Appendix

Appendix A: Reference list

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Appendix B: Notation

K_1	declination tide component
O_1	declination tide component
M_2	principal tide component
S_2	principal tide component
h	water level (m)
Q	discharge (m^3/s)
B	flow width (m)
t	time (s)
x	distance (m)
g	acceleration due to gravity (m/s^2)
A_s	cross-sectional area of the flow (m^2)
R	hydraulic radius (m)
κ	linearised expression for the resistant (s^{-1})
c_f	the friction coefficient (-)
U	flow velocity (m/s)
ω	frequency (rad/s)
σ	ration between resistance and inertia (-)
µ	damping constant (m^{-1})
k	wave number (rad/m)
c	phase velocity (m/s)
δ	phase lag (-)
c_0	wave velocity (m/s)
L	length of the estuary (km)
d	depth (m)
B_s	total width (m)
h^	amplitude of the incoming wave (m)
u^	velocity of the incoming wave (m/s)
L	wave length (m)
r_r	reflection coefficient (-)
r_t	transmission coefficient (-)
N	Canter Cremers estuary number (-)
V_{fl}	flood volume (m^3)
S	steady state salinity (kg/l^3)
S_f	fresh water salinity (kg/l^3)
S_0	salinity at the estuary mouth at HWS (kg/l^3)
Q_f	fresh water flushing (m^3/s)
A	cross-sectional area (m^2)
D	longitudinal dispersion (m^2/s)
D_0	dispersion coefficient at the estuary mouth at HWS (m^2/s)
A_0	cross-sectional area at the estuary mouth (m^2)
a	cross-sectional convergence length (m)
K	dimensionless Van den Burgh's coefficient (-)
h_0	constant tidal average stream depth (m)

E	tidal excursion (m)
H	tidal range (m)
α_0	calibration coefficient equal to D_0/Q_f
L	intrusion length at HWS (m)
T	tidal period (s)
h	stream depth (m)
v	tidal velocity amplitude (m^2/s)
v_f	fresh water velocity (m^2/s)
N_R	Estuarine Richardson number (-)

Appendix C: Geological map (1952)

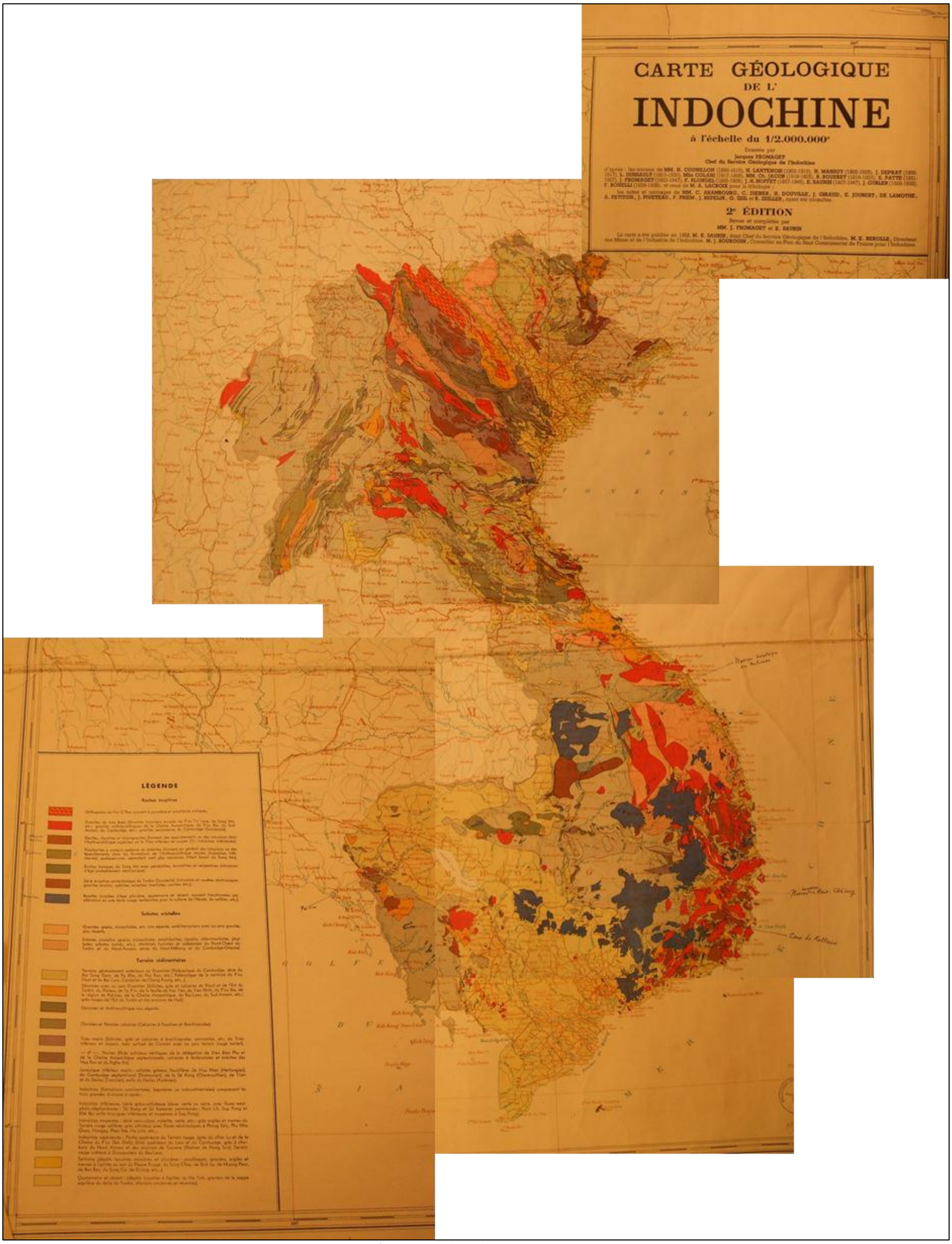


figure C-1: geological map Vietnam

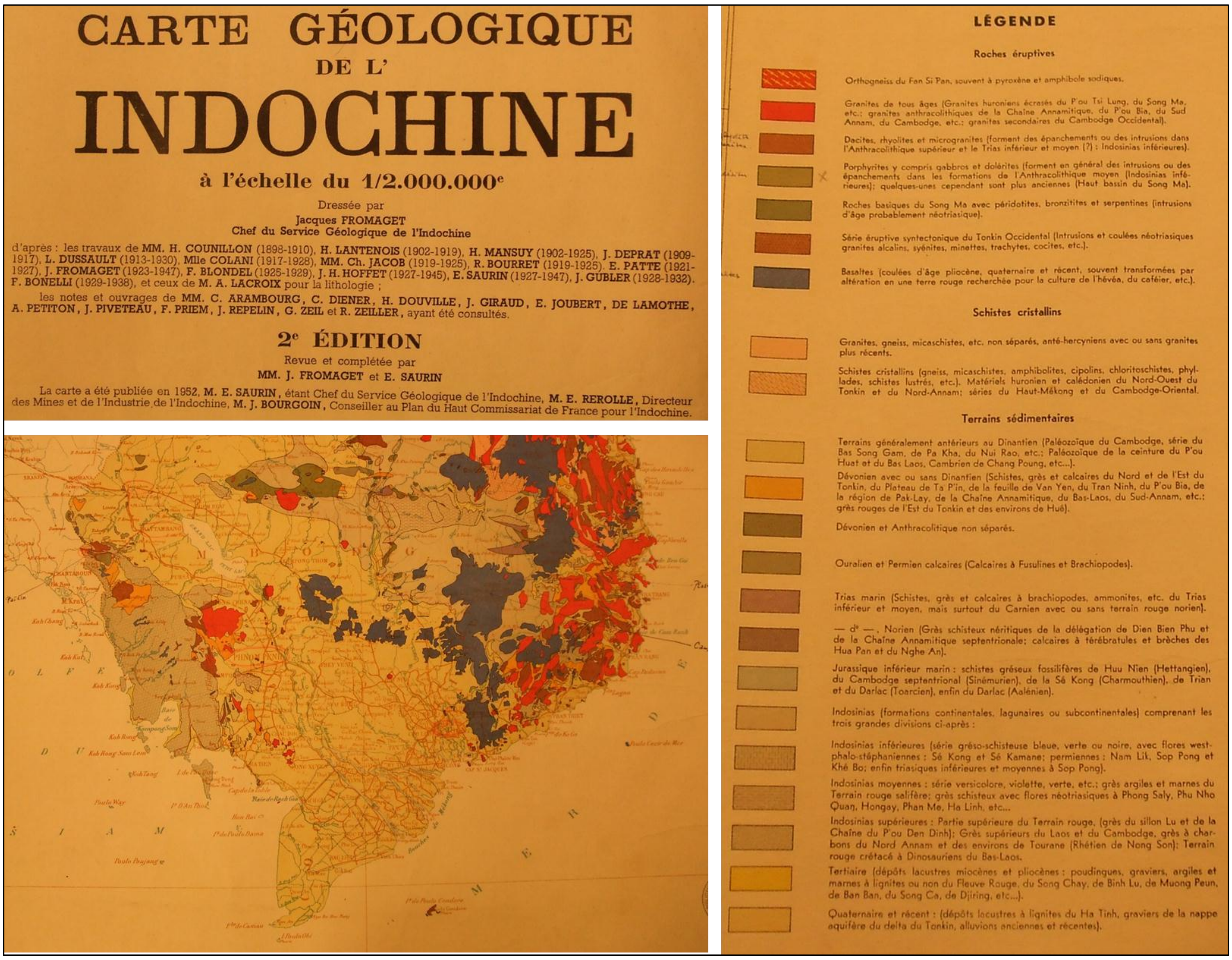


figure C-2: Details geological map

Appendix D: Land subsidence

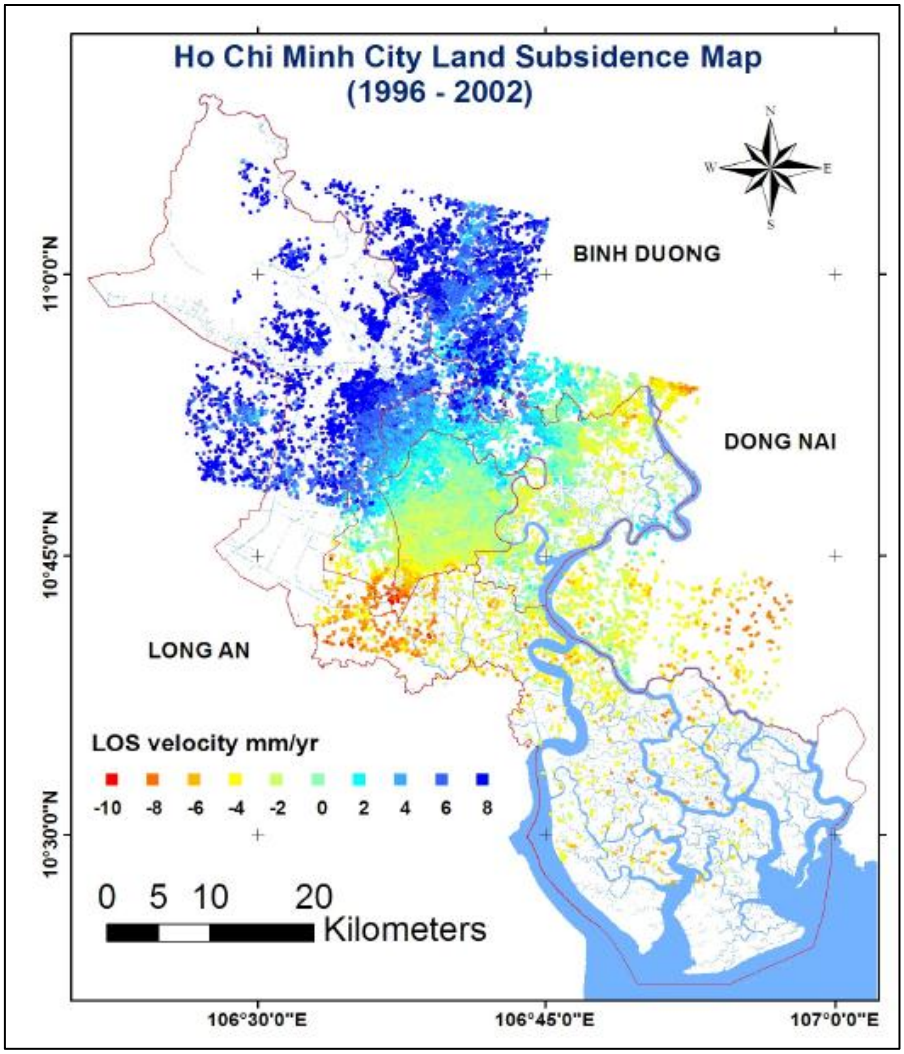


Figure D-1: land subsidence map Ho Chi Minh City (1996-2002)

Appendix E: Propagation and water movement of a tidal wave in an estuary

The tidal waves generated offshore propagate via marginal seas and coastal zones into tidal inlets, estuaries and rivers. A tidal wave is initially a propagating wave with constant amplitude and a wave of which the water level elevation and the flow velocity are in phase. When a tidal wave propagates into an estuary these characteristics change, this is the result of two processes. The first process is loss of energy due to friction leading to decrease in amplitude. The second process is transition and reflection of wave energy when propagating; this is the effect of abrupt changes in cross sections. As a result of reflection the characteristics of the wave change and the water movement can be described as a mix of a propagating wave and a standing wave. Due to non-linear effects and river discharge the shape of the wave and the average water level over time change, see paragraph E.7 and E.8.

E.1 Harmonic method

In order to gain insight in the processes that occur in an estuary and to determine the effect of different variables, simplification is needed. Assuming that the tidal wave which enters the system is sinusoidal, the harmonic method, developed by Lorentz, can be applied. As starting point the continuity equation and linearised equation of motion will be used and the assumption is made that the water level elevation doesn't affect the geometry of the cross sections. In the linearised equation of motion the advection term is neglected and the resistance term is linearised. Because of these assumptions the equations can only be applied for river sections with a limited length and relative low waves.

$$B\frac{\partial h}{\partial t}+\frac{\partial Q}{\partial x}=0$$

$$\frac{\partial Q}{\partial t}+gA_s\frac{\partial h}{\partial x}+\kappa Q=0$$

$$\kappa=\frac{8}{3\,\pi}c_f\frac{U}{R}$$

$$\sigma=\frac{\kappa}{\omega}=\frac{8}{3\pi}c_f\frac{U}{\omega R}$$

Assuming that every cross section is prismatic, Q can be eliminated from the above equations, resulting in:

$$\frac{\partial^2\zeta}{\partial t^2}-c_0^2\frac{\partial^2\zeta}{\partial x^2}+\kappa\frac{\partial\zeta}{\partial t}=0$$

$$c_0=\sqrt{\frac{gA_s}{B}}$$

The general periodic solution of this equation exists of two waves propagating in opposite direction, each damped exponentially in their direction.

$$\zeta(x) = C_+ e^{-px} + C_- e^{px} = \zeta_+ + \zeta_-$$

$$Q(x) = Bc * cos\delta * e^{i\delta}(\zeta_+ + \zeta_-)$$

With two complex constants C+ and C- (each with amplitude and phase) and p indicating the variation over x of amplitude (damping constant μ) and phase (wave number k)).

E.2 Single propagating wave

Reflection is neglected in this paragraph to simplify the situation in order to get more insight in other processes. If reflection is neglected the solution of the above defined equation will reduce to a single propagating wave, which is exponentially damped in the wave direction.

E.3 Simplified model

The complex HCMC river system is simplified as a straight river where a tidal wave enters the river at one side and the other side is closed, without any river discharge. So a one sided closed basin with two given boundary conditions, Figure E-1. The given boundary conditions are a water level elevation at the mouth of the river, M_2-tide (T=12h and 25min, with $\hat{H}$=2,5m) and a river discharge of 0 m^3/s upstream, Dau Tieng reservoir. All branches are excluded, there is no lateral inflow, no storage and the cross-sections are constant and rectangular.

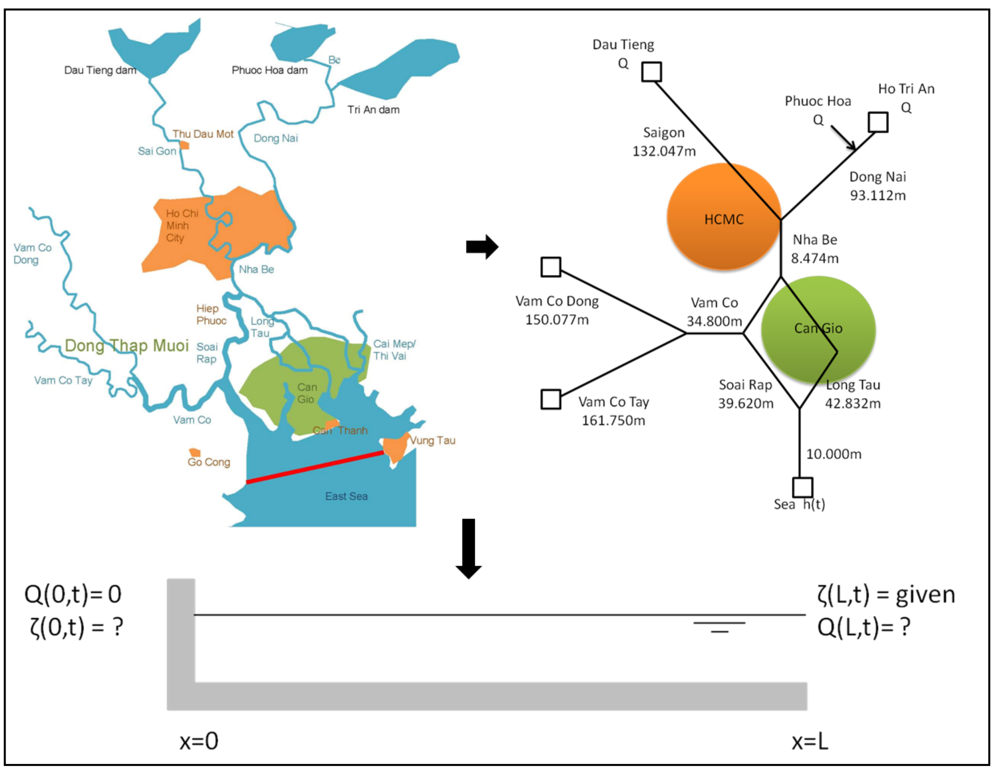


Figure E-1: simplification of system

In table E-1 the dimensions of the simplified system, the used formulas and the results of the calculations are given.

Single propagating wave		
L	200	km
d	13	m
B	700	m
B_{total}	700	m
R	12,53	m
C_0	11,29	m/s
ω	,00014	rad/s
c_f	0,004	-
h^	2,5	m
u^	2,17	m/s
k_0	1,24E-05	rad/m
L	506825	m
σ	4,2	-
δ	38,31	-
k	2,02E-05	rad/m
μ	1,60E-05	m-1
c	6,924	m/s
L	310775	m
k_0L	2,47	rad

$$\omega = \frac{2\pi}{T}$$

$$ud = hc$$

$$k_0 = \frac{\omega}{c_0}$$

$$L_0 = c_0 T = \frac{2\pi}{k_0}$$

$$\sigma = \frac{8}{3\pi} c_f \frac{U}{\omega R}$$

$$\delta = \frac{1}{2} \arctan \sigma$$

$$k = \frac{k_0}{\sqrt{1 - \tan^2 \delta}}$$

$$\mu = k * \tan \delta$$

$$c = c_0 \sqrt{1 - \tan^2 \delta}$$

$$L = \frac{2\pi}{k} = cT$$

table E-1: results of amplitude reduction calculations due to friction

The amplitude of the water level elevation and the discharge reduces exponentially with a factor exp(-μΔx) over distance Δx. Assuming that Δx =10km, the amplitude reduces with 0,85 over that distance. As stated before this method can only be applied if the river sections have a limited length, Δx. Because the reduction is less than 20% over 10km the assumed length of a section is acceptable. In Figure E-2 the damping of the amplitude over the entire estuary is shown, assuming a constant cross-section and constant friction. The amplitude of water level and discharge is reduced from 2,5m at the mouth of the estuary to 0,46m at the end.

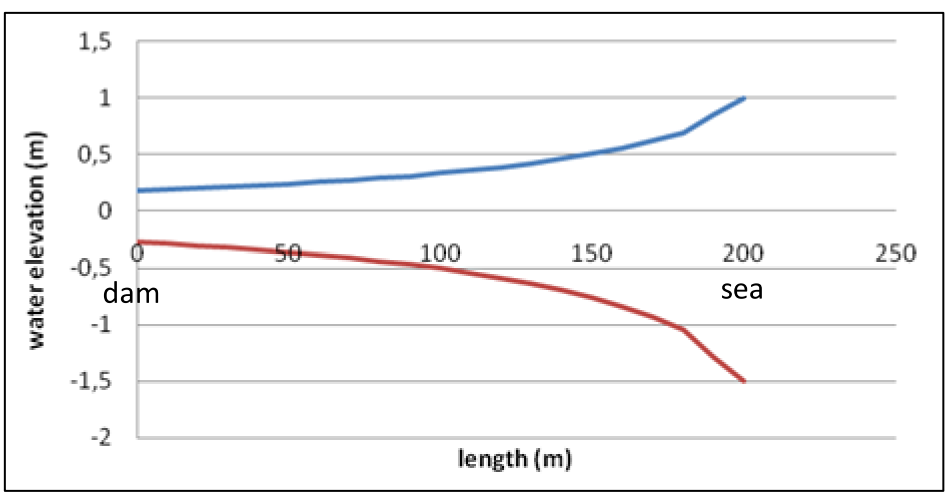


Figure E-2: amplitude reduction due to friction over the estuary

It is assumed that there is no difference between flow width and total width, therefore the wave velocity is constant over the estuary. Due to friction there is a difference between the wave and the phase velocity. This difference is 61% at the mouth of the estuary and decrease s in upstream direction due to reduction of the amplitude resulting in an increase of the phase velocity.

E.3.1 Sensitivity analysis of 1D-river model

In reality dimensions of cross-sections and friction will vary and there will be a difference between the flow and storage width. The sensibility of the results for the different variables will be determined.

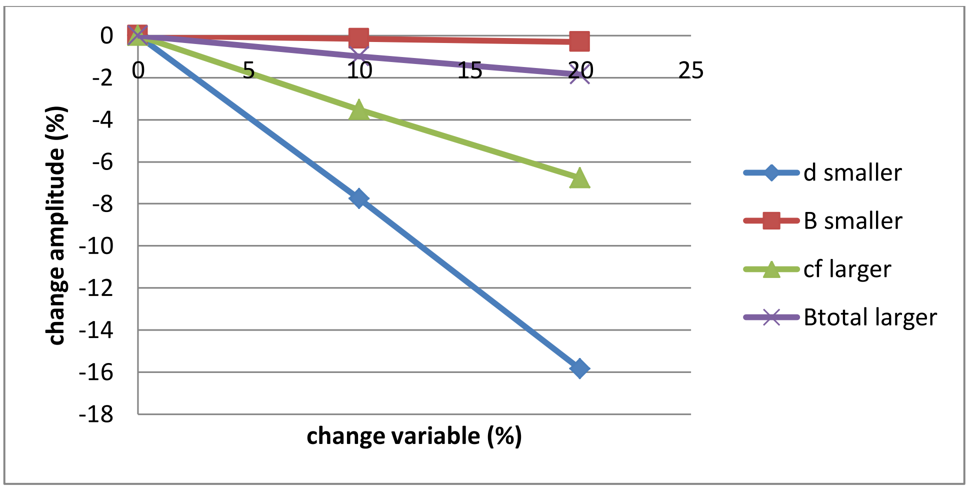
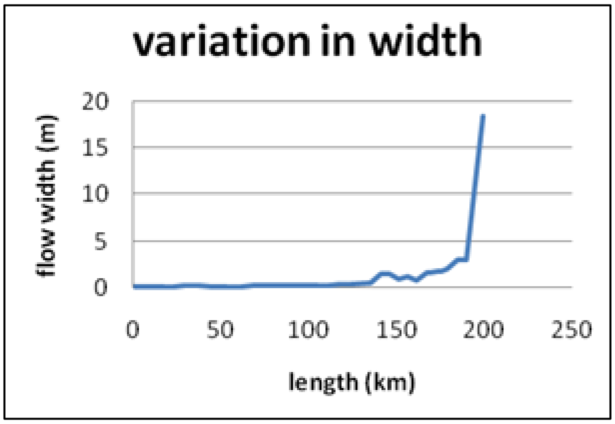


Figure E-3: sensitivity analysis for amplitude reduction calculations due to friction

Figure E-3 shows the results of the sensitivity analysis made for variables used to calculate the amplitude reductive due to friction. It is clear that the results are the most sensitive for a change in the depth (d) and a change in flow width (B) and total width (B_{total}) has almost no effect. In general an estuary becomes shallower and less wide in upstream direction; as a result the amplitude will reduce rapidly when propagating through a natural estuary.

E.3.2 Ho Chi Minh City

In the case of HCMC the dimensions of the cross sections of the estuary vary a lot, see Figure E-4. The trend of the graphs shows indeed a decrease in dimensions in upstream direction. But because of splitting and flocking of rivers the graphs show some irregularities. The shapes of the cross sections are in most cases trapezoidal and sometimes triangular.



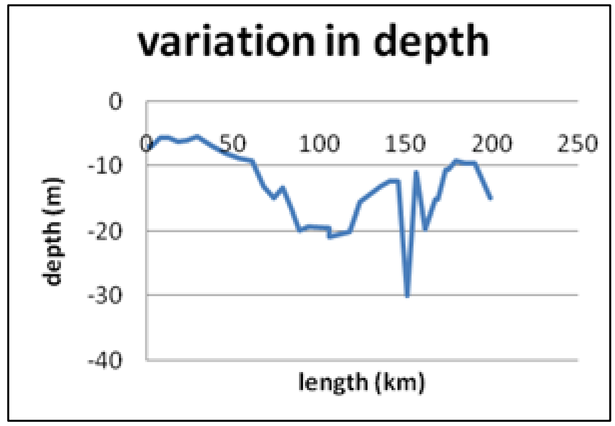


Figure E-4: total width and depth Dau Tieng - Soai Rap

The variation in dimensions of cross sections over the estuary length compared to the wavelength of the incoming tidal waves is substantial and therefore reflection cannot be neglected.

E.4 Partial reflection

The transition between adjacent cross sections influences the propagation and water movement of an incoming wave. If the transition is regular and the change in depth or width over the estuary length is small compared to the wave length of the incoming wave, the phenomenon of shoaling will occur. Due to narrowing of the cross sections the wave velocity decreases, the wave period remains constant resulting in a decrease in wave length. This means that when the energy remains constant (no friction) the wave will become higher.

If the transition is abrupt or the change in depth or width over the estuary length is substantial compared to the wave length of the incoming wave, reflection will occur. Reflection is the phenomenon that occurs when a wave propagates from one prismatic section into an adjacent section which is shallower, smaller or rougher compared to the first section. At the connection point the wave will be partly reflected and partly transmitted. The ratio between reflection and transmission depends on changes in width, velocity and friction. If the friction does not vary between the adjacent sections, γ becomes $\gamma = B_2c_2/B_1c_1$ and there will be no phase difference.

$$\gamma = \frac{B_2 c_2}{B_1 c_1}\frac{\cos\delta_2}{\cos\delta_1}e^{i(\delta_2-\delta_1)}$$

$$r_r = \frac{1-\gamma}{1+\gamma}$$

$$r_t = \frac{2}{1+\gamma} = 1 + r_r$$

If reflection occurs in a system existing of adjacent prismatic sections, the wave can be described as the sum of incoming and reflected waves each damped in their own wave direction, a mix of a progressive and a standing wave.

At the end of the estuary, at the upstream dam there will be complete reflection of the incoming wave. This can result in a standing wave in the estuary.

E.4.1 Simplified model

In the case of HCMC the mouth of the estuary has a width of 18,5 km and narrows till 70 m at the end of the estuary. The wave length of the incoming wave is 506 km so the change in width is substantial compared to the wave length and the phenomenon reflection will occur.

For the calculations it is assumed that the depth is constant, the estuary narrows in upstream direction (18,5 km till 70 m), the shapes of the cross-sections are prismatic and there is no friction; see Figure E-5. The gamma of each adjacent section can be determined and the reflection- and transmission coefficients can be calculated. In Figure E-6 the amplitude of the transmitted wave is shown.

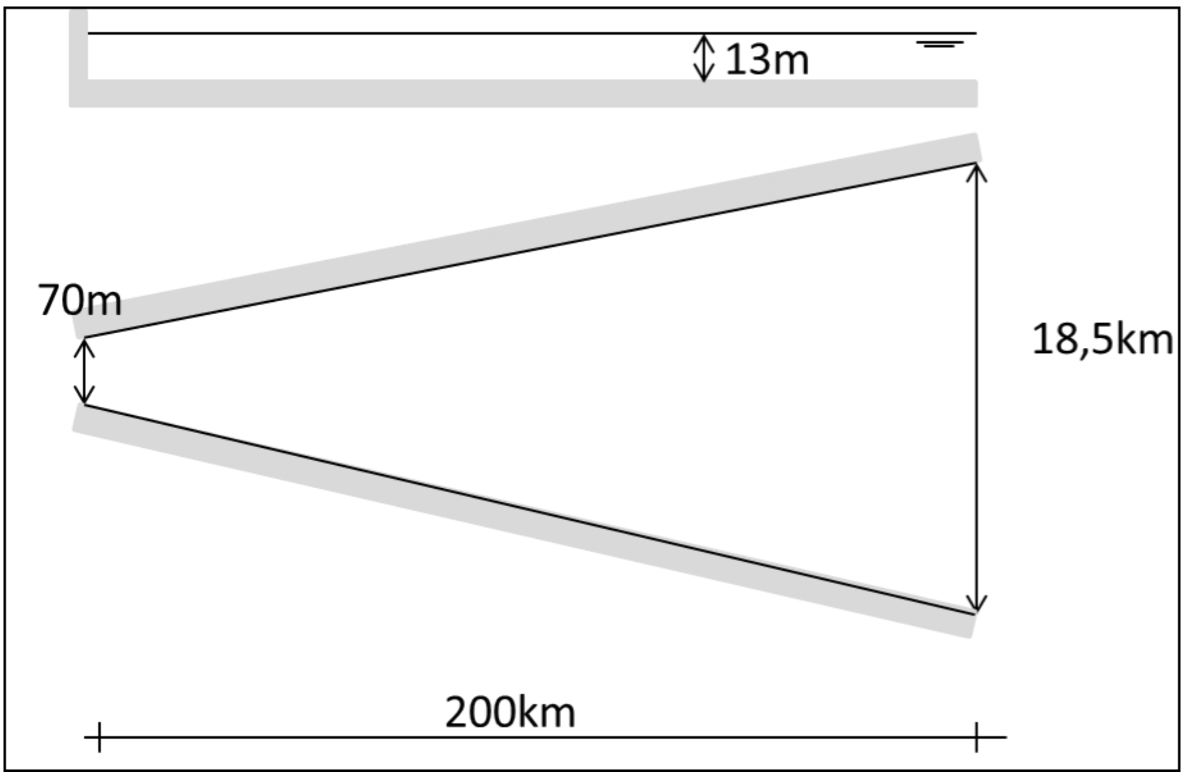


Figure E-5: simplified model used for partial reflection calculations

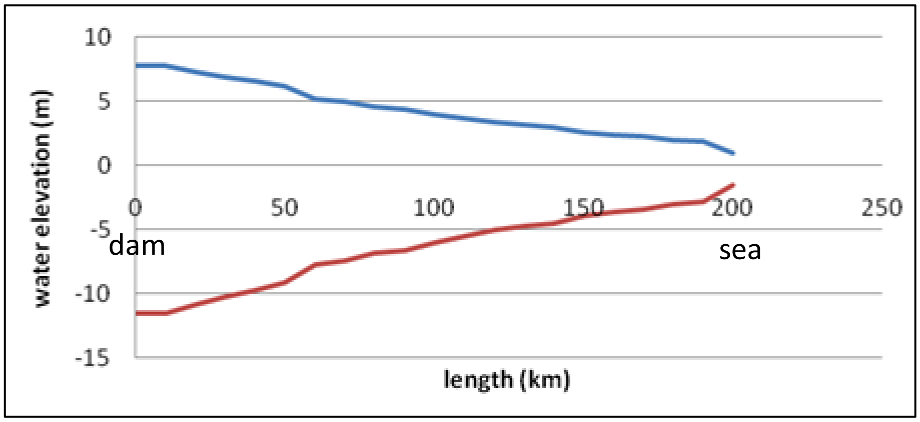


Figure E-6: amplitude change due to partial reflection along the estuary

E.4.2 Ho Chi Minh City

In reality the depth is not constant and also the transmission between different cross sections can be abrupt. Locally partial reflection can play an important role in the propagation and water movement of a tidal wave in an estuary. As discussed in the paragraph E.3.2 splitting and flocking of rivers have effect on the cross-sections of rivers. In Figure E-7 the cross sections of the Nha Be, Saigon en Dong Nai rivers are shown just before and after the point where the Saigon and the Dong Nai flow in the Nha Be. It is clear that there is an abrupt change in dimensions of the cross-sections so partial reflection is important for the propagation and water movement of a tidal wave in an estuary.

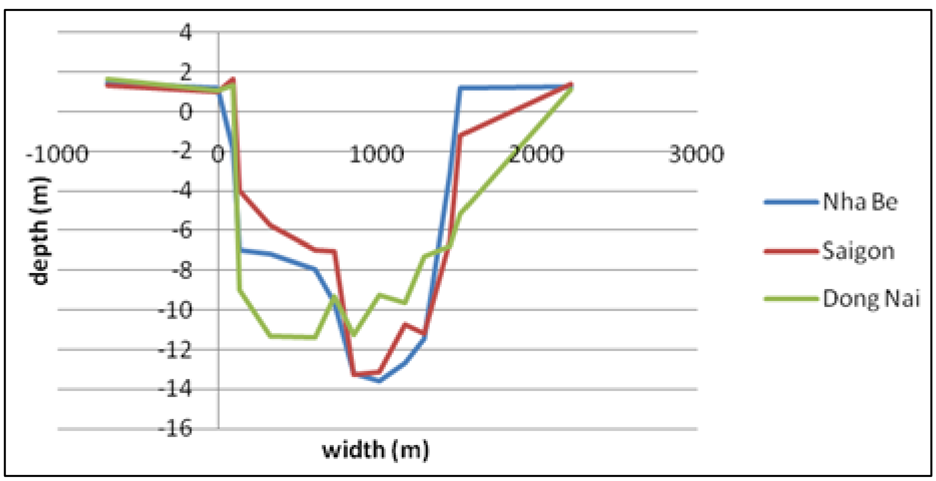


Figure E-7: cross-sections of Nha Be, Saigon and Dong Nai around connection point

E.5 Friction and partial reflection

To get insight in the system processes simplifications were made in paragraphs E.2 and E.4. The effect of friction was described assuming that there is no reflection and the phenomenon reflection was examined assuming there is no friction. In reality both friction and partial reflection take place at the same time and the combinations of these phenomena determine the change in amplitude over the estuary.

E.5.1 1D-river model

In Figure E-8 the effect of both phenomena is combined for the case that the depth is constant, the estuary narrows in upstream direction (18,5km till 70m), the shapes of the cross-sections are prismatic and a constant friction of 0,1. In the mouth of the estuary partial reflection is dominant and the amplitude amplifies, in upstream direction the amplitude reduces due to friction. In these calculations the influence of fricion on reflection- and transmission coefficient is neglected, so there is no phase difference. In reality non-linearity and river discharge play an important role and influences the change of the amplitude over the estuary, see paragraph E.7 and E.8. The graph gives a general idea of the effect of friction and partial reflection on the amplitude of water levels and discharge.

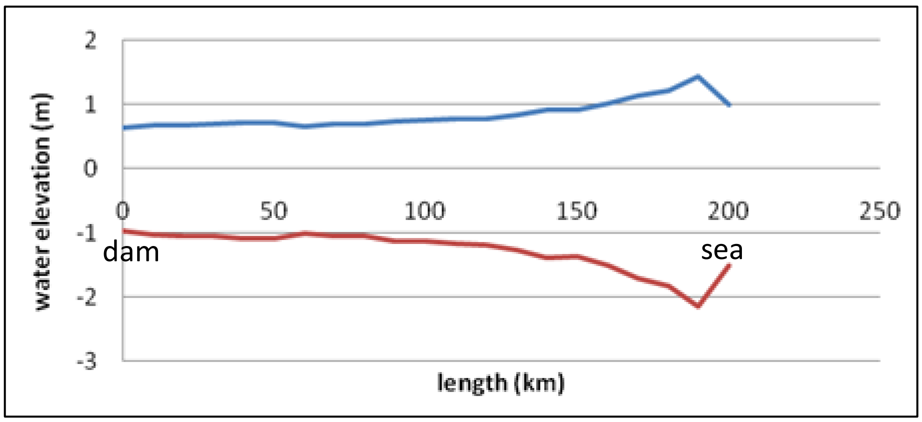


Figure E-8: results of amplitude change calculations over the estuary

E.6 Resonance

In the above calculations it is assumed that a harmonic wave enters the estuary at one side and that the other side is closed, without any river discharges. Because of this assumption the system should be checked on resonance. Resonance occurs when the period of the forced oscillations is equal to one of the natural periods of a system. In theory the movement becomes infinite, but in practice this never happens due to friction. The phenomenon depends on the relative length of the basin and the ratio between resistance and inertia.

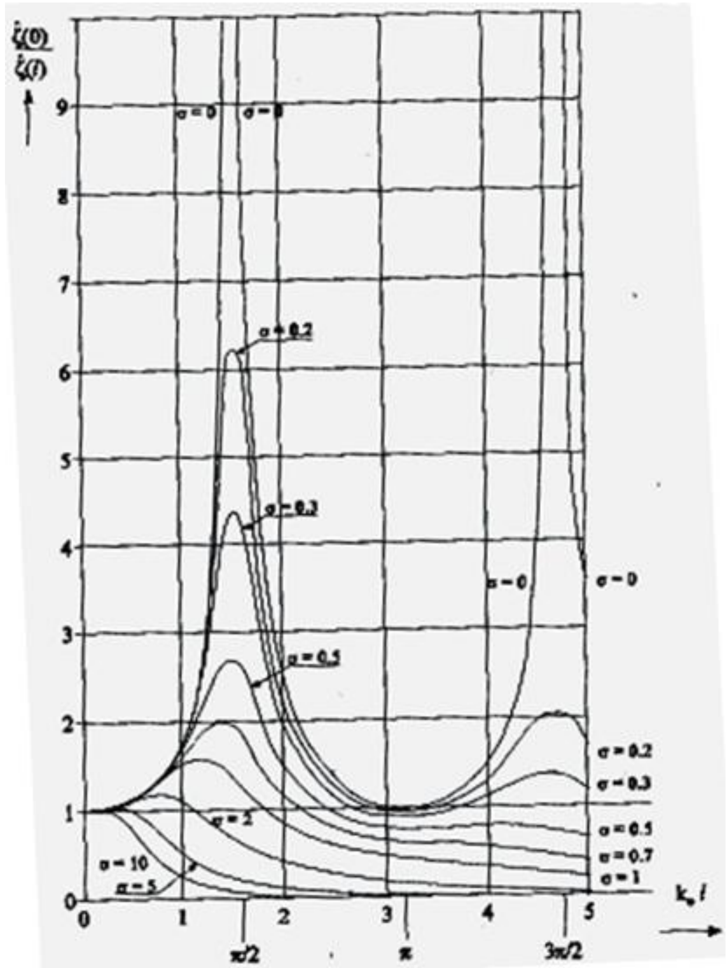


Figure E-9: resonance in one site closed basin

E.6.1 1D-river model

The check on resonance will be performed with the assumptions made in paragraph E.3. The results of the calculations, shown in table E-1 indicate that the relative length of the basin is around 2,47 rad and the ratio between inertia and resistance is 4,20. Figure E-9 shows that the ratio between the wave amplitude at the end and at the mouth of the basin is circa 0,25. So in this case there is no resonance.

E.7 Non-linear effects

There are two kinds of non-linearity. The first one is the terms in the equation of motion that are not linear, such as the advection and friction term. The second one is the non-linearity in the geometry; depth, width and storage. The solution of a linear system will always have the shape of a sinusoidal, even if the forced oscillation is not sinusoidal. In the case of a non-linear system higher harmonics will arise if the forced oscillation is not sinusoidal. Due to higher harmonics the average water level in time will vary with position. Non-linearity also causes deformation of the propagating wave. The combination of non-linear geometry variables determines the reduction or increase in amplitude and the phase difference. Non-linearity increases in shallow water due to the fact that the tidal amplitude is large compared to the depth. When an M_2-tide enters shallow water it generates overtones with double, triple etc. frequencies.

In coastal zones geometry non-linearity plays an important role. In a propagating wave the phase difference between water level and flow velocity is smaller than π/2, so flood occurs when the water level in the estuary is high and ebb occurs when the water level is low. Since during ebb the discharge is similar to the discharge during flood, on average the ebb velocities must be larger than the flood velocities due to the smaller cross-section. This results in asymmetry, with a larger ebb period than flood period. Because of the quadratic relation between flow velocity and resistance the effect of friction increases during ebb. This corresponds to a stronger gradient, resulting in an increase of the average water level in time in upstream direction, see Figure E-10. This phenomenon is independent of river discharge.

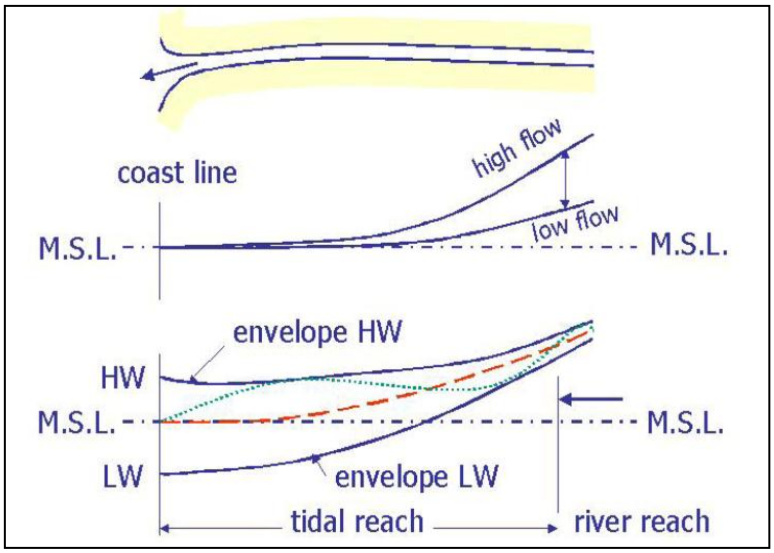


Figure E-10: increase of average water level due to non-linear effects

E.8 River discharge

If a tidal wave enters a river it will meet the water flowing from the upland. River discharge will influence the velocity, the amplitude and the shape of the wave. The average water level also changes; this is more notable upstream, where the influence of the river discharge is larger than at the mouth of the estuary.

E.9 Ho Chi Minh City

Figure E-11 shows some water level measurements at different locations around HCMC in 2000, Vung Tau – Dau Tieng reservoir. These measures show similarities with the theory of a propagating tidal wave in an estuary. Between Vung Tau and Nha Be the amplitude of the incoming wave amplifies a little; this can be explained by partial reflection. Going upstream the amplitude reduces due to friction. What also can be seen is that the average water level is increasing in upstream direction; this is due to non-linear effects and river discharge.

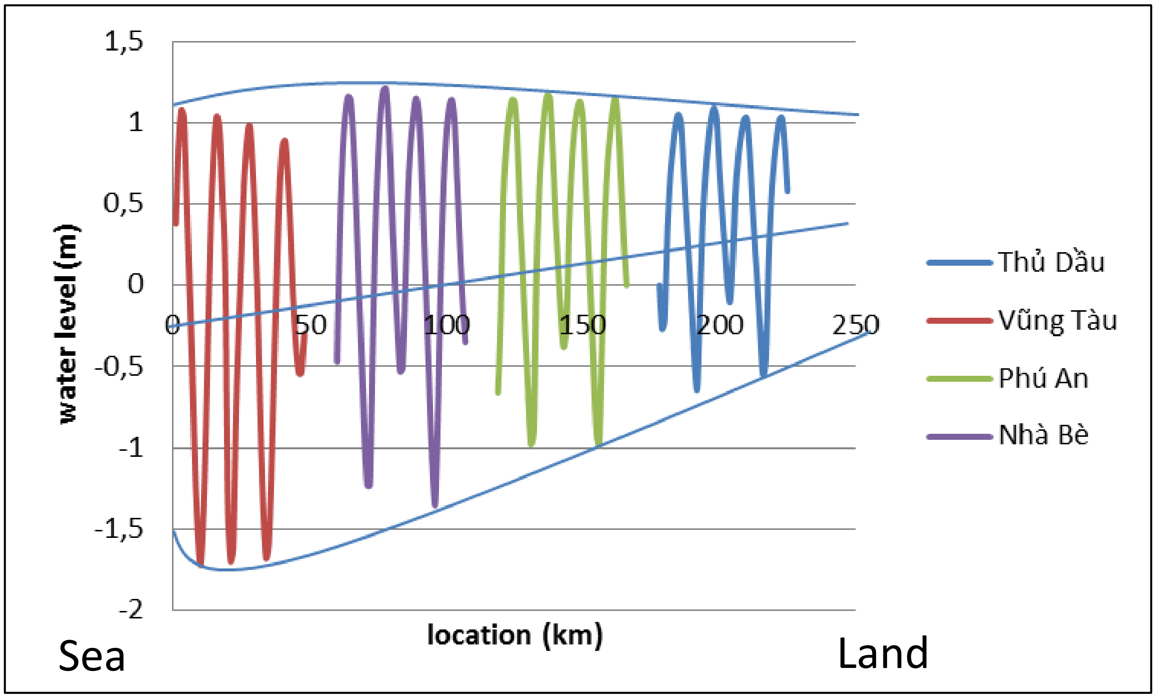


Figure E-11: measured water levels around HCMC

Appendix F: Flood wave

F.1 Introduction

Upstream of HCMC two reservoirs have been constructed and a third one will be constructed shortly. Two of the reservoirs are built for irrigation and flood protection reasons while the last and largest one is built to produce energy. For each of the reservoirs design discharges are determined for 1/200, 1/500 and 1/1000 year. The 1/1000 year design discharge for the Dau Tieng reservoir is 2.800 m^3/s and for the Tri An reservoir and Phuoc Hoa reservoir the design discharge together is 27.000 m^3/s. The duration of the flood waves are in the order of one week.

F.2 Steady approach

A flood wave is a spatial, slowly varying, steady phenomenon. To gain insight in the phenomenon it is assumed that the water level gradient changes so slow that the water mass is able to follow the change and the velocity has adapted rather well to the instantaneously gradient at any moment. As a result the inertia term in the equation of motion can be neglected. In the Sobek-model the inertia term will be taken into account but in hand calculation not. The force due to the gradient is in balance with the resistance, this is the so called steady approach. Q|Q| is replaced by Q^2, this is possible because the flow direction will not change in the considered situations.

$$\frac{\partial A}{\partial t} + \frac{\partial Q}{\partial x} = 0$$

$$g\frac{\partial h}{\partial x} + c_f \frac{Q^2}{A_s^2 R} = 0 \ \ or \ \ -\frac{\partial h}{\partial x} = c_f \frac{Q^2}{g A_s{}^2 R} = i_w$$

Due to neglecting of the inertia term 'dynamic' wave propagation is no longer possible (propagation in two directions). This results in an expression for the discharge and velocity:

$$Q = A_s \sqrt{\frac{gRi_w}{c_f}}$$

$$U = \sqrt{\frac{gRi_w}{c_f}}$$

In the case of a flood wave there is a dominant flow direction and therefore it is useful to highlight the effect of a bed level gradient. The water level gradient term ($\partial h/\partial x$) is expressed as the sum of the bed level gradient ($i_b = -\partial z_b/\partial x$) and the gradient of the flow depth ($\partial d_s/\partial x$).

$$\frac{\partial h}{\partial x} = \frac{\partial z_b}{\partial x} + \frac{\partial d_s}{\partial x} = \frac{\partial d_s}{\partial x} + i_b$$

The equation of motion can be written as:

$$\frac{\partial d_s}{\partial x} - i_b + c_f \frac{Q^2}{g A_s^{\ 2} R} = 0 \ \ or \ \ \frac{\partial d_s}{\partial x} = i_b - i_w$$

With this formula the type of backwatercurve can be determined for a given (constant) Q, Bélanger. The effect of the advective acceleration term is neglected (Fr^2<<1).

F.3 Backwatercurve

For the calculations of the backwatercurve it is assumed that the discharge is constant so it is in fact not a flood wave. As discussed in the previous paragraph the backwatercurve can be calculated with the formula of Bélanger. It is useful to calculate the backwatercurve in order to determine the up- or downstream effect of constructing a hydraulic structure or other interference.

$$\frac{\partial d}{\partial x} = \frac{i_b - i_w}{1 - Fr^2} \ \ if \ \ Fr \neq 1$$

There are two special cases; the numerator or the denominator in the right part of the formula is zero. If the numerator is zero the flow is uniform and the corresponding equilibrium depth can be calculated. If the denominator is zero the flow is critical and the corresponding boundary depth can be calculated.

$$d_e^{\ 3} = \frac{c_f}{i_b} \frac{q^2}{g}$$

$$d_g^{\ 3} = \alpha \frac{q^2}{g}$$

Backwatercurves are classified into types based on the relative bed level gradient (letter) and the type of flow (number). The relative bed level gradient determines if the uniform flow is subcritical or supercritical, based on the Froude-number. In turbulent flows α is circa 1 and in natural waterways the width is much more than the depth, resulting in a Froude-number that is only affected by the relative bed level gradient (i_b) and the friction coefficient (c_f). The distinction is made between different type of slopes; steep (S), critical (C), mild (M), horizontal (H), adverse (A).

3 type of non-unanimous flows are distinguished, indicated with code number 1, 2 or 3, see Figure F-1.

- Type 1: $d>d_e$ and $d>d_g$, so $i_w<i_b$ and $Fr^2<1$
- Type 2: d between d_e and d_g
- Type 3: $d<d_e$ and $d<d_g$, so $i_w>i_b$ and $Fr>1$

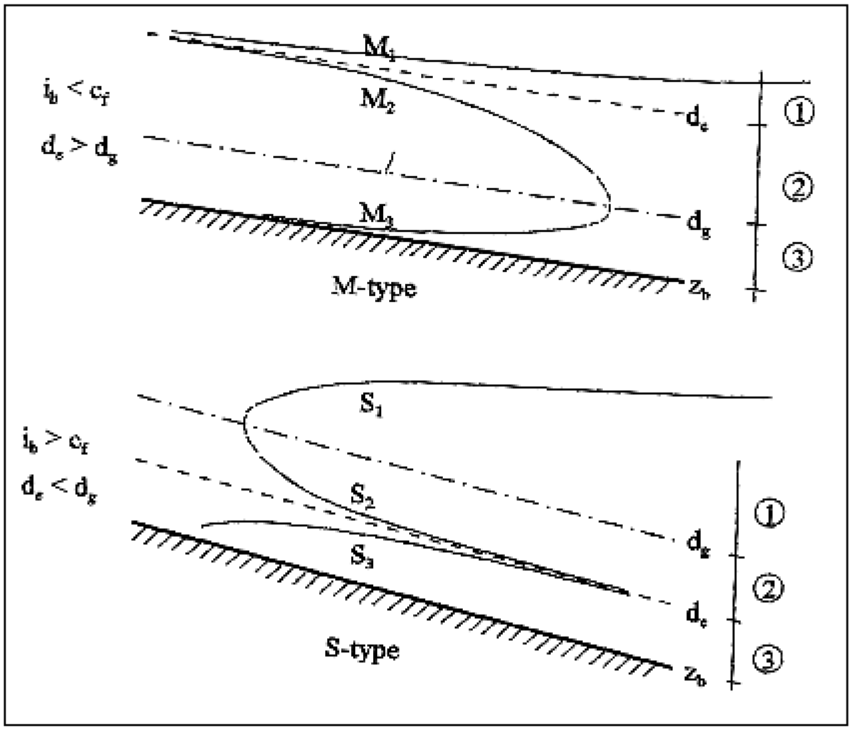


Figure F-1: types of backwatercurves

To calculate the backwatercurve only one boundary condition is needed. Supercritical flow is determined by the upstream boundary condition and subcritical flow is determined by the downstream boundary condition. The condition of the entire section downstream the most downstream located interruption on an M-type slope is uniform (M_e). The condition of the entire section upstream the most upstream located interruption on an S-type slope is uniform (S_e).

The transition of a subcritical to a supercritical flow is only possible if there is a discontinuity in the system, the location is fixed (control section). The transition from a supercritical to a subcritical flow will only take place in a hydraulic jump; the location depends on the boundary conditions.

F.3.1 Simplified model

Also for these calculations the system is simplified, see Figure F-2. The backwatercurve is calculated for the total design river discharge of 30.000m^3/s, it is assumed that the discharge is constant in time.

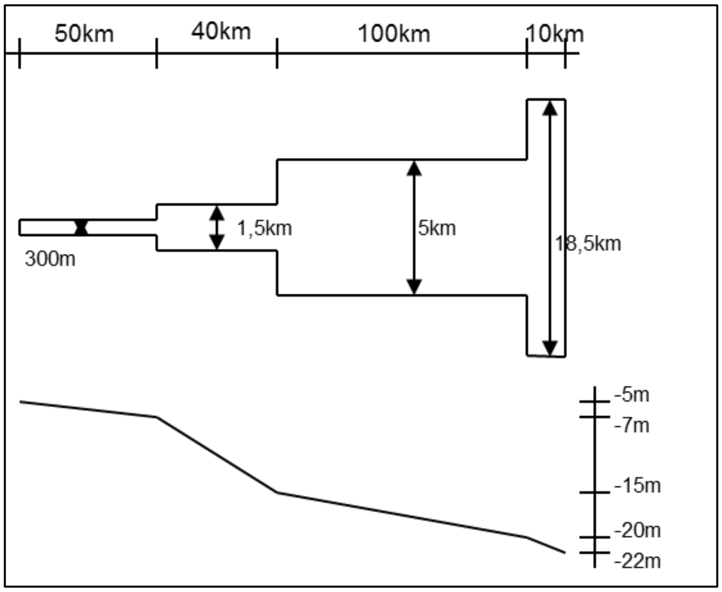


Figure F-2: simplified model

Both the variation in bed level gradient as the change in width affects the backwatercurve. For each section the slope of the bed level is mild and the flow is subcritical. The downstream boundary is assumed as a constant water level at sea of 1,8 m. The backwatercurve is a combination of M1 and M2-type curves. In table F-1 the result of the calculations are shown.

Q	30.000	m3/s		
hsea	1,8	m		
section	1	2	3	4
d_0 (m)	-5	-7	-15	-20
d_1 (m)	-7	-15	-20	-22
L (km)	50	40	100	10
i_b	0,00004	0,0002	0,00005	0,0002
c_f	0,004	0,004	0,004	0,004
B (m)	300	1500	5000	18500
d_e (m)	46,71	9,34	6,65	1,75
d_g (m)	10,06	3,44	1,54	0,64
type	M2	M2	M2	M1

table F-1: backwatercurve calculations for simplified model

The water level will reach the equilibrium depth if the length or the slopes are very long (theoretically semi-∞). table F-1 shows the water levels if indeed the slopes are so long that the equilibrium depth is reached, in reality this will probability not be the case.

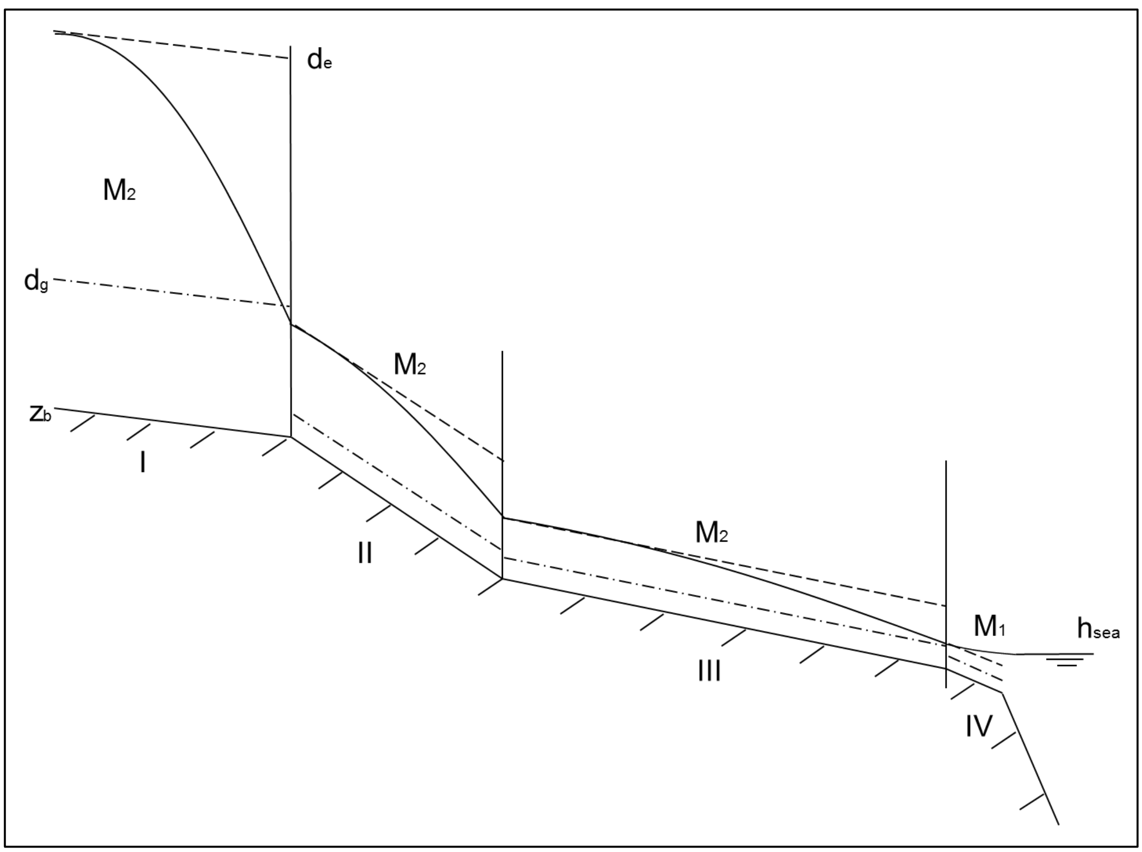


Figure F-3: calculated backwatercurve for simplified model

F.3.2 Sensitivity analysis simplified model

For the backwatercurve calculations the system has been simplified. In reality the transition between the different sections will be smooth, the bed level gradient and friction will vary more and the river discharge will not be constant in time and equal to the design discharge. The sensitivity of the results for the different variables will be determined.

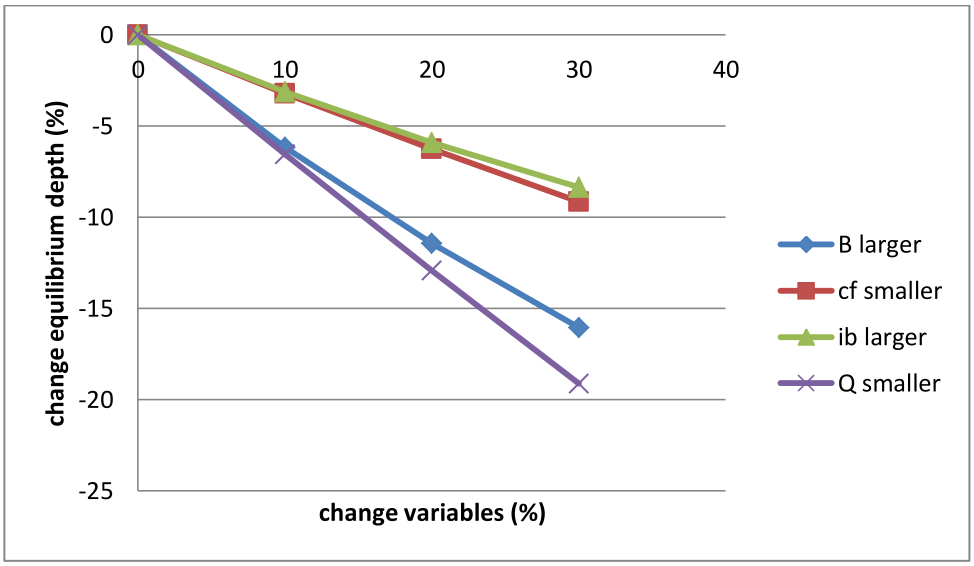


Figure F-4: sensitivity analysis for backwatercurve calculations

Figure F-4 shows the sensitivity of the equilibrium depth for the different variables. The equilibrium depth is very sensible for changes in the river discharge Q and the width of the cross section B and less for changes in friction and bed level gradient. The sensitivity for width is the reason that the water levels in the upstream sections are higher than in de downstream sections. In the calculations it is assumed that the flow width is equal to the storage width, this is not true in reality and the effect will be discussed in the next paragraphs.

F.3.3 1D-river model (Sobek)

As discussed in paragraph F.3.1 the length of the sections will probably not be long enough for the water level to reach the equilibrium depth. To get an idea of the water levels that will be reached in reality Sobek-calculations were made for the same schematisation. The results of those calculations show the same trend, Figure F-5.

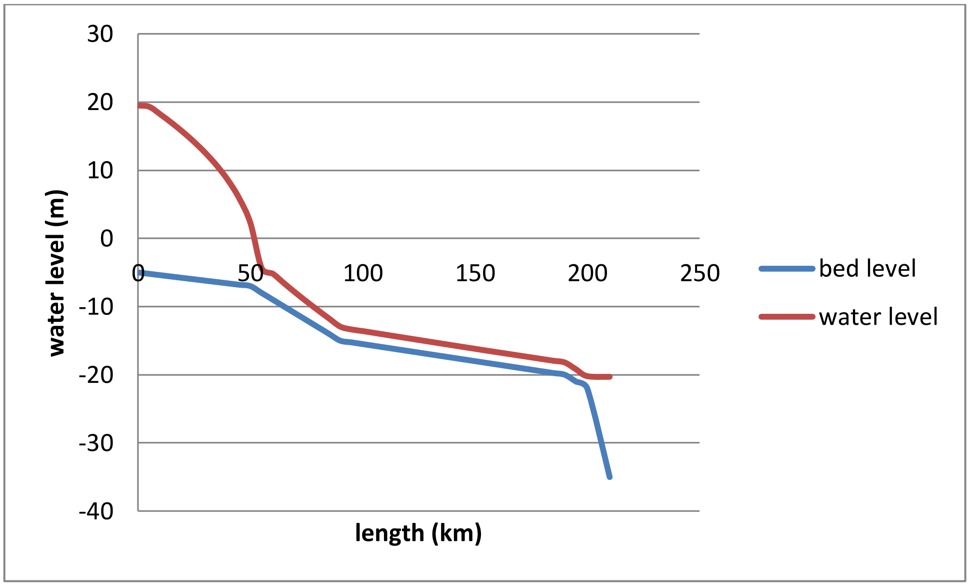


Figure F-5: backwatercurve 1D-river model

The water levels in the most upstream section are very high compared to the other sections and the type of backwatercurve is corresponding to those determined before. The water levels are lower than the equilibrium depth and so the lengths of the sections are indeed not long enough for the water level to reach the equilibrium depth.

F.4 Quasi-uniform approach

In the case of a uniform flow the bed level and the water level have the same gradient. In the case of quasi-uniform flow it is assumed that the effect of the flood wave on the instantaneous water level gradient is small compared to the water level gradient during undisturbed flow. Therefore the water level gradient can be replaced by the bed level gradient in the equation of motion.

$$Q = Q_u = A_s\sqrt{\frac{gRi_b}{c_f}}$$

$$U = U_u = \sqrt{\frac{gRi_b}{c_f}}$$

Due to this assumption the discharge Q depends on the local depth or the dimensions of the cross section. This can be expressed in the continuity equation. The flow velocity of the flood wave can be calculated when assuming that the cross section is constant (also the depth d_s and discharge Q are constant) and B_s and c_f not vary over the depth.

$$c_{HW} = \frac{3}{2}\frac{B}{B_{total}}U_e$$

With the made assumptions it is possible to follow points of the flood wave with constant depth, with a flow velocity which only depends on the depth. c_{HW} is the propagating velocity of the flood wave downstream.

Due to neglecting the inertia term the wave only propagates downstream, this is called a kinematic wave; dynamic doesn't play a role.

As a result of these assumptions the peak of the flood wave will not reduce. The shape of the wave will change because the flow velocity of the wave depends on the water depth d_s. As a result of variation of depth in the leading edge of the high water wave the front of the wave will become steeper in respect to the trailing edge. In time the gradient of the leading edge of the wave will become so large in respect to the bed level gradient that it is no longer valid to neglect the gradient. The model is no longer consistent; this will be discussed in the next paragraph F.5.

F.5 Diffusion model

Due to variation in depth the front of the flood wave will become steeper in respect to the trailing edge. Therefore the flow is higher during rising water than during sinking water with the same water level, this is called hysteresis. As a result the maximum discharge in a high water wave occurs earlier than the maximum water level and the flood wave will be smoothened. This process is comparable with diffusion transport and will therefore be expressed with a diffusion coefficient (K) in the continuity equation.

$$\frac{\partial d_{s0}}{\partial t} + c_{HW} \frac{\partial d_s}{\partial x} - K \frac{\partial^2 d_s}{\partial x^2} = 0$$

The continuity equation has the form of an advection-diffusion equation. To solve this equation one initial condition and two boundary conditions are needed. In reality c_{HW} and K vary with d_s non-linear but for simplification reasons it is assumed that they are constant, the values related to the equilibrium condition of which the high water wave is an interference will be used.

F.5.1 Simplified situation

Hand calculations for the simplified situation as presented in Figure F-6 were made.

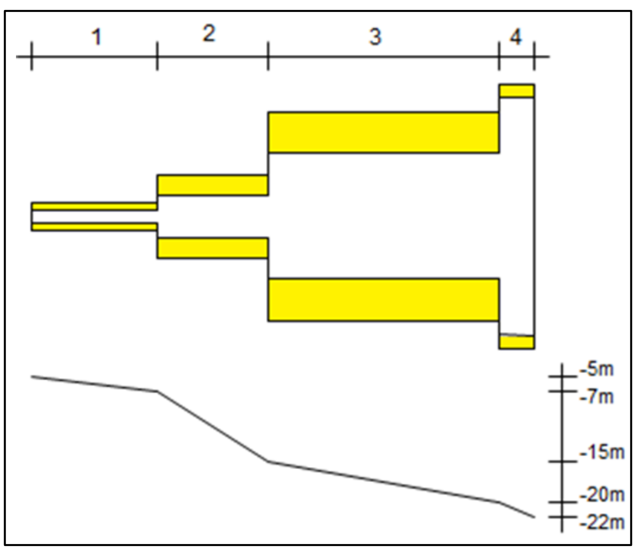


Figure F-6: simplified situation

Initially (t=0) the flow is uniform with a constant depth (d_s); over a small, negligible length (x=0) instantaneously a volume (V) of water is injected. This extra volume of water creates on t=0 a concentrated interference in x=0 on the then uniform flow which will run off after t=0. It is assumed that for all t on a sufficient distance of x=0 the initial condition prevails. With the given initial and boundary conditions the water level during a high water wave can be calculated.

$$d_s(x,t) = d_0 + \frac{V / B_{total}}{\sqrt{2\pi}\sigma_x(t)} \exp\left(-\frac{(x - c_{HW}t)^2}{2\sigma_x^2(t)}\right)$$

$$with\ \sigma_x(t) = \sqrt{2Kt}\ \ and\ \ K = \frac{U_0 d_0 B}{2 i_b B_{total}}$$

The difference between the flow and storage width is affects both the flood wave velocity, c_{HW} as the diffusion coefficient, K. The values are assumed to be constant for each section, in reality they vary with the depth, d_s. The peak of the flood wave propagates with velocity c_{HW}, which is of the same order as the flow velocity because the inertia term is neglected. The velocity of dynamic wave is much more than the flow velocity.

V(m3)	108.000.000			
section	1	2	3	4
d0 (m)	-5	-7	-15	-20
d1 (m)	-7	-15	-20	-22
L (km)	50	40	100	10
cf (-)	0,004	0,004	0,004	0,004
B(m)	300	1500	5000	18500
Btotal (m)	1000	2500	15000	20000
ib (m/m)	0,00004	0,0002	0,00005	0,0002
d0 (m)	1,38	0,87	0,70	0,70
u(0) (m/s)	2,42	0,77	0,29	0,08
K	12500	1000	666,6667	125
cHW (m/s)	1,09	0,69	0,14	0,11

table F-2: diffusion calculations for simplified model

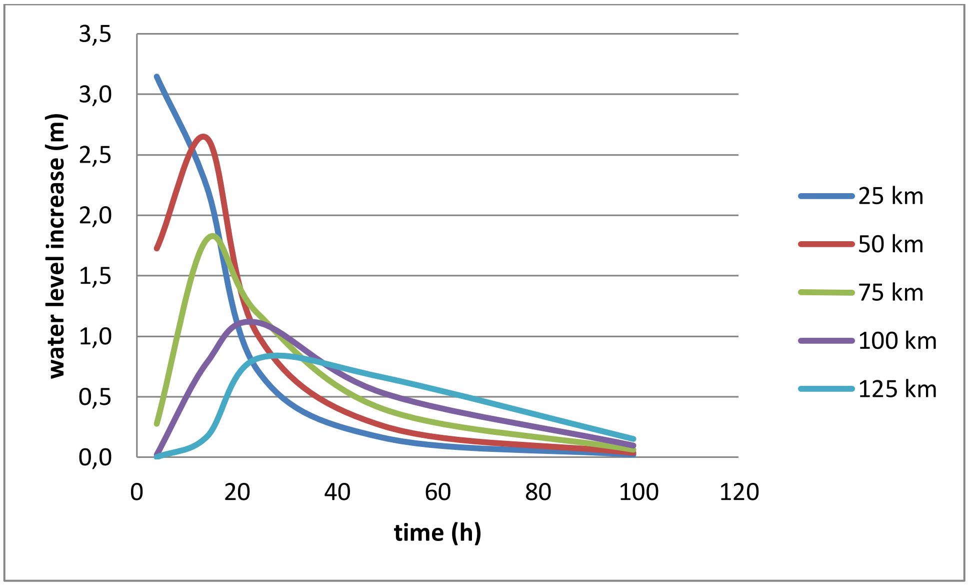


Figure F-7: diffusion results for simplified model

Figure F-7 shows the calculated maximum water levels due to the instantaneous water volume injection of $10,8*10^9 m^3$. The maximum water levels in the sections are different due to the difference in dimensions (B, B_s, i_b) and therefore in values of K and c_{HW}. The length profile of the high water wave has on each moment in time the shape of a Gauss curve. The smoothening of the high water wave is equal to circa √t; if the cross-section is constant rising is faster than dropping.

Figure F-8 shows the discharge in time at different locations. The shape of the graph can be explained by the fact that the flood wave flattens in time (σ increases in time), as a result the rear is flatter than the leading edge. Rising of the water takes more time than lowering of the water.

F.5.2 Sensitivity analysis simplified model

Figure F-8 shows the results of the sensitivity analysis. The advections-diffusion calculations are very sensitive for the chosen initial conditions (u_o and d_0). Therefore it is very important to make a good choice. The calculations are also very sensitive for changes in the bed level gradient (i_b) and changes in flow and total width. The sensitivity for changes to width explains the decrease of maximum water levels in downstream directions.

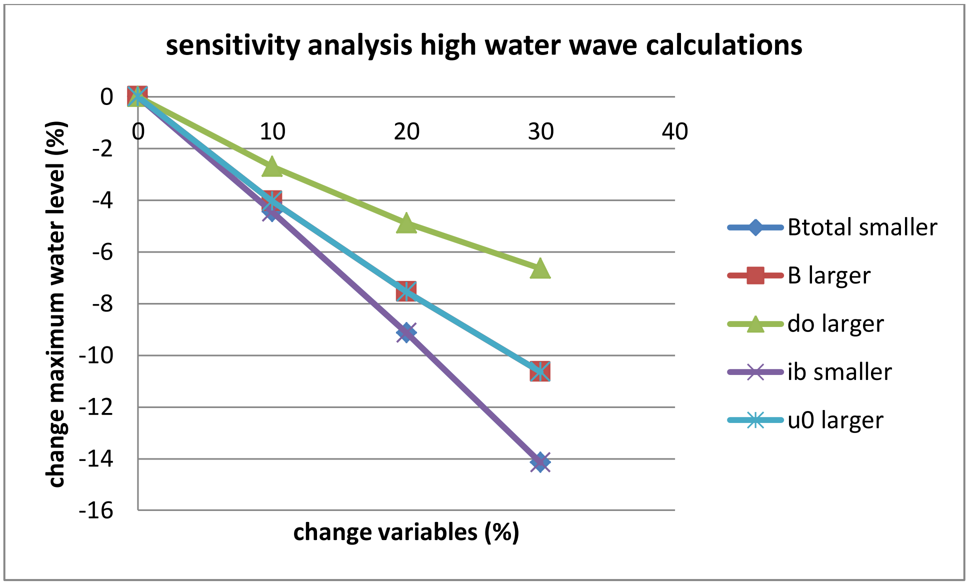


Figure F-8: sensitivity analysis diffusion calculations for simplified model

In reality the situation is much more complex than assumed in the simplified model. Not only are the initial conditions more complex but also the fact that the flow and the total width can vary with the water level (flooding). In the simplified model it is assumed that the flow and total width are constant and do not vary with the water level.

F.5.3 1D-river model

The shape and duration of the flood wave found in the hand calculations and in the Sobek calculations differ a lot, Figure F-7 and Figure F-9. In the hand calculations the assumption was made that the water level gradient changes so slow that the water mass is able to follow the change and the velocity has adapted rather well to the instantaneously gradient at any moment. As a result the inertia term in the equation of motion can be neglected. In the Sobek-model the inertia term will be taken into account. In reality the flood wave is not a spatial, slowly varying, steady phenomenon and the velocity of the flood wave is not in the same order as the flow velocity but much more ($c_{HW} >> u_e$). As a result the flood wave diffuses faster. Figure F-9 also shows that the effect of the flood wave is barely knowable over 50 km downstream, this can be explained by that the river widens from this point.

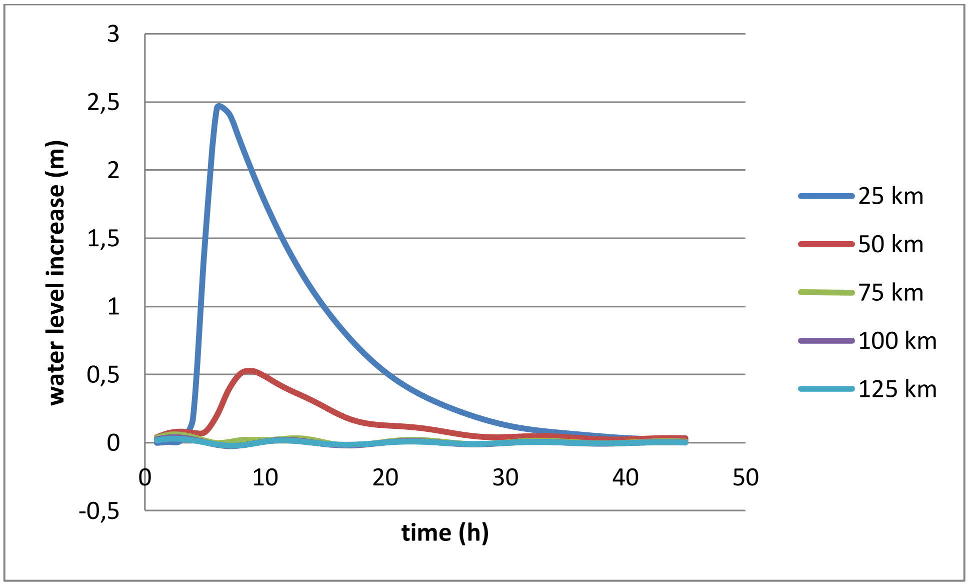


Figure F-9: water levels at different location as a result of a flood wave V=108.000.000m3

Appendix G: Salt intrusion

G.1 Introduction

In an estuary sea water (relative density around 1,028 g/ml) and fresh river water (relative density 1,0 g/ml) meet. There are two main drivers for mixing in estuaries: the density difference and the tide. If there is no tide and the cross-sections of the estuary are regular a salt wedge/tongue is formed. In that case the fresh river water flows out on top of the seawater. Salt wedges are no so common. The phenomenon only occurs in narrow estuaries with a high river discharge and a small tidal range. In most estuaries there is incomplete or (almost) complete mixing of salt and fresh water, the density is constant over the vertical (Savenije, H.H.G., 2006).

G.2 Dominant mixing processes

The two main drivers for mixing in estuaries are: the density difference and the tide. These two mechanisms generate four main mixing mechanisms; turbulent/shear mixing, gravitational mixing, ‘trapping’ and ‘tidal pumping’. Turbulent mixing is the result of the effect of friction on water particles; in general this process is not so important in estuaries. Due to the density difference the hydraulic pressure on the sea-side and on the river-side is not in equilibrium. This causes a residual circulation that carries relatively saline water upstream along the bottom and relatively fresh water downstream along the surface. This phenomenon is called gravitational mixing and is important in parts of the estuary where the salinity gradient is large. The third mechanism is trapping; this is the result of irregularity of the banks of an estuary. If there are for example tidal flats there will be a phase lag between filling an emptying the flats and the flow in the main channel, resulting in density difference. Also this mechanism is more important with a larger salinity gradient. The last mixing phenomenon is tidal pumping. This process is very important near the mouth of a wide estuary but it is the least studied process. It does however not depend on the salinity gradient but it is proportional to the width of the estuary.

G.3 Estuarine Richardson number

The Estuarine Richardson number is a measure for the relative importance of gravitational circulation compared to tidal mixing. It gives the balance between the potential energy deficit and the tidal kinetic energy. The ultimate form of gravitational circulation is the saline wedge, which corresponds with a high Estuarine Richardson number.

$$N_R = \frac{E_m}{E_t} = \frac{\Delta\rho}{\rho}\frac{ghQ_f T}{P_t v_0^2}$$

Observations of real estuaries suggest that, the transition from a well-mixed to a strongly stratified estuary occurs in the range 0,.08<N_R<0,8. In the calculations a distinction is made between the dry and rainy season and the effect on the average river discharge and the Froude number is assumed to be 1.

	dry season	rainy season	
Q	550	3600	m^3/s
T	44700	44700	s
H	2,6	2,6	m
$A_{surface}$	100	100	km^2
V_{fl}	3E+08	2,6E+08	m^3
N_R	0,0876	0,5730	-

table G-1: estuary number

During the dry season as well as during the rainy season the estuary is well mixed, see Figure G-1.

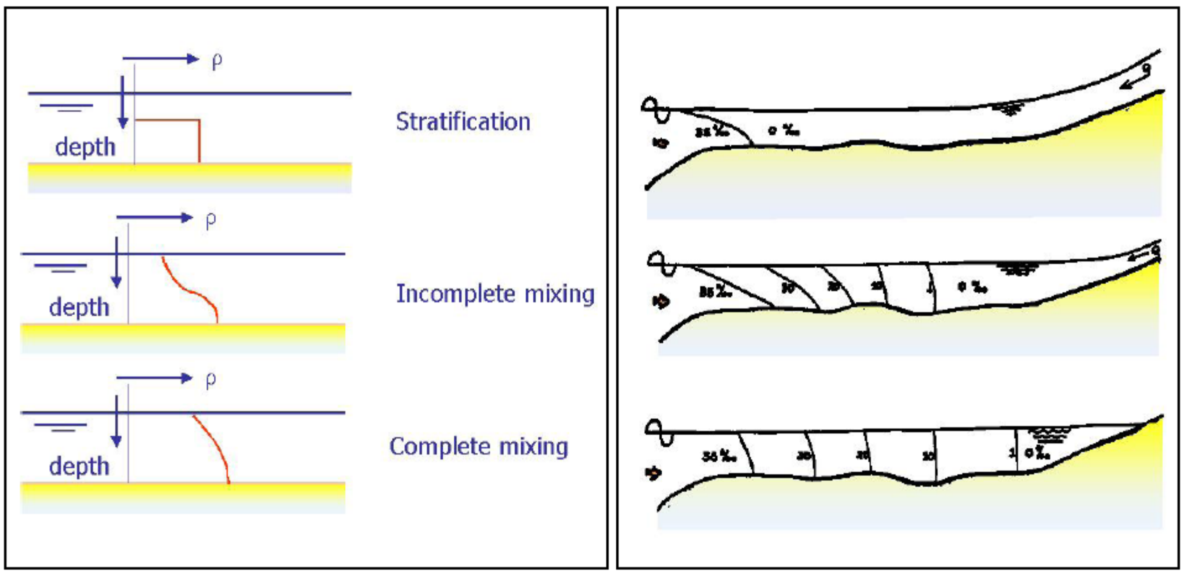


Figure G-1: degree of mixing

G.4 Well-mixed estuary

Due to the mixing process salinity penetrates into well-mixed estuaries while at the same time river discharge flushes it back towards the sea. The eventual salinity in the estuary depends on the dominant mechanism. In the case mixing is dominant the salinity will increase over time, while the estuary becomes fresher when river flow is dominant. If the two mechanisms tie, a steady-state situation occurs where the salinity remains constant over time. Figure G-1 shows the longitudinal distribution of the salinity over a well-mixed estuary. The longitudinal distribution diminishes from sea salinity at the mouth to fresh water at the toe of the salt intrusion curve.

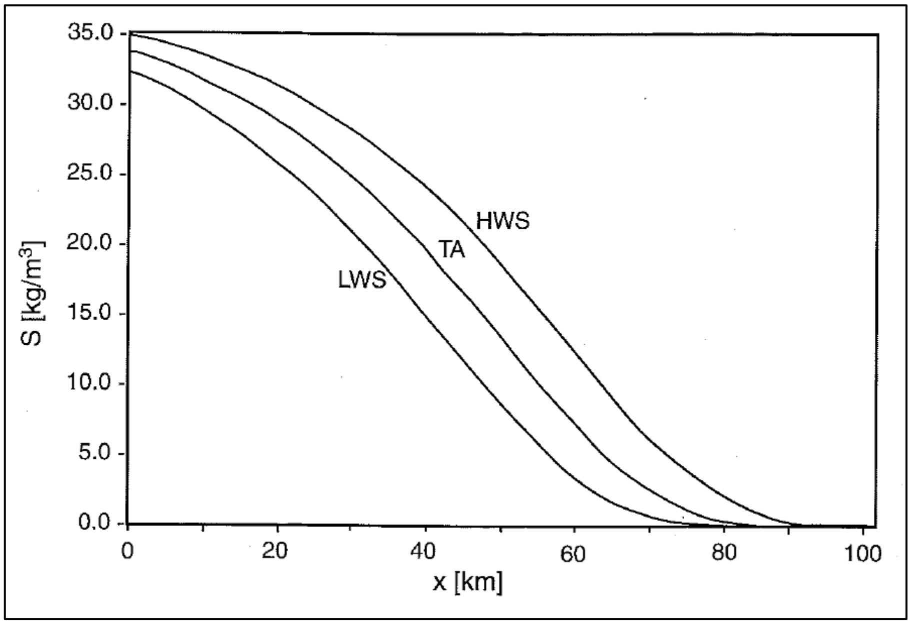


Figure G-2: envelope curves of salinity intrusion at High Water Slack (HWS), Low Water Slack (LWS) and mean tide (TA)

Figure G-2 shows the longitudinal salinity distribution in a well-mixed estuary at high water slack (HWS), low water slack (LWS) and tidal average (TA) condition. There are different intrusion lines; HWS for the maximum salt intrusion, LWS for the minimum salt intrusion and TA for the tidal average situation. The horizontal distance between the HWS and LWS curve is the tidal excursion. As the Figure indicates, salinity moves up and down the estuary following the water particles that travel between HWS and LWS. The shape of the salt intrusion curve depends on the geometry of the estuary. In the case of HCMC the estuary is funnel-shaped but there is a stronger widening near the estuary mouth therefore the shape will be between type 2 and type 3, Figure G-3.

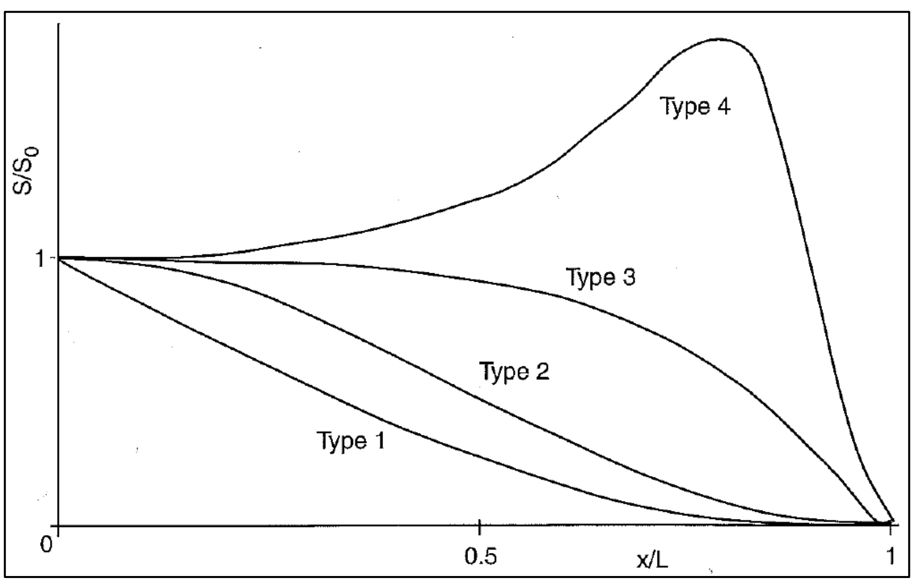


Figure G-3: different shapes of well-mixed salt intrusion curves

G.4.1 Steady state model

For alluvial estuaries the salt intrusion can be described using a steady state model. For steady state there is an equilibrium in any cross-section between the advective transport of salt by the river in downstream direction and the dispersive transport of salt in upstream direction under the effect of mixing which is proportional to the salinity gradient dS/dx.

$$(S - S_f)Q_f = -AD\frac{dS}{dx}$$

Assuming that Q/DA is constant and S_0 is the sea salinity at the estuary mouth (x=L).

$$S - S_f = (S_0 - S_f)\exp(-\frac{Q}{DA}x)$$

In reality the cross-sectional area A is not constant with x; the value of D is not easy to predict because it differs from one estuary to another and varies with x and the salinity for x=0 is not constant but depends on Q. Also there is a difference between the actual records and the calculated results.

Generally the geometric variation of the cross-sectional area can be solved by fitting an exponential function. Savenije (1996) showed that good results can be obtained when considering an 'ideal estuary' with a horizontal bed, an exponentially varying width and hence an exponentially varying cross section:

$$A(x) = A_0 \exp(-\frac{x}{a})$$

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

For the longitudinal variation of the dispersion coefficient D, different researchers have followed different approaches. Savenije (1992) came up with an equation that works well under a wide range of estuaries, of different depths, different convergence lengths and different widths.

$$\frac{D}{D_0} = (\frac{S}{S_0})^K$$

Where K is Van Der Burght's coefficient. Combining the above equations gives:

$$\frac{S - S_f}{S_0 - S_f} = (\frac{D}{D_0})^{\frac{1}{K}}$$

$$\frac{D}{D_0} = 1 - \beta(\exp(\frac{x}{a}) - 1)$$

Substitution that x=L where S=S_f gives:

$$\beta = \frac{Ka}{\alpha_0 A_0}$$

$$L = a * \ln(\frac{1}{\beta} + 1)$$

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.

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

$$K = 6,3*10^{-6}(\frac{h_0}{a})^{1,04}(\frac{E}{H})^{2,36}$$

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

Combination of these equations gives an expression for the intrusion length L at HWS. This relation is a combination of dimensionless ratios; including the densimetric Froude number, the Canter-Cremers number, the Estuarine Richardson number.

$$L_{HWS} = a*\ln(1+\frac{220}{K}\frac{h}{a}\frac{E}{a}\frac{v}{v_f}\sqrt{40N_R})$$

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

G.4.2 Simplified model

In reality a steady state model cannot be used for the Dong Nai – Saigon estuary because the fresh river discharges is not constant in time and depends on the season. Therefore a distinction between the dry and rainy season is made in the calculations with corresponding average river discharges.

The Dong Nai – Saigon estuary is not a perfect alluvial shaped estuary because of the presence of the Can Gio mangrove forest. For the calculations it is assumed that the estuary is an 'ideal estuary' with a horizontal bed (h_0 is constant), an exponentially varying width and hence an exponentially varying cross section. A convergence length can be obtained from fitting measured cross-sections to a line on semi-logarithmic paper, see Figure G-4.

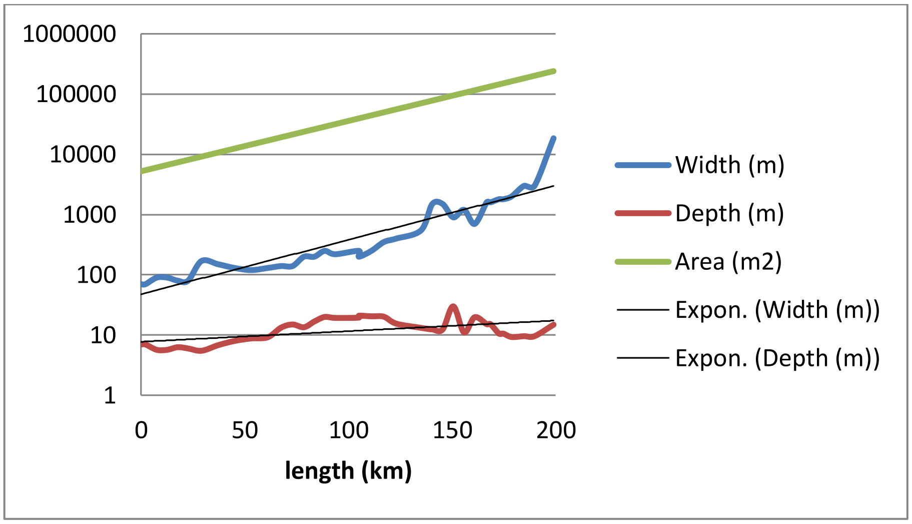


Figure G-4: dimensions 'ideal estuary'

table G-2 shows the result of the calculations for the salt intrusion length. During the dry season, when the river discharge is low the salt intrusion length is in the order of 72 km while during the rainy season the length is only 40 km. So during the dry season the problems regarding to salt intrusion are larger than during the rainy season, this corresponds with the experience of the farmers in the Dong Thuap Muoi region.

	dry season	Rainy season	
a	52000	52000	m
h_0	13	13	m
E	937	937	m
$A_{surface}$	100	100	km^2
H	2,6	2,6	m
h	14	14	m
v	1	1	m
Q	550	3600	m^3/s
A_{river}	490	490	m^2
A_0	277500	27700	m^2
v_f	1,12	7,34	m/s
T	44700	44700	s
V_{fl}	2,6E+08	2,6E+08	m^3
N	0,094	0,619	-
ρ	1	1	mg/l
Δρ	0,028	0,028	mg/l
Fd	0,26	0,26	-
K	0,001222	0,001222	-
N_R	0,36	2,38	-
L_{HWS}	71660	4040	m

$$F_d = \frac{\rho}{\Delta\rho}\frac{v^2}{gh}$$

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

$$N_R = \frac{N}{F_d}$$

table G-2: calculation salt intrusion length

G.5 Water control on coastal areas

The saline intrusion is maximal during dry periods with minimum river discharge; this is the period with a maximum need for fresh water for irrigation. The tolerance limits for irrigation depends on soil, crop and water and soil management. Commonly accepted limits for the use of irrigation water for rice varieties are 350 mg Cl^- per litre in the early stages of growth and 1500 in the later stages when the plants have become more resistant.

In low-lying coastal areas large-scale storage of fresh water can be created by damming off estuaries, coastal lagoons and tidal embayment's with an inflow of fresh water. A large 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. In the next chapter the effect of the construction of a hydraulic structure downstream of HCMC will be described.

The feasibility of such reservoirs depends on:

- length of the period of desalinization
- ultimate salinity of the water in the reservoir after the desalinization
- water balance of the reservoir in connection with the regulation of the normal operational level

G.6 Ho Chi Minh City

Salt intrusion is a problem for the rice production in the Dong Thap Muoi area. During the dry season the salt concentration in the Vam Co Tay and Vam Co Dong is higher than during the rainy season as a result of minimum river discharge. Commonly accepted limits for the use of irrigation water for rice varieties are 350 mg Cl^- per litre in the early stages of growth and 1500 in the later stages when the plants have become more resistant. Figure G-5 shows some salinity measurements in the Vam Co Dong and Vam Co Tay during February till June 2005. It is clear that there are problems with salt intrusion.

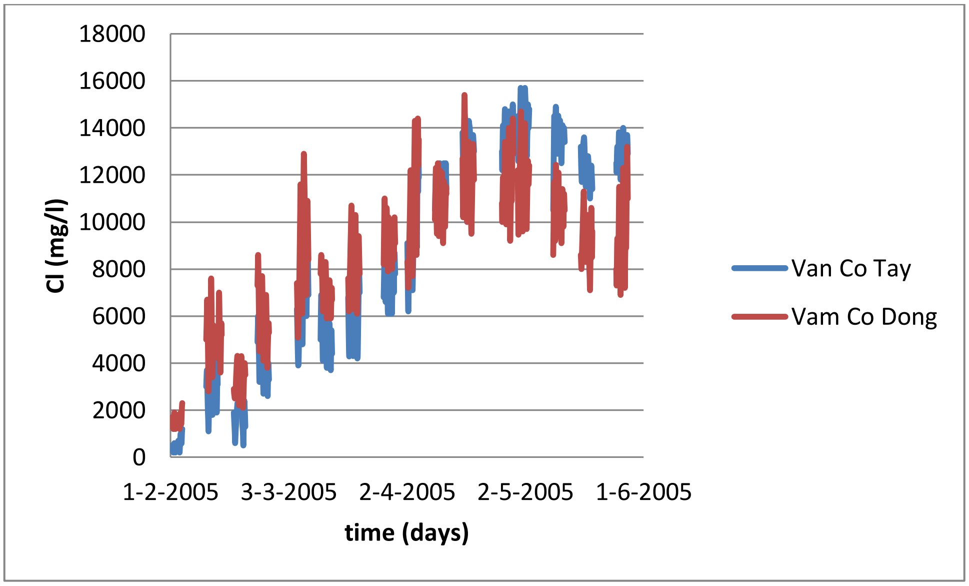


Figure G-5: salinity measurements Vam Co Dong and Vam Co Tay

Appendix H: Functioning of a seaward barrier

H.1 Introduction

A possible solution to reduce the water levels in the rivers surrounding and crossing HCMC is by controlling the in- and outflow in the estuary mouth to a feasible extent, which can be done in several ways. In this chapter the functioning of a barrier with two highly different operational regimes will be described:

- a barrier which is always open, both during normal and during extreme conditions ⇨ reductor.
- a barrier which is open when the water level at the land side of the barrier exceeds the water level at the sea side of the barrier and in all other conditions is closed ⇨ discharge sluice.

H.2 Storage basin approach

To determine the effect on the water level of a barrier which controls the downstream in- and outflow, the system is simplified and the storage basin approach is used, see Figure H-1.

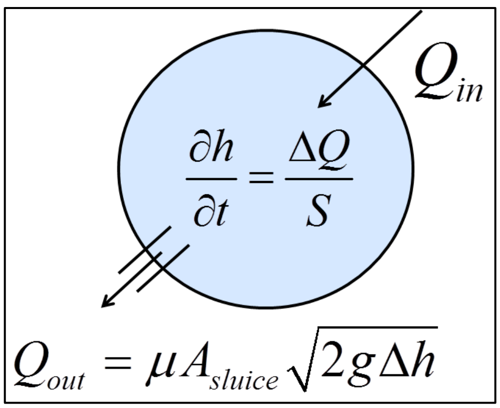


Figure H-1: storage basin approach

The storage basin approach is based on two formulas which form a coupled system; change of water level in time and outflow (or inflow). With this theory the effect of a reductor and a discharge sluice can be determined.

Water level (h) and velocity (v) depend on:

- Tidal movement (Δh)
- Cross section opening (A_{sluice})
- River discharge (Q_{in})
- Storage area (S)

H.3 Reductor

A reductor is always open both during normal conditions as well as during extreme conditions. So there is in- and outflow through the hydraulic structure. However, the amount of water which flows in and out is affected by the construction of the structure because the cross section is reduced compared to the original situation. This will affect the upstream water levels and salt intrusion.

H.3.1 Water level

For the following calculations it is assumed that the system is a basin, so length effects are neglected, Figure H-1. The barrier is always open and there is in- and outflow though the structure, consequently the water level in the lake is affected by the tidal movement at sea.

Due to the construction of the barrier the downstream cross section is reduced when compared to the original situation. This results in less in- and outflow into the system from sea. As a result the water level in the basin is less affected by the tidal movement and therefore the amplitude of the water level variation has been reduced, see Figure H-2.

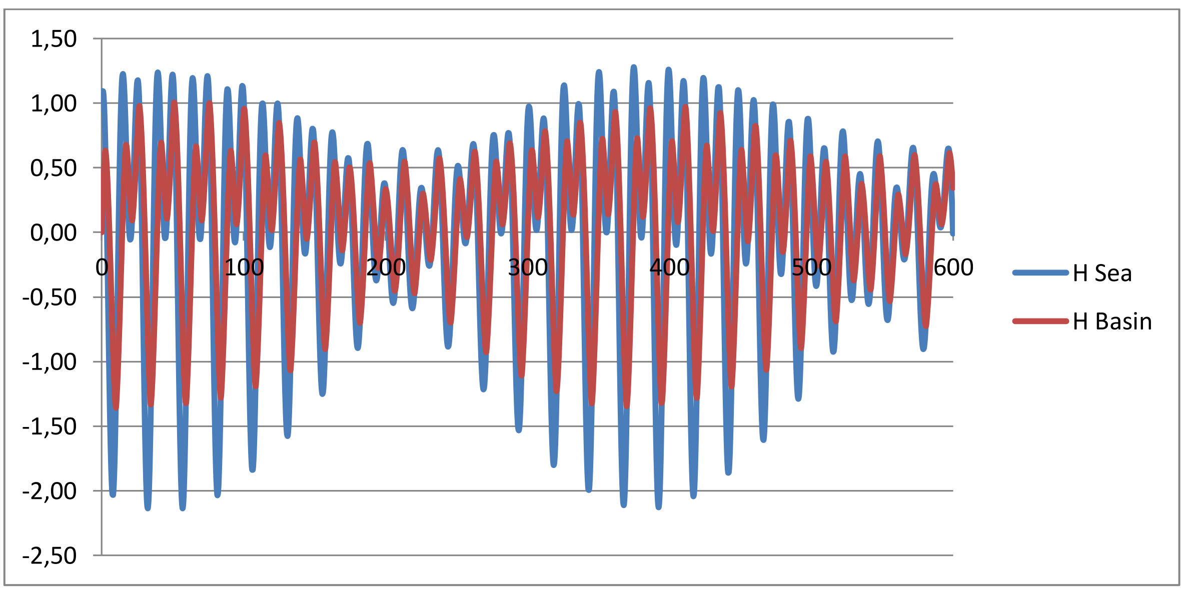


Figure H-2: effect storm surge barrier on water level; Q=1.000m3/s, S=500km2, Ac=10.000m2

For a given water level fluctuation at sea the water level in the basin depends on three variables:

- Cross section opening (A_c)
- River discharge (Q)
- Storage area (S)

The river discharge is assumed to be constant; this corresponds with the worst case scenario, see Appendix I. See Figure H-3 and Figure H-4 for the results.

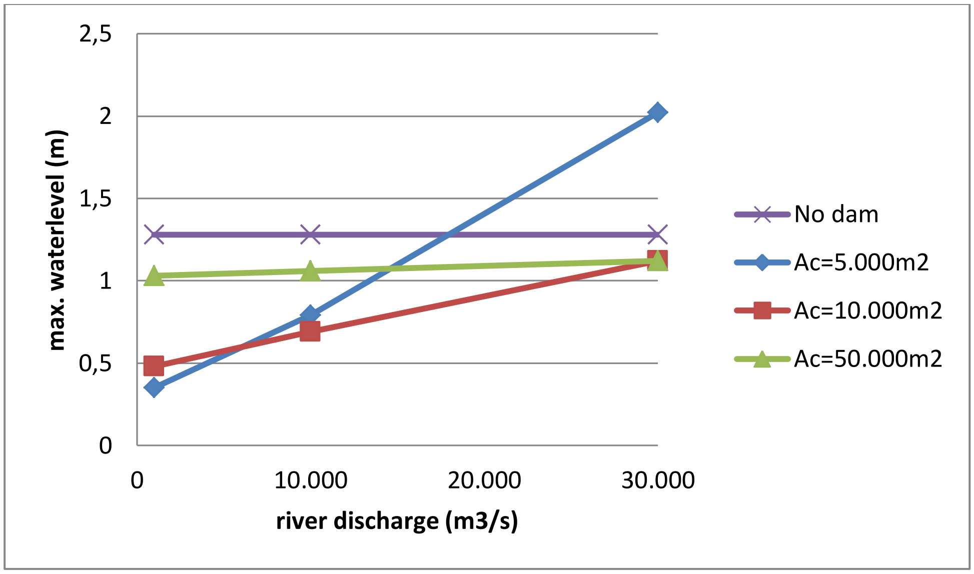


Figure H-3: max. water level in basin as a function of river discharge Q for various opening sizes Ac; S=2.000km2 for a reductor

H.3.1.1 Cross section opening

In these calculations the reductor is always open. Due to the construction of the barrier the cross section has been decreased and as a result the water level in the basin is less affected by the tidal movement at sea. Figure H-3 shows that a small opening size gives more water level reduction during low river discharges than a large opening size, showing the water level in the basin is less affected by the tidal movement at sea for smaller opening sizes. In case of a high river discharge a small opening size will lead to an increase in the basin water level because the outflow is limited and the water is piling up in the basin. If the opening is large enough the water can flow out easily resulting in a water level reduction.

Conclusion; the cross section of the structure should be designed in close coherence with the expected river discharge.

H.3.1.2 River discharge

Figure H-3 shows clearly that larger river discharges lead to less water level reduction, because the river discharge cannot flow out to sea easily enough. So decreasing the design flood wave will have a positive effect on the feasibility of a reductor.

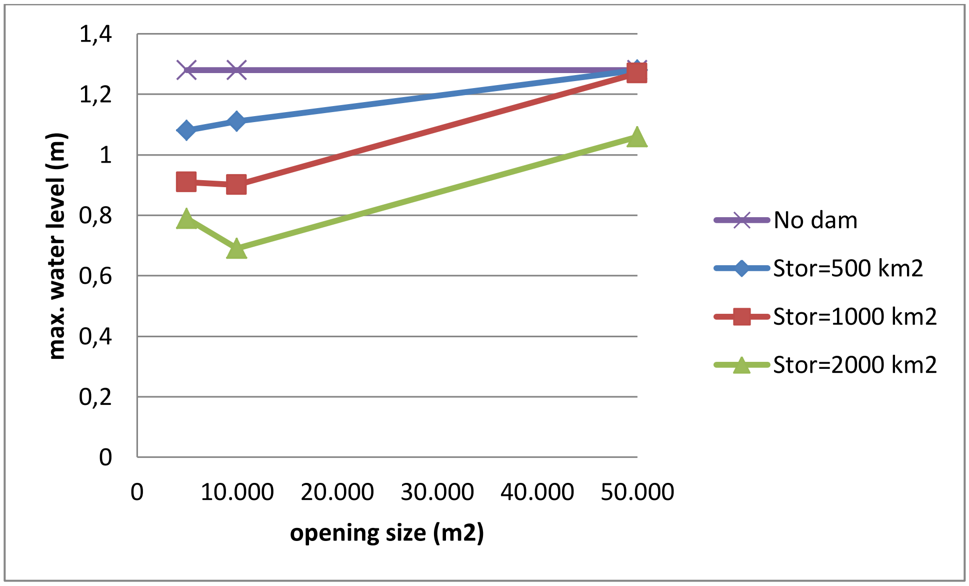


Figure H-4: max. water level in basin as a function of opening size Ac for various storage S, Q=10.000m3/s for a reductor

H.3.1.3 Storage area

Figure H-4 shows the maximum water level in the lake as a function of openings sizes for various storage areas and a constant design river discharge, Q=10.000 m^3/s. It is clear that more storage area leads to more water level reduction in the lake.

	storm surge barrier
	water level reduction
Ac increases	0
Q_{in} decreases	+
S increases	+

table H-1: effect of the variables on the effectiveness of a reductor

H.3.1.4 Sensitivity analysis

The effectiveness of a reductor depends on the three variables (cross section opening, river discharge and storage area) and the combination between these variables. For example, a small opening size results in water level reduction during low river discharge while it results in a water level increase during high river discharges. With a simple approach, Appendix J, the required cross section of the opening and the storage area can be determined for different constant river discharges, see table H-2. It is assumed that the maximum allowable water level is MSL+1,0m. It is clear that the required cross section and storage area are largely affected by the river discharge.

Q (m^3/s)	h_{max} (m)	h_{min} (m)	LW (m)	HW (m)	A (m^2)	S (km^2)
30.000	1	0,5	0	1	15.870	1.185
20.000	1	0,5	0	1	10.580	790
10.000	1	0,5	0	1	5.290	395

table H-2: results simple approach reductor

H.3.2 Flow velocity

In the case of a tidal barrier there is both in- and outflow though the structure. In these calculations it is assumed that flow from the sea into the basin is positive (inflow) and flow towards the sea is negative (outflow).

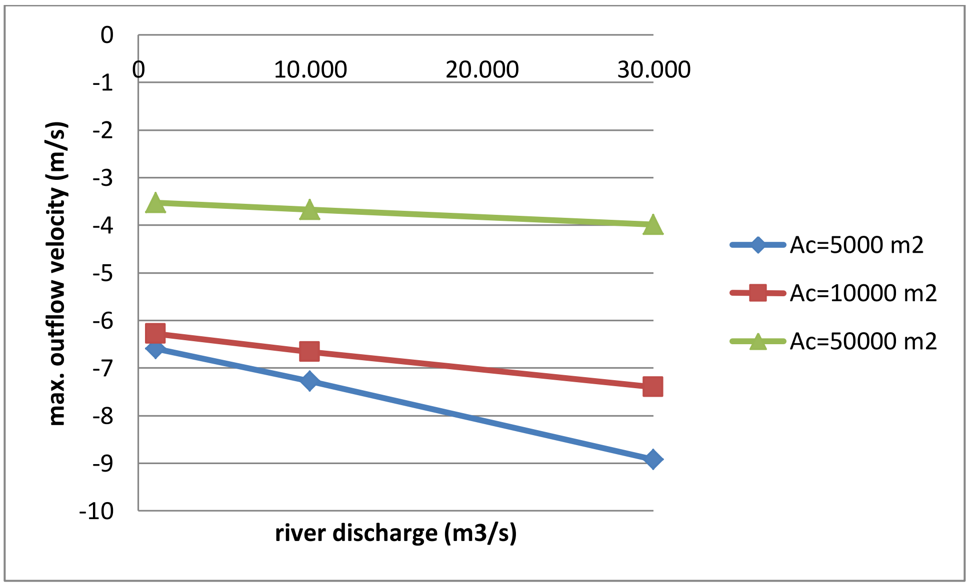


Figure H-5: max. outflow velocity through the barrier as a function of river discharge Q for various opening sizes Ac, S=1.000km2 for a reductor

Figure H-4 shows that the maximum outflow velocity increases when the opening size is reduced and the river discharge increases. Figure H-6 shows that a larger storage area results in higher maximum outflow velocities. Due to the construction of the barrier the water level in the basin is less affected by the tidal movement at sea. This difference between the water level in the basin and at sea increases when the storage area becomes larger resulting in higher maximum outflow velocities.

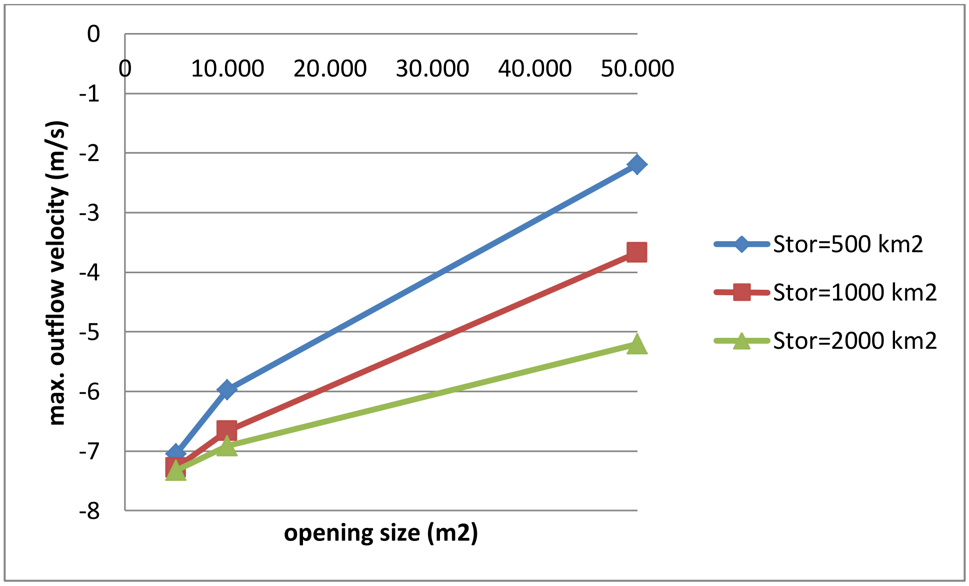


Figure H-6: max. outflow velocity through barrier as a function of opening size Ac for storage areas S, Q=10.000m3/s for a reductor

	storm surge barrier
	max. outflow velocity
Ac increases	lower
Q_{in} decreases	lower
S increases	higher

table H-3: effect variables on max. outflow velocity through the structure

H.3.3 Salt intrusion

In this prefeasibility study the focus is not on the salt intrusion problem and the effect of a structure will only be described qualitative and not quantitative. In the case of reductor there is in- and outflow through the structure, so the water upstream of the barrier will stay salt. The salt intrusion is affected by the construction of a barrier but future study is needed to determine how much the salt intrusion is affected.

H.4 Discharge sluice

A discharge sluice is a hydraulic structure which is only open, in this case, when the water level of the basin exceeds the water level at sea (positive head difference over the structure). Also a certain value for the water level in- or outside can be reason too close or open a discharge sluice. In this case the discharge sluice is open during a positive head difference over the structure. This means that there is no inflow (water from the sea into the basin) but only outflow; this will eventually result in a fresh water basin.

H.4.1 Water level

The water level fluctuation of the basin looks somewhat like a tidal fluctuation. If the water level of the basin exceeds the water level at sea, the discharge sluice is open and water from the basin is discharged into the sea, resulting in lower water levels in the basin. At some point in time the water level of the basin doesn't exceed the water level at sea (tidal movement) and the discharge sluice is closed; then no water is discharged from the basin into the sea. At the same time a river discharges into the basin resulting in an increase of the water level of the basin. At some point in time the water level of the basin exceeds the water level outside and the discharge sluice is opened, etc.

Just like the tidal barrier the functioning of the discharge sluice will be explained with the help of a simplified model. See Figure H-7 for the effect of a discharge sluice on the water levels in the basin.

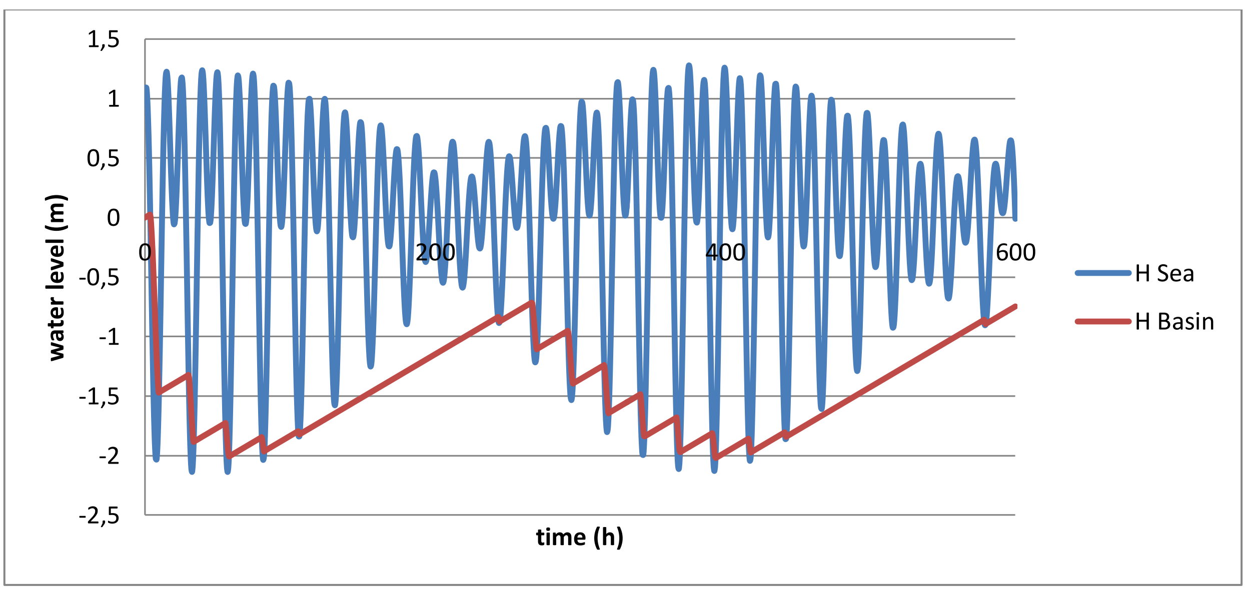


Figure H-7: effect discharge sluice on water level; Q=10.000m3/s, S=500km2, Ac=10.000m2

The water level reduction during spring tide is more than during neap tide. This can be explained by the fact that the tidal range gets smaller going from spring to neap tide. As a result the period in which the water level of the basin exceeds the water level at sea is shorter. Therefore less water is discharged into the sea. The minimum water level in the basin is higher during neap tide than during spring tide, consequently even a small river discharge can lead to an increase of the water levels. These water levels can be higher than the maximum water levels that occur during neap tide in the original situation.

Also for these calculations it is assumed that the river discharge is constant, this corresponds with the worst case scenario, see Appendix I.

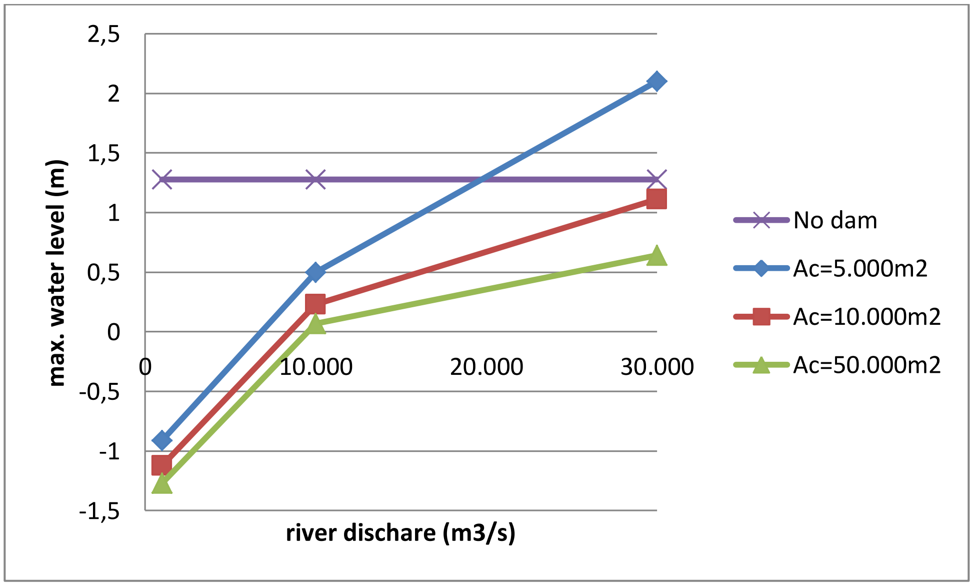


Figure H-8: max. water level in basin as a function of river discharge Q for various opening sizes Ac, S=1.000km2

H.4.1.1 Cross section opening

Figure H-8 shows the maximum water levels in the basin as a function of river discharge for various opening sizes. A large opening results in more water level reduction than a small opening. During high river discharge a small opening can lead to the piling up of the water and consequently a water level increase.

H.4.1.2 River discharge

A discharge sluice leads to water level reduction when the amount of water which is discharged into the sea, when the sluices are open, equals or exceeds the stored inflow and the inflow during the period the sluices are open. The discharge out has to be roughly 4 times the inflow, see Appendix K. Reduction of the design flood wave will increase the feasibility of the discharge sluice.

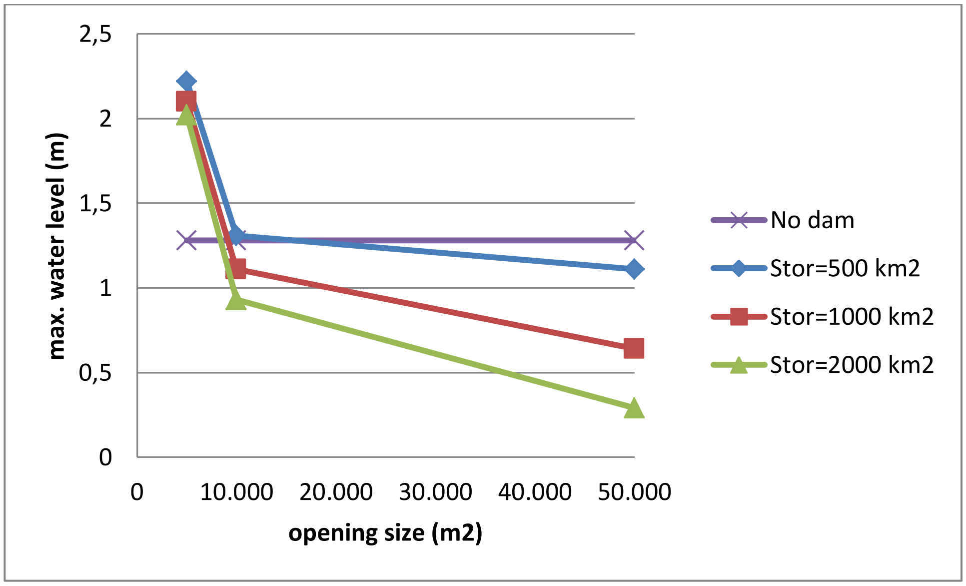


Figure H-9: max. water level in basin as a function of opening size Ac for various storages S, Q=30.000m3/s

H.4.1.3 Storage area

A large storage area has a positive effect on the feasibility of the discharge sluice.

	discharge sluice
	water level reduction
Ac increases	+
Q_{in} decreases	+
S increases	+

table H-4: effect variables on the effectiveness of a discharge sluice

H.4.1.4 Sensitivity Analysis

The effectiveness of a discharge sluice depends on the three variables (cross section opening, river discharge and storage area) and the combination between these variables. table H-5 shows the required storage area and cross section for the opening for various constant river discharges. These are the results of calculations using a simple approach, Appendix K. For HCMC it is assumed that there are no flood problems when the water level in the basin does not exceed MSL+1,0m. For the same constant river discharge a discharge sluice is just as effective as a reductor with less required storage area and opening size.

Q (m^3/s)	h_{max} (m)	S (km^2)	A_c (m^2)
30.000	1	650	27.000
20.000	1	430	18.000
10.000	1	220	9.000

table H-5: results simple approach discharge sluice

H.4.2 Flow velocity

Only when the water level in the basin exceeds the water level at sea, water is discharged into the sea. So there is never inflow but only outflow.

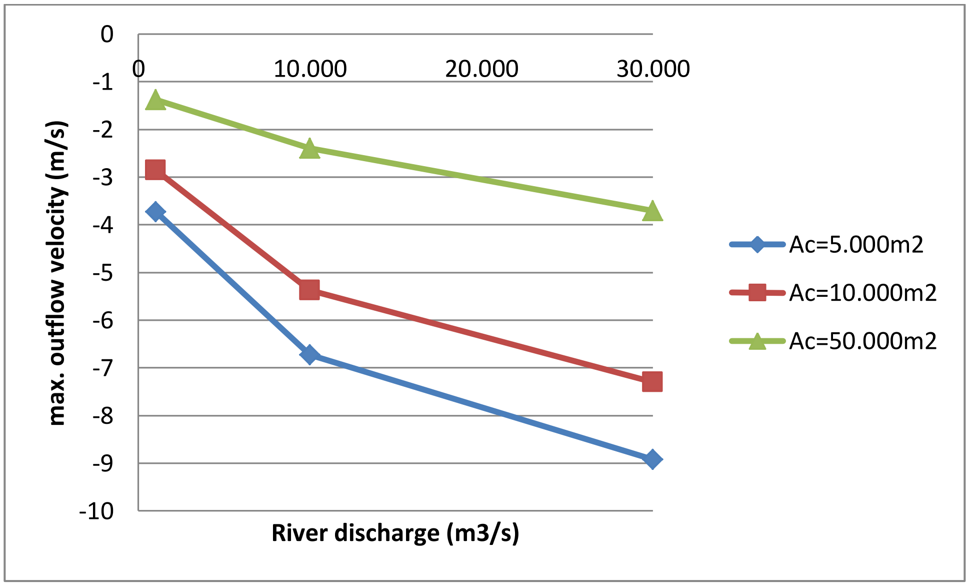


Figure H-10: max. outflow velocity through discharge sluice as a function of river discharge Q for various opening sizes Ac, S=1.000km2

Figure H-10 shows that a large opening size and small river discharge results in lower maximum outflow velocities. Figure H-11 shows that a decrease of storage area results in higher maximum outflow velocities.

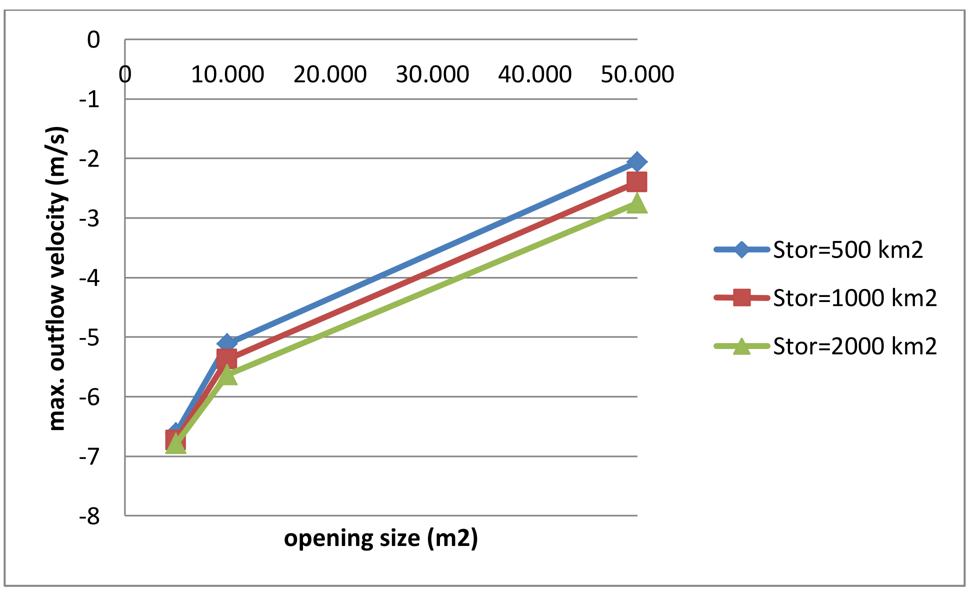


Figure H-11: max. outflow velocity through discharge sluice as a function of opening size Ac for various storage area S, Q=10.000m3/s

	discharge sluice
	max. outflow velocity
Ac increases	lower
Q_{in} decreases	lower
S increases	higher

table H-6: effect variables on max. outflow velocity through the structure

H.4.3 Salt intrusion

In the case of a discharge sluice there is no inflow but only outflow through the structure, eventually this will result in a fresh water situation upstream of the barrier. It depends on the lay out of the discharge sluice and river discharge how long it takes for to create a fresh water situation.

H.5 HCMC

HCMC is frequently flooded due to a combination of high tide, rainfall and high river discharges. A possible solution to reduce the water levels in the rivers surrounding and crossing HCMC is by controlling the in- and outflow in the estuary mouth to a feasible extent. The effectiveness of a barrier depends on the operational regime, the tidal movement, cross section of the opening A_c, river discharge Q and the storage area S.

In order for a reductor to be effective the storage area and cross section of the opening should be large. In case of HCMC the storage area of the system is relatively small. A reductor is therefore not a good option in the case of HCMC. A discharge sluices creates water level reduction by not allowing water to flow from sea into the system. This operation regime can only be feasible for extreme design flood waves if the water level reduction is large enough to store large river discharge temporarily. In the study the feasibility of a discharge sluice will be investigated. Besides this operational regime also the feasibility of a tidal barrier will be investigated. A tidal barrier is a barrier that is closed when at a defined location the water level exceeds a certain value and is open if the water level is below that certain value.

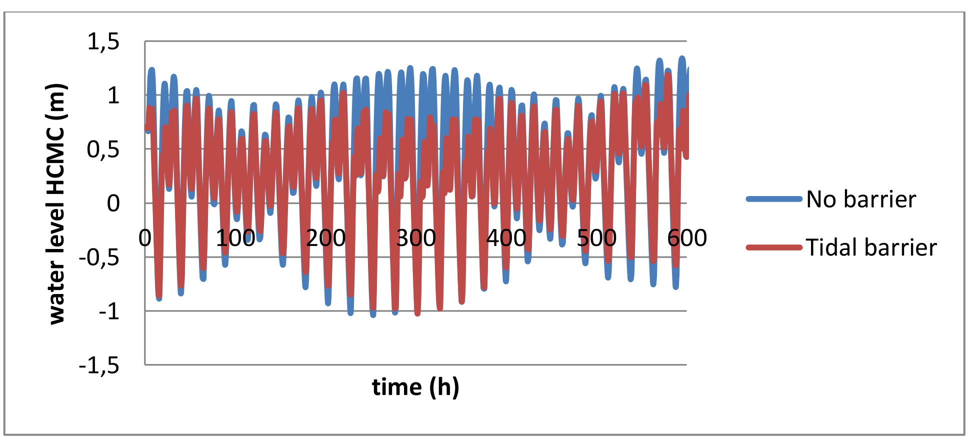


Figure H-12: effect tidal barrier on upstream water level

Appendix I: Duration and occurrence in point in time in tidal cycle of a flood wave

In chapter Appendix H the functioning of a hydraulic structure with two different operational regimes is described. The water level in the basin depends on three variables; cross section opening A_c, river discharge Q and storage area S. For the calculations it is assumed that the river discharge is constant, which corresponds with the worst case scenario. This assumption is based on calculations on the effect of variation of duration and occurrence in point in time in tidal cycle of a flood wave on the water level reduction in the basin. Figure I-1 shows the different occurrence in point in time in tidal cycle of a flood wave.

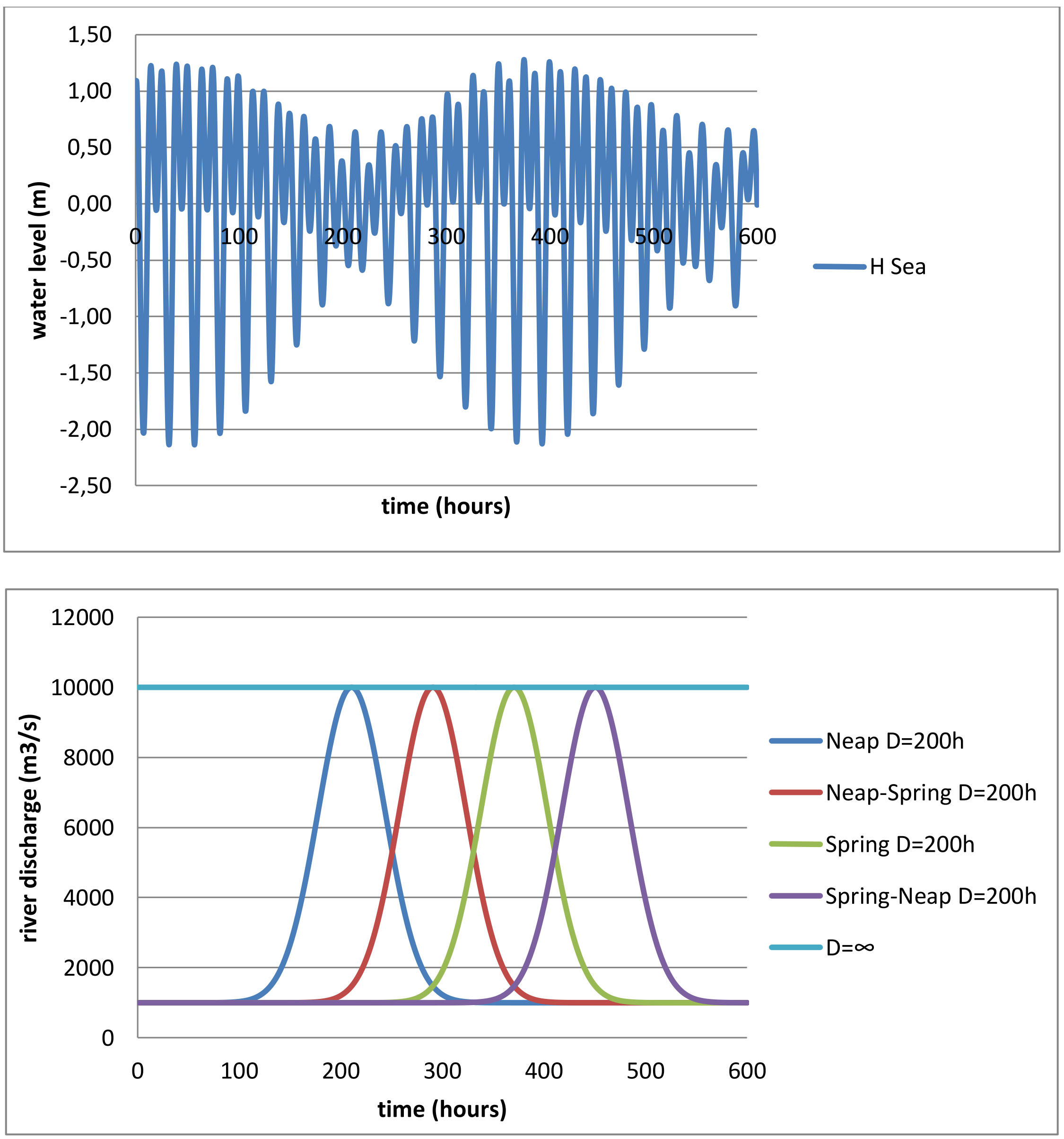


Figure I-1: occurrence in point in time in tidal cycle of a flood wave

I.1 Reductor

In the case of a reductor the barrier is open both during normal conditions and during extreme conditions. In Figure I-2 the effect of variation in duration and point in time in the tidal cycle of a flood wave on the maximum water level in the basin. During spring tide the water level reduction is more than during neap tide (difference between no dam and every other line); this is independent of the duration and the occurrence in point in time in the tidal cycle of a flood wave.

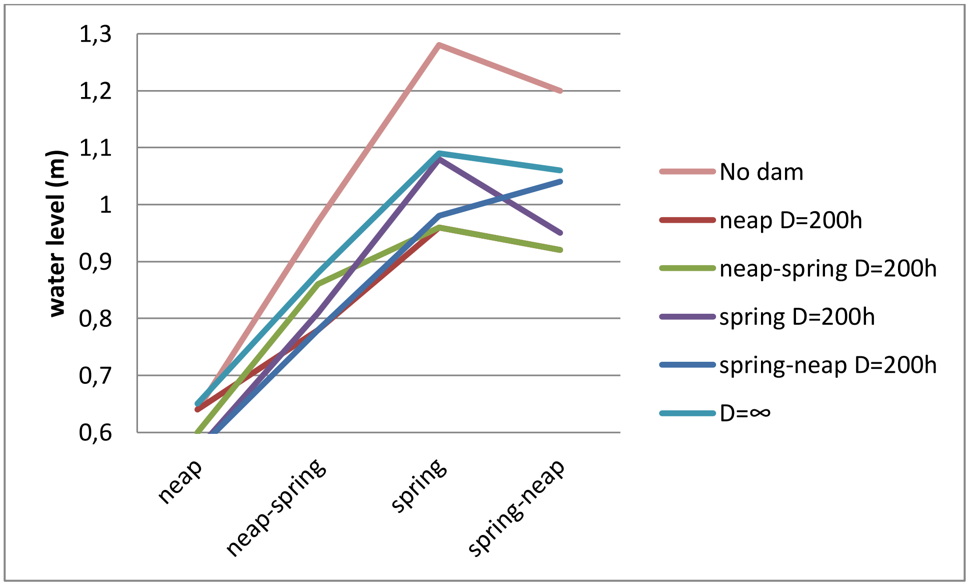


Figure I-2: effect of duration and point in time in tidal cycle of flood wave on max. water level in tidal cycle in the basin; Qpeak=10.000m3/s, S=500km2, Ac=10.000m2

The figure shows that the water level reduction depends on the occurrence of a flood wave in the tidal cycle. As discussed in chapter 4 it is assumed that HCMC has no flood problems as long as the water levels in the rivers around and crossing HCMC are below MSL +1,0 m. So in this case a flood wave during spring tide is the least favourable, the water level at spring tide exceeds MSL +1,0 m. The Figure also shows the effect of a constant extreme discharge. The line D=∞corresponds with the lowest water level reduction during every stage of the tidal cycle and is therefore the worst case scenario. The occurrence of a flood wave during spring tide results in the highest water levels in the basin.

I.2 Discharge sluice

In case of a discharge sluice the water level in the basin is not directly affected by the tidal movement at sea and therefore there is no clear relation between the occurrence of a flood wave at a certain point in the tidal cycle and the water levels during the tidal cycle, Figure I-3. A constant extreme discharge gives the lowest water level reduction at all points in the tidal cycle and gives therefore the worst case scenario. The occurrence of a flood wave during neap-spring tide results in the highest water levels in the basin.

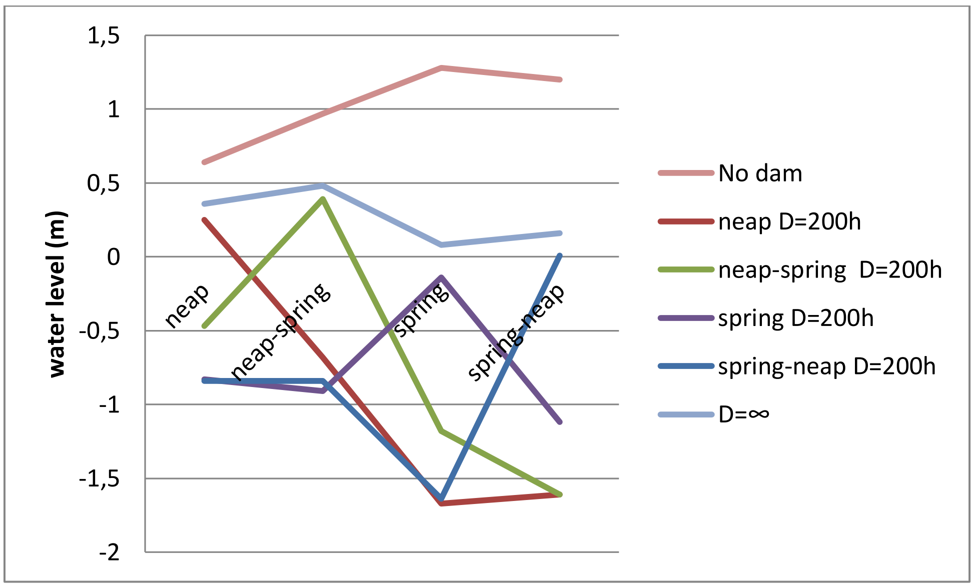


Figure I-3: effect of duration and point in time in tidal cycle of flood wave on max. water level in tidal cycle in the basin; Qpeak=10.000m3/s, S=500km2, Ac=10.000m2

Appendix J: Simple approach reductor

The effectiveness of a reductor depends on 3 variables: cross section opening A_c, river discharge Q_{in} and storage area S. The goal of a reductor is controlling the water level in a basin by limiting the in- and outflow through the structure. Suppose the tide is as a block signal (T=6 hours) and a constant river inflow, see Figure J-1. A reductor is open in every stage of the tidal cycles, both during high and low tide.

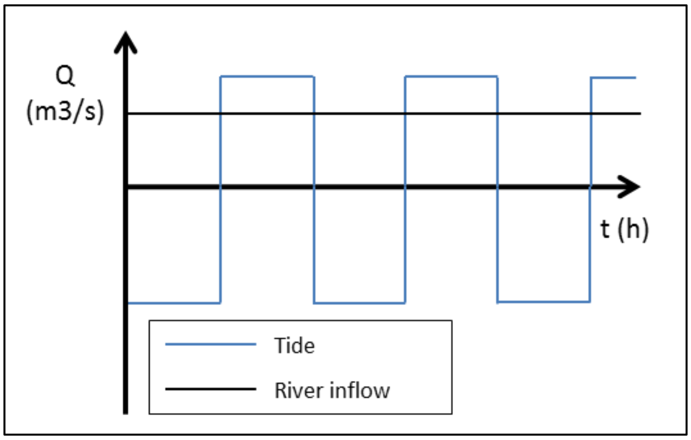


Figure J-1: schematisation of discharge

During HW the water level in the basin rises due to inflow form the river as well as from the sea. During LW the water level falls if the outflow exceeds the total inflow over a tidal period.

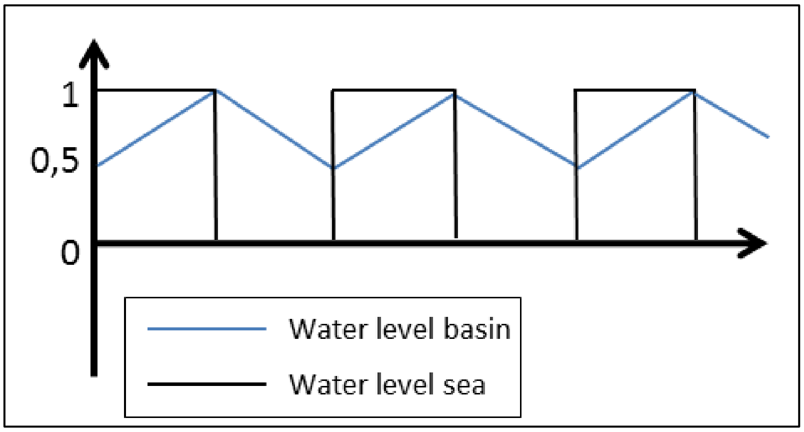


Figure J-2:schematisation of water level fluctuation in basin

The in- and outflow through a structure depend on the cross section opening and on the water level difference in the basin and at sea. In case of a reductor the cross section of the opening is constant over a tidal period. For a reductor to be effective the outflow should at least be larger than the inflow. Therefore the water level difference should be in favour of the outflow, this results in a minimum water level requirement in the basin. It is assumed that the minimum water level requirement for HCMC is MSL+0,5m.

In the case of HCMC the maximum water level in the basin should not exceed MSL+1,0m in order to prevent flood problems. During HW the inflow should be stored temporarily; the storage area of the system should be large enough to make sure the maximum water level in the basin is not exceeded. It is assumed that the tidal movement fluctuates between MSL+0,0m and MSL+1,0m.

$$h_{\max} = \frac{Q_{river}T + \mu A\sqrt{2g(HW - h_{\min})}\frac{T}{2}}{S}$$

$$h_{\max} \leq MSL + 1,0m$$

$$S \geq= \frac{Q_{river}T + \mu A\sqrt{2g(HW - h_{\min})}\frac{T}{2}}{h_{\max}} = Q_{river}T + \mu A\sqrt{2g(HW - h_{\min})}\frac{T}{2}$$

During LW water is discharged into the sea. In order not to exceed the maximum water level in the basin the outflow volume should be equal to the inflow volume over one tidal period.

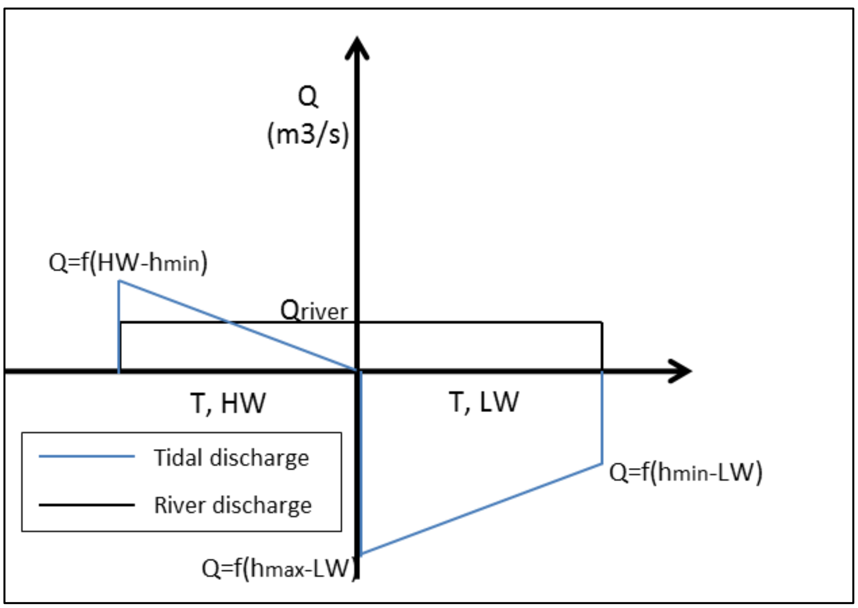


Figure J-3: schematisation in- and outflow during a tidal period

$$V_{in} = Q_{river} * 2T + \mu A\sqrt{2g(HW - h_{\min})}\frac{T}{2}$$

$$V_{out} = \mu A\sqrt{2g(h_{\max} - LW)}\frac{T}{2} + \mu A\sqrt{2g(h_{\min} - LW)}T$$

$$V_{in} = V_{out}$$

$$h_{\max} = MSL + 1,0m$$

$$\mu A_c \geq \frac{2Q_{river}}{\frac{1}{2}\sqrt{2g(h_{\max} - LW)} + \sqrt{2g(h_{\min} - LW)} - \frac{1}{2}\sqrt{2g(HW - h_{\min})}}$$

This simple approach shows relations between different variables:

- A is proportional with Q_{river}
- A is inversely with $\sqrt{h_{max}}$

For T=6 hours table K-1 shows the effect of different river inflows on the required storage area and opening size. The required storage area and cross section of the opening for the same river discharge is smaller compared to the situation with a reductor, see Appendix K.

Q (m^3/s)	h_{max} (m)	h_{min} (m)	LW (m)	HW (m)	A (m^2)	S (km^2)
30.000	1	0,5	0	1	15.870	1.185
20.000	1	0,5	0	1	10.580	790
10.000	1	0,5	0	1	5.290	395

table J-1: examples effect river inflow on required storage area and opening size

In order for a reductor to be effective the storage area should be large compared to a discharge sluice, see Appendix K. The effectiveness depends also on the tidal movement at sea and the required minimum and maximum water levels in the basin. In case the maximum allowable water level is lowered in the basin or the HW at sea increases there is more inflow into the system. This requires more storage area and a larger cross section for the opening. Due to a large cross section both the in- and outflow through the structure increase but the increase in outflow is larger than the increase in inflow. In the case the minimum allowable water level is increased in the basin or the LW at sea is lowered there is more outflow. The required storage area and cross section of the opening reduce.

Appendix K: Simple approach discharge sluice

The effectiveness of a discharge sluice depends on 3 variables: cross section opening A_c, river discharge Q_{in} and storage area S. A discharge sluice can be used to control the water level in a basin.

Suppose the tide is a block signal (T=6 hours) and the river inflow a constant, see Figure K-1. The discharge sluice is closed during high tide and open during low tide.

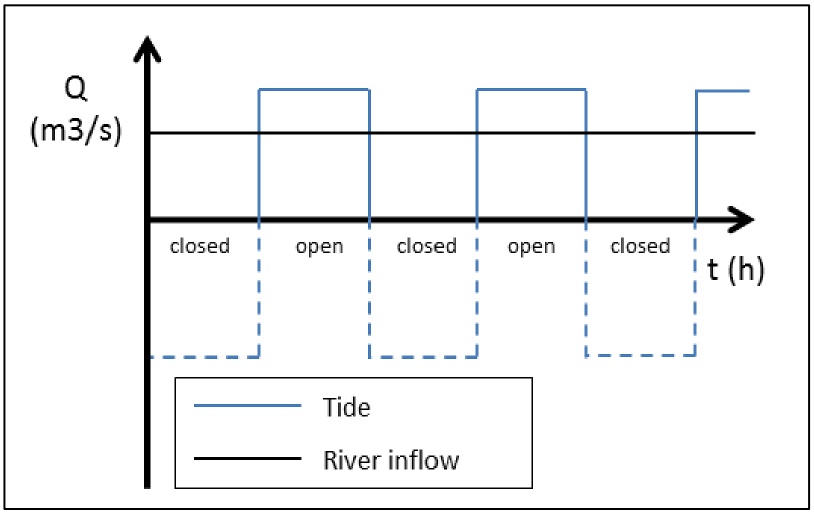


Figure K-1: schematisation of discharge

During HW the water level in the basin rises (sluice is closed) with Q_{in} * T and falls during LW.

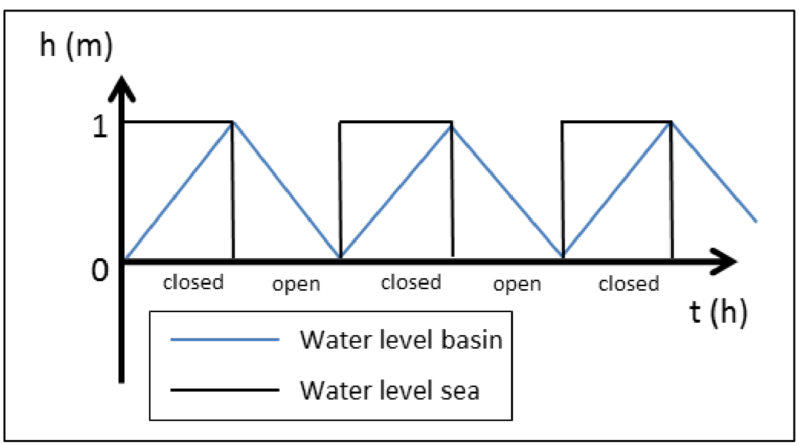


Figure K-2:schematisation of water level fluctuation in basin

In the case of HCMC the maximum water level in the basin should not exceed MSL+1,0m in order to prevent flood problems. During HW (sluice is closed) the inflow should be stored temporarily; the storage area of the system should be large enough to make sure the maximum water level in the basin is not exceeded. It is assumed that the tidal movement fluctuates between MSL+0,0m and MSL+1,0m.

$$h_{\max} = Q_{in} \frac{T}{S}$$
$$h_{\max} \leq MSL + 1,0m$$
$$S \geq Q_{in} \frac{T}{h_{\max}} = Q_{in} T$$

During LW (sluice is open) the outflow capacity should be large enough to discharge the stored inflow and the inflow during the period that the sluice is open. In order not to exceed the maximum water level in the basin the outflow volume should be equal to the inflow volume over one tidal period.

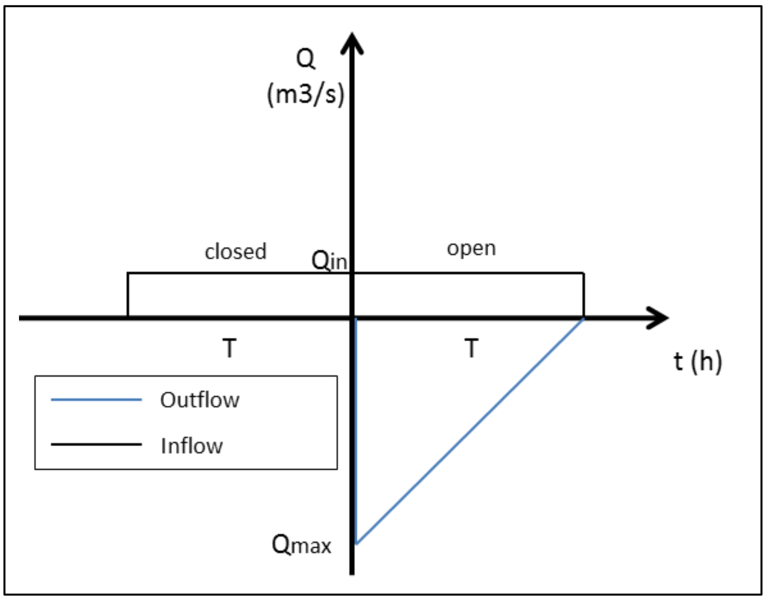


Figure K-3: schematisation in- and outflow during a tidal period

$$V_{in} = Q_{in} * 2T$$
$$V_{out} = Q_{\max} \frac{T}{2}$$
$$so\ Q_{\max} = 4Q_{in}$$

The outflow depends on the cross section opening A_c and the water level difference in the basin and at sea h(t). Given a maximum water level a relation can be made with the required cross section of the opening.

$$Q_{out,\max} = \mu A_c \sqrt{2gh_{\max}} = 4Q_{in}$$
$$h_{\max} = MSL + 1,0m$$
$$\mu A_c \geq \frac{4Q_{in}}{\sqrt{2g}}$$

This simple approach shows relations between different variables:

- h_{max} is proportional with Q_{in}/S
- A is proportional with Q_{in}
- A is inversely with $\sqrt{h_{max}}$

For T=6 hours table K-1 shows the effect of different river inflows on the required storage area and opening size. The required storage area and cross section of the opening for the same river discharge is smaller compared to the situation with a reductor, see Appendix J.

Q (m^3/s)	h_{max} (m)	S (km^2)	A_c (m^2)
30.000	1	650	27.000
20.000	1	430	18.000
10.000	1	220	9.000

table K-1: examples effect river inflow on required storage area and opening size

During HW at sea the discharge sluice is closed and therefore the required storage area and cross section of the opening is not affected by this. A lower LW at sea has a positive effect on the functioning of the discharge sluice and the required storage area and opening size decreases. If the maximum allowable water level in the basin is lower than HW at sea the sluice has less time to discharge the same amount of water and this requires more storage area and a larger cross section of the opening.

Appendix L: Sobek model

The goal of this prefeasibility study is to determine the effect of constructing a closure dam with a hydraulic structure on the water levels in the rivers surrounding and crossing HCMC. For this study it is not necessary to model the entire system in detail, a 1D-model called Sobek was used for the calculations. Sobek is the name of a highly sophisticated software package, which in technical terms is a one-dimensional open-channel dynamic numerical modelling system, equipped with the user shell and which is capable of solving the equations that describe unsteady water flow, salt intrusion, sediment transport, morphology and water quality. In less technical terms Sobek can be described as a flexible, powerful and reliable tool to simulate and solve problems in river management, flood protection, design of canals, irrigation systems, water quality, navigation and dredging.

L.1 Schematisation

For this prefeasibility study it is not necessary to model the entire system in detail, therefore simplifications are made. The schematisation is based on information received from the Institute for Water & Environmental Research (IWER); cross-sections of the rivers, land elevation, measured water levels, measured salt concentrations, measured river discharge. Some of the simplifications made are described below:

- Cross-sections

The data on the cross-sections of the rivers of the Dong Nai - Saigon river system were collected in 2007. The cross-sections were measured with the average yearly MSL as reference, the slope of the rivers can be determined from these measurements. The measured cross-sections of the main rivers (Saigon, Dong Nai, Nha Be, Soai Rap, Long Tau and Vam Co) were used in the model. For simplification reasons the cross section of smaller rivers and tributaries were combined in the schematisation.

- Overland flow

In the model, water will only flow through the indicated cross-sections of the rivers independent of the water levels. This is not the case in reality; water will flow over land if the water level exceeds a certain value. The calculated water levels will therefore be higher than those reached in reality. In a future study the overland flow should be taken into account to make the results more reliable.

- Mangrove

Characteristic for a mangrove forest is that during high tide a part of the mangrove is flooded while during low tide it is not. The cross-sections of the Can Gio mangrove forest were based on the received river cross-sections and land elevation data.

- Connection with sea

The cross-sections of the connection between the main rivers Soai Rap and Long Tau are based on the nautical chart.

- Upstream boundary condition

With some intervals the river discharge of the Saigon, Dong Nai, Vam Co Dong and Vam Co Tay are measured and it is assumed that only these rivers generate fresh water inflow into the system.

- Downstream boundary condition

The measured water level fluctuation at Vung Tau is used as downstream boundary condition.

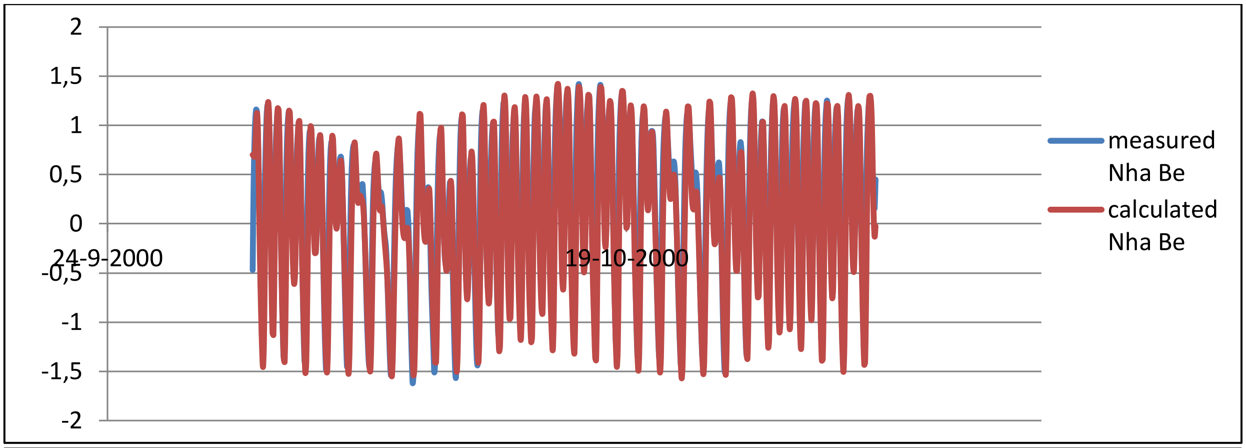
L.2 Calibration and validation

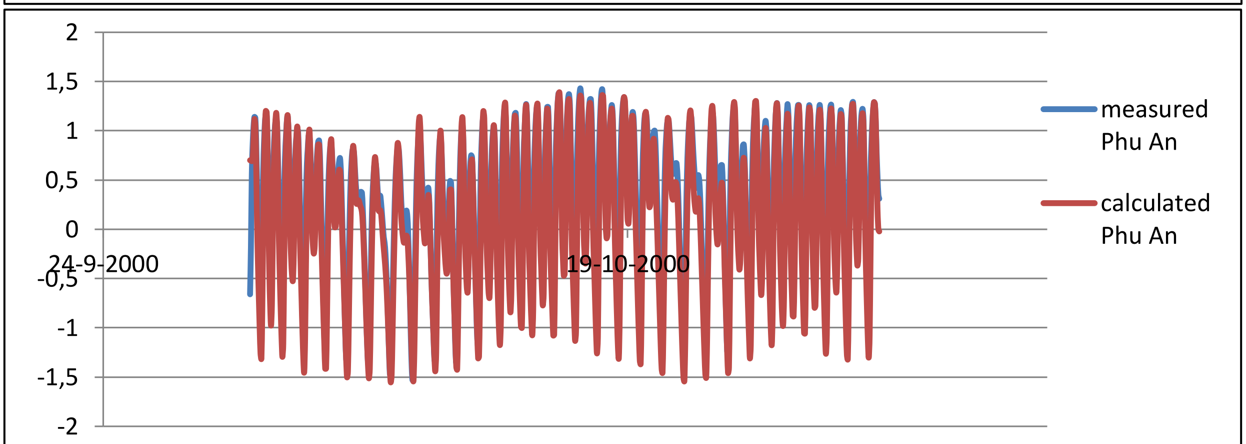
Water levels are measured at 6 locations for a number of periods, see Figure L-1 . These measurements are used to calibrate and validate the model. Roughness is the parameter which is used to make the model as accurate as possible. In this model the roughness is expressed with Manning and all values are between 0,015 and 0,04.

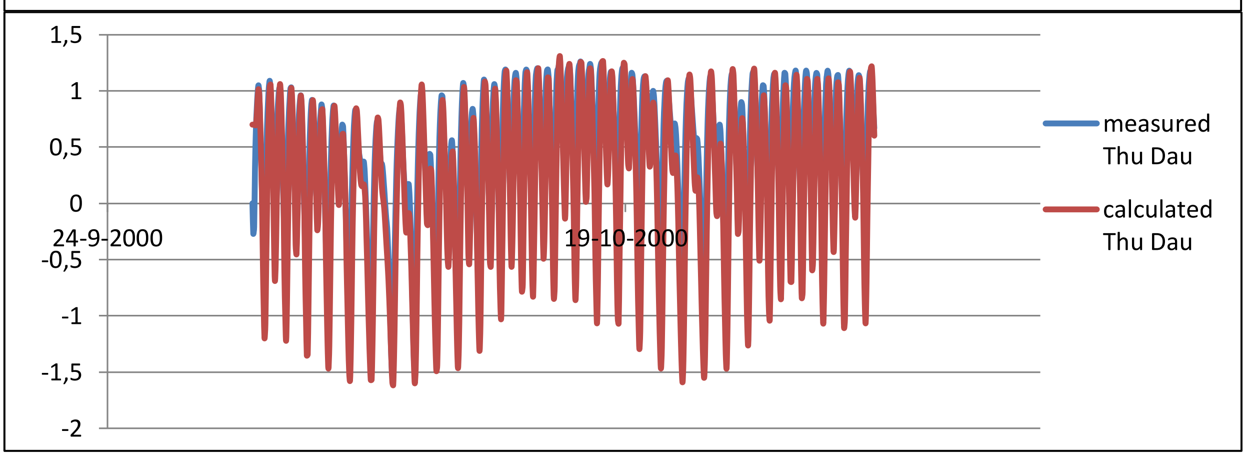
Figure L-1: locations where water level and salt concentration measurements are done

In Figure L-2, Figure L-3 and Figure L-4 the calibration and validation results are shown for different months. For November 2000 at the locations Nha Be, Phu An and Thu Dau the phase and amplitude of the water level corresponds fairly well with the measured water levels (difference for maxima is circa 5 cm). For January 2005 and November 2007 the calculated values corresponds less with the measured values, especially the maxima (difference of circa 20 cm).

In November 2000 en November 2007 there is, at Ben Luc and Tan An, a large difference between the measured and calculated water levels (amplitude and phase correspond well). In January 2005 the calculated water levels correspond fairly well with the measured values. The fact that the measured and calculated water levels correspond fairly well in January and that there is a large difference in November can be related to the difference in river discharge and the effect on the bed level of the rivers. There is however not enough data to explain the difference.


2
1,5
1
0,5
0
-0,5
-1
-1,5
-2
24-9-2000
19-10-2000
measured Nha Be
calculated Nha Be


2
1,5
1
0,5
0
-0,5
-1
-1,5
-2
24-9-2000
19-10-2000
measured Phu An
calculated Phu An


1,5
1
0,5
0
-0,5
-1
-1,5
-2
24-9-2000
19-10-2000
measured Thu Dau
calculated Thu Dau

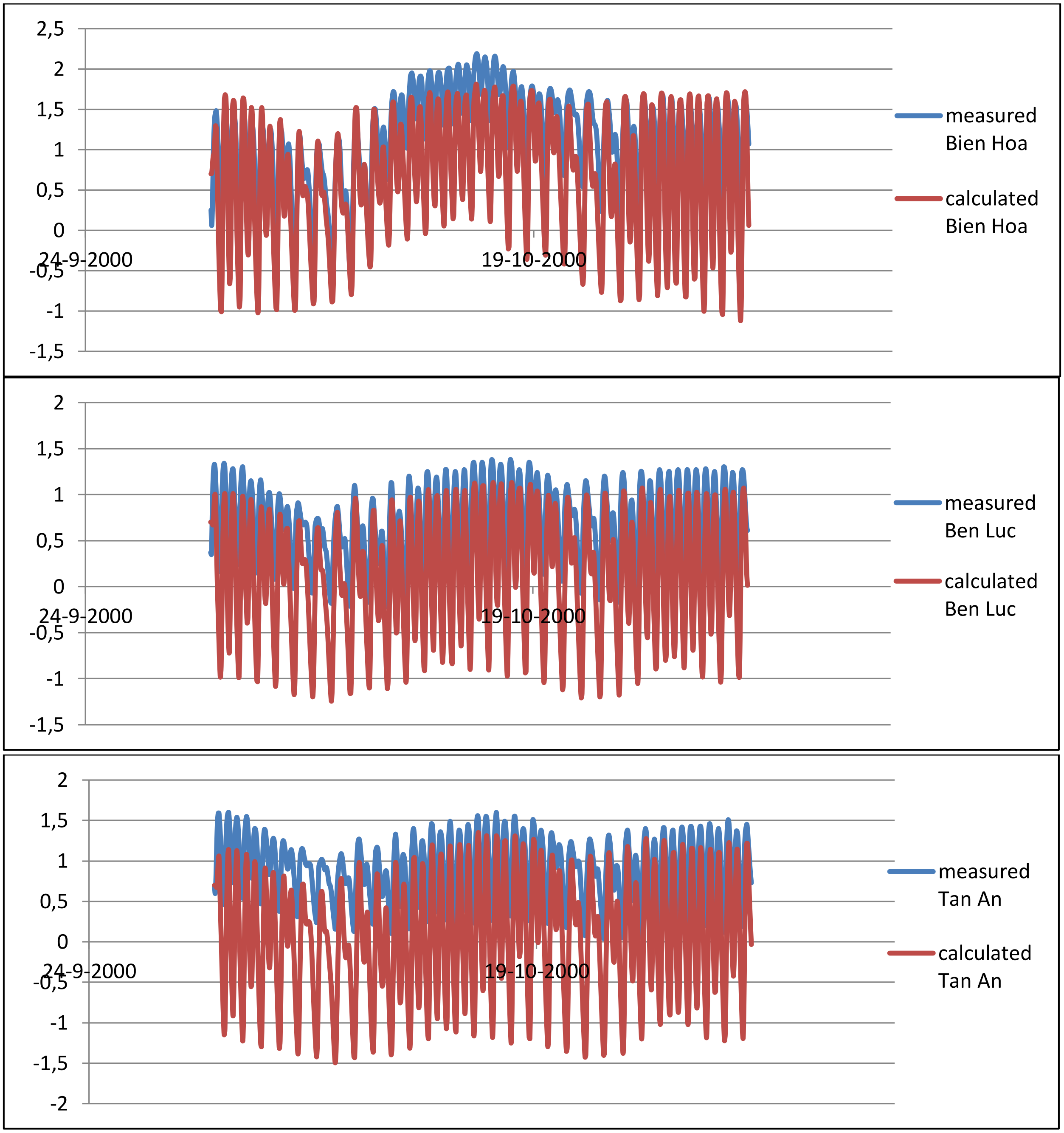
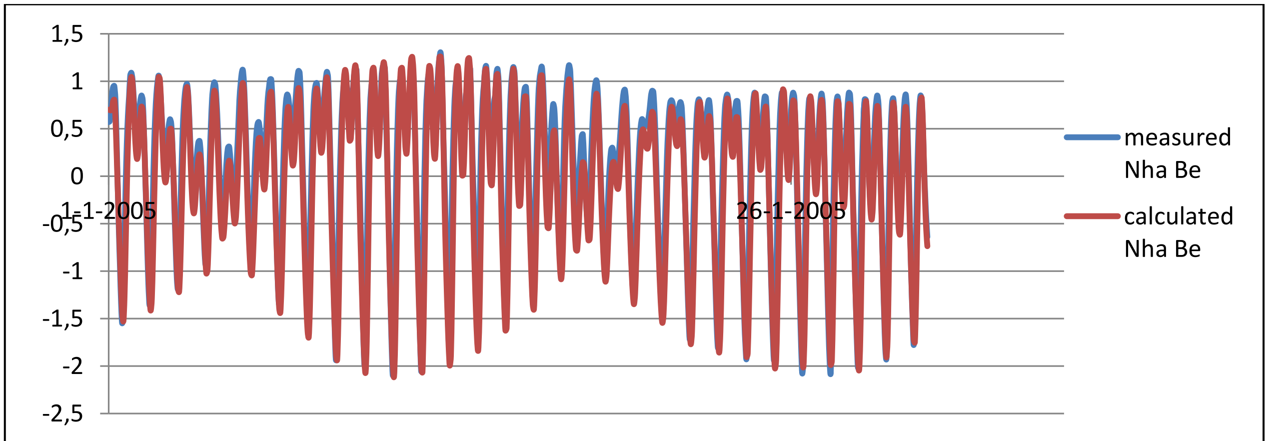
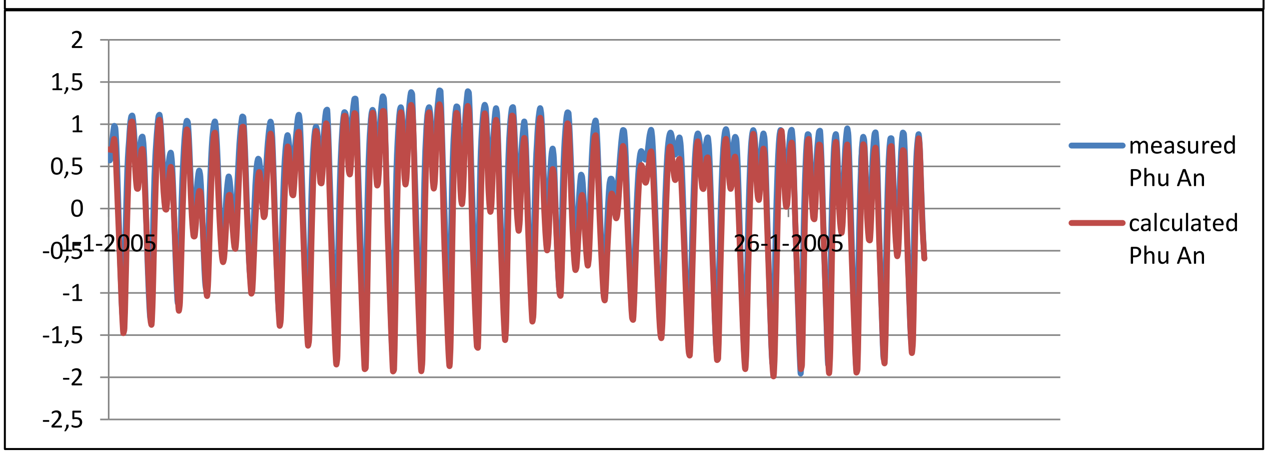
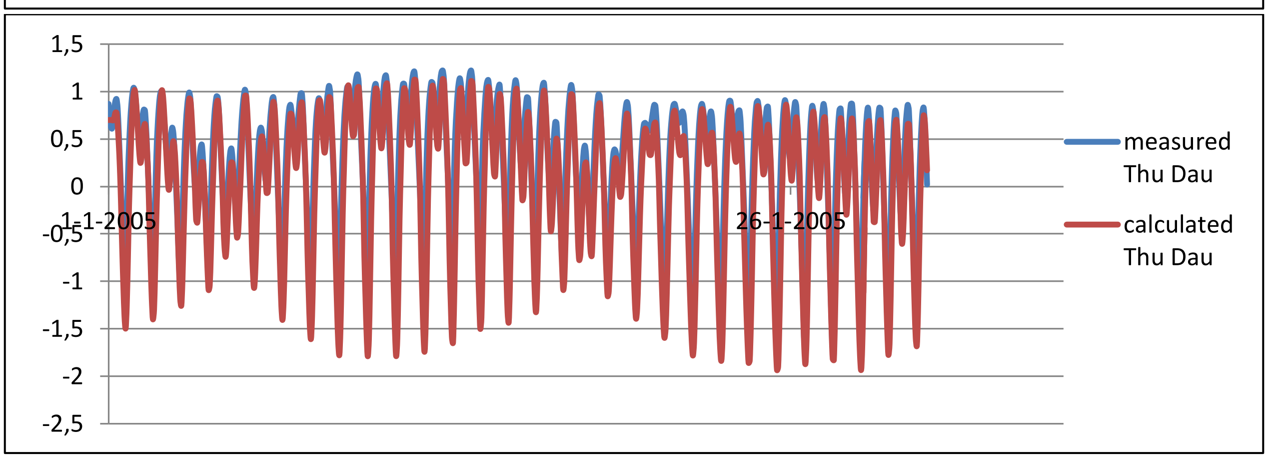


Figure L-2: calibartion November 2000


1,5
1
0,5
0
-0,5
-1
-1,5
-2
-2,5
1-1-2005
26-1-2005
measured Nha Be
calculated Nha Be


2
1,5
1
0,5
0
-0,5
-1
-1,5
-2
-2,5
1-1-2005
26-1-2005
measured Phu An
calculated Phu An


1,5
1
0,5
0
-0,5
-1
-1,5
-2
-2,5
1-1-2005
26-1-2005
measured Thu Dau
calculated Thu Dau

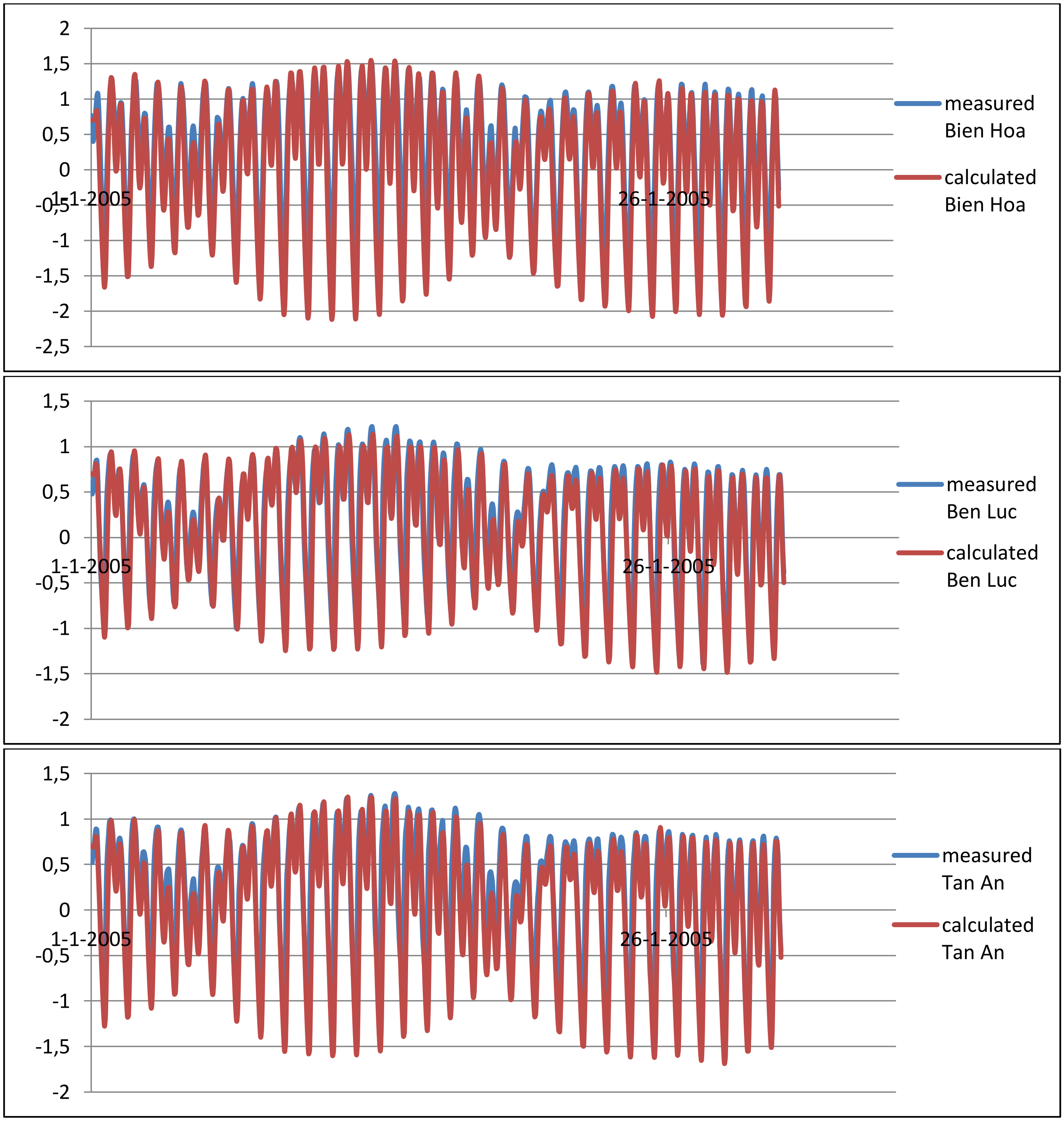
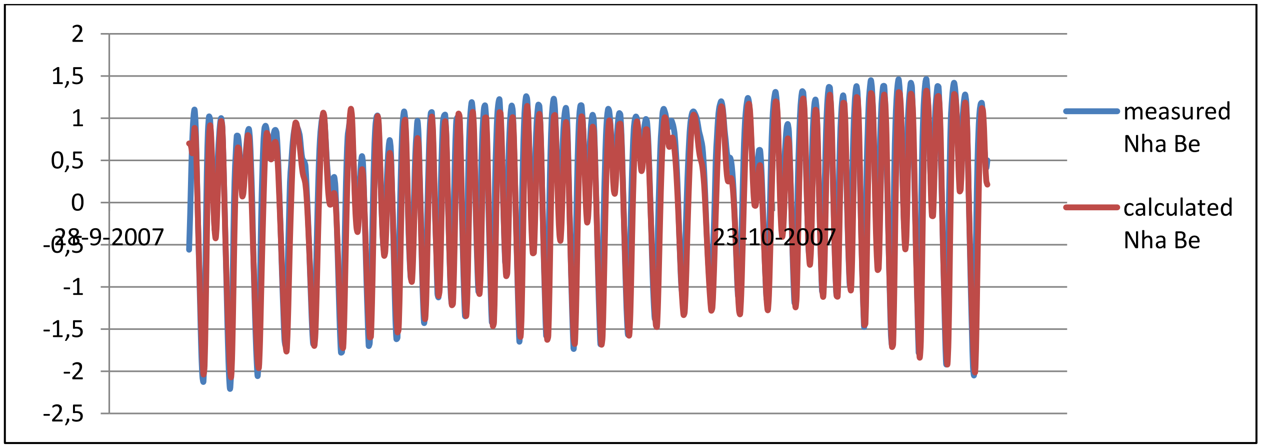
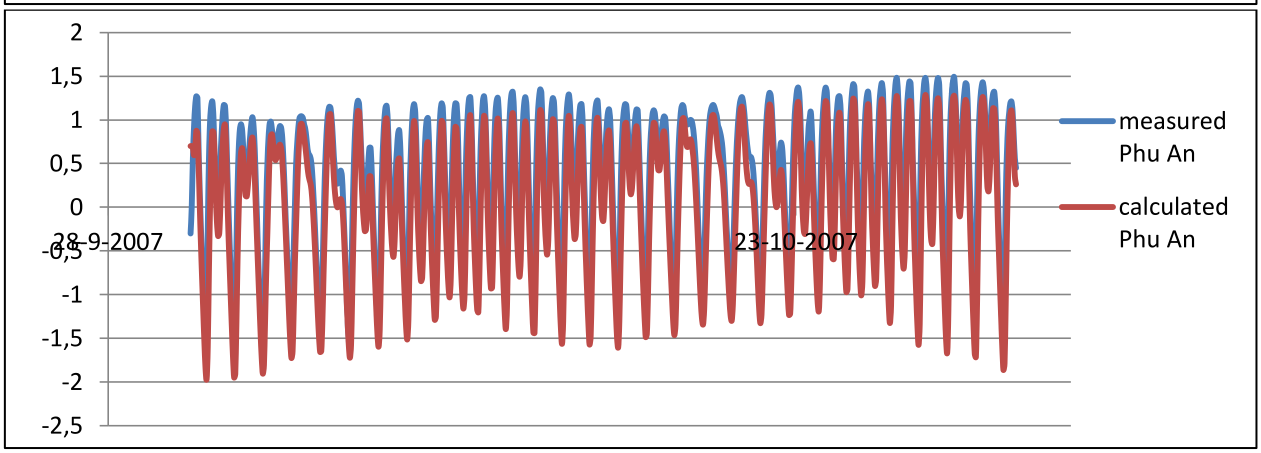
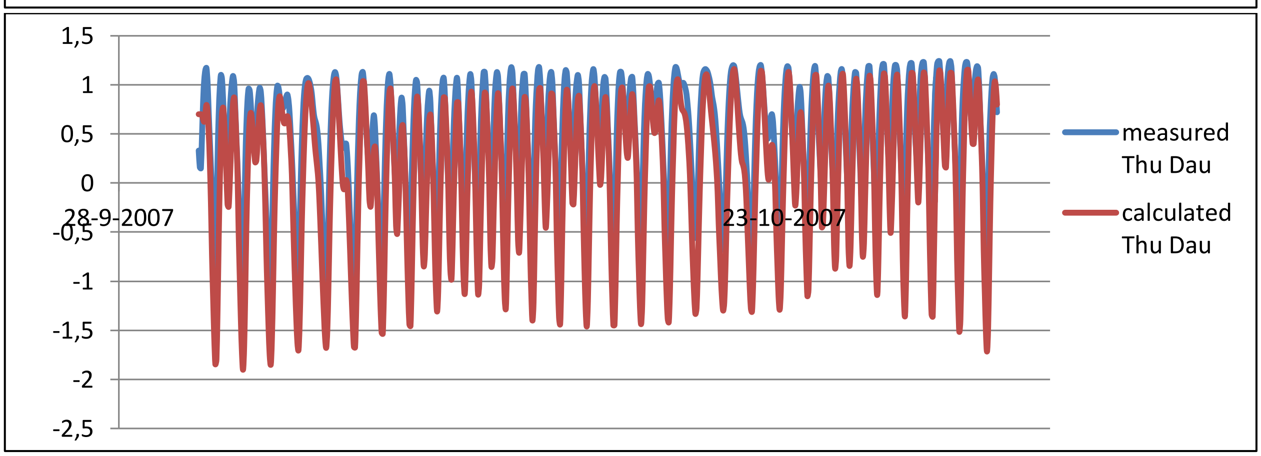


Figure L-3: validation January 2005


2
1,5
1
0,5
0
-0,5
-1
-1,5
-2
-2,5
28-9-2007
23-10-2007
measured Nha Be
calculated Nha Be


2
1,5
1
0,5
0
-0,5
-1
-1,5
-2
-2,5
28-9-2007
23-10-2007
measured Phu An
calculated Phu An


1,5
1
0,5
0
-0,5
-1
-1,5
-2
-2,5
28-9-2007
23-10-2007
measured Thu Dau
calculated Thu Dau

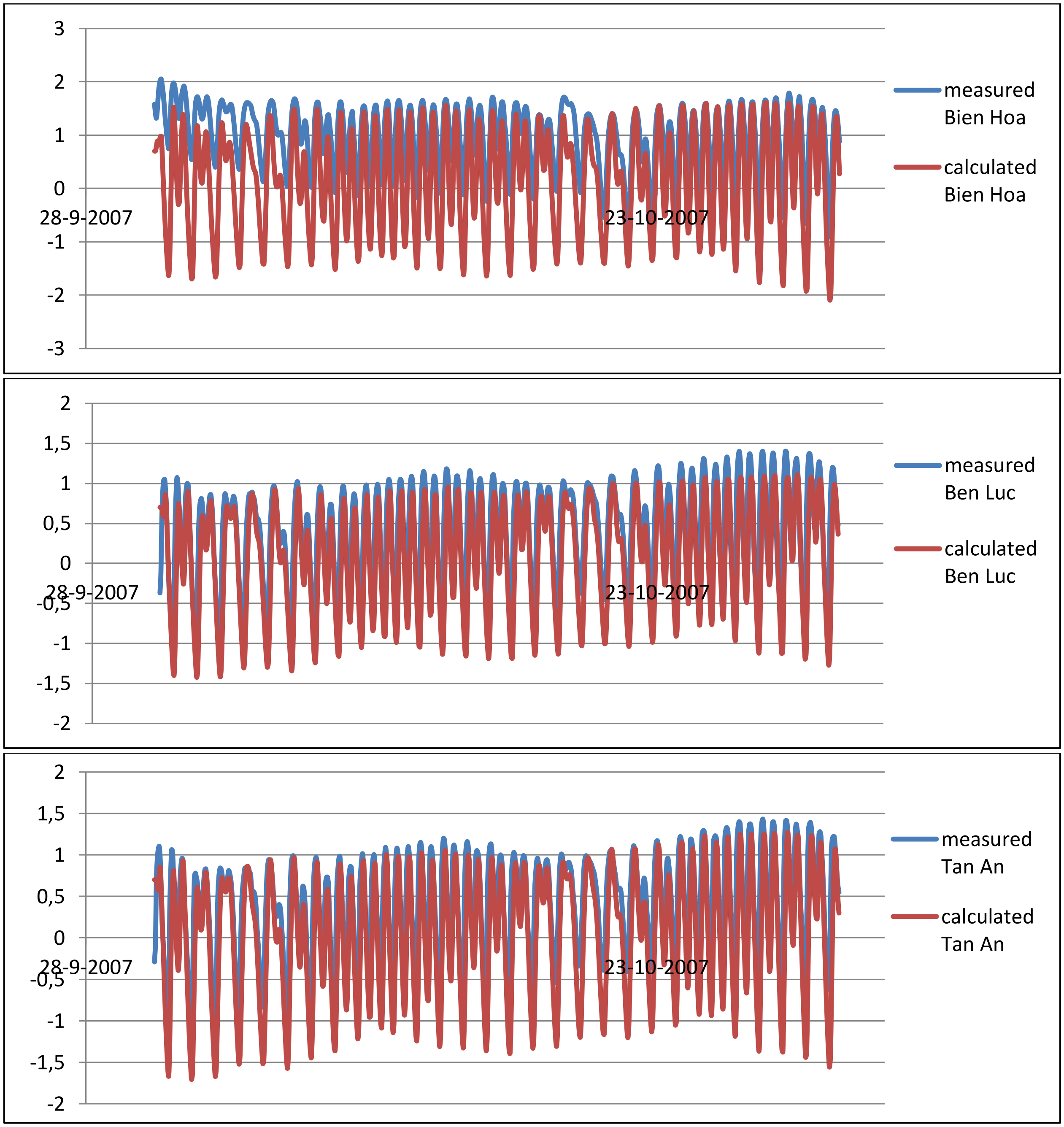


Figure L-4: validation November 2007

L.3 Inaccuracy

As a result of simplifications in the schematisation and a lack of data the model is inaccurate and the model results have a fault margin. For this study we are interested in the maximum water levels in HCMC and the location Phu An is used as reference point. The calibration and validation results show an inaccuracy of 5 to 20 cm for the maximum water levels in the case of normal yearly river discharge, this inaccuracy increases when the river discharges are more extreme. The model results are used to compare the effect of different barriers and the results should not be taken as the truth. With the model the potential of a barrier (with a certain operational regime) can be determined and different barrier options can be compared. For this prefeasibility study the potential of the system is more important than the exact calculated water level. In this stage the inaccuracy of the model is of minor importance but in a further study the accuracy should be improved. This can be done by the following measures:

- Storage area

Appendix H, J and K show that the effectiveness of a barrier downstream of HCMC depends for a large part on the amount of storage area in the system; this is especially the case for a discharge sluice (storage basin approach, Appendix H). In the schematisation of the Sobek model only the main rivers and some smaller rivers and tributaries are included. It is likely that in reality the amount of storage in the system is larger. The calculated flow velocities also indicate that there is not enough storage area included in the model. The maximum calculated flow velocity in the Long Tau is in the order of 0,7 m/s during an average tidal movement at sea (tidal range of 2 m). It is likely that the flow velocities in reality are higher (exceed 1 m/s), which indicates more storage in the system. An increase in storage area will have a positive effect on the effectiveness of a barrier downstream of HCMC. During the calibration and validation of the model it was found that the water levels, in the absence of a barrier, are hardly affected by the amount of storage area. The accuracy of the storage area in the model can only be improved using data on flow velocities. Flow velocities measurements are therefore essential.

- Simultaneously measured data on flow velocity, river discharge and water levels

Another important aspect which is needed to improve the accuracy of the model is simultaneous measurements on flow velocity, river discharge and water levels at different locations in the rivers.

- Overland flow

The fact that in the model-schematisation the elevation of the land is not taken into account makes the model inaccurate. Due to neglecting overland flow the calculated water levels are possibly higher than expected in reality. In order to make the schematisation more accurate overland flow should be included in the model.

- Water level data during extreme river discharges

For this study only water level measurements during normal yearly river discharges were available. Due to the lack of water level data during extreme river discharge the results are less reliable for cases with extreme river discharges. Accurate data on historical floods can be used to improve the accuracy of the model.

Appendix M: Model results

The effectiveness of a barrier depends on the operational regime, the tidal movement, cross section of the opening A_c, river discharge Q and the storage area S. The effect of a barrier downstream of HCMC on the water levels in the city is determined for two operational regimes, a discharge sluice and a tidal barrier, at 10 location options. The location of the main barrier determines the storage area.

M.1 Input

- Operational regime

Two operational regimes were investigated; a discharge sluice and a tidal barrier. See paragraph H.5 for argumentation.

- Tidal movement

With the data of the admiralty chart an average yearly tidal movement near HCMC was computed. Figure M-1 shows this average yearly tidal movement taken the effect of the monsoon seasons on the mean water level into account. The red square indicates the period that was used for the model calculations. This period was chosen because it includes the highest water levels throughout the year but also because the low waters are relatively high. This has a very negative effect on the effectiveness of a discharge sluice. The ADB expects a sea level rise of 24 to 26 cm in 2050, see paragraph 5.9. The effectiveness of the barrier was also determined for 2050. As input the tidal movement for 2010 was used with a mean water level that was shifted 25 cm upwards.

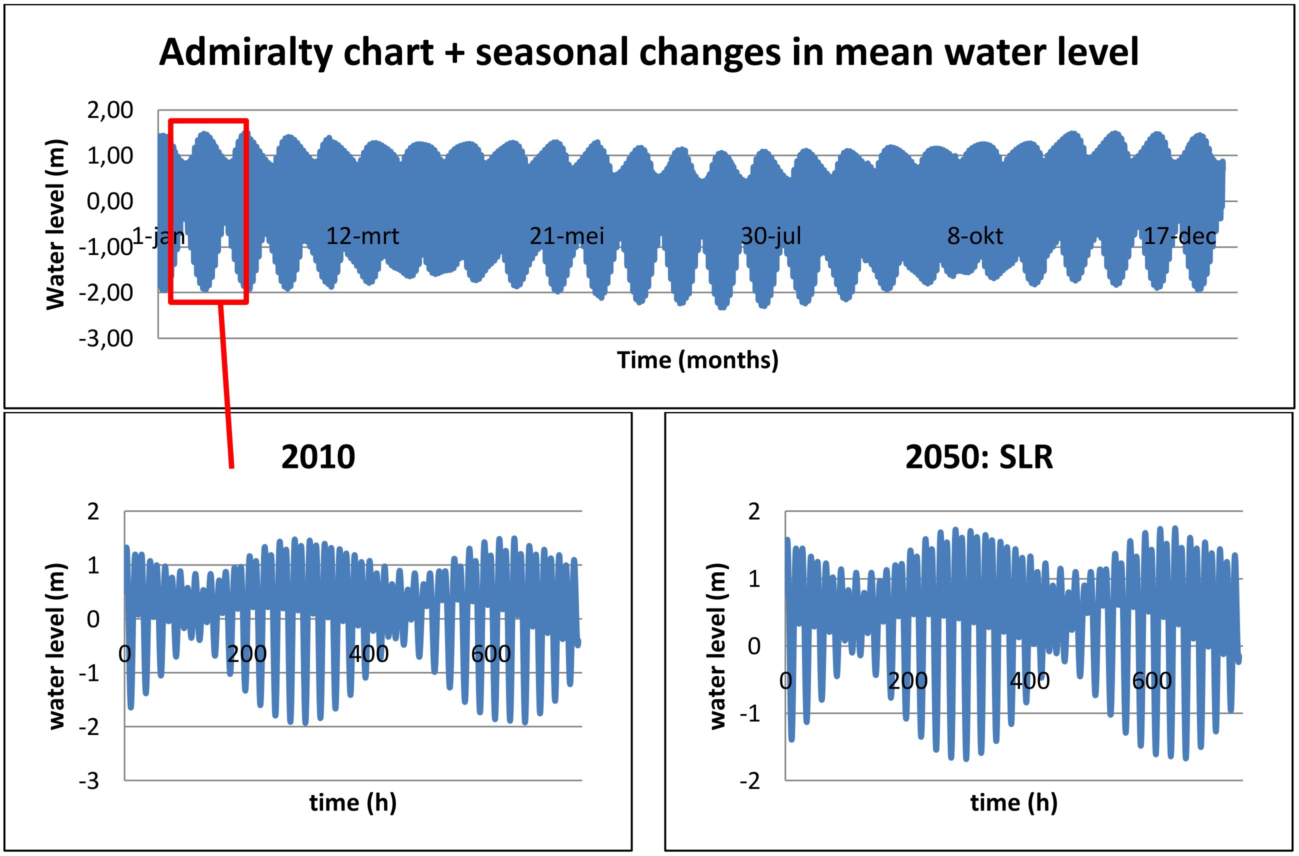


Figure M-1: tidal movement used as input for model calculations

- Cross section opening A_c

The effects of 3 different opening sizes were investigated: 10.000, 20.000 and 30.000 m^2. The sill of each opening is assumed to be 8 m below MSL. These 3 opening sizes were chosen to determine roughly how large the opening should be in order to be effective. In a future study the opening size can be determined more precise.

- River discharge Q

In the model there are 4 rivers which discharge fresh water into the system, see Figure M-2. For every river design river discharge with different probability of occurrence are shown. Appendix I showed that the effect of the discharge sluice is the least in case the flood wave occurs in the tidal cycle during the transmission from neap tide to spring tide. For the model calculations it is assumed that this is the case.

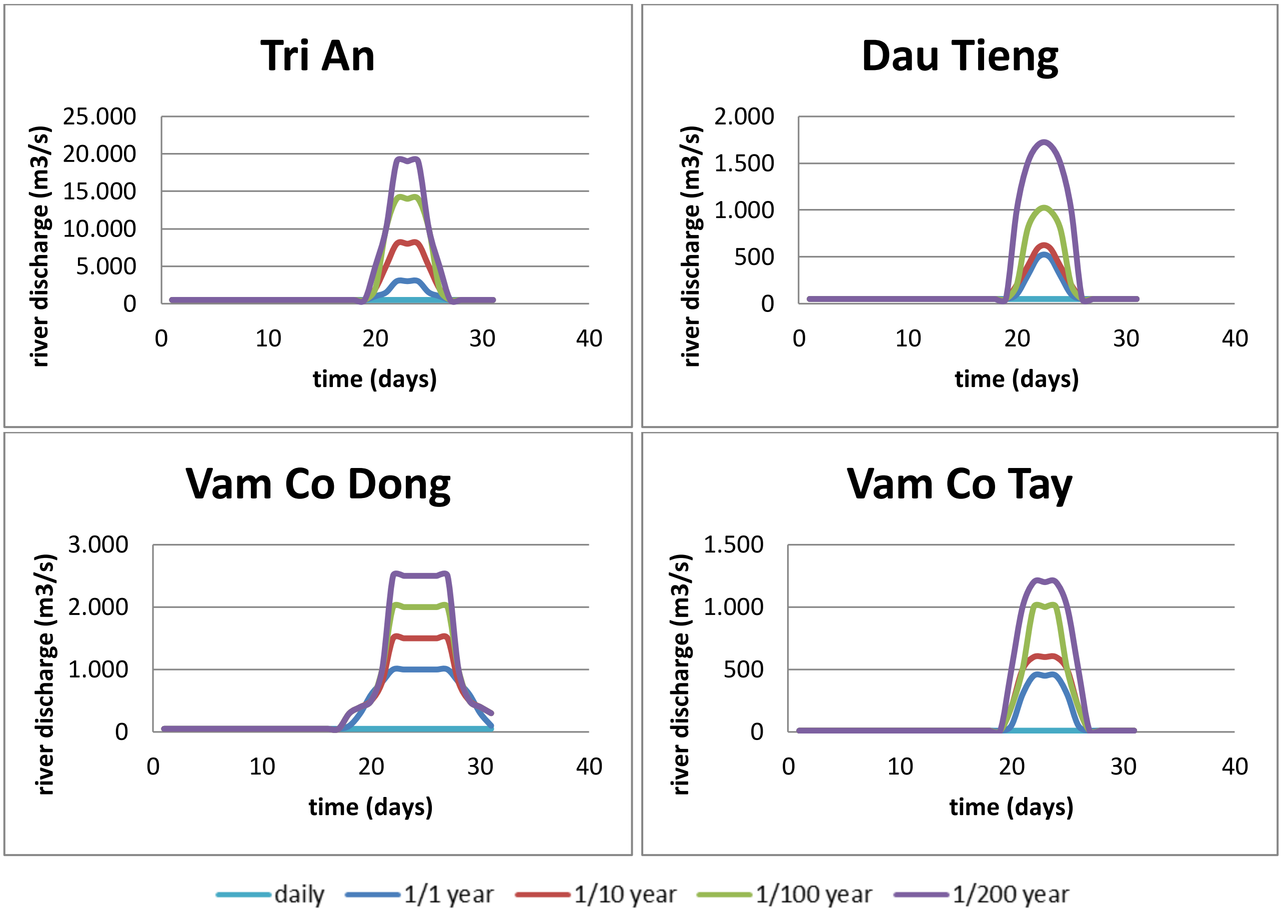


Figure M-2: design flood waves used as input for model calculations

	Tri An (m3/s)	Dau Tieng (m3/s)	Vam Co Dong (m3/s)	Vam Co Tay (m3/s)
daily	500	50	50	10
1/1 year	3000	500	1000	450
1/10 year	8000	600	1500	600
1/100 year	14000	1000	2000	1000
1/200 year	19000	1700	2500	1200
duration (days)	3	2	6	3

table M-1: max. discharge and duration of max. discharge per river

- Storage area S

The storage area depends on the location of the main barrier. For this study 4 different locations for the main barrier were investigated, see paragraph 7.2.

M.2 Results

M.2.1 Discharge sluice

Figure M-3 and Figure M-4 show the results if the main barrier functions as a discharge sluice. The calculations were made for 3 opening sizes (10.000, 20.000, 30.000 m^2), 5 design river discharges (daily,1/1, 1/10, 1/100 and 1/200 year) at 10 location options (4 locations main barrier and 2 options for the secondary barriers) both during 2010 and 2050. For this prefeasibility study the potential of the system is more important than the exact calculated water level. The results that are used to compare different location options contain a certain fault marge.

Location 1

	daily	1/1	1/10	1/100	1/200
No barrier	1,34	1,46	1,52	1,60	1,63
10000	0,60	0,92	1,18	1,50	1,91
20000	0,51	0,90	1,11	1,31	1,57
30000	0,51	0,91	1,13	1,27	1,50

Location 2

	daily	1/1	1/10	1/100	1/200
No barrier	1,34	1,46	1,52	1,60	1,63
10000	0,60	0,88	1,13	1,43	1,76
20000	0,55	0,86	1,05	1,29	1,52
30000	0,54	0,86	1,04	1,26	1,44

Location 3

	daily	1/1	1/10	1/100	1/200
No barrier	1,34	1,46	1,52	1,60	1,63
10000	0,43	0,78	1,05	1,38	1,73
20000	0,35	0,73	0,95	1,23	1,47
30000	0,30	0,68	0,93	1,17	1,39

Location 1a

	daily	1/1	1/10	1/100	1/200
No barrier	1,34	1,46	1,52	1,60	1,63
10000	-1,07	0,79	1,31	2,13	2,71
20000	-1,17	0,57	1,17	1,39	1,70
30000	-1,17	0,37	1,12	1,27	1,60

Location 2a

	daily	1/1	1/10	1/100	1/200
No barrier	1,34	1,46	1,52	1,60	1,63
10000	-1,04	0,72	1,28	1,96	2,63
20000	-1,19	0,40	1,04	1,38	1,72
30000	-1,26	0,24	0,96	1,29	1,51

Location 3a

	daily	1/1	1/10	1/100	1/200
No barrier	1,34	1,46	1,52	1,60	1,63
10000	0,46	0,53	1,26	1,91	2,46
20000	0,35	0,10	0,92	1,37	1,71
30000	0,30	-0,04	0,82	1,23	1,50

Location 1b

	daily	1/1	1/10	1/100	1/200
No barrier	1,34	1,46	1,52	1,60	1,63
10000	-1,31	0,60	1,21	1,58	2,03
20000	-1,30	0,47	1,06	1,33	1,60
30000	-1,24	0,30	1,05	1,27	1,51

Location 2b

	daily	1/1	1/10	1/100	1/200
No barrier	1,34	1,46	1,52	1,60	1,63
10000	-1,28	0,46	1,08	1,49	1,90
20000	-1,33	0,24	0,93	1,27	1,53
30000	-1,34	0,17	0,90	1,23	1,40

Location 3b

	daily	1/1	1/10	1/100	1/200
No barrier	1,34	1,46	1,52	1,60	1,63
10000	-1,30	0,15	0,97	1,47	1,85
20000	-1,46	-0,05	0,80	1,22	1,51
30000	-1,48	-0,18	0,65	1,13	1,38

Location 4

	daily	1/1	1/10	1/100	1/200
No barrier	1,34	1,46	1,52	1,60	1,63
10000	-0,79	0,35	1,13	1,71	2,18
20000	-1,28	-0,10	0,75	1,25	1,61
30000	-1,46	-0,35	0,52	1,06	1,38

- h<1m
- h<1,10m
- h<1,20m
- h<1,30m
- h<1,40m
- h>1,40m
- h>horginal

Figure M-3: results for a discharge sluices at 10 locations for 3 opening sizes and 5 design river discharges, 2010

Location 1

	daily	1/1	1/10	1/100	1/200
No barrier	1,49	1,64	1,70	1,78	1,83
10000	0,75	1,12	1,37	1,74	2,12
20000	0,71	1,09	1,29	1,54	1,76
30000	0,67	1,09	1,27	1,48	1,69

Location 2

	daily	1/1	1/10	1/100	1/200
No barrier	1,49	1,64	1,70	1,78	1,83
10000	0,81	1,10	1,32	1,66	1,97
20000	0,73	1,06	1,25	1,51	1,72
30000	0,73	1,05	1,24	1,47	1,62

Location 3

	daily	1/1	1/10	1/100	1/200
No barrier	1,49	1,64	1,70	1,78	1,83
10000	0,65	1,02	1,27	1,63	1,94
20000	0,58	0,96	1,18	1,47	1,68
30000	0,47	0,91	1,14	1,38	1,59

Location 1a

	daily	1/1	1/10	1/100	1/200
No barrier	1,49	1,64	1,70	1,78	1,83
10000	-0,76	0,99	1,49	2,18	2,89
20000	-0,92	0,72	1,30	1,59	1,90
30000	-0,92	0,58	1,30	1,49	1,81

Location 2a

	daily	1/1	1/10	1/100	1/200
No barrier	1,49	1,64	1,70	1,78	1,83
10000	-0,77	0,90	1,48	2,12	2,72
20000	-0,99	0,60	1,22	1,57	1,92
30000	-1,01	0,44	1,16	1,46	1,68

Location 3a

	daily	1/1	1/10	1/100	1/200
No barrier	1,49	1,64	1,70	1,78	1,83
10000	-0,74	0,73	1,44	2,06	2,61
20000	-1,05	0,37	1,12	1,56	1,91
30000	-1,13	0,20	1,02	1,44	1,67

Location 1b

	daily	1/1	1/10	1/100	1/200
No barrier	1,49	1,64	1,70	1,78	1,83
10000	-1,03	0,86	1,37	1,79	2,23
20000	-1,11	0,60	1,24	1,52	1,80
30000	-1,18	0,52	1,22	1,47	1,68

Location 2b

	daily	1/1	1/10	1/100	1/200
No barrier	1,49	1,64	1,70	1,78	1,83
10000	-1,07	0,68	1,27	1,69	2,08
20000	-1,13	0,44	1,14	1,46	1,72
30000	-1,14	0,40	1,10	1,43	1,60

Location 3b

	daily	1/1	1/10	1/100	1/200
No barrier	1,49	1,64	1,70	1,78	1,83
10000	-1,05	0,44	1,18	1,67	2,05
20000	-1,21	0,20	0,99	1,43	1,70
30000	-1,25	0,06	0,86	1,32	1,58

Location 4

	daily	1/1	1/10	1/100	1/200
No barrier	1,49	1,64	1,70	1,78	1,83
10000	-0,54	0,56	1,32	1,83	2,30
20000	-1,03	0,14	1,02	1,46	1,79
30000	-1,19	-0,09	0,83	1,27	1,58

h<1m
h<1,10m
h<1,20m
h<1,30m
h<1,40m
h>1,40m
h>horginal

Figure M-4: results for a discharge sluice at 10 locations for 3 opening sizes and 4 design flood waves, 2050

Assuming that the water levels in the city may not increase compared to the original situation due to the construction of a barrier, the opening size should be larger than 10.000 m^2 at all locations. At location 1A, 2A and 3A the opening size should even be larger than 20.000 m^2. Figure M-5 shows the results for the options that meet the previous assumption. In case of daily river discharge the water level requirement of h<MSL+1,0m is met for every option. It is clear that a discharge sluice is less effective in 2050, with sea level rise.

		2010					2050				
Location	Opening	daily	1/1	1/10	1/100	1/200	daily	1/1	1/10	1/100	1/200
No barrier	(m2)	1,34	1,46	1,52	1,60	1,63	1,49	1,64	1,70	1,78	1,83
1	20.000	0,51	0,90	1,11	1,31	1,57	0,71	1,09	1,29	1,54	1,76
1	30.000	0,51	0,91	1,13	1,27	1,50	0,67	1,09	1,27	1,48	1,69
1A	30.000	-1,17	0,37	1,12	1,27	1,60	-0,92	0,58	1,30	1,49	1,81
1B	20.000	-1,30	0,47	1,06	1,33	1,60	-1,11	0,60	1,24	1,52	1,80
1B	30.000	-1,24	0,30	1,05	1,27	1,51	-1,18	0,52	1,22	1,47	1,68
2	20.000	0,55	0,86	1,05	1,29	1,52	0,73	1,06	1,25	1,51	1,72
2	30.000	0,54	0,86	1,04	1,26	1,44	0,73	1,05	1,24	1,47	1,62
2A	30.000	-1,26	0,24	0,96	1,29	1,51	-1,01	0,44	1,16	1,46	1,68
2B	20.000	-1,33	0,24	0,93	1,27	1,53	-1,13	0,44	1,14	1,46	1,72
2B	30.000	-1,34	0,17	0,90	1,23	1,40	-1,14	0,40	1,10	1,43	1,60
3	20.000	0,35	0,73	0,95	1,23	1,47	0,58	0,96	1,18	1,47	1,68
3	30.000	0,30	0,68	0,93	1,17	1,39	0,47	0,91	1,14	1,38	1,59
3A	30.000	0,30	-0,04	0,82	1,23	1,50	-1,13	0,20	1,02	1,44	1,67
3B	20.000	-1,46	-0,05	0,80	1,22	1,51	-1,21	0,20	0,99	1,43	1,70
3B	30.000	-1,48	-0,18	0,65	1,13	1,38	-1,25	0,06	0,86	1,32	1,58
4	20.000	-1,28	-0,10	0,75	1,25	1,61	-1,03	0,14	1,02	1,46	1,79
4	30.000	-1,46	-0,35	0,52	1,06	1,38	-1,19	-0,09	0,83	1,27	1,58

h<1m
h<1,10m
h<1,20m
h<1,30m
h<1,40m
h>1,40m
h>horgineel

Figure M-5: results for a discharge sluice

M.2.2 Tidal barrier

A tidal barrier is a less extreme operational regime than a discharge sluice because there is also inflow into the system. A tidal barrier will always be less effective in reducing the upstream maximum water level compared to a discharge sluice. A tidal barrier has a big advantage compared to a discharge sluice, which is the type of structure. A structure which functions as a discharge sluice needs to be permanent because the in- and outflow need to be permanently controlled. A tidal barrier is closed when at a defined location the water level exceeds a certain value and is open if the water level is below that certain value. Therefore the structure can have a more temporary character. When the barrier is closed the river is blocked but when the barrier is open the cross section of the river is similar to the original situation. This has a big advantage for the navigation. However, this type of structures can only be applied when the length is in the order of a few kilometres. In this study a tidal barrier at location 1 was investigated. Location 1A and 1B were not investigated because the effect of secondary barriers is marginal. At all other locations the effect of a tidal barrier was not investigated because the lengths of the main barrier at those locations require a permanent structure. Therefore the advantage for navigation is no longer valid and a tidal barrier will only be a different kind of operational regime for the same kind of structure.

2010

	daily	1/1	1/10	1/100	1/200
no barrier	1,34	1,52	1,52	1,60	1,63
1,1-1,2	1,17	1,25	1,25	1,30	1,61
1,2-1,3	1,29	1,36	1,38	1,38	1,51

2050

	daily	1/1	1/10	1/100	1/200
no barrier	1,49	1,64	1,70	1,78	1,83
1,1-1,2	1,24	1,26	1,27	1,48	1,82
1,2-1,3	1,24	1,36	1,35	1,46	1,69

h<1m
h<1,10m
h<1,20m
h<1,30m
h<1,40m
h>1,40m
h>horginal

Figure M-6: results tidal barrier, location 1, A=30.000m2 for two controllers

Figure M-6 shows the results of a tidal barrier at location 1 with an opening size of 30.000 m^2. The barrier is triggered when the water level near the barrier exceeds a value of 1 m above MSL. At that moment the water level is measured and depending on the operational regime the barrier is closed. Figure M-6 shows the results for two controllers, a barrier which closes if the water level is 1,2 m above MSL and another which closes when the water level is above MSL+1,3 m. The type of controller determines the number of closings, see Figure M-7.

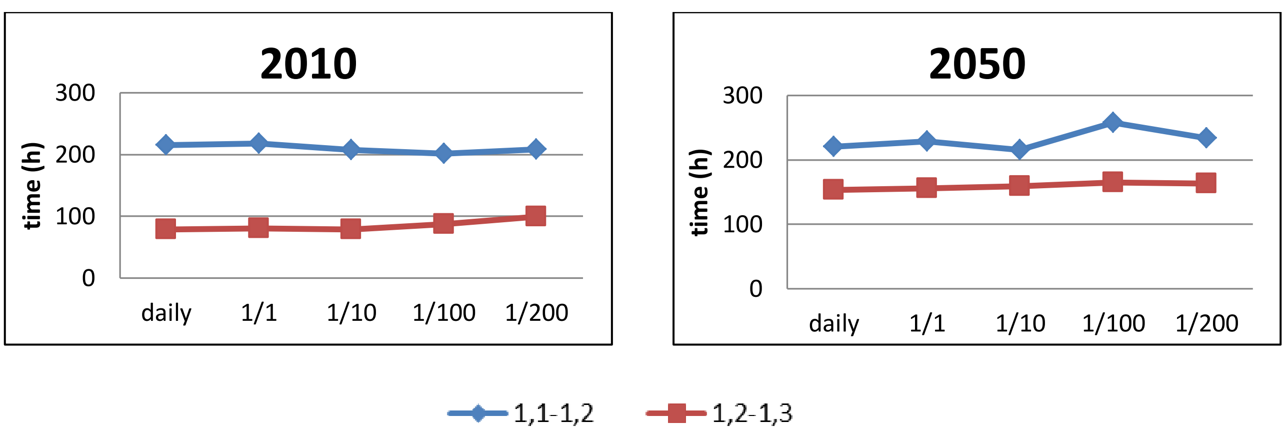


Figure M-7: number of hours the tidal barrier is closed

The results in Figure M-6 show that, even with a daily river discharge, the water level reduction due to a tidal barrier is too little to meet the requirement of h<MSL+1,0 m. Therefore a tidal barrier is not a good option in this situation.

Appendix N: max. flow velocity for navigation

The construction of a barrier downstream of HCMC will reduce the cross section at that location and therefore increase the flow velocity locally. The navigation opening in the structure can be made as a lock or as a permanent opening which can be closed when needed. In the case of a lock, the flow velocities in the lock are limited and are therefore not a problem. This is different in the case of a permanent opening. Due to the construction of a barrier the flow velocities increases locally, which can make safe navigation more difficult. There are no strict guidelines or PIANC publications regarding the maximum flow velocity which is allowed for safe navigation through a hydraulic structure. Based on reference projects (Thames barrier, Venice barrier, St. Petersburg barrier) can be concluded that the maximum allowable flow velocity for safe navigation through a hydraulic structure is 2 m/s for vessels of 100.000 DWT.

N.1 Thames barrier

One of the design criteria for the Thames barrier was unrestricted navigation within the main openings at any state of the tide (Holloway, *et al.,* 1987). This required a gate which, in the open position would not impede the river flow or river traffic. To avoid problems with the regime of the river, the requirement was laid down that the waterway cross section should not be reduced by more than 25% by the structure and during construction not more than 30% (Palmer, Trittor 1994 and Hormer 1987).

The navigational requirements for the Thames barrier were:
(i) Four 61 m (200ft) clear width main navigational openings.
(ii) Two subsidiary navigational openings of 31,5 m (103 ft) clear width.
(iii) Piers to be as narrow as possible consistent with their function.
(iv) Sills of the main openings to be at a level which conforms to the ‘ruling’ depth of the present channel and hence which did not restrict navigation in the Reach, or lead to excessive siltation (Holloway, Miller Richards and Draper, 1987).
A navigational depth of 9,25 m at mean tide level was used to match the guaranteed depth in the Woolwich Reach (Cox and Coombes, 1984).

There were no design criteria regarding the design vessel or the maximum allowed flow velocity. The maximum size of vessels which can be accommodated in the India and Millwall docks have a length of 166 m, a beam of 23,4 m and a draught of 8,0 m (National port council 1978). These docks are located just upstream of the barrier and it can be assumed that is the largest ship that can sail through the barrier (100.000 DWT). Before the barrier was constructed the width of the river at that location was 390 m and the flow velocity was 1,5 m/s (Kendrick, M.P., 1972). One of the requirements was that the cross section should not be reduced by more than 25%, which results in a flow velocity of circa 2 m/s. With this velocity ship navigation is possible. More reduction of the cross section will lead to higher flow velocities and can cause erosion problems.

N.2 Venice storm surge barrier

The Venice storm surge barrier exists of 3 barriers which can close of the three tidal inlets; Lido-San Nicolo (B=400 m, d=-11 m), Malamocco (B=380 m, d=-15 m) and Chioggia (B=360 m, d=-11 m). Also for the Venice storm surge barrier there is no criterion regarding the maximum allowed flow velocity for safe navigation trough the structure. However, there is a criterion for the maximum reduction of the cross section and the tidal prism which can lead a restriction of the average flow velocity.

‘The fixed structures of the barriers will produce a very limited permanent reduction in tidal volume, amounting to about 2%, because the barriers have been designed to avoid changing the cross section of the present channels, which are in morphdynamic equilibrium with the surface of the lagoon (the most relevant parameter affecting the scouring velocity on the sandy bottom of the inlets.’ (Cecconi 1998)

The maximum flow velocity before the construction of the barrier was 2 m/s (Gacic, M *et al,* 2002). The cross sections of the channels will not be changed by the barrier, and the tidal prism will be reduced with circa 2%. Consequently the flow velocity though the barrier will still be circa 2 m/s. A ship of 100.000 DWT can sail safely through the barrier.

N.3 St. Petersburg barrier

The St. Petersburg barrier is a barrier of 25 km with 6 sluice complexes, each with up to 12 steel radial gates 24m wide which allow water flow during normal conditions but can be closed in times of flood. There are also two navigation channels to accommodate marine traffic. The first of these is 200 m wide and 14 m deep and have the capacity to allow ships up to 100.000 TUE through. In order to close this channel, two of the largest hydraulic structures in the world are being built, floating radial steel gates with steel arms to a radius of 130 m each. The second navigation channel is 110 m wide and 6 m deep and will be closed by a single vertical lifting steel gate.

Information on the design criteria regarding the maximum allowed flow velocity for safe navigation through the navigation channels is hard to find. B. te Slaa, who is a Project Manager at Royal Haskoning and has been involved in the design of the barrier, estimated the maximum flow velocity to be 1,5 – 2,0 m/s for save navigation through the navigation opening of 200 m width. Mr. te Slaa assumed that navigation is stopped even before this velocity is reached because of simultaneously occurrence of wind and waves.

Appendix O: Impression lay out barrier

The model results, Appendix L show that for HCMC the best option is a main barrier which functions as a discharge sluice, independent of the location of the barrier. The results also show that the flow velocity through the barrier exceeds 2 m/s and therefore navigation via a permanent navigation opening (which can be closed when needed) is not a safe option and locks are needed. The required capacity of the locks depend on the number and type of vessels and thus on the location of the barrier. The barrier downstream of HCMC will exist of a closure dam, discharge sluices and locks. The design of the barrier is highly effected by the location of the barrier, in this feasibility study only the general lay out of the barrier will be described.

O.1 Closure dam

The length of the closure dam depend on the location of the barrier (varying from 500 m to 30 km). The design of the dam depend on the construction costs and method of closure, see Appendix P and Appendix R.


figure O-1: Saemangeum barrier South Korea and caisson closure

O.2 Discharge sluices

The model results from Appendix L show that the cross section of the opening is in the order of 20.000 m^2 to 30.000 m^2. The design of the discharge sluices is comparable with the discharge sluices of the Afsluitdijk in the Netherlands (25 sluices with a width of 12 m) and of the Saemangeum barrier in South Korea (2 sluices with a total width of 240 m and 300 m), see Figure O-2.

Figure O-2: discharge sluices of the Saemangeum barrier, South Korea and the Afsluitdijk, the Netherlands

O.3 Navigation locks

As described above, the capacity of the locks depend on the number and type of vessels and therefore on the location of the barrier. Figure O-3 show some examples of locks.

Figure O-3: navigation locks in Ijmuiden, the Netherlands and the Afsluitdijk, the Netherlands.

Appendix P: First costs estimation[1]

The costs of flood defences are mainly determined by five cost factors:

- Planning and engineering costs

This concerns the dike design and planning of the flood defence. In case of large uniform sections in rural areas, the unit costs may be low, while in residential areas with non-uniform conditions, the unit costs are relatively high.

- Material costs

The cost of materials is very site dependent. In deltaic regions, there is sometimes scarcity of construction material (e.g. clay in New Orleans; stones for revetments in the Netherlands). This highly influences the unit price and method of construction.

- Labour costs

The cost of labour varies a lot between countries. However, when the cost of labour is low, labour is more intensely used, while in the case of expensive labour, mechanized equipment is more widely applied.

- Costs for implementation in the environment

An important factor concerns the implementation of the flood defence in its environment. Two main factors are:

- Land use by flood defences. The required width of a flood defence usually increases with its height. The required amount of land has to be obtained, which could be financially, legally and challenging and thus a costly and time consuming task. However, in a rural environment, fewer challenges are expected.
- Rural or urban implementation. In an urban environment, space is usually scarce and space-saving solutions are needed for the implementation of flood defence projects. The solutions needed in urban environment (e.g. sheet piles) are usually more expensive than the relatively cheap rural purchases of land.

- Costs for management and maintenance

Organization is needed for management and maintenance of flood defences. This will result in an additional percentage of cost on the total expenses. The management and maintenance in the Netherlands is carried out by so-called Water Boards. In other countries there are usually also (semi-) governmental bodies for management and maintenance.

To give an indication: The yearly costs for management and maintenance for primary flood defences in the Netherlands is estimated to be approximately € 350 billion per year (AFPM, 2006). With a total length of

[1] *Coastal defence cost estimates; case study of the Netherlands, New Orleans and Vietnam*

primary flood defences of about 3600 km the estimated costs for management and maintenance becomes € 100.000 per km flood defence per year.

P.1 Comparison of costs between countries

As mentioned above, the different unit costs vary per country and per location. The differences between locations in a region will be largely determined by the exact design and the implementation. Country specific factors will be related to the local economic situation. The constitution of the categories to the unit price is likely to be different for each county. The costs for material and labour will also affect the selected design, the materials used and the construction method. In countries with low labour cost, and high material cost, another choice is made for e.g. dike revetments than in countries with low material and high labour cost.

The relative contributions may be compared. Though, in case of comparison per country, the development (GDP, specific education etc.) needs to be taken into account. This is illustrated by comparing the derived average unit cost prices for dikes to the FDP per capita (CIA World fact book; assuming exchange rate 1 Euro = 1,34 USD) for the Netherlands, New Orleans and Vietnam, Figure P-1. The figure shows that the estimated unit cost prices for Vietnam are relatively high in comparison with the GDP per capita.

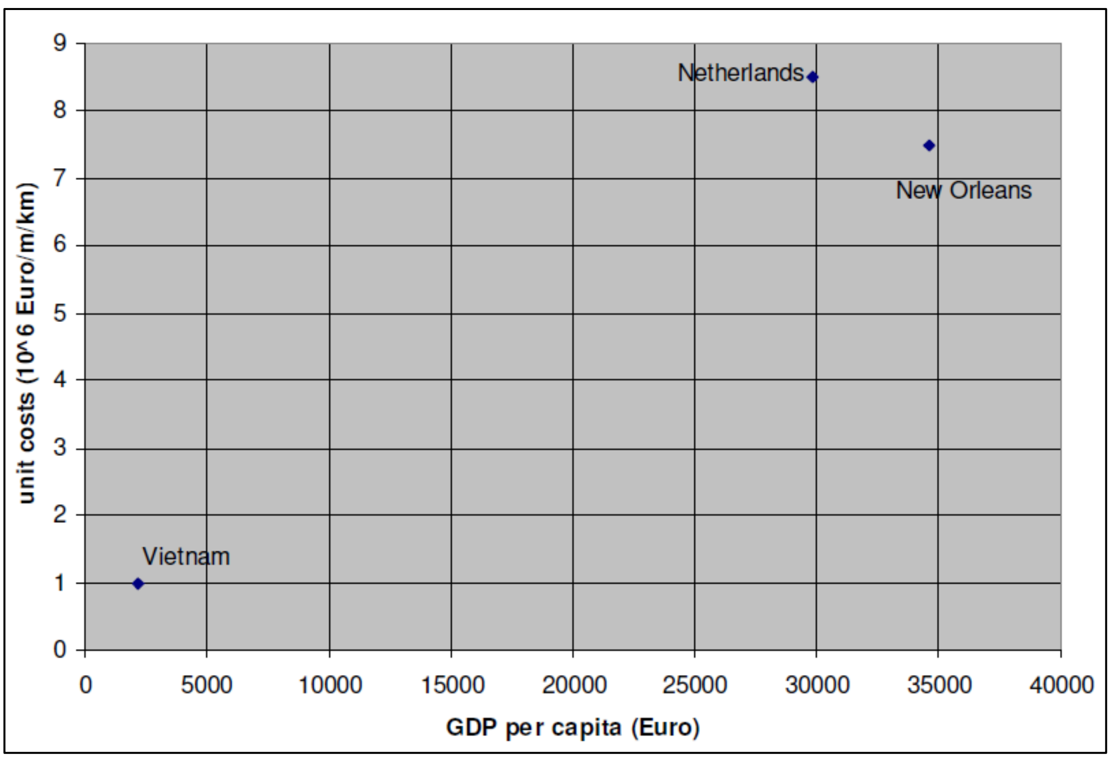


Figure P-1: comparison between the average unit cost prices for dikes and the relationship with the GDP per capita

P.2 Construction costs

The costs of a storm surge barrier depend on many factors, such as the type of barrier/gates, the local soil characteristics, the desired height and hydraulic head over the barrier. In the case of HCMC the barrier will exist of a closure dam, discharge sluices and navigation locks, the length of each section depends of the location. In order to make a first cost estimation a unit cost price of each section will be used.

P.2.1 Closure dam

The most recent and comparable project is the Saemangeum Seawall Project in South Korea. The completed barrier has a length of 33,9 km and two discharge sluices (width 240 and 300 m) which control the in- and outflow into the estuary. table P-1 shows the characteristics of the closure dams. It was found that a closure dam costs on average 39,6 B€/km. This is in line with the unit costs estimation of 24,3 – 40,6 B€/km (2009 price level) by IPCC CZMS (1990) for a study in the Netherlands.

	Length	Height	Dpth	Weight at bottom	M KRW	M€	M€/km (2009)
closure dam no 1	4694	20	102	201	2700	34,90	7,44
closure dam no 2	9936	35	9,6	290	754400	499,60	50,28
closure dam no 3	2693	16	8,5	198	37800	250,66	3,08
closure dam no 4	1.436	20	1	290	534800	354,17	3097
	2.8759					**1.139,32**	**39,62**

table P-1: characteristics sea dikes of the Saemangeum Seawall Project in South Korea

P.2.2 Discharge sluices

In various locations around the world storm surge barriers have been constructed. Storm surge barriers are often chosen as a preferred alternative to close of estuaries and reduce the required dike strengthening behind the dams. Another important characteristic is that they are often partly opened during normal conditions and this will allow the tide and saltwater to enter the areas behind the barrier. An overview of the main characteristics of storm surge barriers around the world is given in table P-2. The costs of a storm surge barrier depend on a broad number of physical parameters. It is proven that the largest impact comes from the water level difference over the barrier during closure (head difference) in addition to the total river depth and river width. The dimensions of the design rely on these parameters as the main forces during closure of the barrier can be found in a relation of these parameters. For instance, the foundation increases by the total weight of the structure, which is directly related to the river depth. Costs of the closure mechanisms, piers and abutments are related to the total height of these components and the forces they need to withstand, for which the head difference is a good measure. It was concluded that a fairly reasonable estimate of the capital costs can be made by relating the investment costs to the length * head difference * depth of a barrier. Results of this approach are given in table P-3. The height indicates the total depth in front of the barrier during maximum surge elevation. Taken all storm surge barriers into account, an average cost estimate of 23.900 €/m^3 is found.

Name barrier	Type	Year	Width [m]	Height [m]	Head [m]	Construction costs [M€]	Construction costs 2009 price level [M€]	Unit cost prices per cubic meter [1000 € /m3]
the Netherlands								
Maeslant barrier (New Waterway, Rotterdam)	Floating secto gate	1991	360	22	5	450*1)	656	16,57
Hartel barrier (Hartel channel)	Vertical lifting gates	1991	170	9,3	5,5	98*2)	143	16,45
Eastern Scheldt Barrier	Vertical lifting gates	1986	2400	14	5	2500*3)	4021	23,93
Ramspol (near IJssellake)	Bellow barrier	1996	240	8,2	4,4	100	132	15,24
Europe								
Ems (Germany)	Sector gates	1998	360	8,5	3,8	290	368	31,65
Thames (Great-Britain)	Sector gates	1980	530	17	7,2	800	1449	22,34
Venice MOSE project (Italy)	Flap gates	2010	3200	15	3	4678	4678	32,49
New Orleans								
Seabrook barrier (New Orleans)	Vertical lifting gates/sector gates	2010	130	8	4	114,7*4)	115	27,64
IHNC barrier (New Orleans) - only gates (excl. floodwall)	Sector gates	2010	250	12	6	518*5)	518	28,78

table P-2: overview storm surge barriers

Remarks table P-2:

1) Maeslant barrier has a relatively low cost price due to heavy competition for the contract.
2) The Hartel barrier has one very large horizontal span which increased the cost price.
3) The Eastern Scheldt barrier is relatively inexpensive due to tits repetitive character.
4) The Seabrook barrier (New Orleans) has two different types of gate in a small span.
5) From the IHNC/St. Bernard storm surge barrier only the parts containing the gates have been take into account, the floodwall was excluded.

P.2.3 Navigation locks

The capacity of the navigation locks depends on the location of the barrier (number of vessels) and type of vessels which have to navigate through the barrier. Currently a new lock at Ijmuiden, the Netherlans is being design and will be ready in 2016. The new lock will have a length of 500 m, a width of 65 m and a depth of 18 m, making it sutable for vessels up to 100.000 DWT. The construction costs are estimated at 750 million euro.

P.2.4 HCMC

The barrier downstream of HCMC will exist of a closure dam, discharge sluices and navigation locks. In order to make a first cost estimation it is assumed that the head difference over the structure is 3,5 m maximum when closed and that the depth of the barrier is for every location 15 m. Regarding the secondary barriers it is assumed that at location 1A and 1B the total length of the secondary barriers is 500 m and at location 2A, 2B, 3A and 3B the total length is 1 km. For location 4 a navigation lock similar to the new lock at Ijmuiden, Netherlands is needed. For location 1, 2, and 3 the number of vessels that have to navigate through the barrier is less and about have the capacity of the new lock at Ijmuiden is needed. Further study is needed to determine the numer and type of vessels in more detail. In this construction cost estimate extra measures that are needed for safe navigation though the barrier (besides locks) are not taken into account.

		length (km)			construction costs (M€)				total
Location	opening (m2)	closure dam	active part with gates	other	closure dam	active part with gates	locks	other	costs (B€)
1	20.000	1,5	2		59	2.510	375		2,9
1	30.000	0,5	3		20	3.764	375		4,2
1A	30.000	0,5	3	dams	20	3.764	375	15	4,2
1B	20.000	1,5	2	discharge sluice	59	2.510	375	359	3,3
1B	30.000	0,5	3	discharge sluice	20	3.764	375	359	4,5
2	20.000	5	2		198	2.510	375		3,1
2	30.000	4	3		158	3.764	375		4,3
2A	30.000	4	3	dams	158	3.764	375	31	4,3
2B	20.000	5	2	discharge sluice	198	2.510	375	717	3,8
2B	30.000	4	3	discharge sluice	158	3.764	375	717	5,0
3	20.000	22,5	2		891	2.510	375		3,8
3	30.000	21,5	3		852	3.764	375		5,0
3A	30.000	21,5	3	dams	852	3.764	375	31	5,0
3B	20.000	22,5	2	discharge sluice	891	2.510	375	717	4,5
3B	30.000	21,5	3	discharge sluice	852	3.764	375	717	5,7
4	20.000	30	2		1.188	2.510	750		4,4
4	30.000	29	3		1.149	3.764	750		5,7

table P-3: first cost estimated barrier downstream of HCMC

To determine the upper and lower bound to this cost estimate is to use the deviating of the average cost estimates from table P-2. For the closure dam and the navigation loccks the lower and upper bound is found assuming a fault barrier of 10% of the costs. For the discharge sluices a standard deviation of the 8 storm surge barrier of 6.000 €/m^3 is found. A lower bound of the cost estimate has been calculated with 17.900 €/m^3 (=23.900-6.000) and an upper bound with 29.900 €/m^3 (=23.900+6.000), see Figure P-2.

		total costs (B€)	
Location	opening (m2)	lower ban	upper band
1	20.000	2,28	3,6
1	30.000	3,18	5,1
1A	30.000	3,19	5,2
1B	20.000	2,54	4,1
1B	30.000	3,45	5,6
2	20.000	2,42	3,8
2	30.000	3,32	5,3
2A	30.000	3,34	5,3
2B	20.000	2,95	4,6
2B	30.000	3,85	6,2
3	20.000	3,11	4,4
3	30.000	4,01	6,0
3A	30.000	4,01	6,0
3B	20.000	3,65	5,3
3B	30.000	4,55	6,9
4	20.000	3,74	5,2
4	30.000	4,64	6,7

Figure P-2: lower and upper bound cost estimation

P.3 OMRR&R costs

The costs for operation, maintenance, repair, replacement and rehabilitation are important design consideration. The main OMRR&R will be:

- Maintenance of all the movable parts of the structure
- Painting of the structure to prevent corrosion
- Dedicated personnel
- Monitoring organization
- Measurement network
- Inspection of various construction parts including submerged parts of the structure

There is no general rule of thumb available but table P-4 shows an overview of the OMRR&R costs for three large storm surge barriers. It is assumed that the OMRR&R costs for a barrier downstream of HCMC will be in the order of 0,5% of the investment costs per year, see table P-5.

	capital costs	OMRR&R costs per year (M€)	Percentage (%)
Maeslant barrier	656	2	0,8
Eastern Scheldt barrier	4021	17,2	0,6
Thames barrer	132	5,4	0,5

table P-4: OMRR&R costs of various large storm surge barriers

		OMRR&R costs (M€/year)	
Location	opening (m2)	lower band	upper bad
	20.000	11,38	18,06
1	30.000	15,88	25,71
1A	30.000	15,95	25,79
1B	20.000	12,72	20,30
1B	30.000	17,23	27,95
2	20.000	12,08	18,75
2	30.000	16,58	26,40
2A	30.000	16,72	26,57
2B	20.000	14,76	23,24
2B	30.000	19,26	30,89
3	20.000	15,54	22,22
3	30.000	20,04	29,87
3A	30.000	20,04	29,87
3B	20.000	18,23	26,70
3B	30.000	22,73	34,35
4	20.000	18,71	25,76
4	30.000	23,22	33,42

table P-5: OMRR&R costs barrier downstream of HCMC

Appendix Q: Multi criteria analysis

The construction of a barrier downstream of HCMC has both positive and negative effects. In order to make a comparison between the possible locations a multi criteria analysis was made. The effect of the barrier is compared with the interests of the main stakeholders and a score is given. Also the construction cost of a barrier is taken into account.

Q.1 Flood problems

The main interest of HCMC is lowering the water levels of the rivers surrounding and crossing the city in order to reduce the flood problems. For this study it is assumed that there will be no flood problems in HCMS as long as the water levels in the river at all locations in the city do not exceed MSL +1,0m. Figure Q-1 shows the effect of a discharge sluice downstream of HCMC for different locations and opening sizes. A score is given for the reduction of the water level in the city.

		2010					2050					flood
Location	Opening	daily	1/1	1/10	1/100	1/200	daily	1/1	1/10	1/100	1/200	problem
No barrier	(m2)	1,34	1,46	1,52	1,60	1,63	1,49	1,64	1,70	1,78	1,83	
1	20.000	0,51	0,90	1,11	1,31	1,57	0,71	1,09	1,29	1,54	1,76	0
1	30.000	0,51	0,91	1,13	1,27	1,50	0,67	1,09	1,27	1,48	1,69	0
1A	30.000	-1,17	0,37	1,12	1,27	1,60	-0,92	0,58	1,30	1,49	1,81	1
1B	20.000	-1,30	0,47	1,06	1,33	1,60	-1,11	0,60	1,24	1,52	1,80	1
1B	30.000	-1,24	0,30	1,05	1,27	1,51	-1,18	0,52	1,22	1,47	1,68	1
2	20.000	0,55	0,86	1,05	1,29	1,52	0,73	1,06	1,25	1,51	1,72	0
2	30.000	0,54	0,86	1,04	1,26	1,44	0,73	1,05	1,24	1,47	1,62	0
2A	30.000	-1,26	0,24	0,96	1,29	1,51	-1,01	0,44	1,16	1,46	1,68	2
2B	20.000	-1,33	0,24	0,93	1,27	1,53	-1,13	0,44	1,14	1,46	1,72	2
2B	30.000	-1,34	0,17	0,90	1,23	1,40	-1,14	0,40	1,10	1,43	1,60	2
3	20.000	0,35	0,73	0,95	1,23	1,47	0,58	0,96	1,18	1,47	1,68	2
3	30.000	0,30	0,68	0,93	1,17	1,39	0,47	0,91	1,14	1,38	1,59	2
3A	30.000	0,30	-0,04	0,82	1,23	1,50	-1,13	0,20	1,02	1,44	1,67	2
3B	20.000	-1,46	-0,05	0,80	1,22	1,51	-1,21	0,20	0,99	1,43	1,70	3
3B	30.000	-1,48	-0,18	0,65	1,13	1,38	-1,25	0,06	0,86	1,32	1,58	3
4	20.000	-1,28	-0,10	0,75	1,25	1,61	-1,03	0,14	1,02	1,46	1,79	2
4	30.000	-1,46	-0,35	0,52	1,06	1,38	-1,19	-0,09	0,83	1,27	1,58	3

Figure Q-1: results discharge sluice and score on flood problem

3	1/10 year; 2010 & 2050
2	1/10 year; 2010
1	1/1 year; 2010 & 2050
0	1/1 year; 2010

Figure Q-1 shows that in case of dialy river discharge every option reduces the water level so that the requirement of h<MSL+1,0m is met. 0 is given if the requirement is met with a 1/1 year flood wave and the tidal movement in 2010. 1 is given if the requirement is met with a 1/1 year flood wave in 2010 and 2050 (SLR). 2 is given is the requirement is met with a 1/10 year flood wave in 2010 and 3 is given if the requirement is met with a 1/10 year flood wave in 2010 and 2050. Figure Q-1 shows that the effect of the barrier increases if it is constructed more seaward.

Q.2 Agriculture

The farmers of the Dong Thap Muoi region as well as the producers of drinking water have interest in reducing the salt intrusion in the Vam Co and the Saigon. The effect of the barrier on the salt intrusion depends on the location of the main barrier and secondary barriers. In case of location 1, 2 and 3 there are no secondary barriers constructed and therefore salt water from the sea is able to flow into the system. In all other cases a fresh water situation is created.

Location	opening	Interest agriculture
No barrier	(m2)	
1	20.000	0
1	30.000	0
1A	30.000	3
1B	20.000	3
1B	30.000	3
2	20.000	0
2	30.000	0
2A	30.000	3
2B	20.000	3
2B	30.000	3
3	20.000	0
3	30.000	0
3A	30.000	3
3B	20.000	3
3B	30.000	3
4	20.000	3
4	30.000	3

3	fresh water situation
2	
1	
0	no change
-1	
-2	
-3	salt water situation

figure Q-2: score agriculture

Q.3 Nature

In order to survive the natural tidal and salinity range is very important for a mangrove forest. Depending on the location of the main barrier and the presence of secondary barriers the natural situation will be changed. Figure Q-3 shows the scores. In case of location 1, 2 and 3 there are no secondary barriers constructed and therefore the natural tidal and salinity range is not changed that much. In case of 1A and 1B the location of the secondary barriers is less seaward, resulting in less change compared to 2A, 2B, 3A and 3B. In case of location 4 in entire in- and outflow into the entire estuary is controlled and a fresh water situation is created.

Location No barrier	opening (m2)	Interest nature
1	20.000	0
1	30.000	0
1A	30.000	0
1B	20.000	0
1B	30.000	0
2	20.000	-1
2	30.000	-1
2A	30.000	-2
2B	20.000	-2
2B	30.000	-2
3	20.000	-1
3	30.000	-1
3A	30.000	-2
3B	20.000	-2
3B	30.000	-2
4	20.000	-3
4	30.000	-3

3	increase of natural tidal and salinity range
2	
1	
0	no change
-1	
-2	
-3	decrease of natural tidal and salinity range

Figure Q-3: score nature

Q.4 Navigation

It is of the stakeholders' interest that delay due to the construction of a barrier is being avoided. Because the flow velocity through the barrier exceeds the value of 2 m/s it is not possible to make a permanent navigation opening and a lock is needed, see Appendix N. Queues should be avoided and therefore the number of locks should be sufficient. Plans to expand port activities towards the sea are not taken into account.

In case of location 4 all vessels have to navigate through the barrier and therefore this option is the least feasible for this stakeholder, Figure Q-4. In case of location 1, 2 and 3 there are no secondary barriers constructed and the situation in the Long Tau has not changed.

Location	opening	Interest navigation
No barrier	(m2)	
1	20.000	-1
1	30.000	-1
1A	30.000	-2
1B	20.000	-2
1B	30.000	-2
2	20.000	-1
2	30.000	-1
2A	30.000	-2
2B	20.000	-2
2B	30.000	-2
3	20.000	-1
3	30.000	-1
3A	30.000	-2
3B	20.000	-2
3B	30.000	-2
4	20.000	-3
4	30.000	-3

3	decrease of handling time
2	
1	
0	no change
-1	
-2	
-3	increase of handling time

Figure Q-4: score navigation

Q.5 Infrastructure

If in the design of a barrier downstream of HCMC a road is included this can reduce the traffic problems in the city. Location 4 gives the most improved connection and this reduces going more landward, see Figure Q-5. The score only depends on the improvement of the infrastructure as result of construction of the barriers, so the effect of a possible bridge is not taken into account.

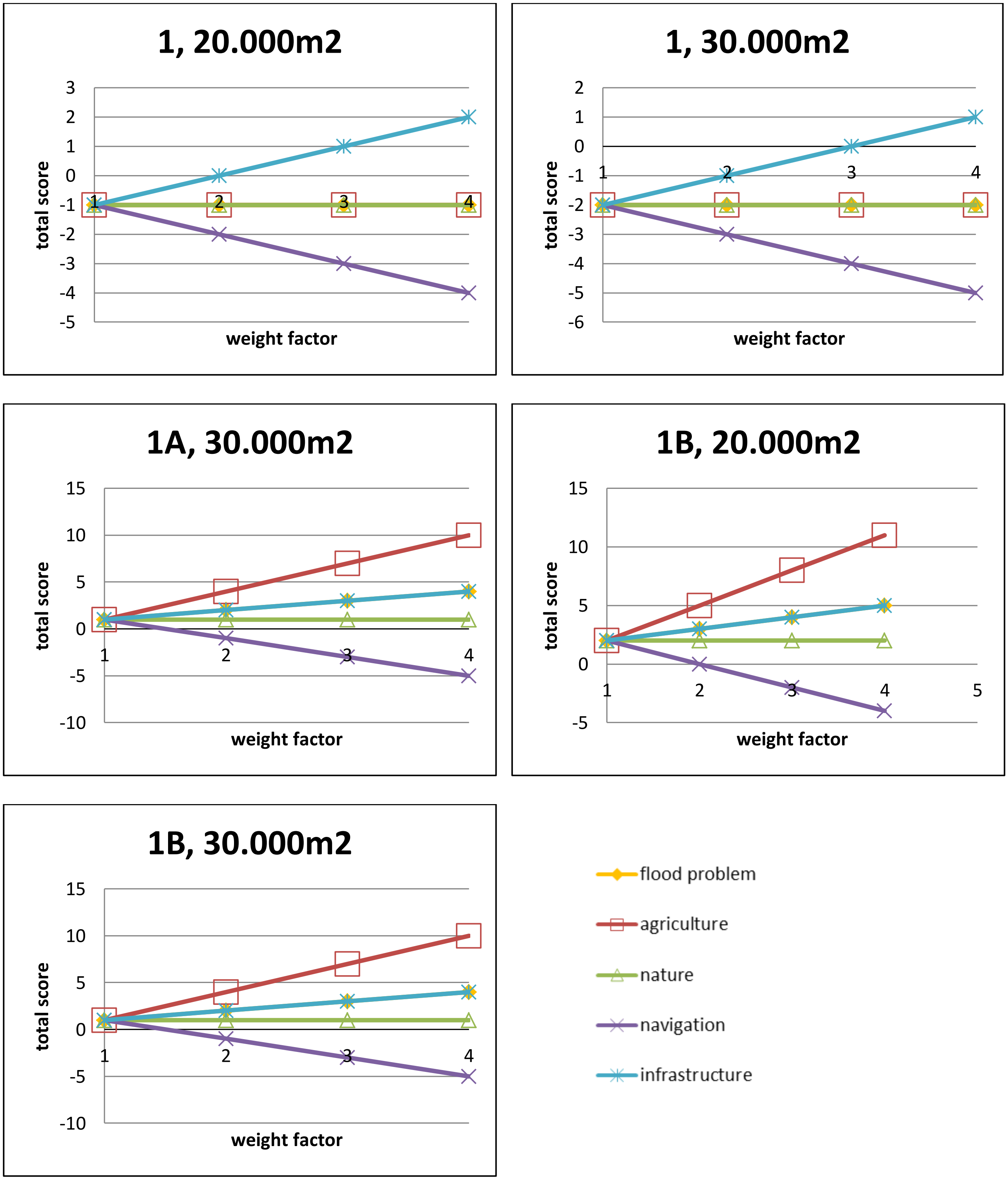
Location	opening	Interest infrastructure
No barrier	(m2)	
1	20.000	1
1	30.000	1
1A	30.000	1
1B	20.000	1
1B	30.000	1
2	20.000	1
2	30.000	1
2A	30.000	1
2B	20.000	1
2B	30.000	1
3	20.000	2
3	30.000	2
3A	30.000	2
3B	20.000	2
3B	30.000	2
4	20.000	3
4	30.000	3

3	improved connection
2	
1	
0	no change
-1	
-2	
-3	deteriorated connection

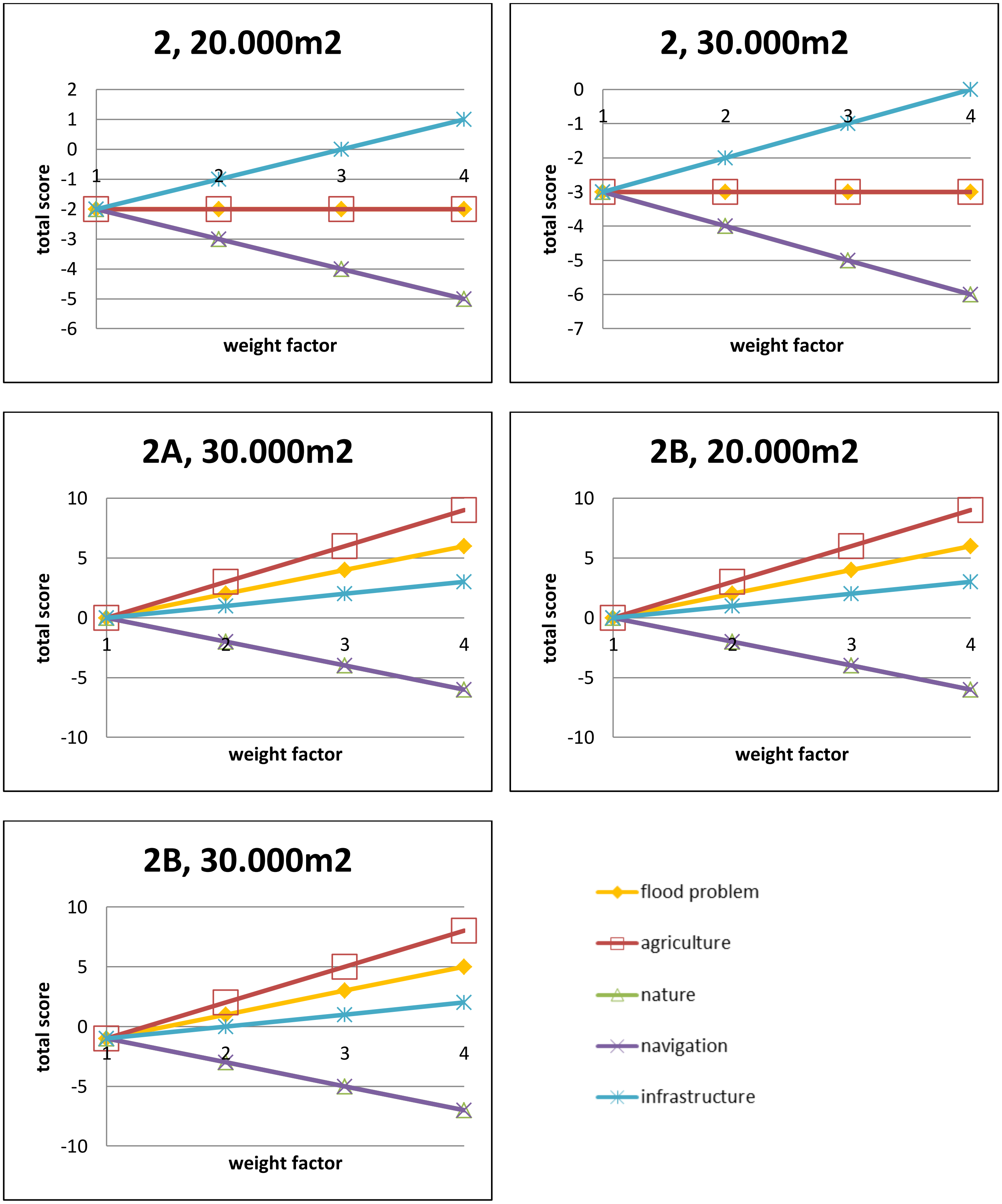
Figure Q-5: score infrastructure

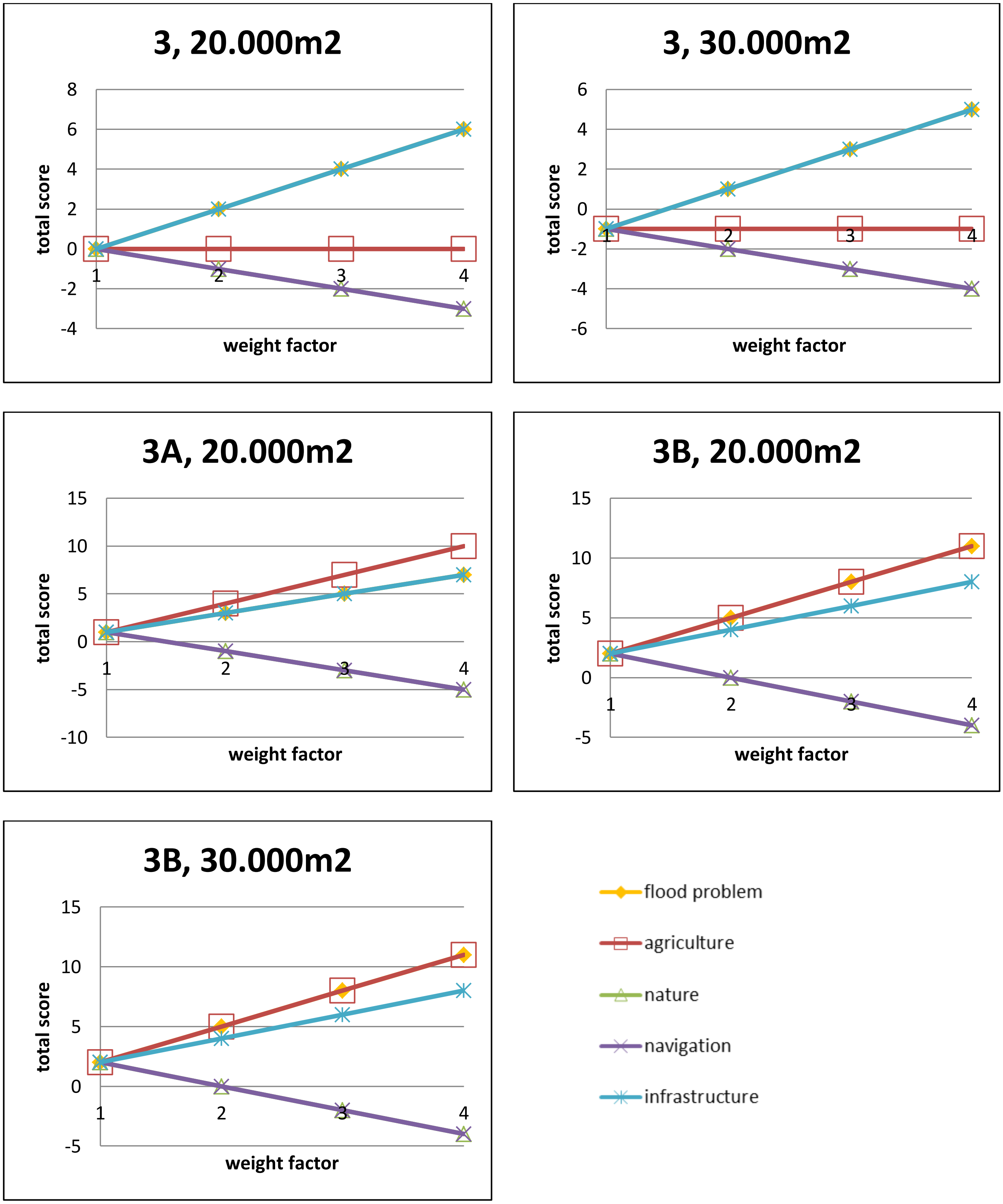
Q.6 Sensitivity analysis

The total score for each barrier option depends on the weight factor given to each interest and to the estimated construction costs. For each barrier option a sensitivity analysis was made. The construction of a barrier can have a positive effect on solving the flood problem, solving the salt intrusion problem for the agriculture and improving the infrastructure. At the same time it can have a negative effect on nature and navigation. For the total score of each barrier option it is beneficial to be sensitive for the interests flood problem, agriculture and infrastructure and be insensitive for the interests nature and navigation. Each barrier option has of course its own estimated construction costs which increases going more seaward.



1, 20.000m2
total score
weight factor
1, 30.000m2
total score
weight factor
1A, 30.000m2
total score
weight factor
1B, 20.000m2
total score
weight factor
1B, 30.000m2
total score
weight factor
flood problem
agriculture
nature
navigation
infrastructure


2, 20.000m2
2, 30.000m2
2A, 30.000m2
2B, 20.000m2
2B, 30.000m2
total score
weight factor
flood problem
agriculture
nature
navigation
infrastructure


3, 20.000m2
total score
weight factor
3, 30.000m2
total score
weight factor
3A, 20.000m2
total score
weight factor
3B, 20.000m2
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weight factor
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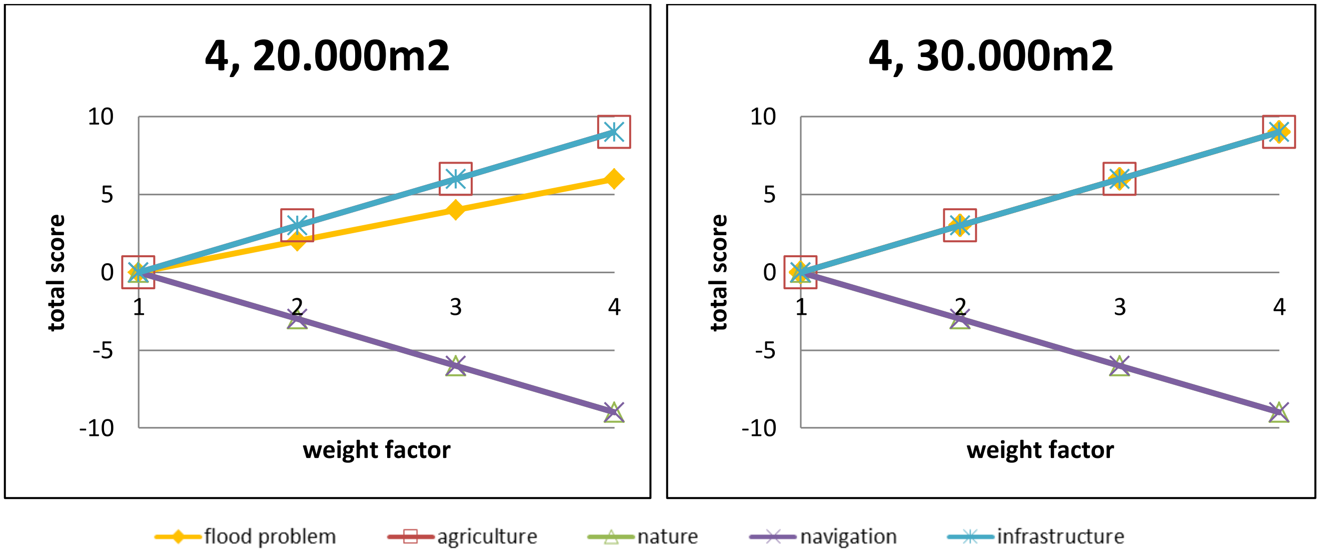
total score
weight factor
flood problem
agriculture
nature
navigation
infrastructure


4, 20.000m2
total score
weight factor
4, 30.000m2
total score
weight factor
flood problem
agriculture
nature
navigation
infrastructure

Appendix R: Methods of closure

The method of constructing a closure dam is related to the used equipment, which is either land based or water-borne. A distinction can be made between horizontal or vertical closure and the possible combination of these methods. Horizontal closure can be done with large elements (caissons).

There are two basic methods of closure:

- Gradual closure

Relatively small size, flow resistant material is progressively deposited in small quantities into the flow until complete blockage is attained. This can be done vertical (horizontal layers), horizontal (sideway) or a combination (sill and then sideway).

- Sudden closure

Blocking of the flow in a single operation by using pre-installed flap gates, sliding gates or by the placing of a caisson or vessel.

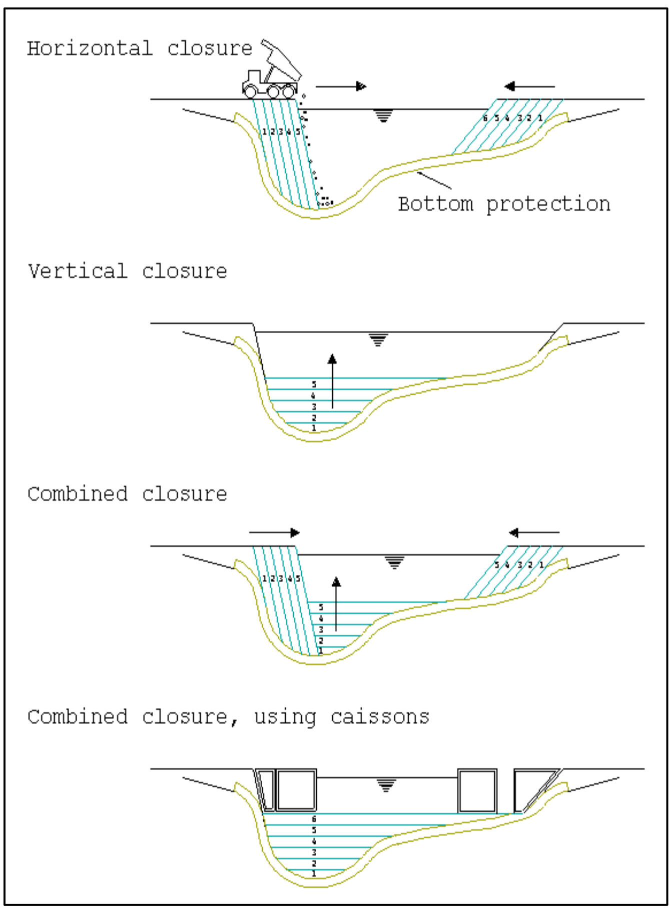


Figure R-1: basic methods of closure

Methods of closure may also be distinguished according to the topography of the gap to be closed:

- Tidal gully closure. Closure of a deeply scoured channel in which high flow-velocities may occur.
- Tidal-flat closure. Closure across a shallow area that is generally dry at low water. This is characterized by critical flow at certain tidal levels.
- Reservoir dam, used in mountainous area. This requires temporary diversion of the flow in order to obtain solid foundation in the riverbed at bedrock level.

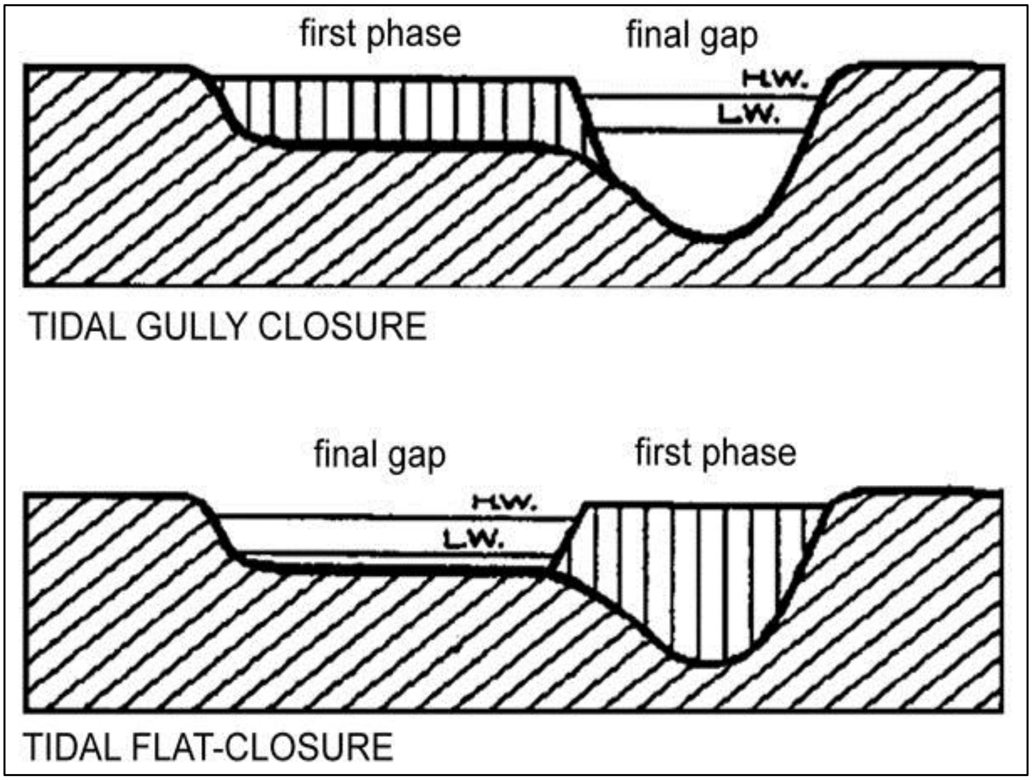


Figure R-2: closure named after topography

It depends on the hydrologic conditions which type of closure is used:

- Tidal-basin closure: characterized by regularly changing flow directions and still water in between; mainly determined by the tidal volumes and the storage capacity of the enclosed basin.
- Partial tidal closure: a closure in a system of watercourses, such that after closure there is still a variation in water-level at both sides of the closure dam.
- River closure (non-tidal): closure determined by upland discharge characteristics and backwater curves.

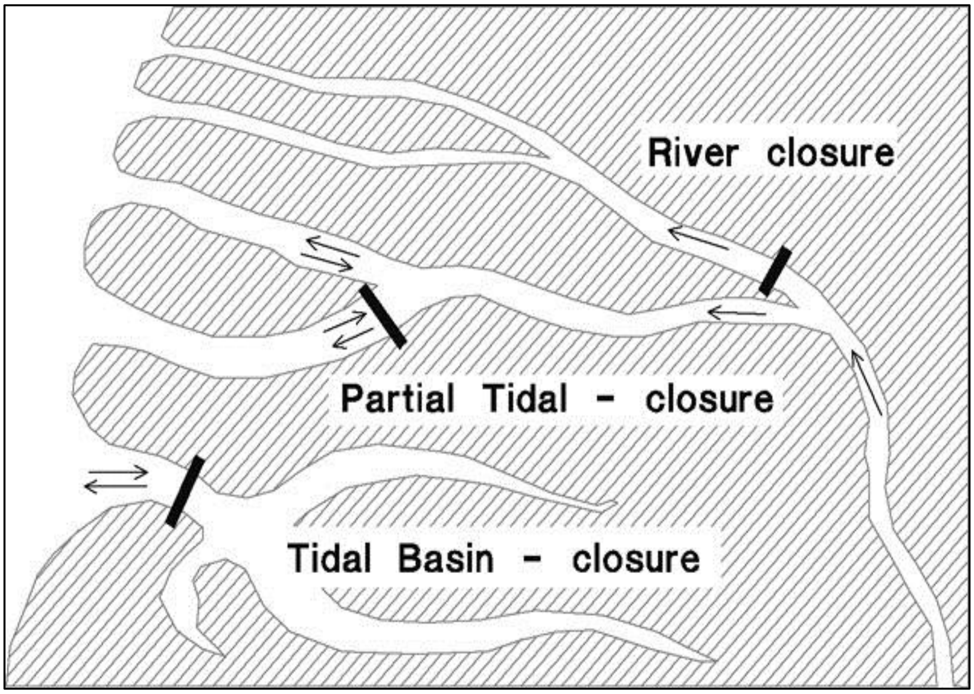


Figure R-3: closure named after hydrologic conditions

The materials used, which may vary according to the method of closure:

- Stacking-up mattresses: closure realized by successively dropping mattresses (made of willow or bamboo faggots, ballasted by clay or cobbles) onto each other.
- Sand closure: closure realized by pumping sand at a very high rate of production.
- Clay or boulder-clay closure: lumps of flow-resistant clay.
- Stone-dam closure: closure realized by dumping rock, boulders or concrete blocks in the gap, either by using dump-barges and floating cranes, or by cableway.
- Caisson closure: closure by using large concrete structure or vessels, floated into position and then sunken in the gap (possibly provided with sluice gates).

Special circumstances leading to typical closure types:

- Emergency closure in characterized by improvisation. The basic idea is that quick closure, even at the high risk of failure, prevents escalation of conditions. The method is mainly used for closing dike breaches quickly which may require strengthening afterward.
- Temporary close is used to influence the conditions elsewhere; for instance, by stepwise reduction of the dimensions of the basin. This type of closure needs to be sufficiently strong during the required period but is easily removable afterward.

R.1 HCMC

In the case of HCMC a tidal basin will be closed off. The barrier will exist of a closure dam and a dynamic part with gates, the length of each section depends of the location. The construction state starts with making a construction pit for the dynamic part of the barrier. From a constructional point of view, building on shoals will always be preferable as the costs will be lower and there will be little disturbance of the tidal action. However, for the final stage additional costs will be incurred for dredging the approach channels. Investigations and study of hydrological and geological data are necessary to determine the best location of the construction pit, construction methods and ways of drainage the construction pit.

There are several methods to construct the closure dam part of the barrier. When building the closure dam the in- and outflow through the narrowed gap will be reduced, causing a decrease in tidal range in the basin, but higher current velocities through the remaining opening. Consequently, the scouring effect on the bottom near the dam will be increased, which endangers the stability of the riverbed and thus the foundation of the structure. In the case of HCMC the bottom consists of easily erodible material and protection by current-resistant material is needed. At the beginning of the construction the velocities will be usually low; therefore, relatively cheap materials may initially be used to narrow the gap (sand or small-sized rubble or clay if possible and available). The gap is narrowed by building out from the sides and by heightening the sill. If the velocities become higher than 2 to 3 m/s, heavier material must be used. For this 3 types of closures are possible: gradual closure, sand closure or sudden closure. The dynamic part of the barrier should be designed and constructed in such a way that after the construction the barrier can be set open so that the velocities during the construction of the closure dam are lower.