in the figure by the dashed line.

To understand in a qualitative way the intricate pattern of levels and flows in tidal river estuaries it is useful to firstly consider the simple case of a tidal basin (without inflow from the river), which is in open communication via a narrow but deep opening with a sea with tides. The tidal variation at sea propagates into the basin. In the opening an alternating current is generated with alternatively inflow (flood) and outflow (ebb).

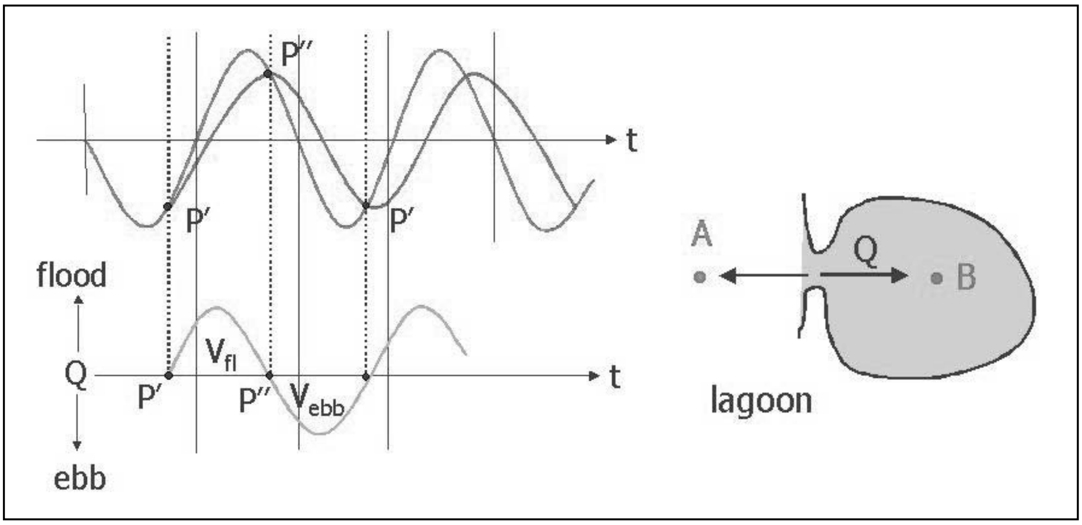

Figure 4.9: Lagoon concept

Because of the time lag in the propagation of the tidal wave, the water levels at A and B are not the same: during the flood the water level in B is lower than the one in A and the reverse is true during ebb. The result is a variation of the water level in B as shown in Figure 4.9 (it is assumed that in the tidal basin the water levels are horizontal; lagoon concept).

The points P' and P" indicate the moments of reversal of the current of slack water;
P' corresponds to low water slack (LWS), and P" corresponds to high water slack (HWS). In this example, where the lagoon concept is applied, HWS occurs at high water (HW) in B. This situation corresponds to a standing wave.

If subsequent tidal ranges are equal (no daily inequality), the two areas indicated by V_{fl} (flood volume) and V_{ebb} (ebb volume) should also be equal.

If a river is debouching into the tidal basin the effect of the inflow Q (supposed to be constant during the tidal cycle) can be superimposed on the flow generated by the tide. The ebb flow is reinforced and the flood flow is pushed back. For reasons of continuity (daily equality):

$$V_{ebb} - V_{fl} = \overline{Q} \cdot T \qquad \textit{Equation 4.2}$$

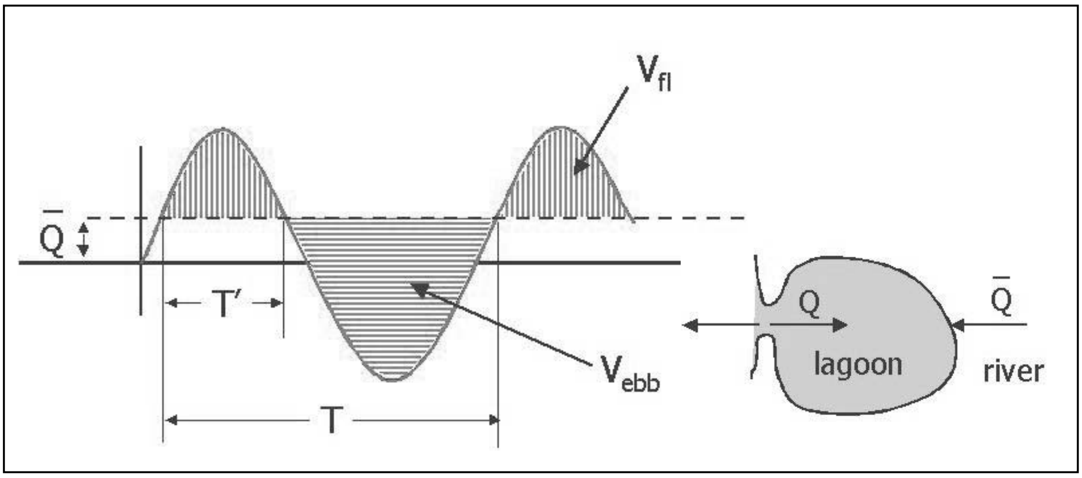

Figure 4.10

An important parameter in salt intrusion models is formed by the flood number or Canter-Cremers number *N*:

$$N = \frac{\text{volume of upland discharge during tidal period}}{\text{volume of the flood}}$$

$$= \frac{QT}{V_{fl}}$$

Equation 4.3

N is dimensionless and indicates the relative importance of the river flow with respect to the tide; if $N = 0$, there is no river flow; if $N \rightarrow \infty$, no tidal influence.

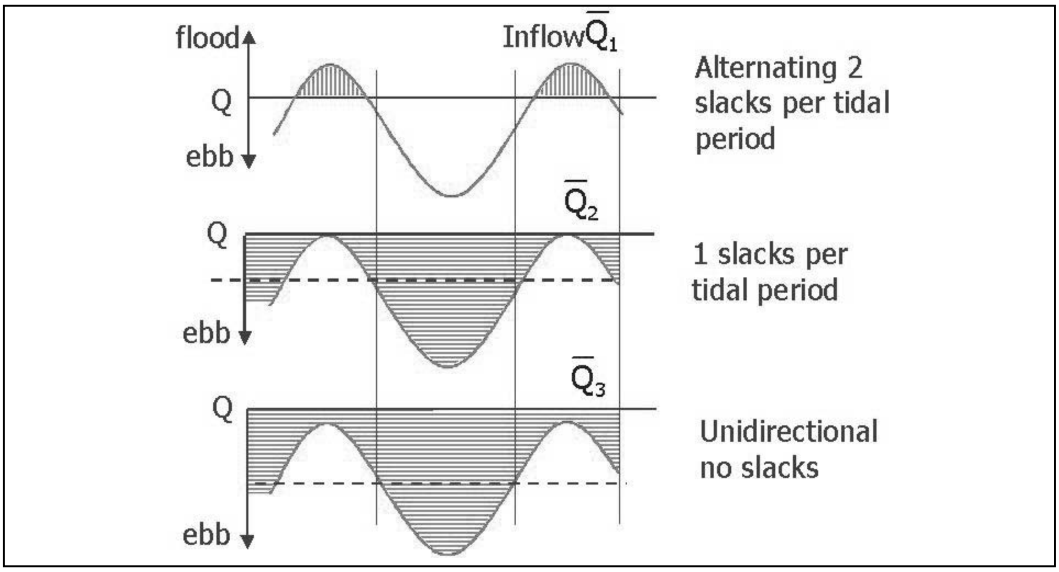

Figure 4.11: Amount of slack per tidal period

Depending on the magnitude of *Q*, the flow in the opening may be alternating or unidirectional as shown below. In the case with Q_2, there is only one slack water during a tidal cycle at which the flow does not change direction.

In an alluvial estuary, the transition between a sea and an alluvial river, the lagoon concept cannot be applied. The estuary is elongated and the time of travel can no longer be neglected. Neither can the friction be ignored.

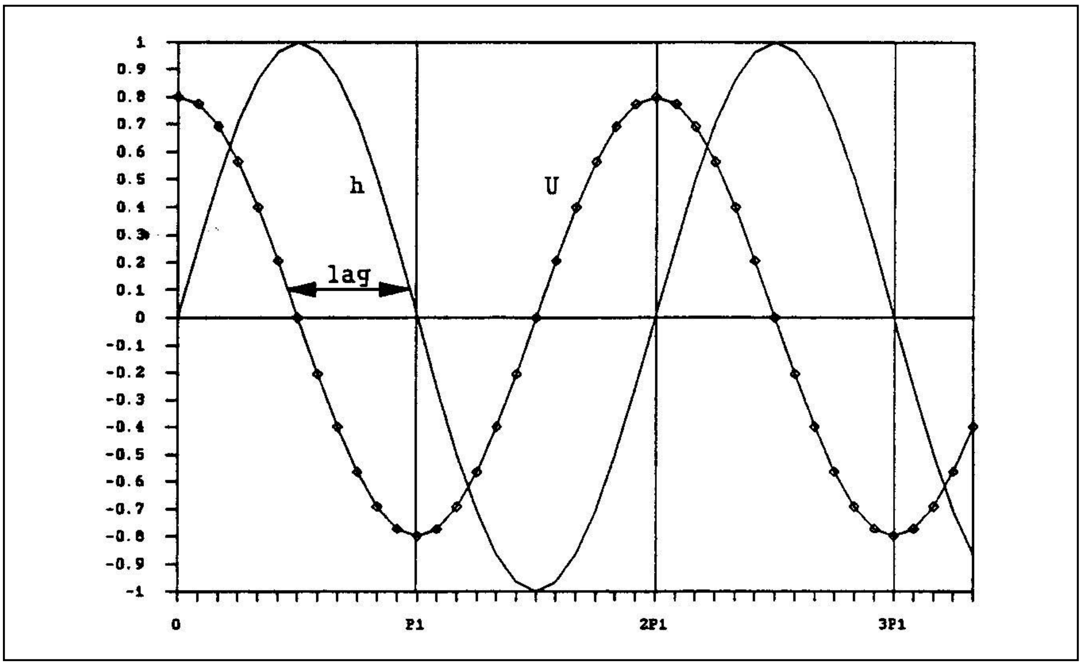

Figure 4.12: Standing wave

Three types of waves may be distinguished:
- a standing wave
- a progressive wave
- a wave of mixed type

Only the latter type of wave occurs in an alluvial estuary, which gradually tapers into an alluvial river. A purely standing wave requires a semi-enclosed body where the tidal wave is fully reflected. Since an alluvial estuary gradually changes into a river, this case does not apply. Standing waves only occur in non-alluvial estuaries or in estuaries where a closing structure has been constructed. In case of a standing wave, extreme water levels are reached simultaneously along the estuary.

Consequently, the "apparent" celerity of propagation c tends to infinity (as extreme water levels occur everywhere at the same time, it seems as if the celerity of propagation is infinitely large). Moreover, HWS coincides with HW (high water) and LWS coincides with LW (low water). The phase lag φ between the fluctuation of water level h and flow velocity U is $\pi/2$ (see Figure 4.12). In Figure 4.12, the positive direction of flow is assumed in upstream direction.

A purely progressive wave only occurs in a frictionless prismatic (constant cross-section) channel of infinite length. Alluvial estuaries do not belong to that category. In case of a progressive, water level and stream velocity are in phase: high water occurs at the same time as maximum flow velocity. The phase lag φ is zero and the wave celerity $c=c_0=\sqrt{(gh)}$ (see Figure 4.13)

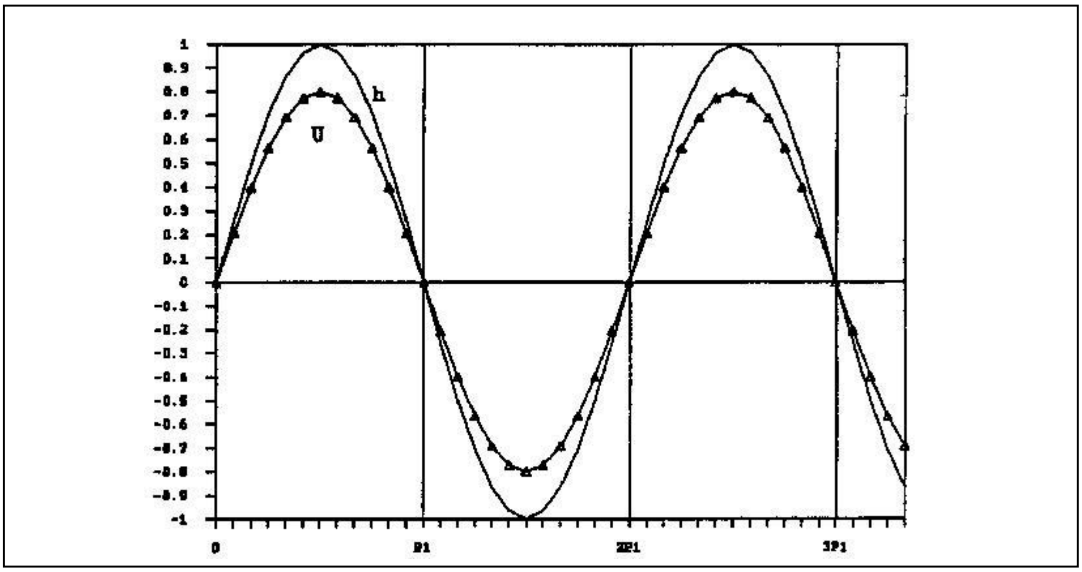

Figure 4.13: Progressive wave

None of these extreme situations occurs in an alluvial estuary. Hence, the tidal wave in an alluvial estuary is of a mixed type with a celerity inferior to c_0 and a phase lag φ which lies between 0 and $\pi/2$.

Another way to determine the phase lag is by looking at the lag time between HW and HWS (or LW and LWS). The resulting phase lag, indicated by ε in Figure 4.14, equals $\pi/2-\varphi$, and is an important parameter indicating the character of an estuary. ε can be easily computed by multiplying the time difference between the occurrence of HW and HWS (or LW and LWS) with $T=2\pi/T$. Like φ, ε varies between 0 and $\pi/2$ (see Figure 4.14).

As a result, in an alluvial estuary, HWS occurs after HW and before mean tidal level; and LWS occurs after LW and before mean tidal level. In an estuary with semi-diurnal tide, HWS occurs approximately 40 to 60 minutes after HW. In case of a diurnal tide the time lag is twice as large.

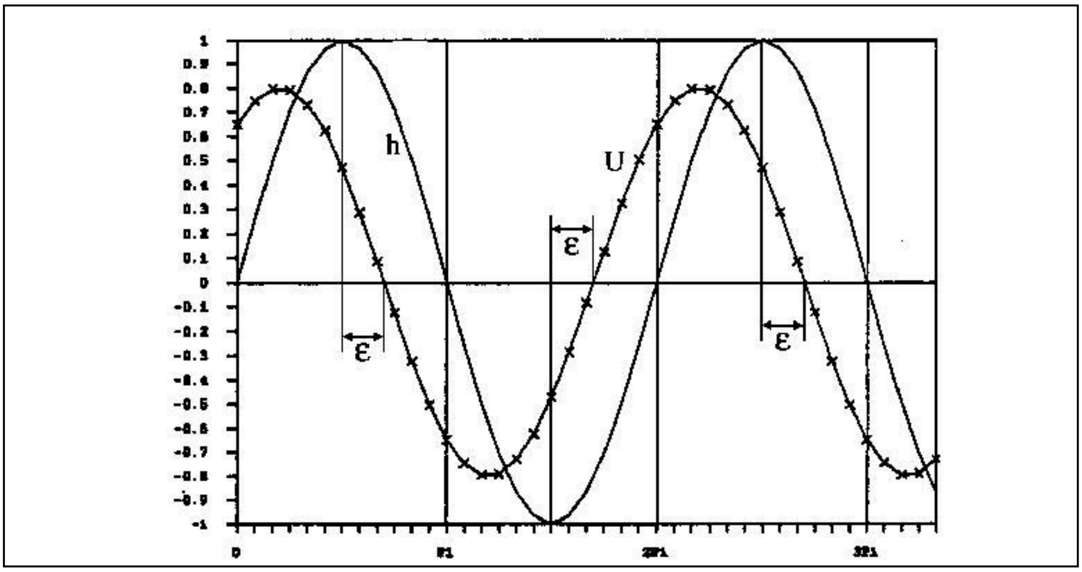

Figure 4.14: Wave of mixed type

Several researchers have approached the phenomenon of tidal wave propagation on the basis of prismatic channels of infinite length (e.g. Ippen, 1966; Van Rijn, 1990). This has led in some cases to incorrect conclusions. Van Rijn (1990) states that bottom friction and river discharge cause the phase lag between horizontal movement (current velocities) and vertical movement (water levels).

The effect of the river discharge is not a phase lag, but a vertical shift of the velocity-time graph, which causes HWS to occur earlier and LWS to occur later. The second cause mentioned, the friction, only has a minor effect on the phase lag. Computer simulation of an estuary with a constant depth of 10 m, a slight funnel shape (width reduction of 18% over 40 km), a tidal range of 3 m, and a Chezy coefficient of 60 $m^{0.5}/s$ showed that the velocity amplitude doubled if the Chezy coefficient was increased to an extremely high value of 1000 $m^{0.5}/s$ (frictionless flow) but that the phase lag φ only decreased from 0.3π to 0.22π and not to zero. A more important cause for phase lag is the shape of the estuary, which, depending on the convergence of the banks and the bottom causes partial reflection of the tidal wave and consequently a tidal wave of the mixed type.

Figure 4.15 shows a longitudinal profile of the water levels in an estuary at three different times t_1, t_2 and t_3, to illustrate the progressive character of the tidal wave. The mean level is slightly above MSL because the friction is higher during the ebb than during the flood.

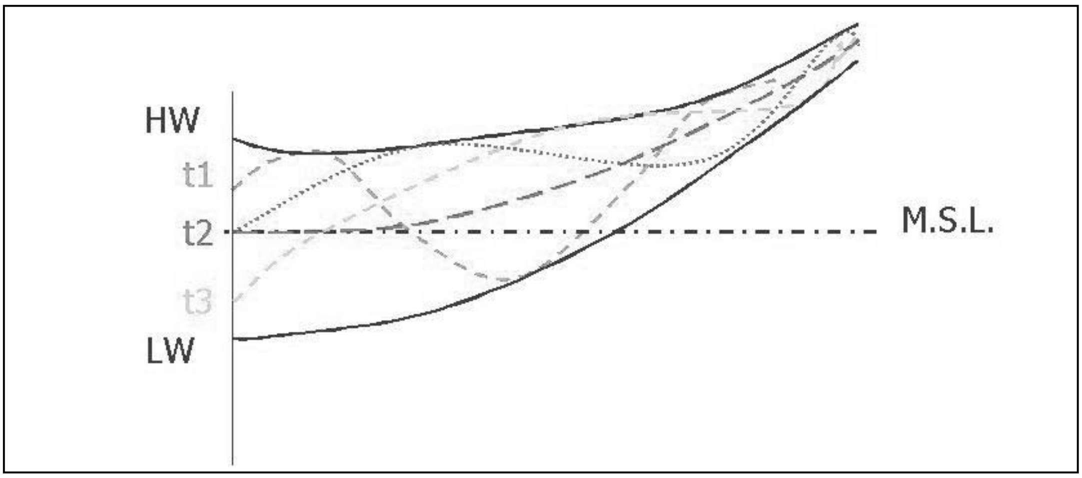

Figure 4.15

Figure 4.16 shows that the longitudinal profile of water levels can be divided into three reaches. The dotted lines represent the levels in the estuaries where current reversals occur. It is observed that the volume of water between the loci (envelopes) of slack water is equal to the flood volume V_{fl} upstream of any cross-section. Thus two methods exist for the determination of the flood volume ε:

- from integration of the discharge over the time between LWS and HWS
- from integration of the water levels between HWS and LWS over the area

Integration of the discharge between LWS and HWS yields:

$$\int_{LWS}^{HWS} Q dt = EA$$ *Equation 4.4*

Integration of *H'* over the surface area yields:

$$\int_{x}^{\infty} H' B dx = \frac{HBb \cos \varepsilon}{1 - b/\delta}$$ *Equation 4.5*

where *H'* is the tidal range between HWS and LWS, *E* is the tidal excursion of a water particle, *A* is the cross-sectional area, *B* is the width, *b* is the convergence length of the width ($B=B_0$ exp($-x/b$)), δ is the damping length of the tidal velocity amplitude ($v=v_0$ exp($-x/\delta$)) and ε is the phase lag between HW and HWS (equal to $\pi/2-\varphi$). Equating these two volumes leads to the following simple expression (Savenije, 1992, 1998):

$$Eh = \frac{Hb \cos \varepsilon}{1 - b/\delta}$$ *Equation 4.6*

and subsequently:

$$\frac{H}{E} = \frac{h_0}{b} \frac{(1 - b/\delta)}{\cos \varepsilon}$$ *Equation 4.7*

It should be noted that - for a constant *Q* - the value of ε differs according to the location in the estuary: ε is minimum in the mouth and infinite in P.

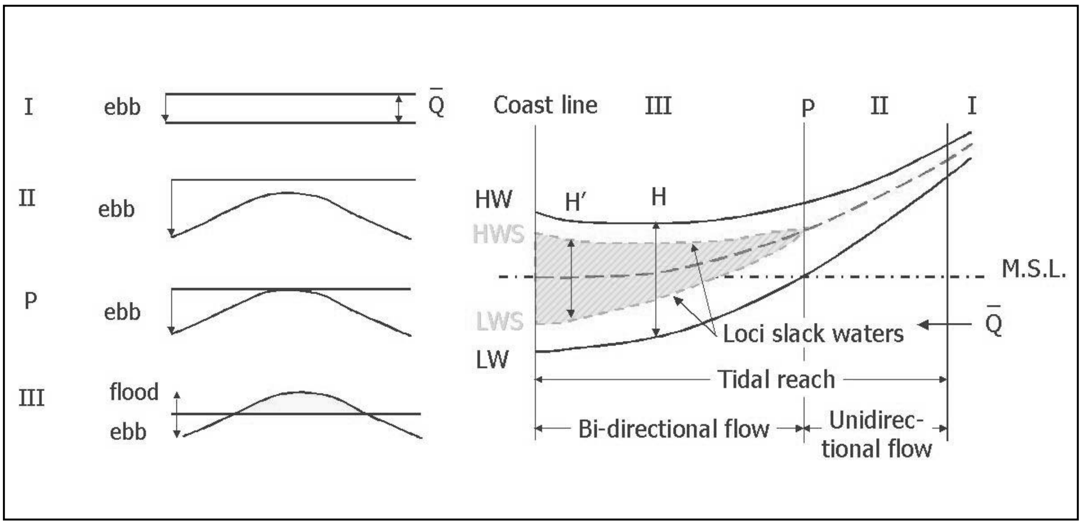

Figure 4.16

The distance over which the tides penetrate into an estuary (theoretically infinite) depends on the tidal range, the upland discharge and the channel characteristics (convergence length, roughness and especially the gradient). At low flows of the Mekong (2000 m^3/sec) the tides from the South China Sea penetrate as far as Phnom-Penh, 300 km from the mouth (gradient 1 to 3 cm per km). In the Gambia estuary the tides even propagate over a distance of 500 km, which is a distance in the same order of magnitude as the wavelength.

With increasing upland discharge the length of the tidal reach decreases: the river flow pushes the tides back.

Because of the effect of the tides on the discharges it is difficult to determine the upland discharge *Q* from measurements in the reach where the flow is alternating. In this reach *Q* has to be computed from the difference between flow and ebb volumes which both may show errors in the order of 5 to 10%. After subtraction of the ebb and flood volumes the relative error may well turn out to be more than 100%.

4.3.2 Storm surges

Storm surge levels also propagate into estuaries. Storm surges are non-periodic long wave of rather short duration especially when generated by tropical depressions. In non-embanked deltas they cause extensive flooding. Since the filling of the estuaries and especially of the land areas takes time and requires gradients, the highest water levels inside may remain much lower than the maximum water level at sea (case α). Only for storm surges of long duration (several days) may the areas be filled up with water to the same extreme level as at sea (case β).

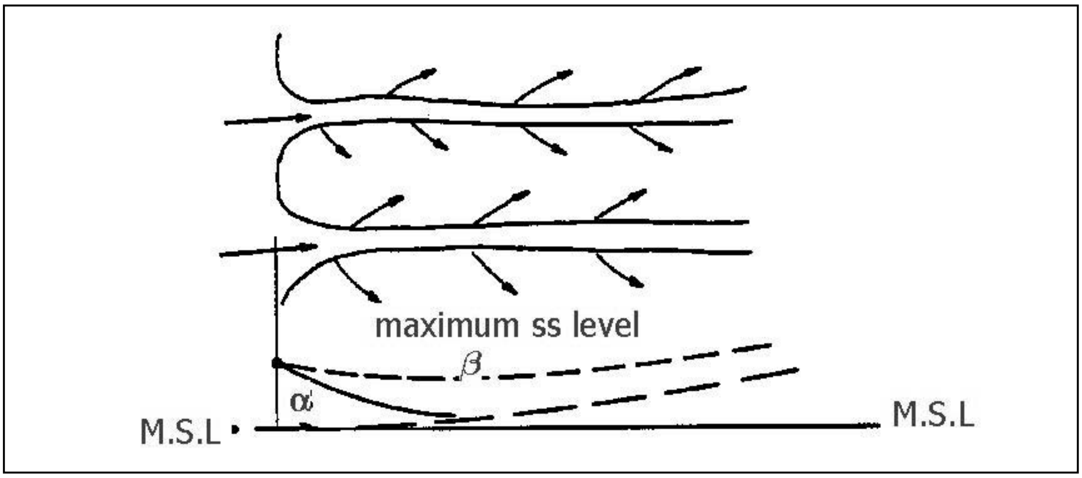

Figure 4.17

From this follows that embanking in the coastal strip brings about a rise of the water levels in the estuaries in case of storm surges of short duration, since in that case the land areas can no longer store water.

4.4 Salt water intrusion

4.4.1 Introduction

Deltaic and other low-lying coastal areas are exposed to the intrusion of salt both in surface water and groundwater. The result may be that the water in the rivers and in the canals cannot be used for consumption. Saline water can originate from various sources:

1. Salt water intrusion from the sea into open estuaries.
2. Seepage of brackish or saline ground water originating from old marine deposits.
3. Seawater entering through navigation locks between the sea and a fresh water canal.
4. Leakage of saline water through sluices and locks.
5. The salt load of a river (natural salinity or brackish effluents from agricultural drainage and industrial wastes).
6. The salt contained in rainwater and the saline spray in coastal regions (usually a minor amount).
7. Saline groundwater coming to the surface because of irrigation without proper drainage (e.g. Iraqi), or clearing of forest (e.g. Australia)

The first source of salt is the most general source in coastal areas.

4.4.2 Sea water intrusion into open estuaries

General

The penetration of seawater (relative density around 1.028) into rivers with a fresh water flow (relative density 1.0) is due to the 2.8% difference in density. If there are no tides and the estuary cross-sections are regular the intrusion assumes the shape of a sort of wedge (or tongue) with stagnant seawater. The fresh river water flows out on top of the seawater (see Figure 4.18).

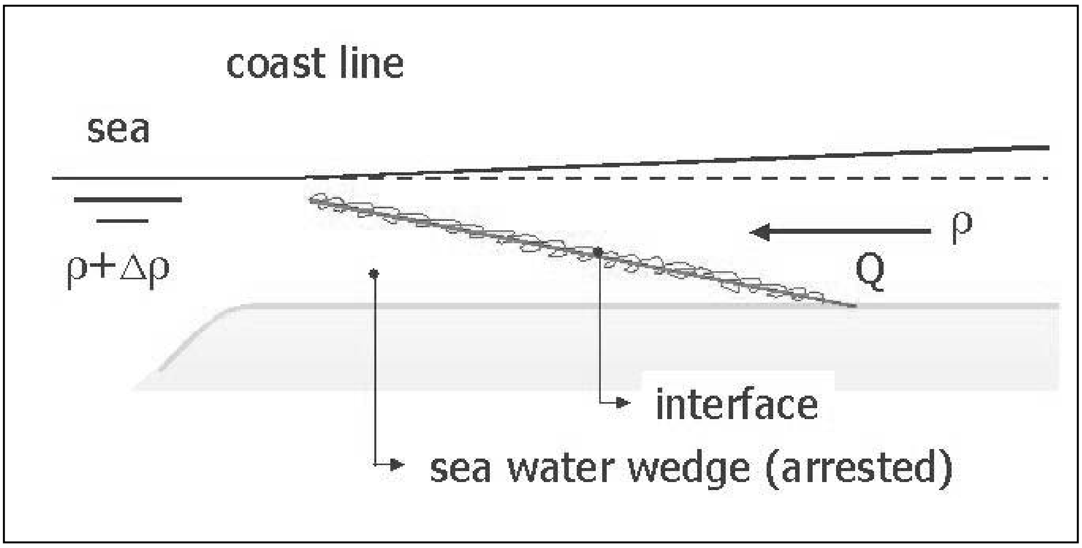

Figure 4.18

Saline wedges, however are very uncommon. They only occur in narrow estuaries, often man-made or non-alluvial estuaries, with a high river discharge and a small tidal range. Tides promote the turbulent mixing at the interface resulting into an estuary where incomplete or (almost) complete mixing of the waters near the bottom and near the surface occurs. Mixed estuaries are most common, particularly during periods of low river flow.

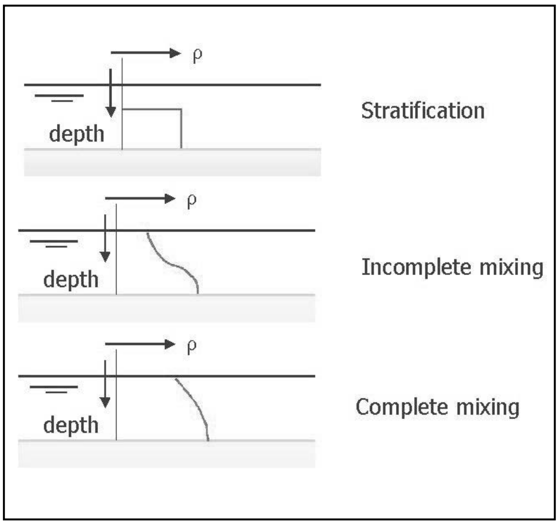

Figure 4.19

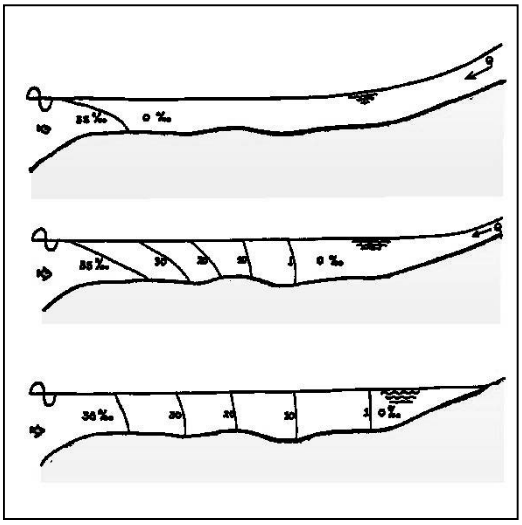

Figure 4.20: Mixing degree

The degree of mixing largely depends on the flood number N, also called the Canter-Cremers number:

$$N = \frac{QT}{V_{fl}} \qquad \text{Equation 4.8}$$

Roughly it can be assumed that if:

$N \geq 1.0$: the estuary is stratified
$0.1 < N < 1.0$: the estuary is partly mixed
$0 < N < 0.1$: the estuary is well mixed

It can also be seen from Figure 4.19 and Figure 4.20 that there is no significant bottom slope in the part of the estuary where the tide dominates over the river discharge. Observations in estuaries have shown, however, that there is a slight increase of the water level in the upstream direction over the length of the salt intrusion.

Balance of forces

This phenomenon can be explained by a balance of hydraulic forces over the intrusion length of the salinity. This analysis is applicable to both the mixed and stratified situation (see Figure 4.21 and Figure 4.22)

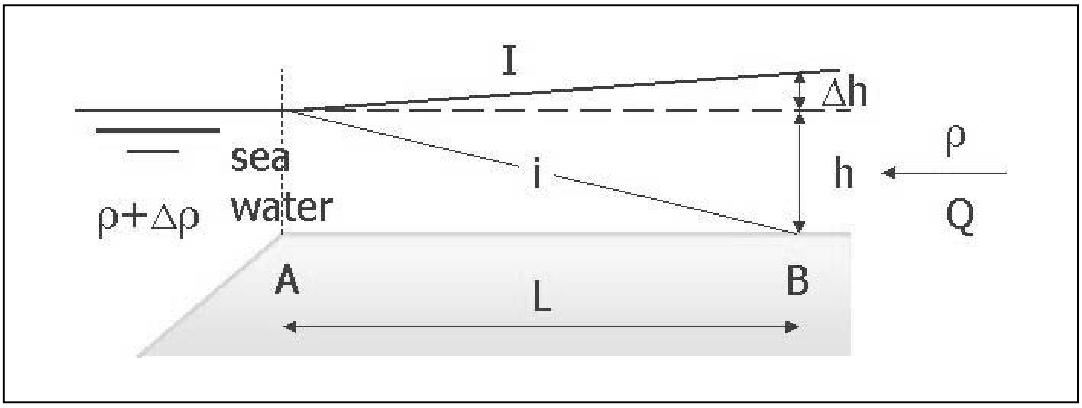

Figure 4.21 The balance of hydrostatic pressure over a reach of salt water intrusion (mixed or stratified)

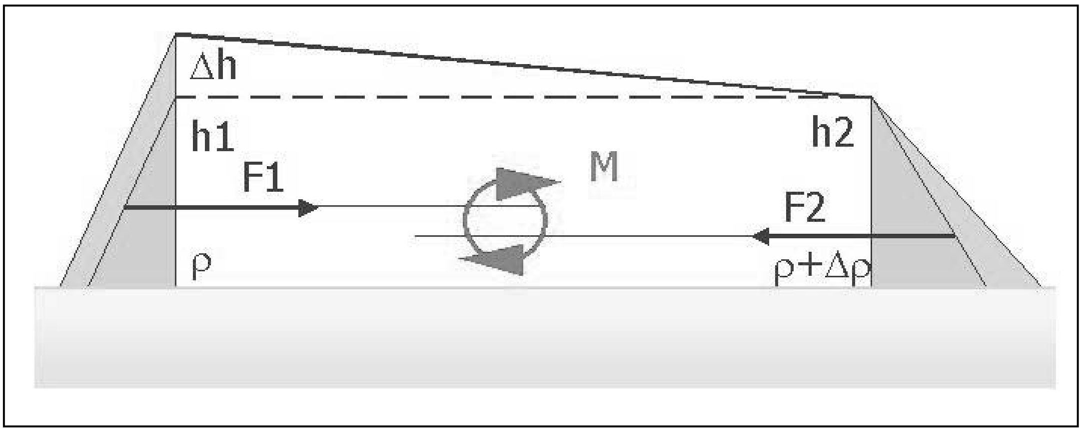

Figure 4.22 After the hydrostatic forces make horizontal equilibrium, a moment of forces remains driving vertical circulation and mixing

Since the sea water is not moving, and the flow lines in point B me be assumed to be parallel, the forces resulting from the hydrostatic pressures in A and B should be equal, or:

$$\frac{1}{2}gh^2\left(\rho+\Delta\rho\right)=\frac{1}{2}g\left(h+\Delta h\right)^2\rho \qquad \textit{Equation 4.9}$$

After disregarding higher orders of Δh, this yields:

$$\Delta h = \frac{\Delta\rho}{2\rho}\mathrm{h} \quad \text{and} \quad \mathrm{I} = \frac{\Delta h}{L} = \frac{\Delta\rho}{2\rho}\frac{h}{L} = \frac{\Delta\rho}{2\rho}i \qquad \textit{Equation 4.10}$$

Although the two forces cancel out, there is a residual moment M that results from the fact that the working lines of the forces don't agree (see Figure 4.22). The arm of the moment is $\Delta h/3$. Hence, the moment equals:

$$M = \frac{1}{2}gh^2\rho\frac{\Delta\rho}{2\rho}\frac{h}{3} = \frac{1}{12}g\Delta\rho h^3 \qquad \textit{Equation 4.11}$$

In a stratified estuary, this moment is counteracted by a shear stress in the interface, which keeps the interface in place. In a mixed estuary, this moment is counteracted by friction. The moment drives the vertical circulation and mixing process.

Stratified estuary

Thijsse (1952) developed a formula for the intrusion length for stratified flow based on the assumption that the flow of fresh water over the salt-fresh interface can be described by an adjusted Chézy equation. The flow of the fresh water is then governed by:

$$Q = C'A'\sqrt{h'I} \qquad \textit{Equation 4.12}$$

In this formula C' is referring to the conditions at the interface and A' and h' vary in the flow conduit. All three are virtually unknown and Thijsse replaced A' and h' by the known parameters A and h (full cross-section) and put the uncertainties in the value of C' to be determined from actual field records (canals in the Netherlands). Under these conditions a value of C' is 21.2 $m^{½}$/sec showed acceptable agreement with the observations and the formula reads:

$$Q = 21.2A\sqrt{hi\frac{\Delta\rho}{2\rho}} \quad \text{and since} \quad \mathrm{i} = \frac{h}{L} \qquad \textit{Equation 4.13}$$

$$L = \frac{225A^2h^2}{Q^2}\frac{\Delta\rho}{\rho} = \frac{225B^2h^4}{Q^2}\frac{\Delta\rho}{\rho} = \frac{225h^2}{v^2}\frac{\Delta\rho}{\rho} \qquad \textit{Equation 4.14}$$

for a rectangular cross-section with width B and depth h.

The formula should be used with suspicion and not without due checking but it does show the great influence of the depth h (effect of dredging for estuarine harbours) and the freshening effect of the upland discharge Q.

Most estuaries only have stratified flow during extremely high flows. Estuaries normally have mixed salt intrusion. For these estuaries Savenije (1993) introduced in an analytical formula for the total salt intrusion length at HWS (L) on the basis of a number of dimensionless ratios. This theory is presented in the following paragraphs.

Mixed estuary

In a mixed (or partially mixed) estuary the salinity in a given station varies not only with the upland discharge but also with the tide.

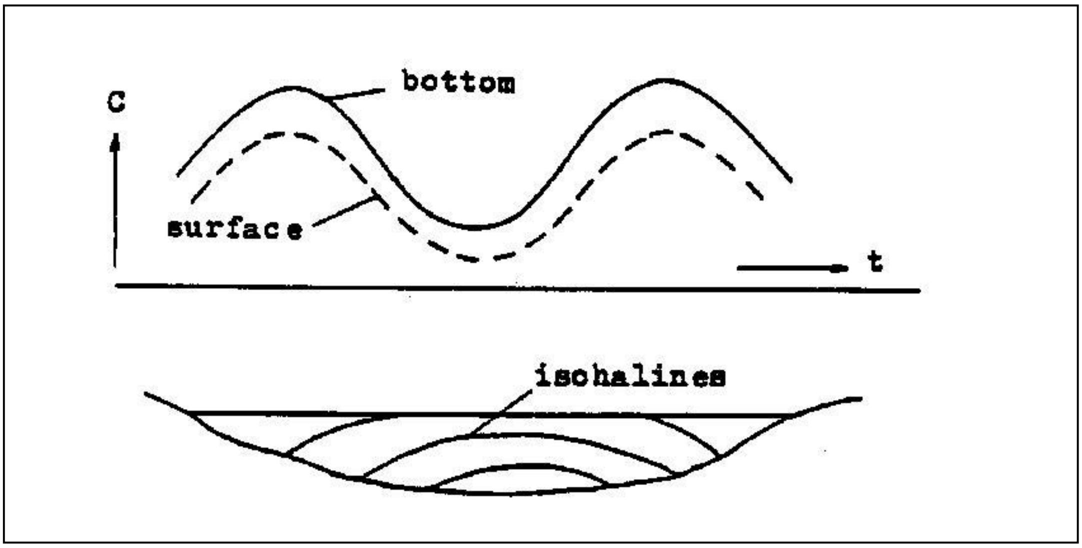

Figure 4.23

The concentrations in a certain station and at a certain moment can vary over the cross-section so that a sample taken from the bank does not always show the average salinity. During measurements one should be aware of this, so as to make certain that a representative sample is taken.

Salinity can be determined in a simple way by measuring the electrical conductivity (calibration with a limited number of samples with chemical analysis). Probes can be attached to a cable so that the vertical distribution can quickly be determined.

The average salinity in a cross-section S is a function of time t and space x. When plotted against the longitudinal x-axis, three characteristic situations can be distinguished: HWS, LWS and TA (tidal average). At HWS, when the direction of flow changes from upstream into downstream. In case of a progressive or mixed-type wave the point in time at which HWS occurs — some time after reaching high water (HW) — increases as the tidal wave moves upstream. Hence at each point along the estuary HWS occurs at a different time.

The best way to carry out a reconnaissance salt intrusion measurement is by moving boat at HWS; this is for the following reasons:

1. The moment that HWS occurs is easily determined. The observer measures the salinity, when the in-going current slacks. Although the same advantage applies to LWS, the accessibility at LWS is generally poor. Sometimes, especially in natural estuaries, it is very difficult to reach the waterside. Inaccessible mud flats often separate the river from the banks.

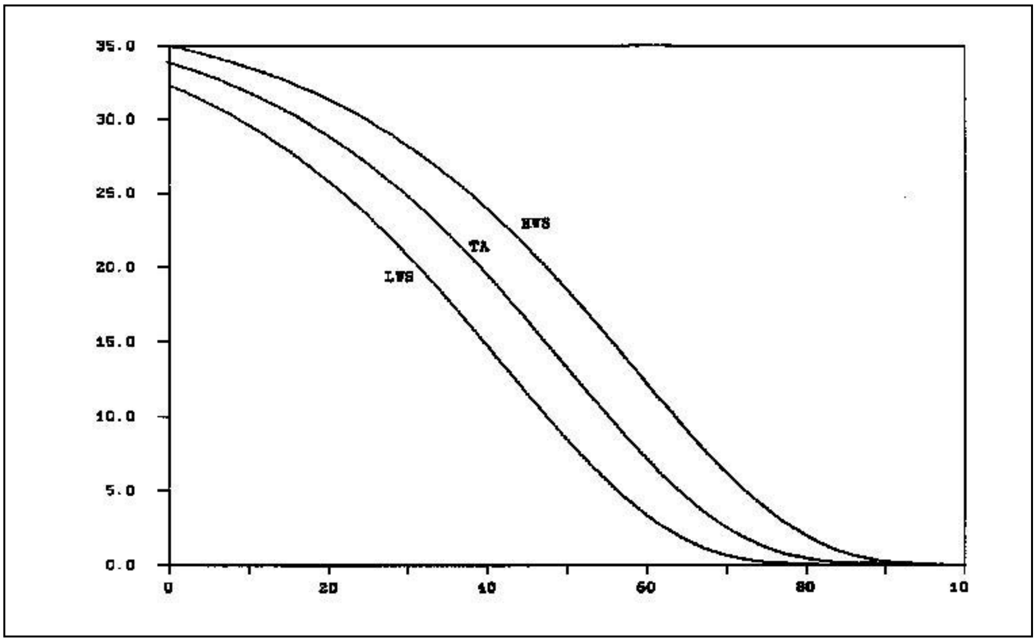

Figure 4.24: Envelope curves of salinity intrusion at High Water Slack (HWS), Low Water Slack (LWS) and mean tide (TA)

2. If the salinity at the downstream boundary is not known, it can easiest be estimated at HWS. At HWS, the salinity at the estuary mouth is generally equal or almost equal to the ocean salinity, which is not, or not much, affected by the fresh water discharge from the estuary.

3. At HWS the salt intrusion is at its maximum. Generally, it is the maximum intrusion that is of interest to planners.

4. A single observer in a small outboard driven boat can travel with the tidal wave and measure the entire salt intrusion curve at HWS.

5. In a moving boat it is easy to measure a full vertical in the center of the stream which is the best location to measure the salinity; if necessary a check in another vertical can be made to see if there is much lateral variation.

6. If the intrusion length is not too long, the observe can (and should) return to the estuary mouth and repeat the measurement for LWS.

Another method, which is often recommended in handbooks but which is not advisable, is to station one or two permanent observes at strategic points along the estuary to record continuous variations of the salinity. The information obtained by this method is not very valuable because from the shore it is difficult to determine the salinity at midstream and to measure the variation over the depth. In addition, it is impossible to install too many observers along the estuary so as to obtain a sufficient amount to draw accurate longitudinal profiles. Moreover, the most important information to be collected is the range over which the salinity varies; the moving boat method just measures this range and hence is much more efficient.

Analysis

In most alluvial estuaries (except exceptionally large estuaries such as the Gambia, or estuaries where there is no fresh water inflow during the dry season) a steady state model can be used to describe the salt intrusion.

For steady state, there should be an equilibrium in any cross-section between:

- the advective transport of salt by the river flow in downstream direction $Q(S-S_f)$, where S_f is the salinity of the river water;
- and the dispersive transport of salt in upstream direction under the effect of mixing which is proportional to the concentration gradient dS/dx.

The dispersive transport is generally assumed to be equal to

$$DA\frac{dS}{dx} \qquad \textit{Equation 4.15}$$

where: A = cross-sectional area
D = dispersion coefficient

Hence:

$$\left(S-S_f\right)Q_f = -AD\frac{dS}{dx} \qquad \textit{Equation 4.16}$$

If we assume, for the purpose of illustration, that *Q/DA* is constant, then

$$S-S_f = \left(S_0-S_f\right)\exp\left(-\frac{Q}{DA}x\right) \qquad \textit{Equation 4.17}$$

where S_0 is the sea salinity at x=0 (the estuary mouth).

In this simple case, the salinity plots as straight lines on semi-logarithmic paper.

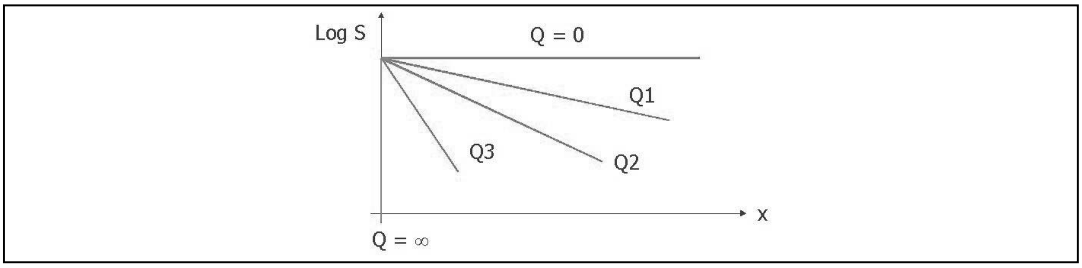

Figure 4.25

The following considerations make that the solution is not that simple:

1. The cross-sectional area A is not constant with x;
2. The value of D cannot be easily predicted; it differs from one estuary to another; moreover it varies with x;
3. Actual records show that the lines deviate from straight lines;
4. The concentration for x = 0 is not constant but depends on Q;

The objective is to have a formula derived from the available and usually small amount of data and with as much physical foundation as possible which would enable to predict for a wide range:

- the effect of upland discharge;
- the salinity at any station along the estuary;
- the effect of deepening or shoaling of the channels.

The geometric variation of the cross-sectional area can generally be solved by fitting an exponential function. Savenije (1986) showed that very good results are obtained by considering an "ideal estuary" with a horizontal bed (the depth h_0 is constant), exponentially varying width (easy to measure from a map), and hence an exponentially varying cross-section:

$$A(x) = A_0 \exp\left(-\frac{x}{a}\right) \qquad \textit{Equation 4.18}$$

where A_0 is the cross-sectional area at the estuary mouth and a is a convergence length to be obtained from fitting measured cross-sections to a line on semi-logarithmic paper.

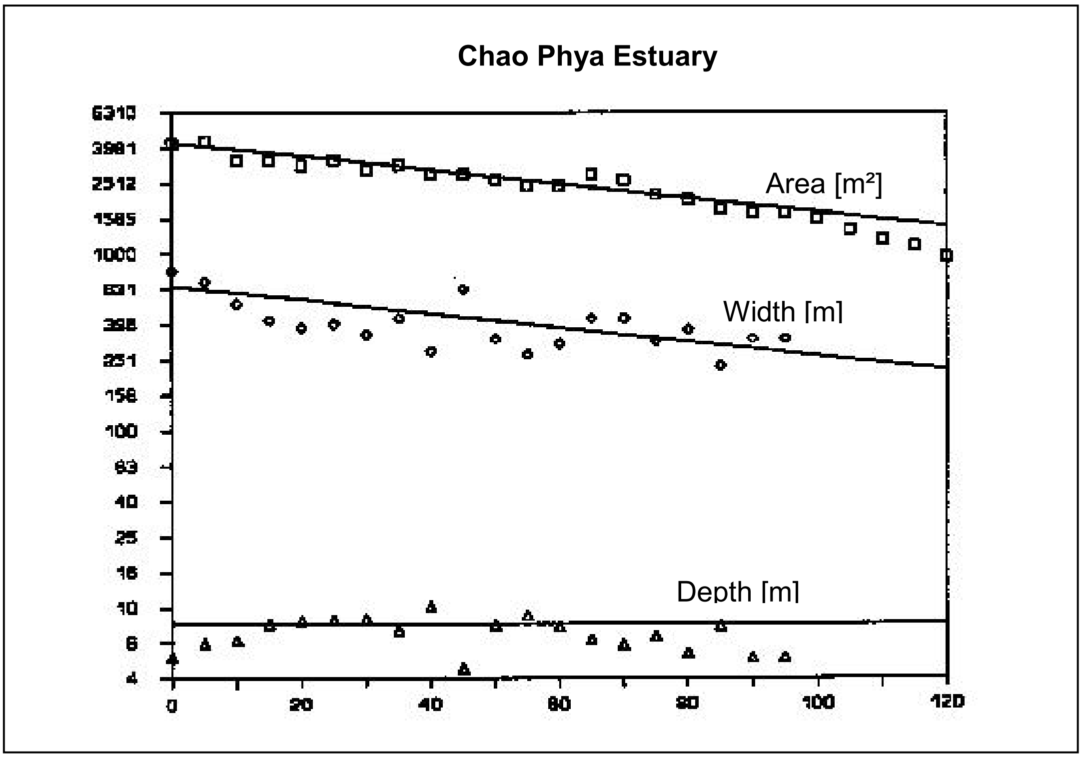

Figure 4.26

For the longitudinal variation of the dispersion coefficient different researchers have followed different approaches. Of the different types of relations tried in the literature, Prandle (1981) gave a good overview. He tried out the following relations:

$$D = D_0$$

$$D \propto \frac{\partial S}{\partial x} \qquad \textit{Equation 4.19}$$

$$D \propto \left(\frac{\partial S}{\partial x}\right)^2$$

which can be summarized as:

$$D \propto \left(\frac{\partial S}{\partial x}\right)^{k}$$ *Equation 4.20*

with k = 0 ,1, 2 respectively.

These relations appear to work only in very specific conditions. For instance, the first equation performs better in estuaries with a pronounced funnel shape, whereas the second and third equations perform better in narrow and prolonged estuaries.

However, Savenije (1992) showed that the following equation works well under a wide range of estuaries, of different depths, different convergence lengths and different widths.

$$\frac{D}{D_0} = \left(\frac{S}{S_0}\right)^{K}$$ *Equation 4.21*

This approach corresponds with the method of Van der Burgh (1972), who based on a large number of measurements in Dutch estuaries found that:

$$\frac{\partial D}{\partial x} = -K\frac{Q_f}{A}$$ *Equation 4.22*

where K is Van der Burgh's coefficient.

Combination of Equations 4.16, 4.18 and 4.22 yields:

$$\frac{S - S_f}{S_0 - S_f} = \left(\frac{D}{D_0}\right)^{\frac{1}{K}}$$ *Equation 4.23*

$$\frac{D}{D_0} = 1 - \beta\left(\exp\left(\frac{x}{a}\right) - 1\right)$$ *Equation 4.24*

Moreover, substitution that $x = L$ where $S = S_f$ yields:

$$\beta = \frac{Ka}{\alpha_0 A_0}$$ *Equation 4.25*

$$L = a \cdot \ln\left(\frac{1}{\beta} + 1\right)$$ *Equation 4.26*

where L is the intrusion length at HWS, and α_0 is a calibration coefficient equal to D_0/Q_f, where D_0 is the dispersion coefficient at the estuary mouth at HWS.

In addition empirical formulae have been derived by Savenije (1992) to predict the values of the calibration coefficients K and α_0:

$$K = 6.3 \cdot 10^{-6} \left(\frac{h_0}{a}\right)^{1.04} \left(\frac{E}{H}\right)^{2.36}$$ *Equation 4.27*

$$\alpha_0 = 220 \frac{h_0}{a} \sqrt{\frac{ETgh_0}{-Q_f A_0}}$$ *Equation 4.28*

Combination of these equations yields an expression for the intrusion length L at HWS. This relation is a combination of dimensionless ratios. These ratios are:

1. The densimetric Froude number F_d :

   $$\mathrm{F_d} = \frac{\rho}{\Delta\rho} \frac{\upsilon^2}{gh}$$ *Equation 4.29*

   where υ is the tidal velocity amplitude. The ratio of $\Delta\rho$ to ρ is related to the residual moment of the hydrostatic forces, described in Figure 4.21, which relates to stratified flow. In mixed estuaries, the balance of forces is essentially the same and the resulting moment is the driving power behind the salt intrusion through mixing.
2. The Canter-Cremers number relating fresh water discharge to the tidal volume:

   $$N = \frac{QT}{V_{fl}} = \frac{QT}{EA_0}$$ *Equation 4.30*

3. The Estuarine Richardson number, which is generally used as a better indicator than N to determine the degree of stratification:

   $$N_R = \frac{N}{F_d}$$ *Equation 4.31*

4. A simpler form of the Canter Cremers number being the ratio of the tidal velocity amplitude υ to the fresh water velocity v_f
5. The ratio of the depth h to the convergence length of the cross-sectional area a
6. The ratio of E to a
7. The coefficient of Van der Burgh K

The resulting equation is:

$$L_{HWS} = a \ln\left(1 + \frac{220}{K} \frac{h}{a} \frac{E}{a} \frac{\upsilon}{v_f} \sqrt{40 N_R}\right)$$ *Equation 4.32*

This equation is the most accurate equation available to date (see Savenije 1993) and is applicable in all estuaries, provided they are alluvial.

Preventive measures

The saline intrusion can be halted or reduced by:

a) Increasing the upland discharge (releases from reservoir). This requires relatively high amounts of fresh water, which cannot be used for other purposes like irrigation.

b) Decreasing the depth of the estuary. This is an expensive operation, which may only be suitable in special cases (Rotterdam Waterway).

c) In case of small tides and a stratified estuary a low submerged sill effectively halts the saline wedge without offering a significant obstacle to flood flows.

d) Damming off estuaries by building an enclosing dam equipped with sluices and gates to remove excess water to the sea (estuarine reservoirs, see section 4.5.2).

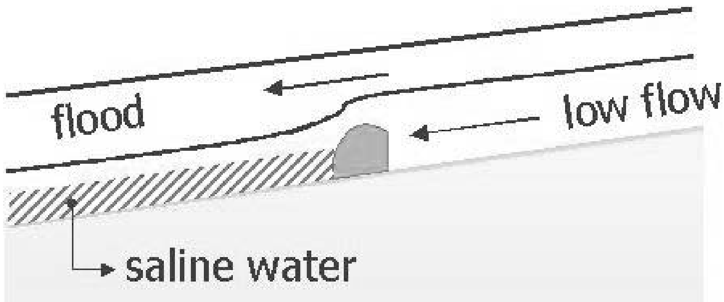

Figure 4.27: damming off estuaries by building an enclosing dam.

If preventive measures are not feasible other measures as well as repressive measures can be considered such as:

a) Shifting of intakes of fresh water (for water supply or irrigation) to points upstream of the saline reach.

b) Rinsing of flushing of canals exposed to saline intrusion with fresh water.

c) Over-irrigation in combination with adequate drainage to leach the soils.

4.4.3 Seepage of brackish ground water

The ground water under coastal areas is often saline or brackish especially under the coastal portions. The saline water originates from the marine transgressions at the time of deposition (primary aquifer salinization). If differences in the elevation of the surface waters are created (by reclamation and empoldering) a seepage flow is generated which carries the saline ground water to the surface ('oozing') thus forming a source of salt. In principle the rate of supply of salt is the product of the seepage rate and the concentration of the ground water. However the seepage flow changes the distribution of the salinity. In simple flow models (an aquifer covered by a semi-pervious top layer) the flow trajectories (or paths), the travel times and the isochrones can be computed and hence the changes in the salinity distribution. In this respect, reference is made to the lectures on ground water.

Fresh water aquifers under coastal areas may become saline because of intrusion of seawater. This may be caused by overdraft on fresh water pockets, reclamation of deep low-lying areas close to the sea and intrusion into aquifers of river water from the saline reach (secondary aquifer salinization).

Seepage of brackish ground water also contributes to soil salinization. The most important cause of soil salinization, especially in arid zone coastal areas), is the use of irrigation water in the absence of any drainage (primary soil salinization). The explanation lies in the fact that irrigation always contains some salt and that evaporation does not remove any salt. In this way a progressive increase of the salinity of the pore water occurs.

Shallow ground water (within say 2 metres below the surface) also acts as a source of salt (secondary soil salinization). The pore water moves upward by capillary forces and in the top layer the water goes into the atmosphere and salt remains in the top layer. Leaching with fresh water gives a temporary relief but produces a rise of the ground-water level, thus increasing the capillary ascend. The only remedy is to apply drainage in combination with leaching and removal of the saline leachate. By drainage (trenches or tile drains) the phreatic table is maintained at 1.2 to 1.5 m (even lower in the U.S.A. and U.S.S.R.) below the surface and a leaching of around 10% of the consumptive use is applied.

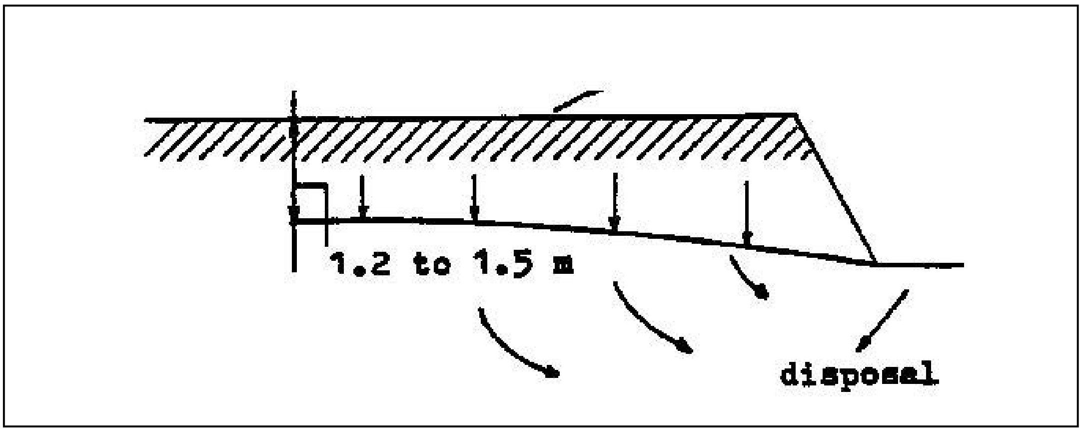

Figure 4.28

4.4.4 Sea water entering at navigation locks

Exchange processes

Distinction should be made between the amount of salt admitted by filling the lock chamber when the level of the sea is higher than the level of the fresh water canal and the amount of salt resulting from the exchange of water between the sea and the chamber filled with fresh water when opening the outer gates.

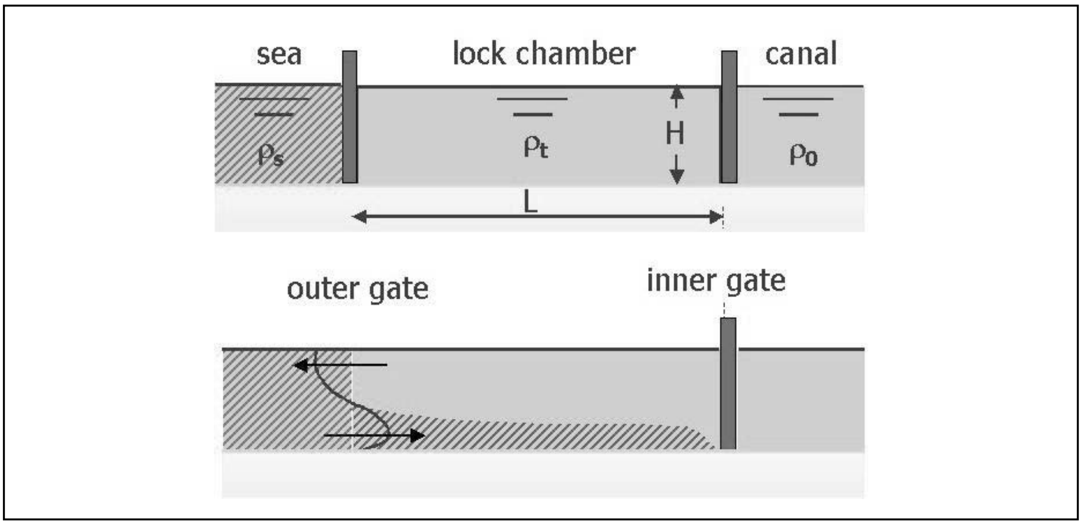

Figure 4.29

ρ_s = relative density of sea water
ρ_o = relative density of fresh water
ρ_t = average relative density of water in the chamber at a time t after opening of the outer gates
H = depth of the chamber
L = length of the chamber

then the exchange at the time t can be characterized by the dimensionless factor:

$$u_t = \frac{\rho_t - \rho_0}{\rho_s - \rho_0} \qquad \textit{Equation 4.33}$$

if $\rho_t = \rho_s$: $u_t = 1$
$\rho_t = \rho_o$: $u_t = 0$

Experiments in the Netherlands at a number of ship locks of various dimensions have led to the following semi-empirical formula.

$$u_t = \tanh\left(\frac{t}{4L}\left(\frac{\Delta\rho}{\rho_0} gH\right)^{\frac{1}{2}}\right) \qquad \textit{Equation 4.34}$$

where $\rho = \rho_s - \rho_o$

If the dimensionless parameter $\frac{t}{4L}\left(\frac{\Delta\rho}{\rho_0} gH\right)^{\frac{1}{2}}$ is indicated by T:

$$u_t = \tanh(T) \qquad \textit{Equation 4.35}$$

Example

L = 100 m
t = 5 min = 300 sec = opening time of gates
$\Delta\rho/\rho$ = 0.02
H = 4 m
T = 300/400 $(0.02 \times 9.8 \times 4)^{0.5}$ = 0.664
u_t = tgh T = tgh 0.664 = 0.58

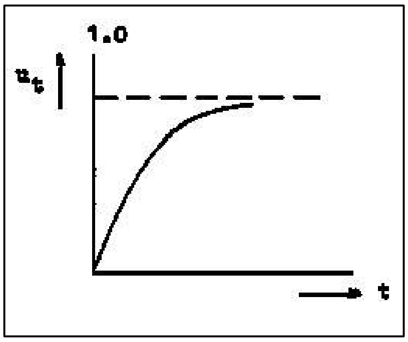

Figure 4.30

So the degree of exchange 5 minutes after opening of the outer gates is 58%. A similar process takes place when afterwards the inner gates are opened.

Preventive measures

There are several ways of preventing or reducing the entrance of sea water at navigation locks.

a) A drastic but expensive way is to pump back the saline water in the lock chamber to the sea and to replace this water by fresh water from the canal. The saline water is removed through openings in the floor of chamber and the fresh water is added on the surface.

b) An economic way of obtaining a reduction of 70 - 50% is found in the injection of air bubbles during the time that the gates are open thus hampering the exchange.

c) The saline water entered at a lock can be collected in a sump at the canal side of the lock and removed from there by pumping or gravity (Seattle U.S.A.).

d) To reduce the amount of salt entering the lock chamber can be subdivided so that smaller boats can be locked through without using the entire chamber. Also a second lock of smaller size can be provided.

4.5 Water control in coastal areas

4.5.1 Problems of quantity and quality

The main problems of water control in low-lying coastal areas are the problem of the salt-water intrusion and the problem of the floods.

The saline intrusion is maximum during dry periods with minimum river flow. This is exactly the period with a maximum need for fresh water for irrigation and other purposes.

The possibility of using water with a high salinity is indicated by the tolerance limits for the use of water as drinking water or industrial water or irrigation water.

According to the recommendations of the World Health Organization (WHO) drinking water should not contain more than 200 mg of Cl^- per litre, in special cases not more than 250 (sea water contains 18,000 to 19,000 mg Cl^- per litre and about 36,000 mg per litre of total salts).

The tolerance limit for process water in industries depends, of course, on the type of product. Water for food processing and breweries should not contain more than 100 to 150 mg Cl^- per litre.

The tolerance limit for irrigation depends on soil, crop and water and soil management and should be fixed in consolation with specialized agronomists.

Commonly accepted limits for the use of irrigation water for high yielding rice varieties are 350 mg Cl^- per litre in the early stages of growth (nurseries and seedlings) and 1500 in the later stages when the plants have become more resistant.

For irrigation of flowers and sensitive fruits and vegetables in hothouses with an artificial climate (Netherlands) limits of 100 to 150 mg Cl^- per litre have been set forth.

In many cases irrigation water with a much higher salinity is accepted. This is applied to salt resistant varieties on pervious soils and under conditions of over-irrigation to leach the soils. This implies adequate drainage facilities to remove the salt and to prevent any accumulation.

Early human settlements along river branches in deltas are often found beyond the saline reach. In the saline reach drinking water could be found in fresh ground-water pockets under dunes, former sandy beach ridges, natural levees and filled-in tidal creeks. Fresh water from excess rainfall during wet periods can be stored in tanks and ponds.

Nowadays desalinization of seawater is applied at increasing rate, especially in arid zones but also in temperate zones. In spite of the high costs (0.3 to 1.0 US $ per m^3) it may be economical for drinking water and water to irrigate valuable crops (flowers, fruits and vegetables in arid zones).

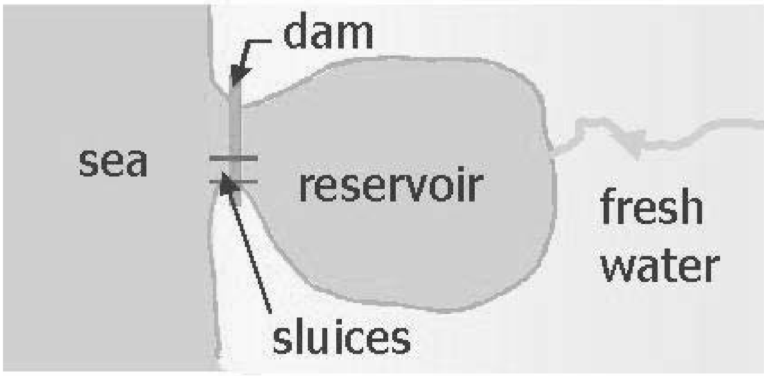

Figure 4.31

Possibilities for large-scale storage of fresh water in low-lying coastal areas can be created by damming off estuaries, coastal lagoons and tidal embayments with an inflow of fresh water. In this way an estuarine or coastal reservoir is formed which is separated from the sea by a dam equipped with sluices for removal of excess water from the reservoir. The originally saline water is replaced by the fresh water from the river. These reservoirs are dealt with in section 4.5.2.

The most generally applied means of flood protection in coastal areas consists of throwing up embankments or dikes. Embanking interferes in the natural hydrological conditions and may have a number of serious side effects and environmental impacts, which will be considered in section 4.5.3.

4.5.2 Coastal reservoirs

General design

In the context of these lectures, only the hydrological aspects of coastal reservoirs will be considered, not the structural aspects of dam building and closing operation.

Coastal reservoirs have been built in a number of countries (Japan, the Netherlands, Korea, Hong Kong, Bangladesh, Sri Lanka, England, etc.). These are multiple purpose reservoirs:

1. by shortening the coast line the salt water intrusion is reduced;
2. fresh water from the river can be stored;
3. a better defence of the adjacent low-lying areas against storm surges is obtained;
4. the drainage of these areas can be improved.

The technical feasibility of such reservoirs depend on:

1. The length of the period of desalinization.
2. The ultimate salinity of the water in the reservoir after the desalinization.
3. The water balance of the reservoir in connection with the regulation of the normal operational level.

The hydrologic design of coastal reservoirs requires the establishment after the water balance of a so-called salt balance in which incoming and outgoing amounts of salt are considered to determine the salinity of the water in the reservoir.

Water balance

In the water balance the following items have to be considered:

Table 4.1: Water balance

IN	OUT
River discharge	Evaporation
Drainage on the reservoir	Drainage to the sea
Rain on the reservoir	Abstraction of water
Decrease in storage	Increase in storage

The water balance is especially important during periods with river floods with a maximum of inflow of water to the reservoirs and during dry periods when water is withdrawn from the reservoir for water supply to the neighbouring areas.

In most cases the normal operational level of the reservoir is around MSL so that excess water can be drained off to the sea by gravity using the period of low water at sea (tidal drainage). During the period of high water at sea the sluice gates remain closed and the reservoir level rises due to the inflow of water to the reservoir.

On those coasts where under the effect of on-shore winds a set-up of the water level occurs, drainage may be hampered during periods with abnormally high sea levels. Then the inflow must be stored and a temporary high reservoir level can occur. Thus the area of the reservoir is an important factor governing the maximum level. This level can be reduced if during the wet season with the floods the normal operational level is maintained as low as possible (still above LW level) so that empty storage space is available to accommodate the floodwaters.

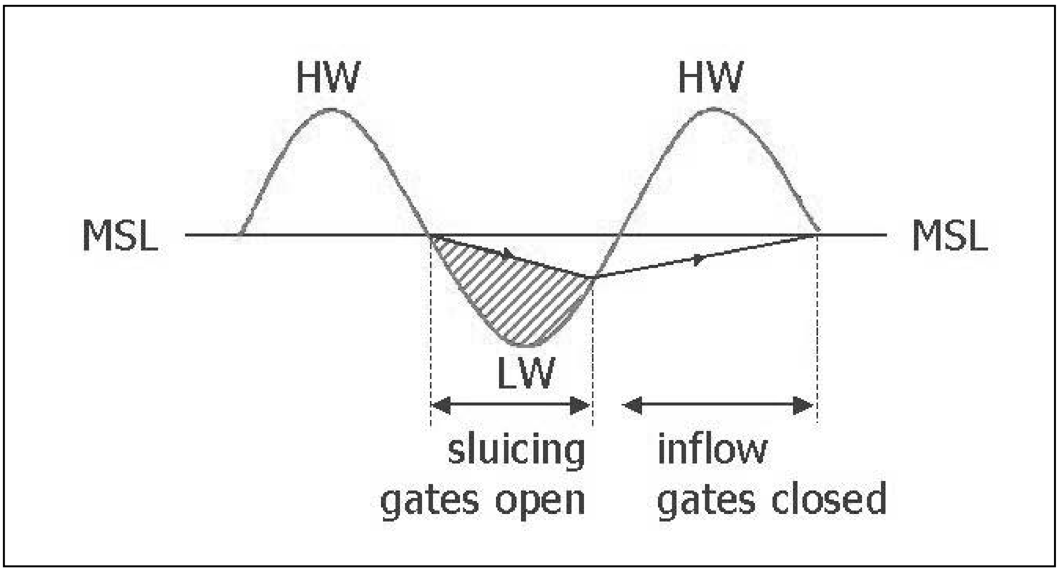

Figure 4.32

During dry periods the gates remain closed. The reservoir level is going down because of abstraction of water from the reservoir and evaporation. The lowest water level can be reduced if during the dry season the normal operational level is maintained at a high elevation so that a maximum amount of water is available to meet the demands. This leads to reservoir regulation somewhat similar to that of upstream reservoirs in the mountains.

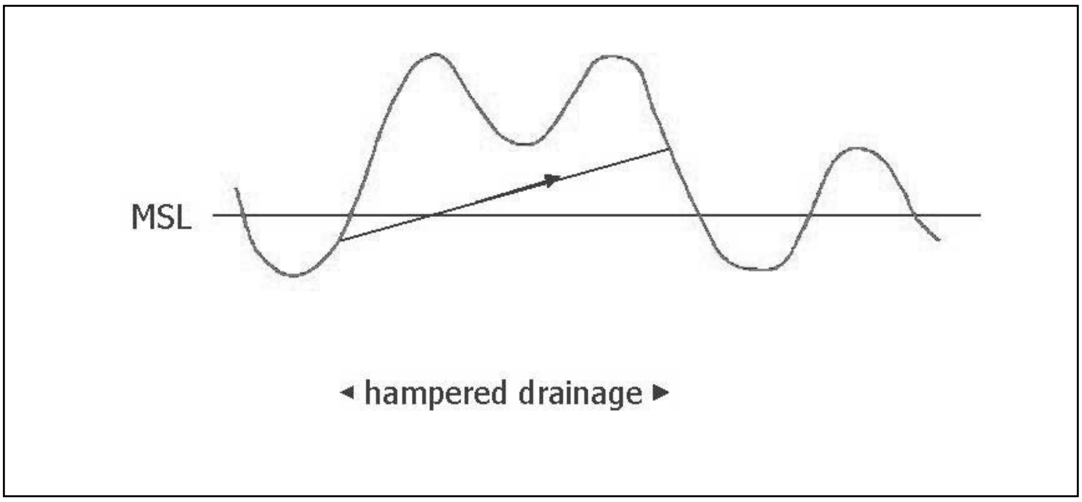

Figure 4.33

Salt balance

Coastal reservoirs are exposed to salt water intrusion and in the salt balance (after the initial period of desalinization) the following items should be considered:

Table 4.2

IN	OUT
Salt load of the river	Drained off to sea
Drainage of brackish water on the reservoir	Abstraction of water from the reservoir
Underground inflow of saline water	Increase in amount of salt stored
Diffusion of salt from the bottom	
Locking of ships	
Leakage of sluice gates	
Decrease in amount of salt stored	

No salt is removed from the reservoir by evaporation. In principle the ultimate salinity of the water in the reservoir under steady conditions is given by the equation:

$$c_{\infty} = \frac{\text{amount of salt reaching the reservoir}}{\text{amount of water drained off}}$$ *Equation 4.36*

A numerical example is given at the end of this section.

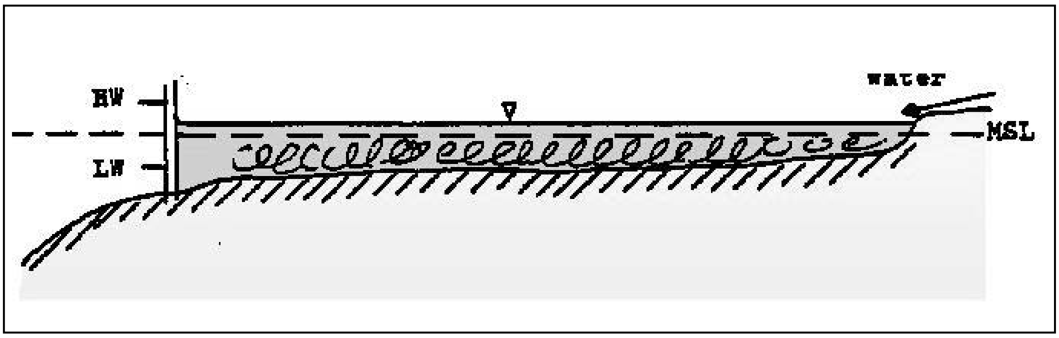

Figure 4.34

The initial desalinization of the reservoir after the closing of the dam can be achieved in two ways:

1. Removal of the saline water by pumping to the sea. The only case known is the Glover Cove Scheme in Hong Kong.

2. Gradual desalinization by draining off to the sea the mixture of the saline water with the fresh water supplied by the river. This is generally applied. The mixing is effectuated by winds and waves on the reservoir and turbulent motion. This works well in shallow reservoirs (up to 4 or 6 m) with a flat bottom.

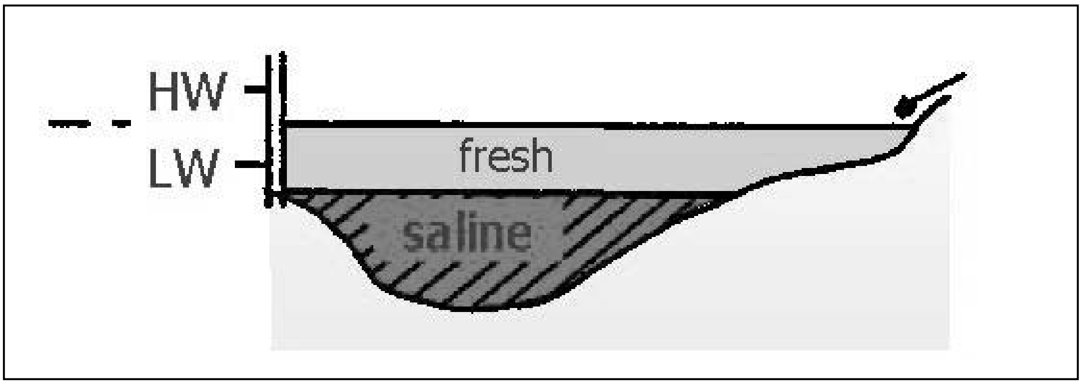

Figure 4.35: In the Hong Kong case saline water would remain stagnant in the deep pocket behind the dam.

The variations of salinity with time (including the desalinization) can be described by a fundamental equation, the mixing equation of theoretical physics (see Figure 4.36). This equation is the same for a container, a coastal reservoir or a column of soil:

q	= rate of supply of water with a concentration c'
e	= net evaporation rate (evaporation minus rainfall)
d	= drainage rate
V_o	= volume of water
c	= concentration of water in the reservoir $c = c(t)$

At the time $t = 0$ $c = c_o$.

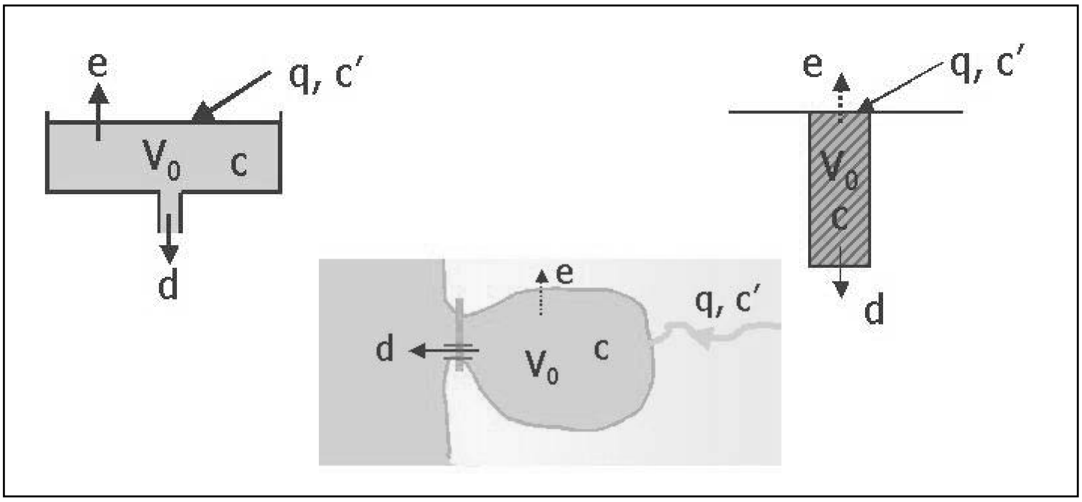

Figure 4.36

The rate of supply of salt is *q.c*'. If q is expressed in $10^6 m^3$ per month and *c*' in kg/m^3 *q.c*' is the rate of supply of salt expressed in 10^6 kg per month. In the case of the coastal reservoir *q.c*' stands for all inflows of salt mentioned in the salt balance.

To obtain a simple analytical solution it is supposed that *q*, *c*', *e*, *d* and V_o are constant and that only *c* varies with time. It is also assumed that there is complete mixing and hence a homogeneous distribution of the concentration.

Then for a time increment d*t*, the water balance reads:

$$q \cdot dt = e \cdot dt + d \cdot dt \qquad \text{Equation 4.37}$$

or

$$q = e + d \qquad \text{Equation 4.38}$$

and the salt balance:

$$qc' \cdot dt = -dc \cdot dt + V_0 dc \qquad \text{Equation 4.39}$$

being, respectively, the amount supplied, the amount drained and the increase in storage. Per unit time this yields:

$$V_0 \frac{dc}{dt} = -dc + qc' \qquad \text{Equation 4.40}$$

In order to solve this equation, first the reduced equation is solved:

$$V_0 = \frac{dc}{dt} = -dc \qquad \text{Equation 4.41}$$

Since, apart from constants, the function c must be equal to its derivative the function is an exponential one, hence:

$$c = K \exp\left(-\frac{d}{V_0} t\right) \qquad \text{Equation 4.42}$$

The non-reduced equation then has the form:

$$c = K \exp\left(-\frac{d}{V_0}t\right) + N$$ *Equation 4.43*

The value of *N* is found by substitution:

$$N = \frac{qc'}{d}$$ *Equation 4.44*

Substitution of the boundary condition $c=c_0$ when t=0 yields an expression for *K*:

$$K = c_0 - N$$ *Equation 4.45*

hence the total equation reads:

$$c - c_\infty = (c_0 - c_\infty)\exp\left(-\frac{d}{V_0}t\right)$$ *Equation 4.46*

c_4 = *N* = the ultimate salinity of the water in the reservoir as mentioned above.

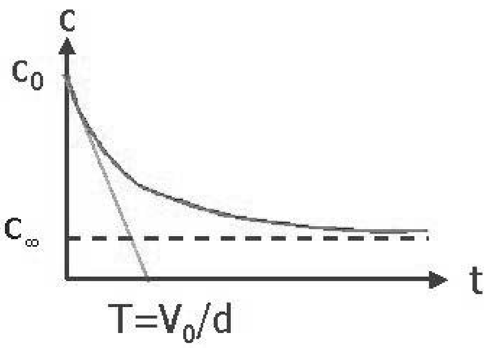

Figure 4.37

The equation yields the following special values:

$t = 0$ $\quad c = c_0$
$t = \infty$ $\quad c = (qc')/d$
$d = 0$ $\quad c = c_0 + (q.c')/ V_0 * t$

The last case is a direct integration of the differential equation with d = 0. It is the case of progressive increase in salinity in the absence of any drainage of salt (soil salinization where in arid climates irrigation water of otherwise low salinity is applied but where lack of drainage leads in the long run, to an accumulation of salt).

For $d \Rightarrow \infty$ (and hence $q \Rightarrow \infty$) $c = c'$, which is logical.
The solution shows that rate of desalinization depends primarily on the ratio d/V_0 (drainage rate in relation to the volume).

If d = drainage rate in 10^6 m^3/year
V_o = volume in 10^6 m^3
t = years

then desalinization expressed as a fraction of the original concentration proceeds as follows:

Table 4.3

After	1 year	2 years	5 years	10 years	20 years
$d = V_0$	0.37	0.135	0.006	0.000	0.000
$d = 0.5\ V_0$	0.61	0.37	0.08	0.006	0.000
$d = 0.1\ V_0$	0.90	0.82	0.61	0.37	0.135

If the period is e.g. 10 years or more the project may not be economically feasible.

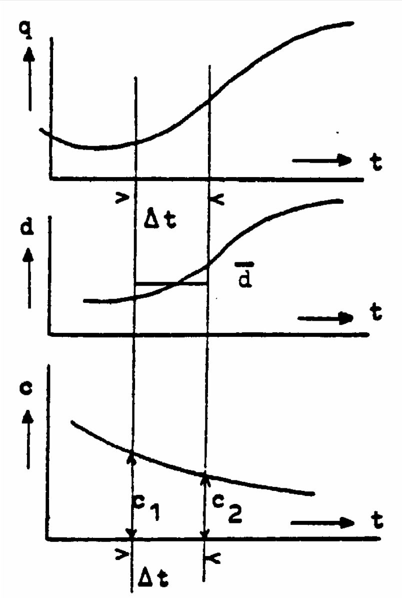

Figure 4.38

In general, however, *q, c', e, d* and V_o are functions of time (seasonal variations) and then discrete periods of time Δt (e.g. = 1 month) can be considered and the water and salt balance be drawn up for each period:

$$V_1 + q\Delta t = V_2 + (e + d)\Delta t$$ *Equation 4.47*

$$c_1 V_1 + qc'\Delta t = c_2 V_2 + \frac{c_1 + c_2}{2} d\Delta t$$ *Equation 4.48*

$$\frac{c_1 + c_2}{2}$$ *Equation 4.49*

From this set of equations c_2 can be determined for each time step:

$$c_2 = \frac{c_1 (V_1 - D/2) + T}{V_2 + D/2}$$ *Equation 4.50*

where $D = d\Delta t$ and $T = qc'\Delta t$

In this way the concentration c_2 at the end of each interval can be computed step by step.

Operation of coastal reservoirs

The requirements of drainage of excess water from floods and supply of water during dry periods leads to reservoir regulation with rule curves for the normal operational level (NOP):

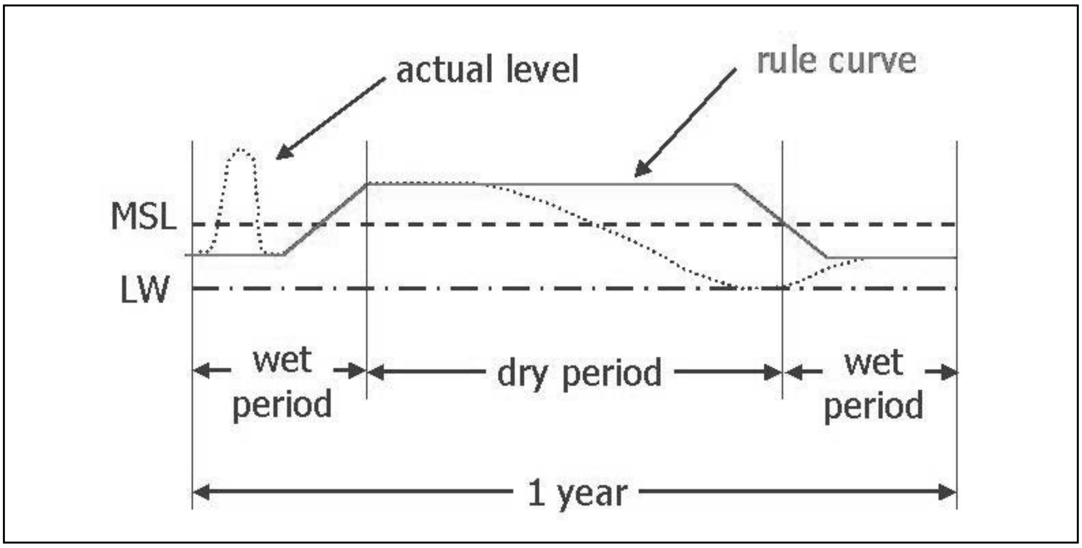

Figure 4.39: Operation rule curves

This works well in a climate with distinct wet and dry seasons. If there is the possibility of the incidence of a flood during the irrigation season (typhoon in Japan) then the operational level will always be low in order to have empty storage space.

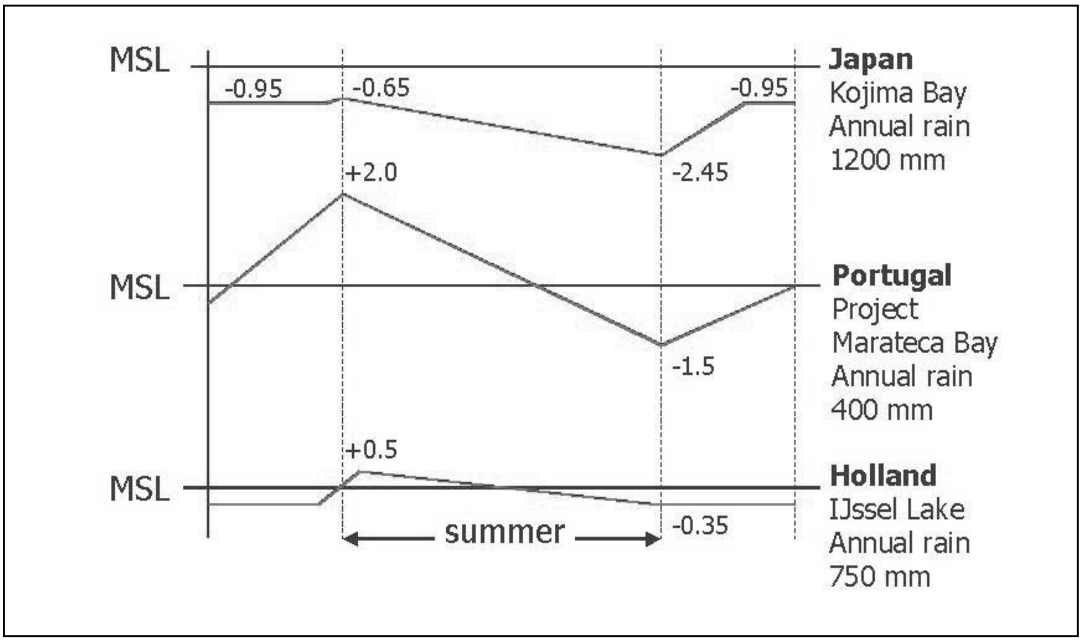

Figure 4.40

Case histories

The tables below show the water and salt balance after desalinization for the IJssel Lake in the Netherlands. The figures refer to the final size of the reservoir (area 1200 km^2) and to conditions of an average year.

Table 4.4: Water balance (in 10^9 m^3 per year)

IN		OUT	
River discharge (IJssel)	10.14	Evaporation	0.80
Drainage high ground	3.30	Abstraction irrigation water	2.00
Drainage polder areas	2.60	Abstraction flushing water	2.00
Rainfall on the lake	0.90	Drainage to the sea	12.14
Locking, leakage	p.m.		
	16.94		16.94

Table 4.5: Salt balance (in 10^6 kg Cl^- per year)

IN		OUT	
Salt load river	1097	Total evacuation	2448
Drainage high grounds	130		
Drainage polder areas	900		
Rainfall on the lake	9		
Locking, leakage	212		
Diffusion	100		
	2448		2448

From this Table it follows that the ultimate salinity of the reservoir for a year with average conditions is:

$$c_\infty = \frac{2448\times10^6 \text{ kg Cl}^-}{(12.14+2.00+2.00)\times10^9 \text{ m}^3} = 152\times10^{-3}\frac{\text{kg Cl}^-}{\text{m}^3} = 152 \text{ mg Cl}^-/\text{l} \qquad \textit{Equation 4.51}$$

The initial desalinization was a matter of 5 years (1932-1937) with at that time V_o = 13 x 10^9 m^3 and d = 11.4 x 10^9 m^3/year.

It is clear that the low ultimate salinity is due to the high amount of water, which is drained off to the sea and the relatively small salt load, factors which are characteristics for temperate climates.

Thus the transformation of the brackish Zuiderzee into the fresh water IJssel Lake was quite a success. As an entirely different case stands the failure of the Braakman, a small coastal reservoir in the south-west of the Netherlands which did not become fresh after damming off and initial (partial) desalinization.

As appears from the water and salt balances below the cause of the high salinity lies in an inflow into the reservoir of ground water with salinity almost equal to that of seawater together with a small drainage to the sea. The ground-water flow did not exist before the enclosure but was generated when after damming off a normal operational level of M.S.L. - 0,6 m was adopted, which is 0.6 m lower than the average level before. This was done to reclaim some tidal foreland without embanking.

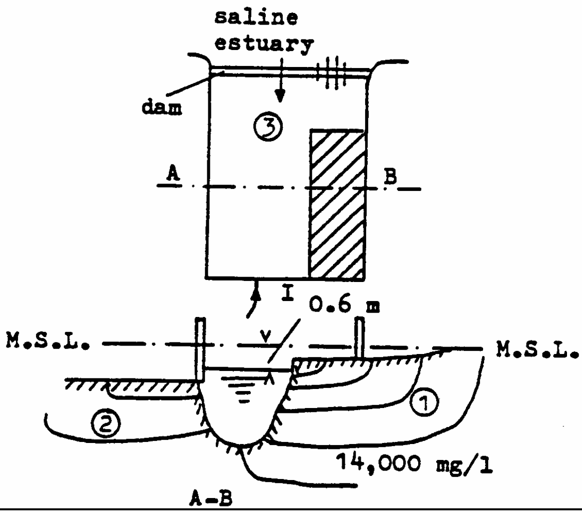

Figure 4.41

The reservoir area is 150 ha $V_o = 5 \times 10^6$ m^3.
The catchment is 1500 ha.

Table 4.6: Water balance for an average year in 10^6 m^3

IN		OUT	
Inflow I from the south	3.00	Loss by seepage (2)	1.64
Rain on reservoir	1.05	Drainage to the sea	2.77
Ground-water inflow (1)	1.09	Evaporation	0.90
Seepage under dam (3)	0.18		
	5.32		5.32

Table 4.7: Salt balance for an average year in 10^6 kgs Cl^-

IN		OUT	
Inflow I from the south	0.3	Loss by seepage (2)	6.68
Rain on reservoir	0.02	Drainage to the sea	11.54
Ground-water inflow (1)	15.34		
Seepage under dam (3)	2.56		
	18.22		18.22

From these tables it follows that the mean salinity in an average year is:

$$\frac{18.22\times10^{6}\ \text{kgs Cl}^{-}}{\left(1.64+2.77\right)\times10^{6}\ \text{m}^{3}} = 4.1\frac{\text{kg Cl}^{-}}{\text{m}^{3}} = 4100\ \text{mg Cl}^{-}/\text{l} \qquad \textit{Equation 4.52}$$

This water is unfit for most uses.

As can be seen from the balances the inflow (1) of ground water from the East plays a relatively small role in the water balance (20% of total inflow) but forms 85% of the total salt load. The generation of this flow after enclosure had been overlooked in the hydrological design.

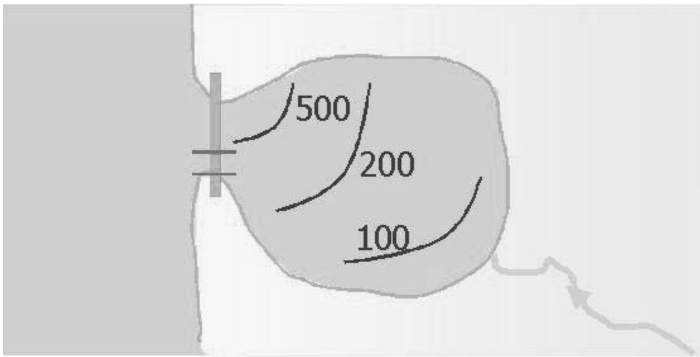

Figure 4.42

The assumption of complete mixing is an approximation. Usually higher concentrations occur near the sources of salt. The concept of the water and salt balance is still valid (continuity) but the computed salinity refers to the concentration of the water that is drained off and not to the average concentration and concentration in any point can be derived from the salinity distribution before enclosure.

4.5.3 Effect of embanking on the hydrological conditions

Flood protection by embanking is a very ancient, simple, and, in most cases, very economical way of preventing flooding. Hence its almost universal application. In many cases embanking provides the only practicable way of flood protection. However, embanking may entail a number of side effects, which may be of such a magnitude that this means of flood protection is not technically feasible. It is often a controversial issue. Embanking has repercussions in the following nine fields:

A. Hydrological effects
1) The longitudinal overland flow.
2) The over-bank storage.
3) The flooding of the strip between the channel and the dike.

B. Morphologic effects
4) The necessity of river training.
5) The natural process of delta building up.
6) The position of the river bed.

C. Environmental effects
7) The hygienic conditions of the land areas.
8) The water management in the land areas.
9) The cropping pattern and farm management.

Failures of flood protection schemes with embankments are often due to the fact that these effects had not been predicted or had been deliberately neglected.

The following considerations refer to the non-tidal reach in a delta and the flood pain.

1. Embanking eliminates the longitudinal overland flow

Before embanking the floodwater is discharged both by channel flow (1.0 to 2.5 m/sec) and by overland flow (0.1 to 0.3 m/sec). Owing to the small depth and high rugosity the velocity of the latter is small but it occurs over a considerable width.

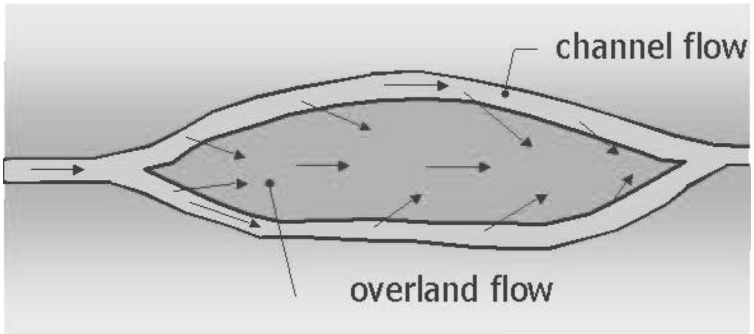

Figure 4.43

Embanking produces a constriction of the flow to the cross-section between the dikes. The channel flow increases and the flood levels are going up. If the flows are known the rise can be roughly estimated starting from a downstream control point, like the M.S.L.

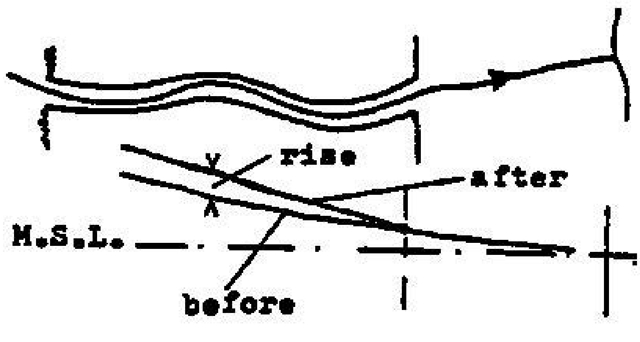

Figure 4.44

The rise can be reduced by setback or retired embankments at some distance away from the channel but this means sacrificing the best (level) soils.

2. *Embanking eliminates the over bank storage*

When during a flood the riverbanks are overtopped water flows in a lateral direction to fill the flood plains. This flow reduces the downstream channel flow from upstream A to downstream B. Thus the flow in B is smaller than in A as long as the water level on the land is still rising.

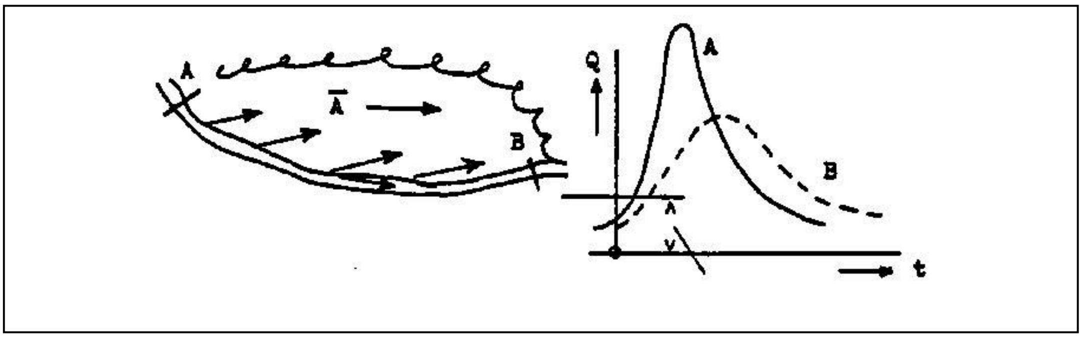

Figure 4.45

The lateral inflow Q_{lat} at any time is given by:

$$Q_{lat} = A\frac{dh}{dt}$$ *Equation 4.53*

where:

A = area of the flood plain
h = depth of flooding of the land

When the flow is receding the hydrograph in B becomes more similar to the one in A. The result is an increase of the peak flow not only between A and B but also downstream of B.

The effect is particularly significant in case of flash floods (rapid rise and sharp peak). When the flood is gentle and has a flat peak the effect is smaller because the flood plain is then filled to almost the same level as the maximum level in the river.

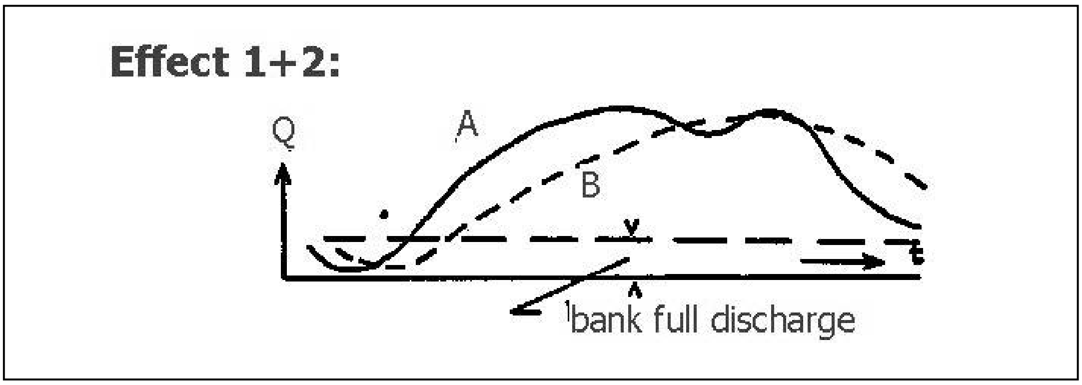

Figure 4.46

The hydrological effects 1 and 2 can roughly be estimated as indicated above. To make a more correct assessment physical or mathematical models can be applied. They require for their calibration extensive data on levels (channels and land) and discharges of floods under the original conditions.

The magnitude of the two effects together may be considerable like in the Pampanga valley and delta where according to a physical model test in case of a major flash flood a rise (upstream) of 2.5 to 3.5 m would occur.

3. Embanking causes more frequent flooding of the strip between the channel and the alignment of the dike

This is precisely where, because of higher elevation and better soils, most people are living and where fruit trees can grow and vegetables be raised.

4. Embanking may render river training necessary

Embanking, because of higher flows, tends to increase river meandering and hence bank erosion. Local protection is often inadequate and river training (groynes upstream) may be necessary. This is many times more expensive than the construction of dikes. Local setback dikes (as 2nd line of defence) are often applied when river training is not feasible (lack of stones).

5. Embankments exclude the deposition of silt and halt therefore the further building-up of the land areas

The much mentioned "fertilizing effect" of the silt is often small in terms of chemical nutrients. The rate of deposition in the lowest parts of the back swamps is often small.

6. Embankments may produce a rise of the river bed

Before embanking the lateral flow carried water as well as sediments to the flood plain. After embanking the channel between the dikes has to carry more material in downstream direction than before. This may entail silting-up. On the other hand the sediment-carrying capacity of the river has also increased (higher flow) and whether actually a silting-up or a scouring will occur depends on which of the two factors prevails.

It should be noted that under natural conditions a rise of the river bed occurs anyhow as a result of delta extension into the sea and hence lengthening of the river course.

7. Embanking affects the hygienic conditions of the land areas

One of the beneficial effects of river floods is the periodic flushing or rinsing with clean and fresh water. All kind of wastes, dirt, human disposal and sometimes salinity accumulated during the dry season is effectively removed. After embanking the water becomes stagnant and special provision may have to be taken to get the required periodic washing out.

8. Embanking affects the water management of the land areas

After embanking a drainage system with sluices or pumping stations and canals has to be installed to remove excess water from local rainfall. This may entail over-drainage and reduction of water conservation so that less water is present in the area at the beginning of the dry season. Supplementary during the dry season by flooding is also eliminated.

A new water management system has to be created with new perspectives for more intensive land use and crop diversification but also a new and unknown exploitation of the farmlands with higher costs and higher returns.

4.5.4 Drainage of level areas

Embanked areas in coastal areas and low-lying coastal areas where the water levels inside can be controlled are called 'polders'. The areas are very flat and horizontal (level areas).

Removal of excess water from local rainfall on these polders often poses serious problems because of the high levels of the receiving waters and the absence of any natural land gradients.

The land in coastal areas has been built up by the river (and the sea) and the general configurations shows natural levees or ridges along the river channels consisting of sediments deposited during the floods and the back swamps or depressions which occupy most of the areas in between.

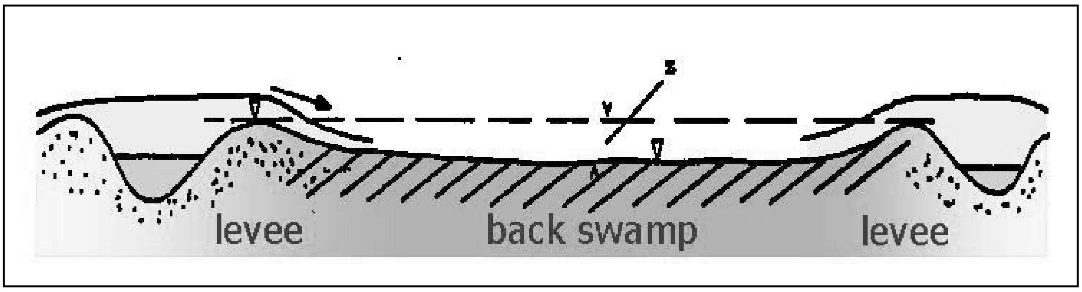

Figure 4.47

Upstream the difference z in elevation between the crest of the natural levee and the back swamp may be as much as 3 to 4 metres. This difference decreases in downstream direction and is almost absent near the coast line (tidal flooding). The natural levees are sandy; in the back swamps heavy soils occur. In the coastal strip the texture is more homogeneous and heavy.

In the river reach the crests of the natural ridges are built up to a level around the annual flood or slightly higher. In the tidal reach the level is around mean high tide. The back swamp is lower, especially upstream.

During floods higher than the annual flood the whole area would be flooded. Embankments can prevent this but the problem of the removal of excess water from local rainfall remains because of the high river level.

It is possible, of course, to remove excess water by pumping but this is expensive. If the floods have short duration, excess water can temporarily be stored in the polders and released later but this is not possible if the high flood levels persist for several weeks.

As can be seen from the longitudinal profile there is a strip near the coast where the land elevation of the back swamp is higher than the low water level of the river even when there is a flood so that gravity drainage is possible (tidal drainage). The distance d must be above a certain minimum.

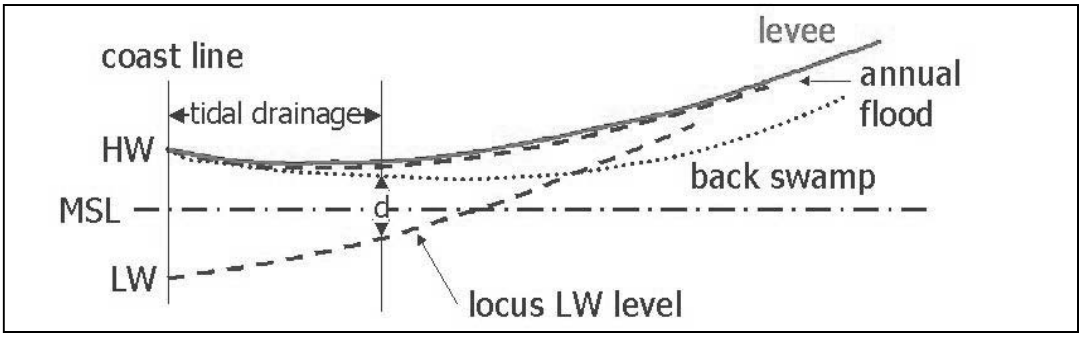

Figure 4.48

Tidal drainage is an intermittent drainage using the period that the water level of the recipient basin is lower than the water level of the canal at the inner side of the sluice. During the high water the gates remain closed and the water level inside may rise due to inflow of drainage water.

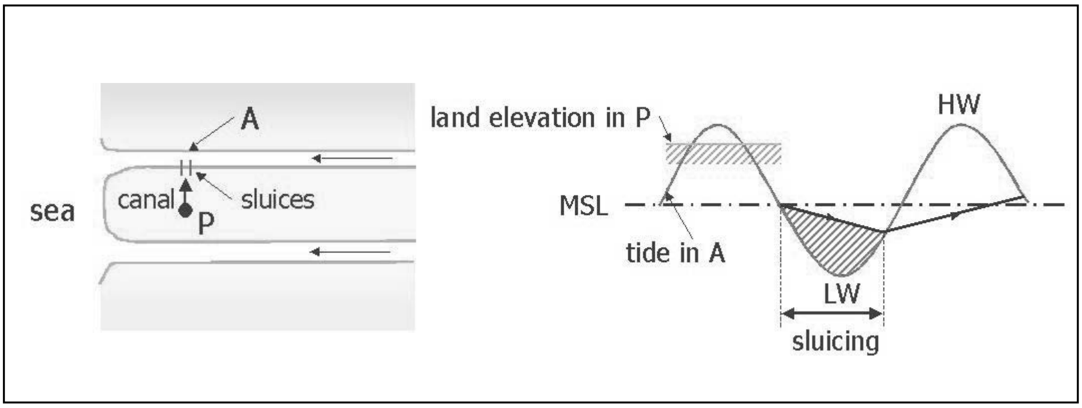

Figure 4.49

The flow through the sluice may be critical or sub-critical depending on the depth of the sill with respect to the tidal level.

In P the water level in the minor drainage at the time of maximum flow should be 1.0 to 1.2 m below ground elevation for dry soil crops and 0.2 to 0.4 m for paddy. The gradient for the flow in the drainage canals from P to A is taken as small as possible, e.g. 5 x 10^{-5}. For a distance of, for instance, 10 km this means 0.5 m. The head in the sluice at the time of maximum flow around LW is, say, 0.3 m. In conclusion the minimum distance d must be 1.8 - 2.0 m for dry soil crops and 1.0 to 1.2 m for paddy.