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Development of a methodology for a preliminary design of port and harbour layout

Toon Laureyssens
Student number: 01502464

Supervisor: Prof. dr. ir. Peter Troch
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Master's dissertation submitted in order to obtain the academic degree of
Master of Science in Civil Engineering

Academic year 2019-2020

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Toon Laureyssens, May 2020

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31 May 2020

Development of a methodology for a preliminary design of port and harbour layout

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Abstract

The optimal layout of a harbour is difficult to determine due to the complex conditions and the interaction between the typical design aspects. Because of the complexity, the various disciplines involved and the design aspects to consider during the design process, only general guidelines, practical recommendations or “rule of thumb” regarding the layout of the harbour are present in literature. Currently there is no such thing as a consistent methodology which can be applied during the preliminary stage of a harbour layout design. The main objective of this master thesis is to identify the main design aspects to be considered during the preliminary phases of port planning and harbour layout design. Based on these design aspects, a methodology is proposed which can be used in a general case to define the harbour layout and to find various alternative harbour configurations. This methodology is then implemented in a tool which can be a useful instrument for harbour and port engineers during the preliminary phase of the master plan and port layout design. In a short period of time, the tool can provide different alternative harbour configurations which need to be optimised in a further design stage. The tool investigates the validity of the proposed solutions, including its benefits and possibilities. Finally, the tool performs a fast sensitivity analysis on the influence of the main geometrical aspect of the harbour considering the analysis on the wave agitation inside the port, navigational aspect and costs for the different proposed alternatives.

Keywords: Harbour layout, port planning, preliminary design, breakwaters, design aspects, methodology, engineering tool.

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***Abstract* — Nowadays, due to its complexity and multidisciplinary aspects involved in, there are only general guidelines and practical advices in literature for preliminary design stages of port planning and harbour layout design. Therefore, there are no sequential and integrated approaches that include the several steps needed to perform an adequate preliminary planning and design of the harbour layout. The present paper describes the development of an integrated approach that can assist engineers during the preliminary phase of harbour layout design. The design aspects and the sequential approach to consider during the preliminary design of a harbour layout are identified and used to develop a methodology integrated with an engineering tool. This tool can be used during the primary stages of layout design to generate and evaluate various alternative configurations in a relative short period of time before the use of advanced numerical models.**

***Keywords* — Harbour layout, port planning, preliminary design, breakwaters, design aspects, methodology, engineering tool.**

I. Introduction

The design of port and harbour layout is a complex activity. The design aspects involved are both multidisciplinary and complicated, resulting often in general guidelines, practical recommendations or "rule of thumb" regarding the layout of the harbour present in the literature. Therefore, there are no sequential and integrated approaches that include the several steps needed to perform an adequate preliminary planning and design of the harbour layout.

The main objective of this master's dissertation is thus to develop one consistent methodology to follow for a preliminary design of a harbour layout based on the current state-of-the-art. To do this, the main design aspects to consider during port planning and the design of the harbour layout are identified. Based on the proposed sequential approach, an engineering tool is developed, which can assist harbour and coastal engineers during the different steps of the port layout, before using physical or numerical modelling. This tool suggests different alternative harbour configurations in a relative short computation time. These alternatives are then evaluated and compared, and should be further optimised during the next, more detailed design stages. This paper mainly focusses on the preliminary design phase of the harbour layout. During this phase, the designer gives form to the harbour layout in general and several alternative harbour configurations are generated and analysed considering various design aspects.

II. Harbour Configuration

The layout of the harbour mainly consists of the protective breakwater structures surrounding the harbour basin. These breakwaters provide an area of water which is sheltered during storm events and/or accommodate economical facilities [1]. Figure 1 shows an example of a possible harbour layout.

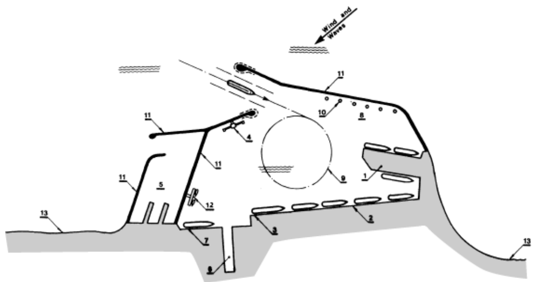

Figure 1 Example of a harbour layout [2]

The majority of harbours have one of those typical breakwater layouts: converging breakwaters with or without inner breakwaters, a coastline parallel attached breakwater with or without a secondary breakwater, a detached breakwater or a river harbour layout [3]. These typical harbour configurations are shown in Figure 2.

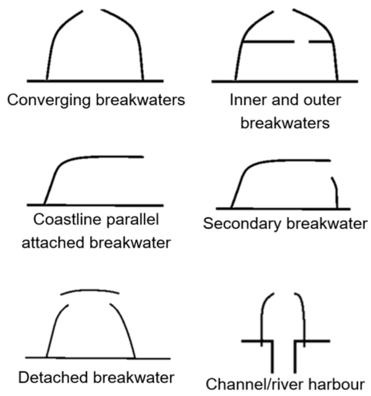

Figure 2 Typical harbour configurations

III. Design Aspects

The main design aspects to be considered during the preliminary design of a harbour layout are here identified and discussed. Based on these relevant design aspects, a methodology is proposed and integrated with an engineering tool.

A. Port location

The port location, or port category, is a classification of the harbour based on its geographical location relative to the shoreline [1][2]. Some harbour locations have geographical features which already provide limited wave sheltering and reduce the need of artificial breakwaters. Such location is for example a bay or a lagoon, which can compromise the choice of specific harbour configuration.

B. Metocean data

The Metocean data consists of the wave, wind and current conditions at the harbour locations. These conditions affect the navigation of the vessels, the wave penetration and agitation into the harbour basin and the operation of the vessel at the quay wall. The mooring conditions indicate the limiting wave heights, up to which a safe vessel operation in the harbour basin is ensured. Here, two different situations can be distinguished. In a first situation, the vessel has to stop its operations such as (un)-loading but can remain berthed at the quay wall. This is called the operational limit. A second situation, where the vessels can't stay moored and have the leave the harbour basin to fly to open sea, is called the limit state condition [4].

C. Navigation

A vessel entering in the harbour has to navigate itself safely towards the vessel berths. Therefore, enough space, called manoeuvring areas, needs to be available. The manoeuvring areas to be specified in the design are further explained. The *approach channel* is any stretch of channel, inside or outside the harbour basin, that connects the open sea with the inner harbour and turning basin. The vessels enter the harbour basin through the *harbour entrance* which needs to provide a safe access. After entering the harbour basin, the vessel needs to reduce its speed before berthing, this stopping manoeuvre is performed in the so-called *stopping area*. To align itself with the berths, a turning manoeuvre is performed inside the *turning basin*. Finally, the *berthing area* provides space to berth [1][2][5].

D. Morphological aspects

The construction of any coastal structure, including breakwaters, influences the sediment transport along the shoreline. The breakwater structures disturb a balanced situation and due to this, large accretion or erosion can take place at neighbouring sites [6][7].

Three different processes affect the harbour layout [4]:

1) *Littoral transport*
2) *Siltation of the approach channel*
3) *Sediment transport into the harbour*

E. Bathymetry

The bathymetry at the harbour location has a large effect on the wave propagation processes, the port category, the selection and alignment of breakwaters. Several guidelines advice to reduce the depth along the breakwaters and to follow the bathymetric contour lines, in order to avoid excessive construction material [8]. Furthermore, the depth often influences the selection of the type of breakwater, as for small to medium water depths a rubble mound breakwater is generally more advantageous from an economic point of view, while at larger depths vertical caisson breakwaters are more optimal [9].

F. Geotechnical aspect

The geotechnical characteristics of the subsoil below the breakwater structures are important for the stability and the planning of dredging operations. It is better to avoid subsoils with significant settlement, liquefaction or sliding [10]. Moreover, rubble mound breakwaters are less sensitive to differential settlement of soft subsoils compared to vertical breakwaters, which need sufficient replacement of subsoil, or other foundation improvements. Note that at this stage of development of the tool, it is assumed that the subsoil conditions below the breakwater are adequate to construct both types of breakwaters. However, it is worth underlying that this design aspect is crucial during a more detailed design phase of the harbour layout.

G. Preliminary cost

The cost of the breakwater structures is related to all previous mentioned design aspects and is thus determined by many different parameters. However, not all parameters can be assessed during the preliminary design of the harbour layout. Important parameters for defining the preliminary cost of the harbour project are the breakwater construction costs and the dredging costs. This preliminary cost can be determined for each alternative harbour configurations and can be used to evaluate and compare them.

IV. Methodology and Tool

Based on the selected main design aspects, a methodology is suggested and implemented in a tool. The engineering tool is a combination of an Excel spreadsheet and a MATLAB script and uses the proposed methodology to generate and evaluate multiple harbour configurations in a short period of time.

First, the required input parameters are entered in the tool, such as the harbour requirements, the bathymetry data, the Metocean data, the design vessel, etc.. Based on these input parameters the tool determines the harbour dimensions and an initial harbour layout is proposed by the tool. In the next step, the tool generates multiple alternative configurations and plots them together with the initial configuration. These initial steps are shown in Figure 3.

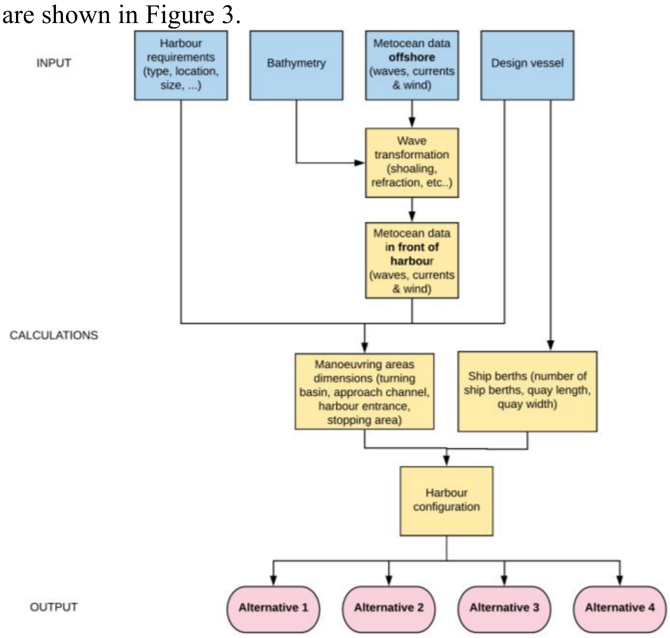

Figure 3 Scheme of tool (part 1)

After generating and plotting the alternative harbour configurations, the tool computes the wave agitation inside the harbour basins using diffraction, reflection and transmission calculations. Further, the tool determines a preliminary cost indication based on the breakwater costs and dredging needs.

The harbour configurations are then evaluated and compared based on three criteria: the preliminary costs, the navigational aspect and the wave agitation inside the harbour basin. These final steps are shown in Figure 4.

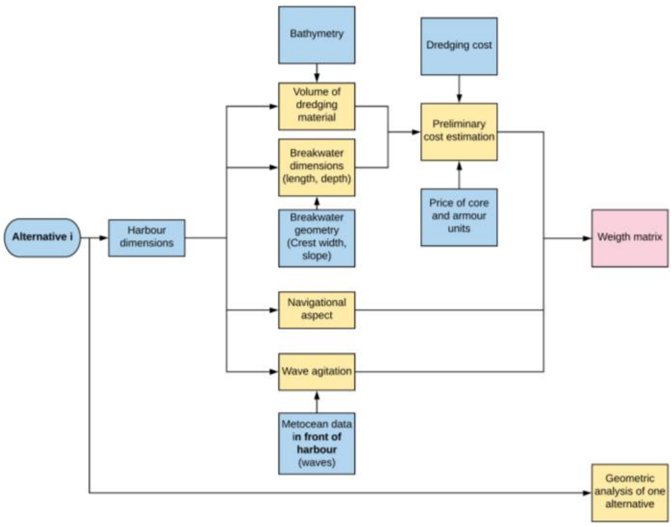

Figure 4 Scheme of tool (part 2)

A. Input of the tool

As a starting point, a series of the input parameters need to be collected and entered in the tool. Here, the tool requests the detailed harbour requirements including the harbour type, the port location, the number of ship berths and the presence of tugboats.

To determine the dimensions of the manoeuvring areas and the mooring conditions, the characteristics of the design vessel are requested. Further, the Metocean conditions need to be entered. Therefore, the wave conditions in front of the harbour, such as the significant wave height H_s, the peak wave period T_p, the water depth d and the wave direction θ are needed as input. If wave conditions in front of the harbour are not available, a simplified method is included in the tool in order to define those conditions based on the deep-water wave conditions offshore of the port.

Additionally, the conditions in the approach channel, such as the crosswind velocity V_{cw}, the cross-current velocity V_{cc} and the longitudinal current velocity V_{lc} are requested. Further, information on water level, the sediment transport, the bathymetry, the breakwater requirements and unit price for breakwater design and dredging operation are requested by the tool in order to estimate the preliminary cost of the different alternatives.

B. Calculations

The present section describes the most important design steps and calculations implemented in the tool.

1) Metocean conditions

To determine the main dimensions of the harbour and compute the wave conditions in front of the harbour, calculations are done regarding the wave, wind and current conditions. When wave conditions are known in deep water, the wave transformations due to bathymetry and currents need to be estimated. To estimate the wave shoaling and refraction processes, the method based on the linear wave theory as proposed by CEM [11] and Sorensen [12] is implemented in the tool. The wave height H will increase together with the steepness s due to wave shoaling. However, this is limited by the water depth d and the wavelength L. This maximum wave height is called the breaking wave height H_{br}.

Memos [2] and Ligteringen [4] suggest to locate the breakwaters past the breaker zone, the zone where waves are breaking, in order to limit the sediment transport and the load-impact of waves. The breaker zone can be estimated by combining the shoaling, refraction and breaking calculations following a method proposed by Kamphuis [6]. Therefore, Eq. 1 and Eq. 2 are implemented in the tool.

$$\gamma_b d_{br} = H_0 \sqrt{\frac{1}{\left(1+\frac{4\pi d_{br}/L}{\sin(4\pi d_{br}/L)}\right)\tanh(2\pi d_{br}/L)}}\sqrt{\frac{\cos\alpha_0}{\cos\alpha_1}} \tag{1}$$

$$x_{br} = m\, d_{br} \tag{2}$$

where γ_b is the breaker index, d_{br} is the depth at breaking, H_0 is the deep-water wave height, L is the wavelength, α is the angle between the wave crest and straight bottom contour, x_{br} is the distance form the shoreline to the breaker zone and m is the beach slope. Please note that the method provides a rough estimation, as it assumes parallel and straight bottom contours and should then also be applied with caution in case of more complex bathymetries.

2) Harbour dimensions

The harbour dimensions are defined by the manoeuvring areas dimensions, the minimum length of the quay wall and the alignment of the breakwaters. The required harbour dimensions computed by the tool with a simplified representation of the harbour layout are shown in Figure 5.

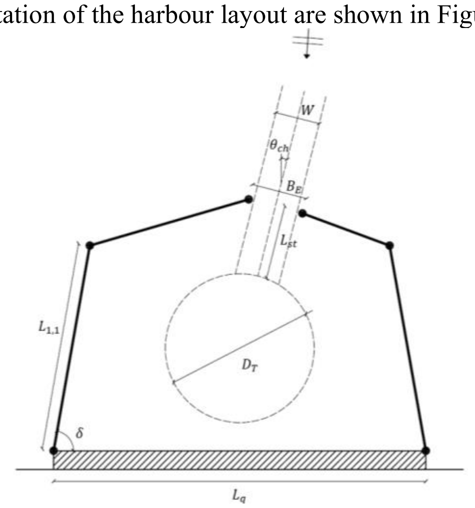

Figure 5 Harbour dimensions

In this figure, D_T is the turning basin diameter, L_q the quay length, δ the angle between the first part of the breakwater and the quay wall, L_{st} the stopping area, B_E the harbour entrance width, W the approach channel width, θ_{ch} the orientation of the approach channel and $L_{1,1}$ the length of the first part of the main breakwater. The tool determines the manoeuvring area dimensions based on the recommendations in the literature, such as PIANC [13], Ligteringen [4], ROM [14] and more. Further, the angles δ and θ_{ch} are requested as input parameters.

3) Wave agitation inside harbour basin

Three different phenomena will determine the wave agitation inside the harbour basin: wave diffraction, wave reflection and wave transmission. Upon entering the harbour entrance, the waves will face the breakwater structures, undergo diffraction and the wave crests will disperse into the shadow zone in the sheltered area of the breakwaters. The simplified method proposed by Goda [15] for wave diffraction is implemented in the tool. Goda [15] proposed diffraction diagrams based on random waves in order to estimate the diffraction pattern in the harbour. Therefore, these diagrams can give a quick and useful view of the real situation during the primary design phases of the harbour layout. Diffraction diagrams were generated for different entrance widths and directional spreading parameters, s_{max} equal to 10 and 75. An example of the diagram is shown in Figure 6.

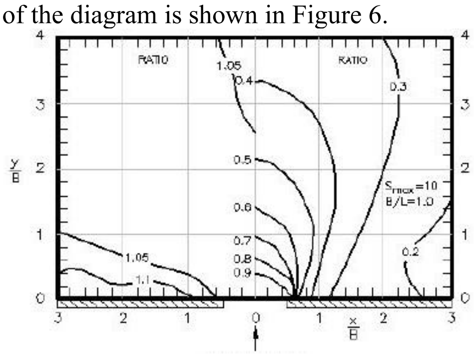

Figure 6 Diffraction diagram for a gap, with a relative entrance width B/L equal to 1.0 and a spreading parameter s_{max} equal to 10 [15]

In case of an oblique incidence wave direction, an additional deviation $\Delta\theta$ of the diffraction lines should be taken into account based on the relative harbour entrance B/L and s_{max} [15]. Furthermore, an apparent entrance width looking from the oblique incidence wave direction is used as in Eq. 3.

$$B_a = B_E \cos(\theta + \Delta\theta) \tag{3}$$

where B_a represents the apparent opening, B_E the real harbour entrance, θ the wave direction and $\Delta\theta$ the wave deviation. Based on the directional spreading parameter s_{max}, the relative apparent opening B_a/L and the amount of breakwaters, the tool selects the correct diffraction diagram and plots it inside the harbour basin as shown in Figure 7.

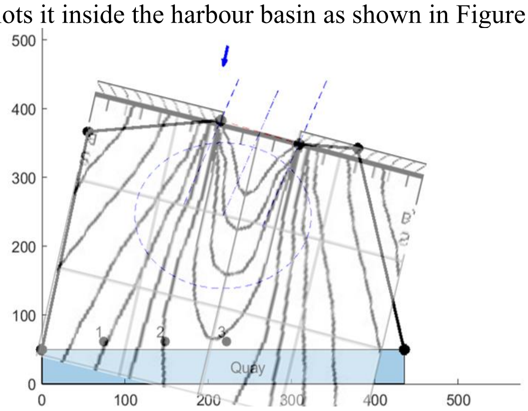

Figure 7 Diffraction diagram applied to harbour basin

The mentioned spreading parameter s_{max} indicates the degree of directional spreading of the wave energy and can be determined based on the wind speed U, if known, using Eq. 2.25 of Goda [15], the deep-water wave steepness H_0/L_0 using Figure 2.13 of Goda [15] or can be entered as an input parameter in the tool. After plotting the diagram, the tool interpolates the data between the diffraction coefficient lines.

To take into account the wave reflection in front of the quays inside the harbour, a mirror-image method proposed by Goda [15] is implemented in the tool. In this method, the harbour layout is transferred in the plane of a mirror image along the quay wall, and the diffraction diagram is again drawn and interpolated inside this mirrored harbour basin. The reflected waves are now treated as waves that develop in the mirror image plane.

An example of the mirror-image method is shown in Figure 8.

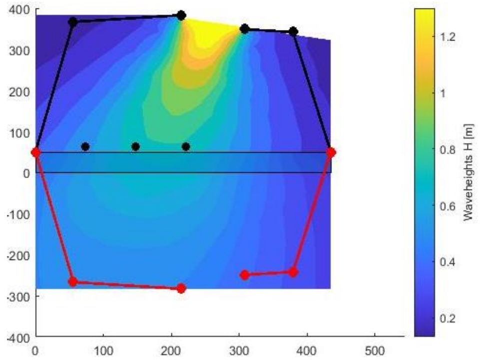

Figure 8 Mirrored harbour layout with interpolated diffraction lines

Finally, the transmission of wave energy through the breakwater and due to overtopping is estimated. Here, equations proposed by the EurOtop [16] are implemented in the tool. The user has to enter the allowable overtopping discharge q as an input parameter in order to determine the minimum required crest freeboard R_c of the breakwater using respectively Eq. 5.13 and Eq. 7.2 from EurOtop [16], for resp. a rubble mound breakwater and a vertical breakwater. The transmission coefficient K_t for respectively a rubble mound and a vertical breakwater can be found be respectively using Eq. 4.8 and Eq. 4.9 from EurOtop [16]. These equations account for the design and safety assessments of the breakwater. The transmitted wave height through overtopping can then be found with means of Eq. 4.

$$H_t = K_t * H_{inc} \tag{4}$$

with H_t the transmitted wave height and H_{inc} the incident wave height.

Finally, the total wave height at any location inside the harbour basin can be found be using the principle of summation of energy components as in Eq. 5.

$$H_s = \sqrt{H_{dif}^2 + H_{ref}^2 + H_t^2} \tag{5}$$

with H_{dif} the diffracted wave height and H_{ref} the reflected wave height.

4) Preliminary cost estimation

For preliminary cost estimation, two components are considered: the dredging cost and the breakwater cost. The cost to dredge per m³ is an input parameter and the required depth of the harbour basin is determined by the tool. Based on the required depth, the bathymetry slope m and the harbour layout, the volume and the cost to dredge can be computed. The total length of the breakwaters can be determine based on the proposed harbour layout and depth along the breakwaters. Together with the input of the required crest width, B_{crest}, the breakwater slope, α, the computed crest freeboard, R_c and the depth along the breakwaters, the volume of core material and the volume of armour units can be defined. These volumes are then multiplied by the unit price of the rocks and armour units to determine the total cost of the breakwaters.

C. Output of the tool

1) Harbour configurations

Based on the computed manoeuvring area and breakwater dimensions, the tool generates four different harbour layouts.

The initial layout is computed based on the output of the Excel spreadsheet, two more are computed by varying the approach channel orientation and a last configuration is generated with only one main breakwater. An example is shown in Figure 9.

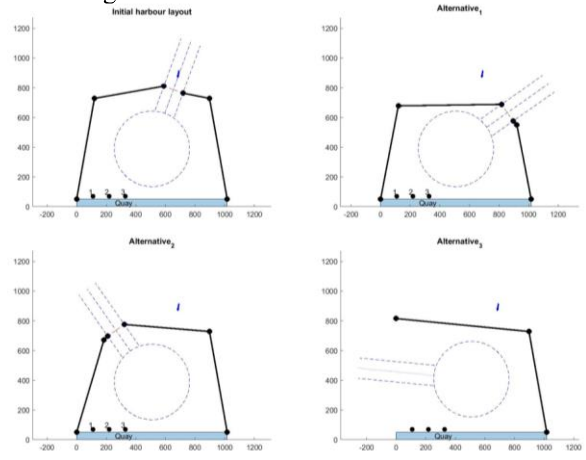

Figure 9 Harbour configurations

2) *Wave agitation inside harbour basin*

For each of the harbour configurations, the tool determines the wave agitation inside the harbour basin based on the diffraction, reflection and transmission calculations. Furthermore, the disturbance coefficients inside the harbour basin are computed using Eq. 6.

$$K_d = \frac{H_d}{H_{inc}} \tag{6}$$

where K_d is the disturbance coefficient, H_d is the disturbed wave height and H_{inc} the incident wave height outside the harbour. An example of the wave agitation inside the harbour basin is shown in Figure 10 for an incident wave with wave direction 10 °N.

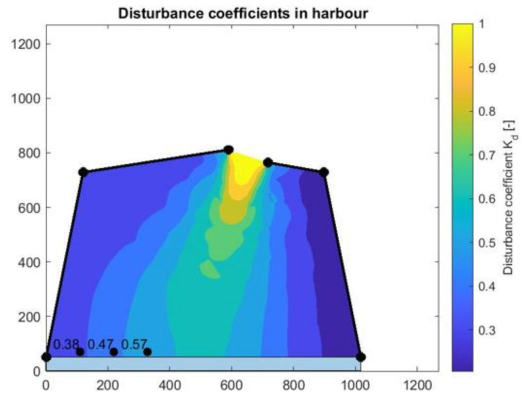

Figure 10 Disturbance coefficients in harbour basin

3) *Preliminary costs*

For each of the harbour configurations, the preliminary costs of the breakwaters structures and dredging needs are estimated. It is worth underlying that this estimation is only a rough preliminary approximation and intended to form a first impression of the possible costs in order to compare the different proposed alternatives.

4) *Decision matrix*

To form a clear overview, the results for each alternative configuration is summarised in a table. Further, the preliminary costs, the wave agitation inside the harbour basin and the navigational aspects are compared. In this phase, the user has to allocate a weight to each of these three criteria and the tool will rank them. Finally, a total score is computed.

D. *Layout optimization*

After the calculations regarding the wave agitation and preliminary costs of the alternative harbour configuration, the tool will perform a quick analysis of one chosen harbour layout. The tool allows an easy modification of the harbour geometry and automatically computes the wave disturbance in the modified harbour basin. Two different analyses can be performed: (i) the effect of the variation of incident wave direction and (ii) the effect of the variation of the geometry for the same incident wave condition.

1) *Effect of different wave directions*

As an example, the wave direction in Figure 10 is changed to 35 °N. The wave agitation for the new wave direction is shown in Figure 11. Due to the variation in wave direction, the incident wave height H_s is reduced from 0.91 m to 0.80 m and the wave penetration is more orientated towards the ship berths, increasing the disturbance coefficients K_d at the berths. However, due to the lower incident wave height H_s and the orientation of the wave penetration, the wave heights at the berths will not significantly change.

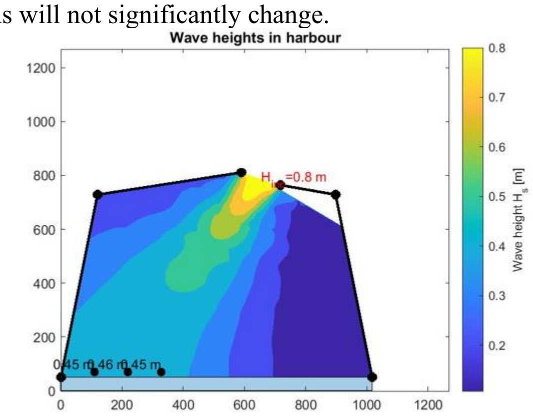

Figure 11 Wave heights for a wave direction of 35 °N

2) *Geometrical optimization*

As a first geometrical modification, the approach channel is rotated and again, the wave agitation inside the harbour basin is computed. As an example, the approach channel in previous figures is rotated by 20° and the wave agitation is shown in Figure 12.

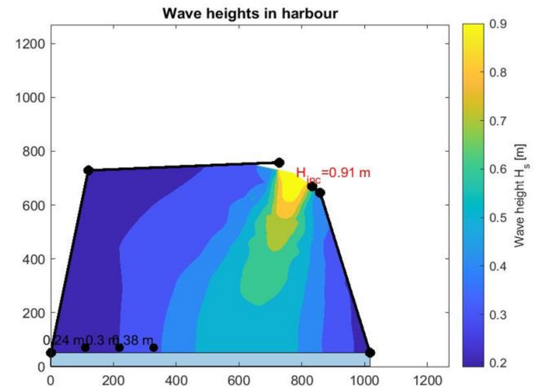

Figure 12 Wave heights for a rotate approach channel (20°)

Due to the rotation, the wave penetration is more shifted to the right side of the harbour basin and the wave heights at the berths are slightly reduced. The angle between the approach channel and mean wave direction is enlarged, which has a negative influence on the vessel’s manoeuvring. Further, the modified harbour layout is more compact, reducing the preliminary costs.

Secondly, the main breakwater is rotated. In this example, the main breakwater is rotated over 10° and the wave agitation is shown in Figure 13. Due to this rotation, the harbour entrance width is enlarged leading to more wave energy penetrating the harbour basin and the wave agitation will slightly increase. Although increasing the harbour entrance will ease the manoeuvring of the vessels, rotating the breakwater structure to deeper water will increase the cost.

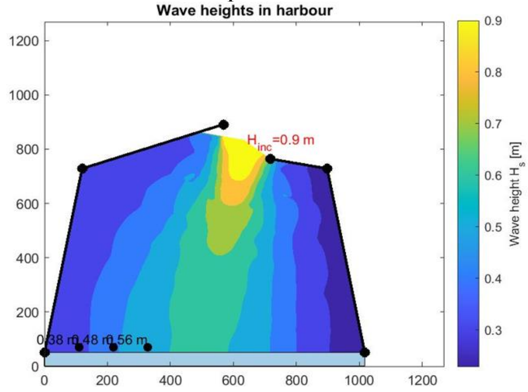

Figure 13 Wave heights for rotated main breakwaters (10°)

Next, the tool will investigate the effect of extending the main breakwater. An example is shown in Figure 14. By doing this, the harbour entrance is narrowed, and less wave energy will penetrate. The wave heights at the berths are significantly reduced. However, this modification complicates the manoeuvring of the vessels when entering the harbour basin. Further, this will increase the total structure cost.

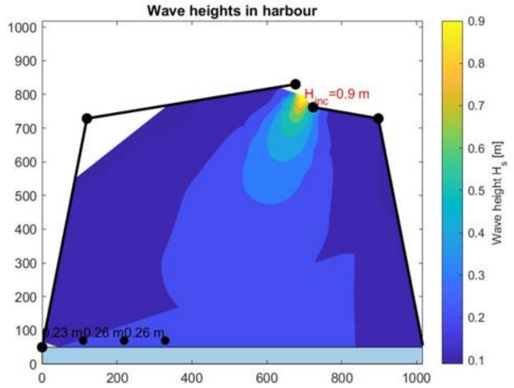

Figure 14 Wave heights for an extended main breakwater

Finally, the approach channel and harbour entrance width is enlarged. An example of this modification is shown in Figure 15. Again, by enlarging the harbour entrance more wave energy will enter the harbour basin and increase the wave agitation. However, this modification will benefit the harbour costs and favour the manoeuvring of the entering vessels.

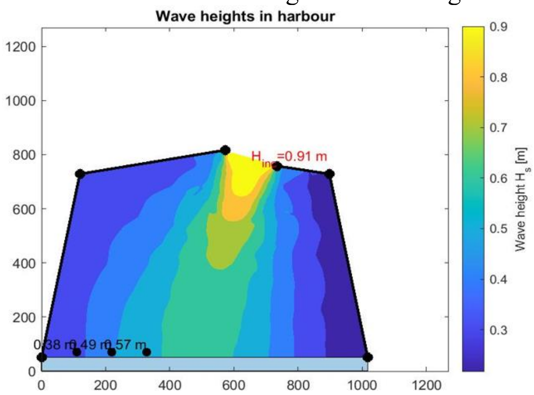

Figure 15 Wave heights for a widened harbour entrance

V. Conclusion

In this master thesis the main design aspects of the preliminary design of a harbour layout were identified and used to develop a consistent methodology. By following the methodology in a general case, multiple harbour configurations can be found. This methodology was then successfully implemented in a handy engineering tool. With help of this tool, the user can generate and evaluate multiple harbour configurations in a short period of time. It computes a primary insight of the wave agitation and costs of the harbour layout. The different harbour configurations should be further optimized during the next, more detailed design phases. This by using physical and numerical models and working towards one final harbour layout.

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LIST OF ABBREVIATIONS AND SYMBOLS

Abbreviations

AtoN	Aids to Navigation
CD	Concept Design
DD	Detailed Design
DGPS	Differential Global navigation satellite Positioning Systems
DWT	Dead Weight Tonnage
ECDIS	Electronic Chart Display and Information System
IMO	International Maritime Organization
LAT	Lowest Astronomical Tide
LNG	Liquefied Natural Gas
LPG	Liquefied Petroleum Gas
MSL	Mean Sea Level
RoRo	Roll-on/roll-off
UKC	Under Keel Clearance
VTS	Vessel Traffic Service

Symbols

Lower case

a	Factor depending on the vessel type for the determination of the channel widening	[-]
b	Width of the wave crest	[m]
d	Water depth	[m]
d_0	Deep-water depth	[m]
d_{br}	Wave breaking depth	[m]
d_{inner}	Required depth of inner approach channel and harbour basin	[m]
d_{outer}	Required depth of outer approach channel	[m]
f	Wave frequency	[Hz]
f_p	Peak wave frequency	[Hz]
g	Gravitational acceleration constant	[m/s²]
k	Wave number	[-]
m	Bathymetry slope	[-]
m_0	Zero-order moment of the variance spectrum	[m²]
n	Number of ship berths	[-]
n_0	Ratio of wave group to phase celerity of a deep wave	[-]
n_1	Ratio of wave group to phase celerity of the wave in front of the harbour	[-]
q	Allowable overtopping discharge per meter structure width	[l/s/m]
s	Wave steepness	[-]
s_0	Deep-water wave steepness	[-]

s_{max}	Directional spreading parameter	[-]
t_y	Time of ship yawning	[s]
x_{br}	Distance from coastline to breaker zone	[m]

Upper case

A	Factor for determining allowable wave height	[-]
A_q	Operational land area	[m²]
B	Factor for determining allowable wave height	[-]
B_E	Harbour entrance width	[m]
B_G	Width of central turning rectangle	[m]
B_a	Apparent harbour entrance width	[m]
B_{crest}	Width of breakwater crest	[m]
B_{gap}	Length of gap between moored ships	[m]
B_q	Quay width	[m]
B_v	Design vessel's beam	[m]
C	Factor for determining allowable movement	[-]
C_0	Celerity of deep-water wave	[m/s]
C_1	Celerity of wave in front of harbour	[m/s]
C_B	Block coefficient	[-]
C_{back}	Backfilling cost	[€/m³]
C_{core}	Cost of core rocks	[€/m³]
C_{dredge}	Dredging cost	[€/m³]
C_{g0}	Group velocity of deep-water wave	[m/s]
C_{g1}	Group velocity of wave in front of harbour	[m/s]
C_{mob}	Mobilisation/demobilisation of one dredge	[€]
C_r	Reflection coefficient	[-]
C_{unit}	Cost of armour units	[€/m³]
C_{vert}	Cost of vertical breakwater	[€/m³]
D	Factor for determining allowable movement	[-]
D_T	Diameter of turning basin	[m]
E	Total wave energy per unit crest width	[J]
$\bar{E}$	Wave energy per unit of length of wave crest and per wavelength L	[J/m]
E_{hor}	Horizontal movement of vessel	[m]
E_{goal}	Allowable movement of vessel	[m]
F_s	Ship related factor	[-]
H	Wave height	[m]
$H_{allowable}$	Allowable wave height	[m]
H_{br}	Breaking wave height	[m]

H_d	Disturbed wave height	[m]
H_{dif}	Diffracted wave height	[m]
H_{inc}	Incident wave height	[m]
H_{m0}	Significant wave height from spectral analysis	[m]
H_{ref}	Reflected wave height	[m]
$H_{s,0}$	Deep-water significant wave height	[m]
H_s	Significant wave height	[m]
H_{sr}	Wave height resulting from shoaling and refraction	[m]
H_t	Transmitted wave height	[m]
K	Distance from the pivot centre to the ship's bow or stern as a fraction of the vessel's length L_v	[-]
K_d	Disturbance coefficient	[-]
$K_d(f,\theta)$	Diffraction coefficient of a regular wave component with frequency f and wave direction θ	[-]
$(K_d)_{eff}$	Diffraction coefficient of irregular waves	[-]
K_r	Refraction coefficient	[-]
K_s	Shoaling coefficient	[-]
K_{sr}	Combined shoaling and refraction coefficient	[-]
K_t	Transmission coefficient	[-]
L	Wavelength	[m]
$L_{1,1}$	Length of first part of main breakwater	[m]
$L_{1,1y}$	Length of first part of main breakwater perpendicular to coastline	[m]
L_G	Length of central turning rectangle	[m]
L_p	Wavelength calculated with peak wave period	[m]
L_q	Quay length	[m]
$L_{st,l}$	Lower bound of stopping length	[m]
$L_{st,u}$	Upper bound of stopping length	[m]
L_{st}	Stopping length	[m]
L_v	Design vessel's overall length	[m]
P	Installed power	[kw]
$\bar{P}$	Wave power per unit of length of wave crest	[W/m]
R	Radius of vessel's path while turning	[m]
R_b	Bend radius	[m]
R_c	Crest free board	[m]
R_{cr}	Radius added to central turning rectangle	[m]
R_{min}	Minimum bend radius	[m]
RP	Return period	[years]
R_{sr}	Radius of turning basin	[m]
$S(f,\theta)$	Frequency spectrum	[m²s]

T	Wave period	[s]
$T_{m-1,0}$	Spectral wave period	[s]
$T_{p,0}$	Deep-water peak wave period	[s]
T_p	Peak wave period	[s]
T_v	Design vessel's draught	[m]
U	Wind speed	[m/s]
V_0	Speed of stopping vessel	[kn]
V_c	Longshore current by breaking waves	[m/s]
V_{cc}	Cross-current velocity	[kn]
V_c^n	Normal component of current velocity	[kn]
V_{cw}	Cross wind velocity	[kn]
V_{lc}	Longitudinal current velocity	[kn]
V_{min}	Minimum vessel speed when entering the harbour	[kn]
V_s	Vessel speed	[kn]
V_v	Approximation of vessel's volume	[m³]
V_w^n	Normal component of wind velocity	[kn]
V_α^n	Normal component of speed drifting out course under action of metocean conditions	[kn]
W	Approach channel width	[m]
W_B	Bank clearance	[m]
W_{BG}	Bank clearance on the 'green' side	[m]
W_{BM}	Basic manoeuvring lane width	[m]
W_{BR}	Bank clearance on the 'red' side	[m]
W_i	Additional widths	[m]
W_{one}	Width of one-way channel	[m]
W_p	Passing distance	[m]
W_{two}	Width of two-way channel	[m]

Greek symbols

α	Breakwater slope	[°]
α_0	Angle between the wave crest of the deep-water wave and the straight bottom contours	[°]
α_1	Angle between the wave crest of the wave in front of the harbour and the straight bottom contours	[°]
α_b	Bend angle	[°]
β_b	Direction of wave propagation on breaker line	[°]
β_y	Yawning angle	[°]
γ^*	Influence factor for overtopping for different types of armour layers	[-]
γ_b	Breaker depth index	[-]

γ_f	Influence factor for overtopping for permeability and roughness	[-]
γ_β	Influence factor for overtopping of oblique waves	[-]
δ	Angle between coastline and breakwater	[°]
θ	Wave direction	[°]
θ_0	Wave direction of deep-water wave	[°]
θ_1	Wave direction in front of harbour	[°]
θ_{ch}	Approach channel orientation	[°]
ξ_{0p}	Breaker parameter	[-]
ρ	Water density	[kg/m³]
Δ	Vessel's displacement	[tonnes]
ΔB_E	Safety parameter for the determination of the harbour entrance width	[m]
ΔW	Channel widening	[m]
$\Delta\theta$	Deviation of approach channel of wave direction	[°]

1 INTRODUCTION

1.1 Harbour layout

A harbour is defined as an area of water enclosed by natural features or by artificial structures, or a combination of both. It is a place of refuge, safe berths and protection of vessels during severe storm events and/or accommodations for economical activities. In case a harbour is used for the latter, it is called a port (Tsinker, 1997). A typical harbour layout consists of several elements: the protection structures which are usually breakwaters, the terminals with ship berths or quays and the ship manoeuvring areas (Liu & Burcharth, 1999). Figure 1.1 shows an example of a harbour layout.

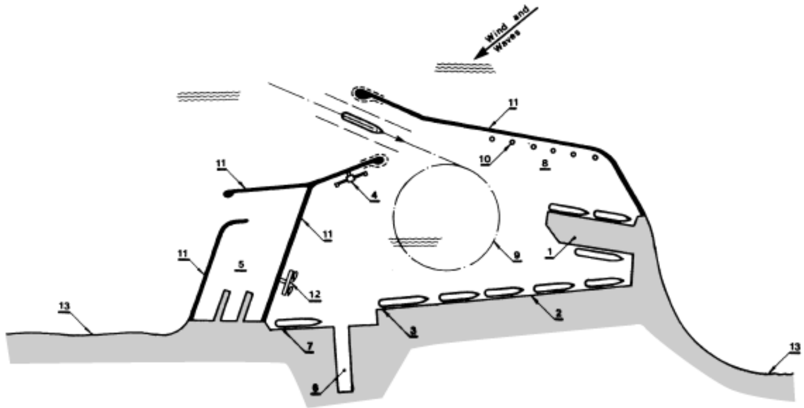


Figure 1.1: Example of a harbour layout (Memos, 2018)

The harbour layout is mainly influenced by the breakwaters locations. The main function of the breakwaters surrounding the harbour is to provide shelter from wave agitation as well as safe manoeuvring and mooring of the vessels inside the harbour. The other possible functions of breakwaters can be the provisions of dock or quay facilities, guiding currents and prevent siltation of the harbour entrance and basin (PIANC, 2016).

There are different types of breakwaters, and they can be divided into different categories according to their structural appearances as shown in Figure 1.2: rubble mound breakwaters, vertical caisson breakwaters and composite types (Verhagen & van den Bos, 2017; PIANC, 2016; Sawaragi, 1995). There exist also many unconventional types of breakwaters such as perforated or floating breakwaters. However, those nonconventional structures will not be discussed further on in the thesis.

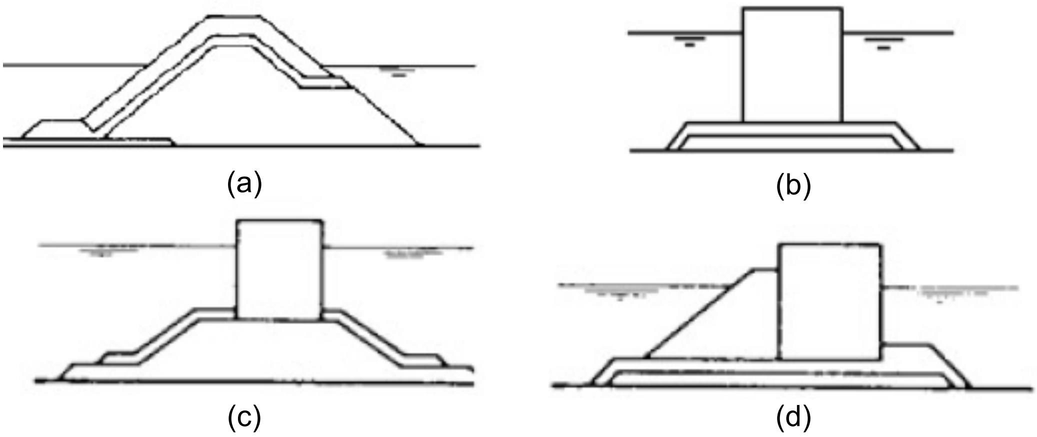


Figure 1.2: Breakwater types: (a) Rubble mound breakwater, (b) Vertical caisson breakwater, (c) Vertical composite breakwater and (d) Horizontal composite breakwater (Verhagen & van den Bos, 2017)

The terminal is the area provided for cargo handling, including space for the ships to berth, storage of the cargo and necessary accommodation (Tsinker, 1997). Today, these terminals serve a very specific purpose, such as handling oil, dry bulk, containers, passenger traffic and others. As mentioned before, these terminals are equipped with berths to provide safe mooring of the vessels, together with cargo handling equipment. A harbour can have different terminals, each serving its own purpose as shown in Figure 1.1.

A vessel which enters a harbour has to navigate itself safely towards the vessel berths. Therefore, enough space needs to be available to provide a safe passage of the vessels inside the harbour. This space is called the manoeuvring areas. Following manoeuvring areas need to be specified in the design of a harbour layout and are each shown in Figure 1.3:

- Approach channel
- Harbour entrance
- Stopping area
- Turning basin

The approach channel is any stretch of channel, inside or outside the harbour, that connects the open sea with the inner harbour and turning basin. When the vessels enter the harbour, the harbour entrance needs to provide a safe access. The vessel needs to reduce speed before berthing, the space provided for this stop manoeuvre is called the stopping area. Further, the vessel has to align itself with the berths, this turning manoeuvre is performed in the turning basin of the harbour basin. Multiple turning basins can exist in large ports (Puertos del Estado, 2007; PIANC, 2014).

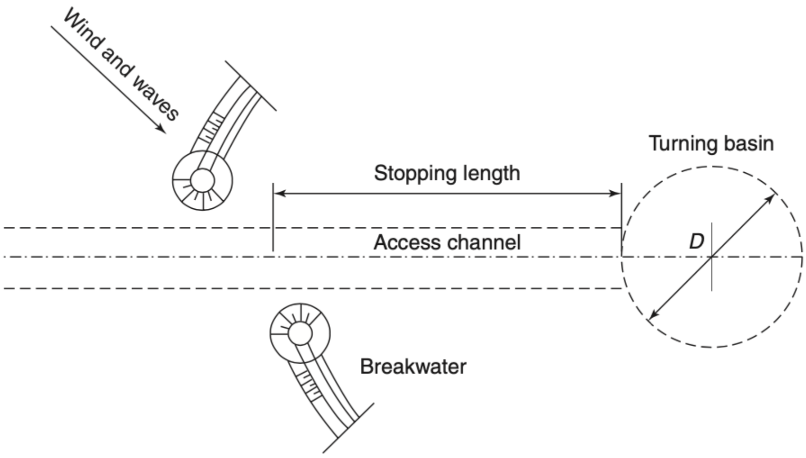


Figure 1.3: Manoeuvring areas in a harbour (Thoresen C. A., 2014)

The designing and construction of a harbour layout has thus two main objectives:

1. to design a large water surface area with appropriate dimensions at all times that provides shelter for vessels, and
2. to provide the means and accommodations for economical activities.

1.2 Design process of a harbour layout

Table 1.1 gives an overview of the different design phases of a harbour layout and the level of design. This thesis focusses on the preliminary design of the harbour layout. During the preliminary design phase, the designer gives form to the harbour layout in general, including determining the functionality of the components and the definition of requirements at the level of the elements (Verhagen & van den Bos, 2017; CIRIA/CUR/CETMEF, 2007). During the preliminary design phase several alternative harbour configurations are generated and analysed considering various design aspects.

Table 1.1: Schematization of the design phases (Verhagen & van den Bos, 2017)

Phases	Abstraction Level		
	System	Component	Element
Initial	Purpose		
Feasibility	Functionality	Purpose	
Preliminary Design	Shape	Functionality	Purpose
Final Design	Specifications	Shape	Functionality
Detailed Design		Specifications	Shape

A project starts with the project definition stage, the initial stage, which defines the project needs and objectives. These project objectives will determine the requirements and restrictions (Verhagen & van den Bos, 2017; CIRIA/CUR/CETMEF, 2007).

After the initial stage and prior to the preliminary design stage, the feasibility of the harbour plan is assessed during the concept design stage. This phase consists of identifying the functions, limitations and information requirements. This might include the collection of physical conditions data required and permissions needed for the design (Verhagen & van den Bos, 2017; CIRIA/CUR/CETMEF, 2007).

The preliminary harbour layout should be further optimized in more detailed design stages, with means of physical and numerical model tests. It involves the development of all structural elements, including in-depth investigations, physical and technical data of the harbour site. The outcome of this stage is the production of drawings, specifications and cost estimations (Verhagen & van den Bos, 2017; CIRIA/CUR/CETMEF, 2007). After the detailed design stage construction of the harbour structures can begin.

Figure 1.4 shows an example of a design process regarding the harbour approach channel layout according to PIANC (2014).

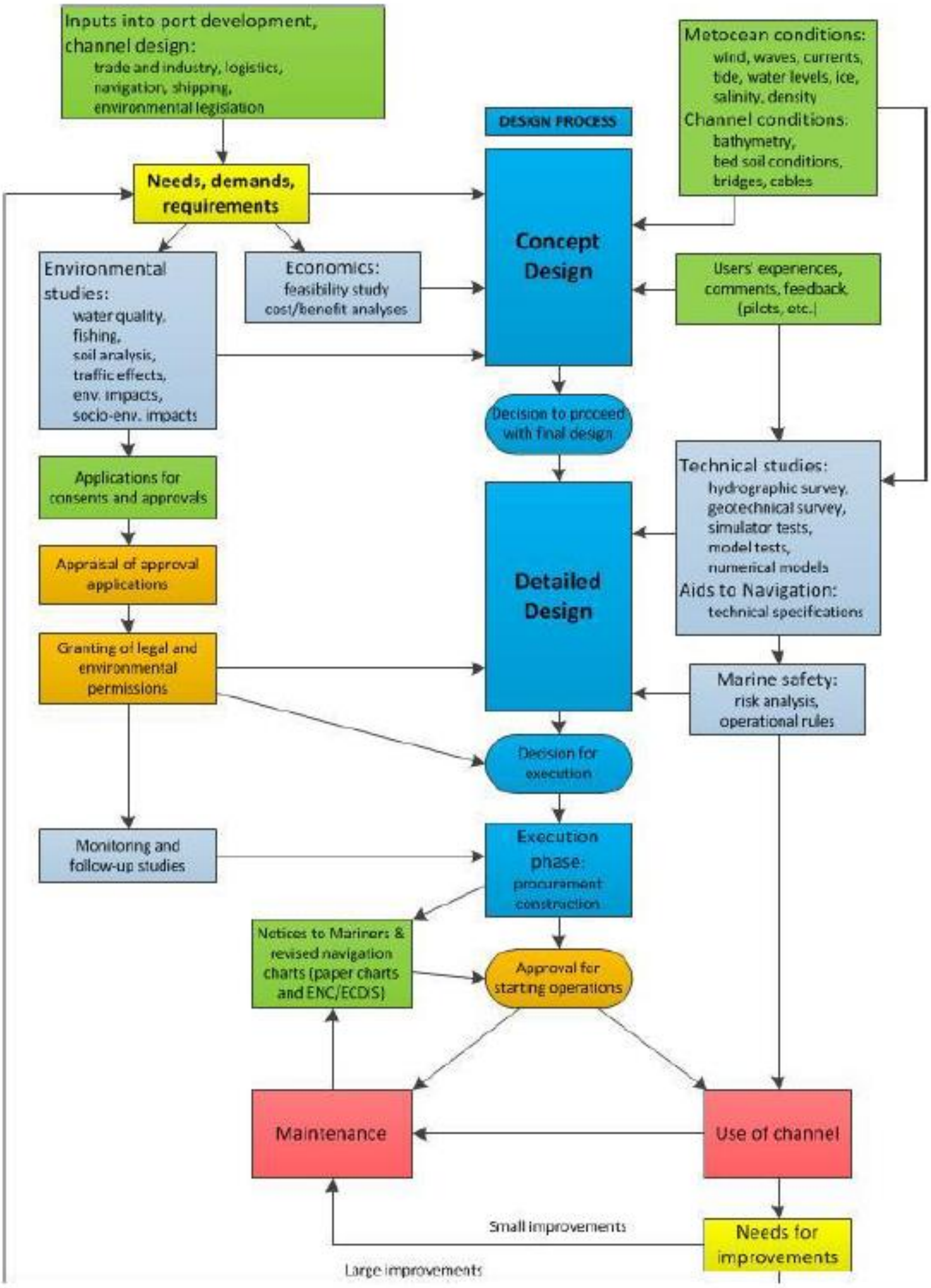


Figure 1.4: Overall channel development process (PIANC, 2014)

1.3 Thesis outline

This thesis is structured as follows:

Chapter 1 gives an introduction on the preliminary design of the harbour layout, the harbour layout itself and the different components of a harbour layout. Further, an overview of the design processes is briefly discussed.

Chapter 2 contains the current state of the art and literature review of the different guidelines that cover the topic of the harbour layout and port planning. The purpose of this literature review is to gather all the latest design principles, the experiences and advices in the design of a harbour layout, the so-called “state-of-the-art”.

Chapter 3 defines the main motivation and objectives of this thesis.

Chapter 4 describes the different existing harbour configurations with their characteristics, advantages and disadvantages. An example of an existing port with the relevant layout is given at the end of each harbour configuration description. By describing the different harbour configurations, an overview is created of what a harbour layout design should look like. This is done with the help of literature.

The identification of the main design aspects to be considered during the preliminary design of a harbour layout is discussed in Chapter 5. These aspects are further investigated in detail and several design methods and approaches are provided. The chapter provides a clear view on the main design aspects that need to be considered in the preliminary design stage and the requirements that are acquired.

Starting from these design aspects, the parameters can be determined that need to be present for the preliminary design of the harbour layout. In Chapter 6 these parameters will be used to develop a methodology and implement it in a tool. Here, the formulas, decision aspect and the methodology adopted in the tool will be discussed. The tool will be an interaction of an Excel spreadsheet and MATLAB scripts and is used to generate alternative harbour configurations.

After the description of the tool, Chapter 7 shows an example of the tool indicating the capability, limitations and the future investigation lines to improve it.

Chapter 8 provides the main conclusions of this master thesis and recommendations.

2 STATE OF THE ART

Many authors have provided recommendations and guidelines on the port planning and harbour layout design. Most of these existing guidelines have their own approaches and/or focus on particular aspects. The approaches provided by the different guidelines, and used in the further elaboration of this thesis, will be discussed in Chapter 5 Design aspects. In the following, the current state of the art in harbour layout design is discussed briefly.

Port development – A handbook for planners in developing countries (UNCTAD, 1985) is one of the first reference books regarding the development of ports. It provides recommendations regarding the master planning and port zoning of new and existing ports. The UNCTAD (1985) suggests already some methods which will be used further on by other references like the port classification. The UNCTAD (1985) also distinguishes several design aspects to be considered such as the different field investigations performed, water area requirements, dredging, breakwaters, quays and jetties and cost estimation. However, the UNCTAD (1985) uses very simplified approaches and does not provide any detailed recommendations regarding the design of manoeuvring areas and harbour configuration.

Ports and terminals (Velsink, 1994) provides a detailed description of the types of vessels, including the dimensions, transport capacity, propulsion, engine and ship manoeuvrability. It describes the framework of port planning, the evaluation of a port masterplan and port zoning. Figure 2.1 shows the different port planning processes according to Velsink (1994). Further, Velsink (1994) already provides more detailed methods regarding the design of the approach channel and stopping length. However, the design of the turning basin is still done based on general recommendations. Also, Velsink (1994) provides a detailed method to evaluate the mooring conditions of the vessels inside the harbour, although these are based on the vessel movements, which are too complex to evaluate in a preliminary phase. In Ports and terminals, several chapters are devoted to the different types of terminals and their requirements.

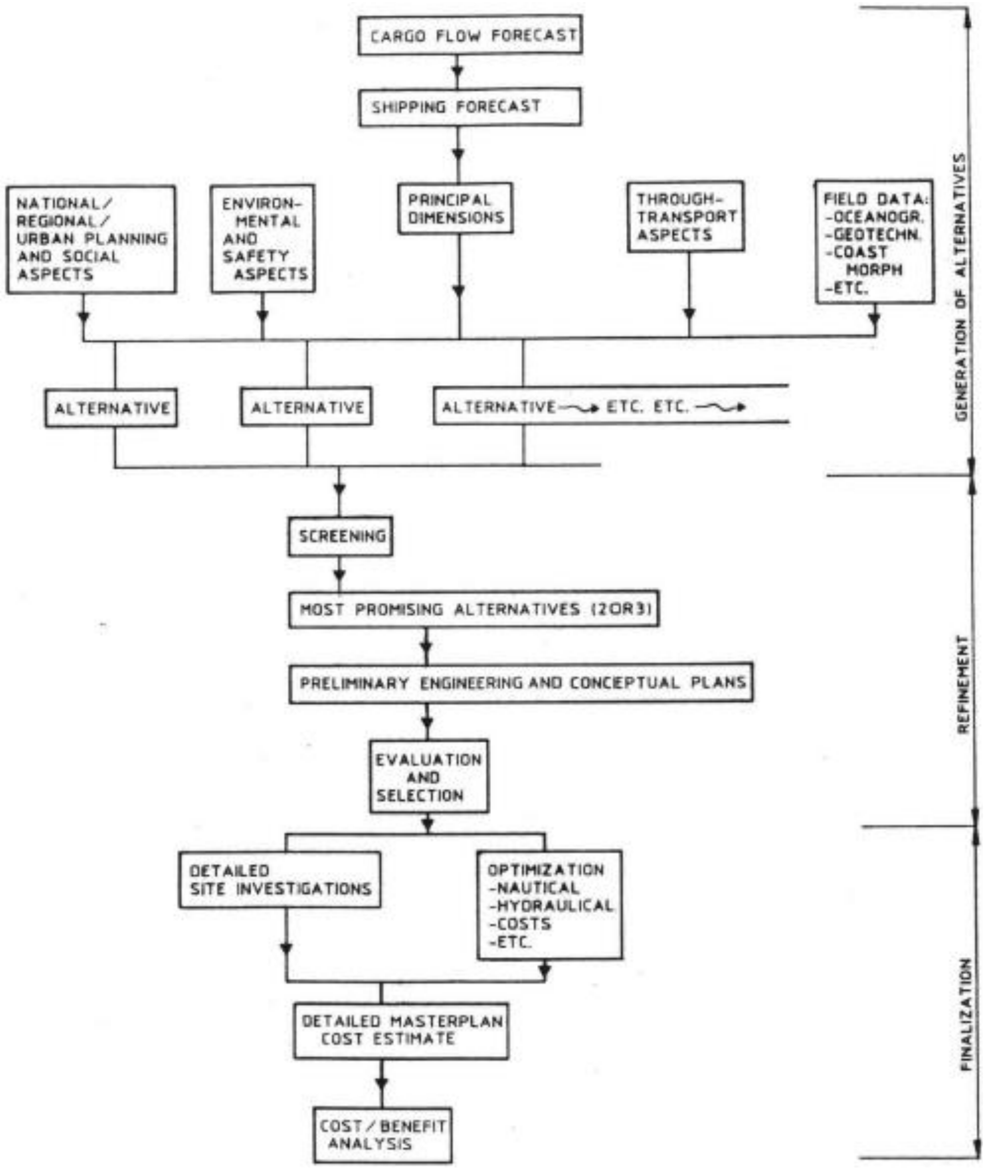


Figure 2.1: Scheme of port planning processes according to Velsink (1994)

Guidelines for the hydraulic design of harbour entrances (McBride, Smallman, & Allsop, 1996) provides recommendations for the design of harbour layouts based on various numerical and physical models. These recommendations are especially for wave reflections induced by vertical caisson breakwater structures. The advices can be taken into account in the harbour layout design to provide save navigation of the vessels within the harbour basin and approach channel. However, the guideline only contains recommendations on the harbour entrance and breakwater alignment and not on the different manoeuvring areas.

Handbook of port and harbour engineering (Tsinker, 1997) describes the environmental conditions such as waves, wind and currents, and their effect on the harbour layout design. A classification of ports is suggested, as shown in Figure 2.2 and the different port components are described. Further, Tsinker (1997) provides detailed recommendations and the design aspects to consider for the design of the approach channel, the harbour entrance and stopping length. However, the design of the turning basin is based on general recommendations regarding the design vessel and nothing is mentioned about mooring or limit state conditions inside the harbour basin.

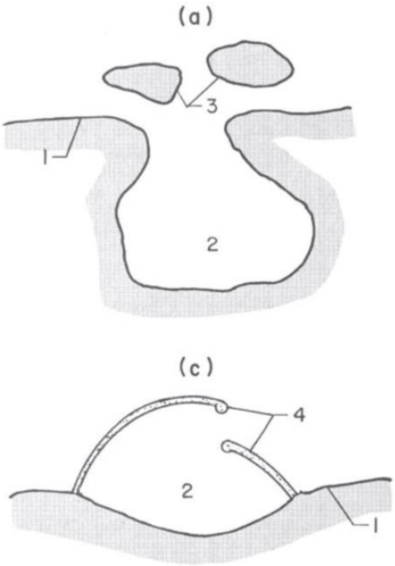


Figure 2.2: Port layout classification; a) naturally protected harbour; b) lagoon protected by detached breakwater; c) converging shore-connected breakwaters; d) shore-connected and detached breakwaters. 1: Shoreline; 2: harbour basin; 3: islands; 4: shore-connected breakwater; 5: detached breakwater (Tsinker, 1997)

Port engineering (Liu & Burcharth, 1999) are course notes of the university of Aalborg. It provides a brief description of the metocean conditions which effect the vessels manoeuvring in- and outside the harbour basin and of the different components of a port. The design of the harbour layout, recommended by Liu & Burcharth (1999), is based on very general advices on the dimensions of manoeuvring areas. During the design of these manoeuvring areas only the design vessels overall length L_v is considered. Liu & Burcharth (1999) describe the typical breakwater layouts in a harbour configuration, as shown in Figure 2.3.

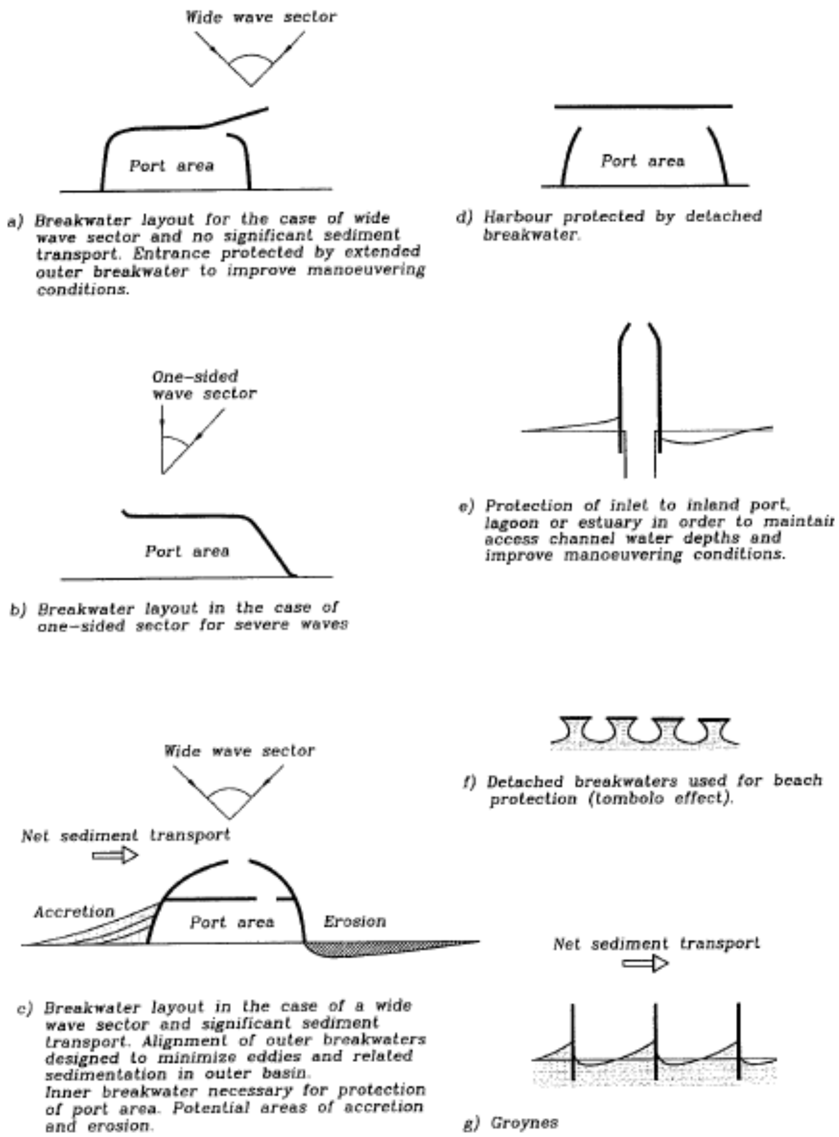


Figure 2.3: Typical harbour layouts (Liu & Burcharth, 1999)

The **British standards (BS 6349, 2000)** provide recommendations and guidance for the planning, design, construction and maintenance of all kinds of coastal structures and consist of 7 different parts. The first part considers the environmental aspects of the design of coastal structures, such as the metocean conditions, the bathymetry, the geotechnical aspects and more. It also describes the different aspects to consider during the design of the manoeuvring areas inside the harbour, although no precise methods are provided. Part 7 of the BS 6349 (2000) discusses the design and construction of breakwaters including the harbour layout planning where different design aspects are considered, such as the navigational aspects, wave penetration, overtopping and transmission, bathymetry, metocean conditions and more. Figure 2.4 shows a schematization of the design process according to BS 6349 (2000). BS 6349 (2000) does not provide methods for determining the dimensions of manoeuvring areas, nor for certain mooring conditions or limit state conditions.

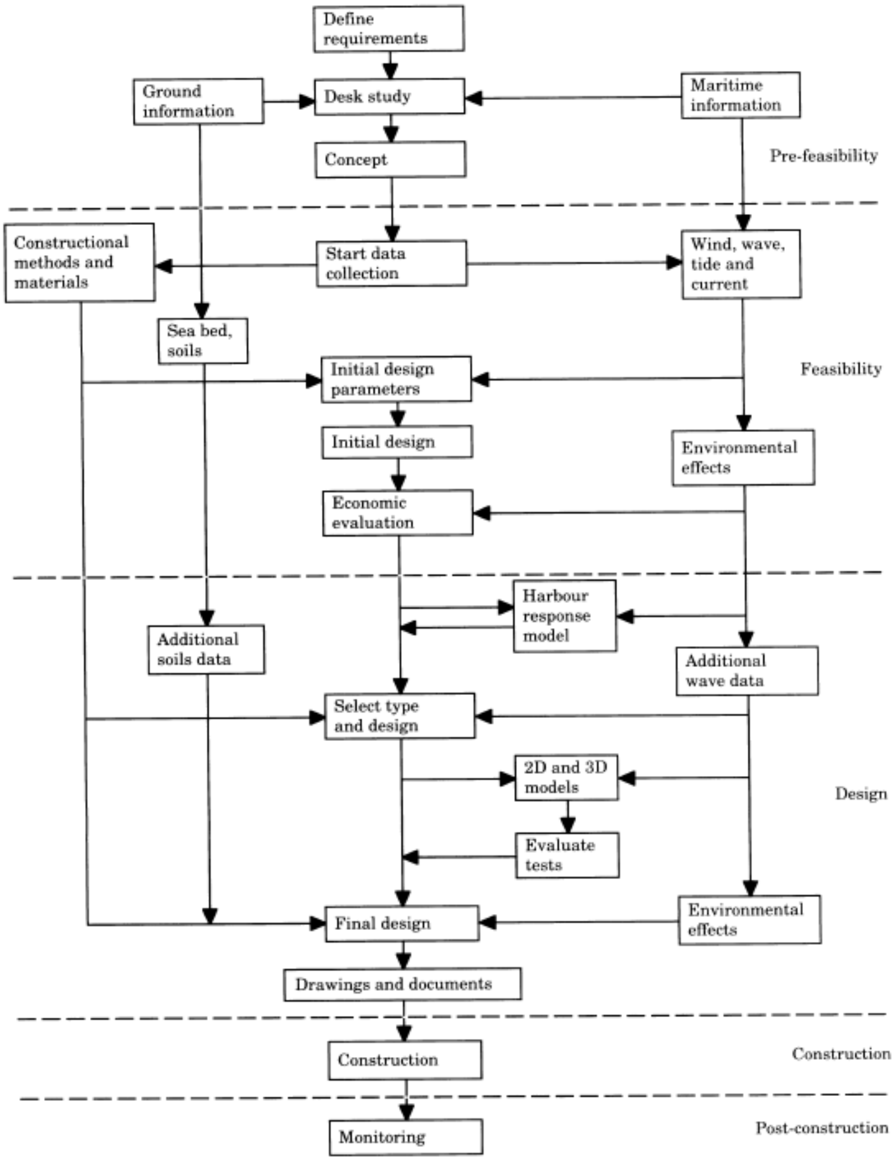


Figure 2.4: Scheme of the design process according to BS 6349 (2000)

The **coastal engineering manual (CEM) (US Army Corps of Engineers, 2000)** collects, as itself describes, the current state-of-the-art regarding coastal engineering. This manual provides methods for the solution of several coastal engineering problems, such as water wave mechanics, coastal processes, coastal planning, coastal flood sties, shore protection and navigation projects. Part V: Chapter 5 (2002) provides recommendations for the different manoeuvring areas, however these are not very detailed. Further, CEM (2000) does not provide any guidelines regarding the design of the breakwater layout.

Planning and design of ports and marine terminals (Agerschou, et al., 2004) describes the facilities requirements, the economic and financial feasibility of a port. Further, the design of the manoeuvring areas, the design of harbour structures such as breakwaters and berths, dredging and environmental considerations are discussed. Agerschou et al. (2004) provide a detailed method for the design of the approach channel. However, no method is provided for the design of the stopping area and turning area. Further, it gives useful recommendations regarding the alignment and layout of the breakwater structures of the harbour layout.

ROM 3.1-99 (Puertos del Estado, 2007) provides extended design methods regarding the design of the different manoeuvring areas, the harbour basin and quay. The ROM 3.1-99 is one of the few references which describes the design of the stopping area in full detail. Further, it provides a detailed method for the design of the turning area, taking more and different aspects into account than any other reference. It also describes the vessel manoeuvrability characteristics, actions on manoeuvring vessels, tugboat characteristics and requirements of the channel cross section.

Technical standards and commentaries for port and harbour facilities in Japan, MLIT (2009) focusses, as the title of the guideline says, on the coastal structures in Japan. The metocean conditions, geotechnical conditions and materials are discussed based on the Japanese circumstances and their experiences. One chapter is devoted on the seismic activity in Japan, which introduces an additional design aspect which will not be considered in this thesis. In the guideline, MLIT (2009) only considers the vertical caisson breakwater type or composite breakwaters and no classical rubble mound breakwaters. This is due to the fact that Japan is an island and the water depths surrounding the coastal structures are relative deep.

Planning & design guidelines for small craft harbors (ASCE, 2012) considers the planning, environmental and financial aspects of small craft harbours, also called marinas. Figure 2.5 shows the planning process scheme suggested by ASCE (2012). It should be stressed that the methods and recommendations provided here are developed for marinas and cannot be applied directly to other types of harbours. ASCE (2012) gives design methods regarding the harbour entrance, approach channel and turning basin. Further, a detailed method for determining the mooring conditions is provided.

Figure 2.5: Schematization of the planning process according to ASCE (2012)

Port designer's handbook (Thoresen C. A., 2014) is one of the most complete guidelines considering harbour layout. It describes the different planning procedures such as setting the scope and the project needs, performing impact studies and site evaluation and the final design of the harbour layout. Thoresen (2014) describes the metocean conditions and their impact on vessels, the port planning process, the manoeuvring areas, the types of vessels and the different harbour structures such as berth structures, quay wall, breakwaters, terminals and more. However, Thoresen's recommendations considering the manoeuvring areas and quay length are mostly general and only based on the overall length of the design vessel L_v. On the other hand, the method regarding the mooring conditions is very detailed, making it too complex to use for the preliminary design of a harbour configuration.

Ports and terminals (Ligteringen, 2017) describes the need for maritime transport, the different port functions, the port planning process and the different types of port terminals. Further, bathymetry, climate, wave conditions, tide and current conditions, sediment and soils characteristics are considered as the design aspects of a harbour layout. Ligteringen (2017) provides detailed design recommendations regarding manoeuvring areas, port basin dimensions, breakwater alignment, mooring conditions, sediment transport and port layout optimization.

Port planning (Memos, 2018) gives recommendations regarding port development and port planning. Memos (2018) describes the general principles of a harbour layout and provides several recommendations for the design of manoeuvring areas. These are rather general and based on the overall length of the design vessel L_v. Further, he considers the layout of the breakwaters, the quay and docks and the effect of the environmental conditions on the vessels.

PIANC, the World Association for Waterborne Transport Infrastructure, has developed several useful reports with guidelines for the design of a harbour layout. These reports refer to the best international practices and bring together the advice of the best experts in the technical, economical and environmental disciplines.

Report n° 121 (PIANC, 2014) describes, in detail, the design of the approach channel, including the width, depth and limiting operational conditions. Here, the PIANC distinguishes two stages: the Concept Design (CD) and the Detailed Design (DD) for the design of the approach channel. The Concept Design consist of the preliminary design of the manoeuvring areas and during this stage many alternative configurations may be developed. During the Detailed Design more in-depth studies and refinement of the Concept Design is done. The design guideline suggested by the PIANC is a so-called 'Design Ship' Concept, where the dimensions of the manoeuvring areas are determined by the type of the design vessel. Further, it describes the different influence factors such as the metocean conditions, the bathymetry, geotechnical aspects, etc..

Report n° 158 (PIANC, 2014) provides guidelines for the port masterplans of existing ports. This report explains the importance of ports and the challenges they encounter. PIANC (2014) describes the concept of masterplanning and port zoning. It considers the different design aspects such as bathymetry, metocean conditions, geotechnical aspects and more. However, PIANC (2014) does not provide any methods regarding the design of manoeuvring areas or harbour dimensions.

Report n° 196 (PIANC, 2016) describes the different types of breakwater structures, the general characteristics of rubble mound and vertical breakwaters and provides recommendations for the selection of the breakwater type. This selection is based on functional criteria (Figure 2.6), site environmental conditions and conditions for construction. The report provides different examples of breakwater design and the safety aspects related to the design. The PIANC (2016) focusses on the breakwater layout, however PIANC does not provide any recommendations regarding the harbour layout or mooring conditions inside the harbour basin in this report.

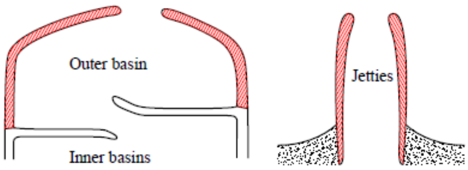
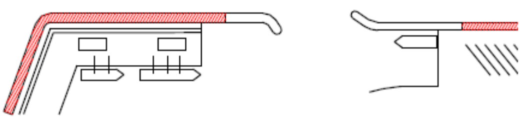
Functional type	Functional constraints (wave transmission, overtopping, access, space)
I Outer breakwaters 	Large overtopping may be acceptable, limited by tolerable wave transmission. Normally no access to breakwaters and space needed
II Breakwaters protecting basins with moorings 	Relatively large overtopping may be acceptable, moderate transmission may be tolerated. Normally no access to breakwaters and space needed
III Breakwaters with berthing facilities 	Only limited overtopping is acceptable. Access to breakwaters and space on breakwaters needed
IV Breakwaters protecting sheds, parking and storage areas. 	Very limited overtopping is acceptable. Access on or just behind breakwaters as well as space behind breakwaters needed

Figure 2.6: Functional classification of breakwaters (PIANC, 2016)

MarCom WG Report n° 185 (PIANC, 2019) gives several guidelines regarding the preparation and application of masterplans for ports on greenfield sites. It gives guidance on how to identify, develop and review the needs of the greenfield site, on the evaluation of potential sites and the preparation of development options.

2.1 Conclusion

The optimum layout of breakwaters for harbour protection is difficult to determine due to the complex conditions and interaction in between the design parameters typically involved. Due to the complexity and the divergent disciplines involved and the different aspects to consider during the design process, only general guidance, practical advices or “rule-of-thumb’’ relative to harbour layout are available in literature. However, there is currently not a consistent methodology to be applied in general case for preliminary design of port layout that can be use in a practical way.

3 MOTIVATION AND GOAL

The literature review makes clear that there does not exist one precise guideline containing a well-established methodology to follow during the preliminary design of a port and harbour layout, and to come up with different alternative configurations. Up until now, different guidelines and advices, available in literature, have to be combined to consider all multidisciplinary aspects of a harbour layout design.

The main goal of this thesis is to develop one consistent methodology to follow for a preliminary design of a harbour configuration based on the current state-of-the-art. This methodology will be based on the design aspects to be considered in port planning and harbour layout, which will be identified first. The proposed methodology will then be used to develop a tool which can assist coastal engineers during the different steps of the design of a port layout, before using physical or numerical modelling.

The tool will suggest different alternative harbour layout configurations in a short computation time. These different alternatives should be assessed and optimised during the next, more detailed design stages, working towards one final configuration that will be constructed.

To summarize the different objectives are listed below:

(i) Performing an extended literature review regarding all the different aspects regarding the preliminary design of the port planning and harbour layout;
(ii) The identification of the principal design aspects to be considered during the preliminary design of a harbour layout;
(iii) A proposal of a consistent methodology, which can be adopted to find different harbour configurations and alternatives, which would be examined during further design phases;
(iv) Implement the methodology in a tool that can be adopted for preliminary design of the harbour layout.

4 HARBOUR CONFIGURATIONS

As mentioned in previous chapters, the breakwater structures will mostly define the layout of the harbour. Most harbours have one of following typical breakwater configurations, shown in Figure 4.1 (Liu & Burcharth, 1999):

- Converging breakwaters
- Converging breakwaters with inner and outer breakwaters
- Coastline parallel attached breakwater
- Coastline parallel attached breakwater with a secondary breakwater
- Detached breakwater
- Channel/river harbour

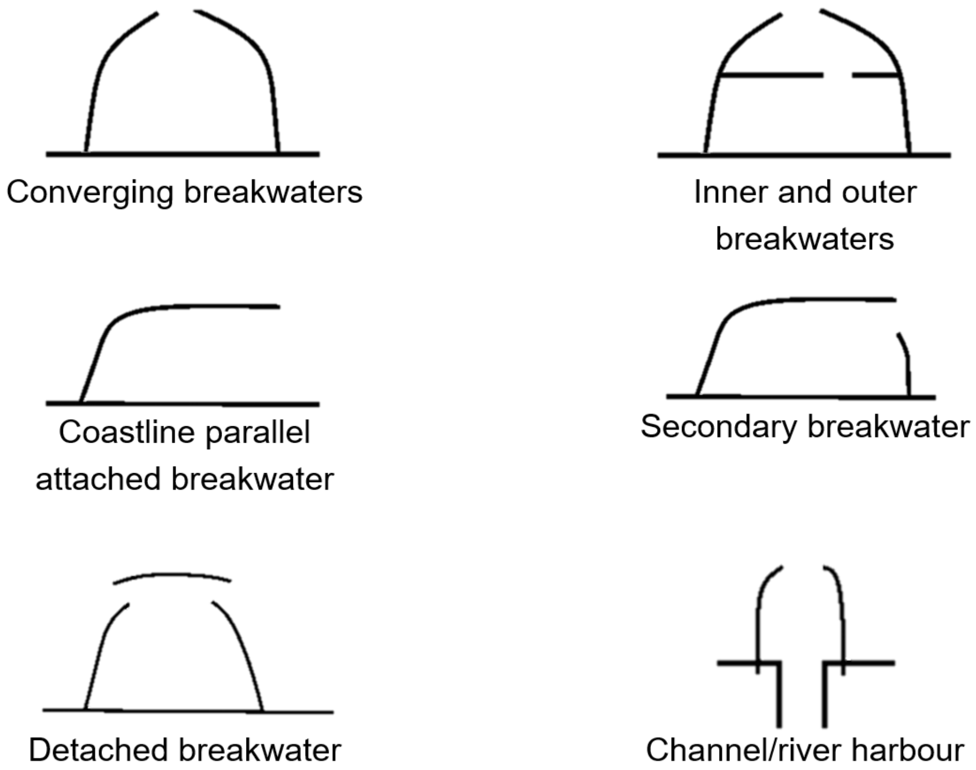


Figure 4.1: Typical harbour configurations

In what follows, the main characteristics of these typical harbour configurations will be discussed more in detail, together with the advantages and disadvantages of each configuration.

4.1 Converging breakwaters

In a harbour layout with converging shore-connected breakwaters (Figure 4.2), two breakwaters will extend from the open coastline towards deeper water, converging to each other and almost equally sized. An overlap of the two breakwaters is possible, where the main breakwater faces the direction of the predominant wave. This overlap will reduce the amount of penetrated wave energy (Thoresen C. A., 2014; Memos, 2018; Mangor, Drønen, Kærgaard, & Kristensen, 2017).

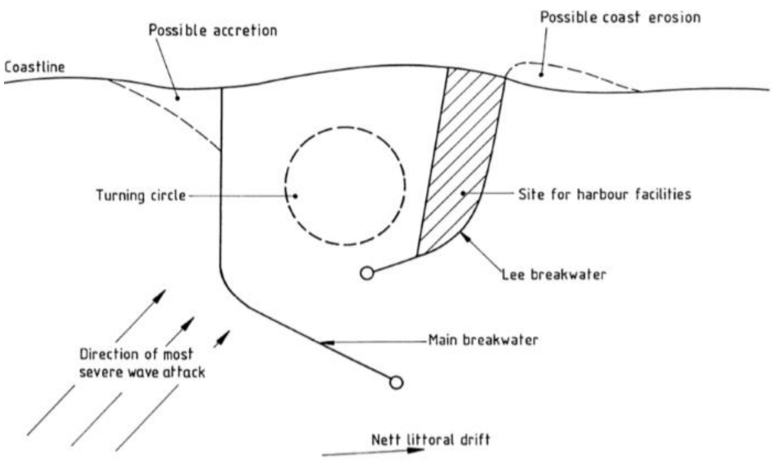


Figure 4.2: Converging breakwater layout (BS 6349, 2000)

The great advantage of this layout is the possible wide wave sector due to the overlapping breakwaters. Another advantage of this configuration is the large harbour basin with a large turning area. As will be discussed in paragraph 5.4, this configuration gives also the possibility to control the longshore sediment transport and minimize the impact of the harbour on the coastal processes. However, due to the large length of the two breakwaters, this type of configuration will be more expensive. This cost will increase even more in the case of a steep sloping bathymetry, which causes large breakwater depths. An example of this harbour configuration is the harbour of Zeebrugge (Belgium) (Figure 4.3).

Figure 4.3: Harbour of Zeebrugge (Belgium) (Google Maps)

4.1.1 Converging breakwaters with inner and outer breakwaters

In case inner breakwaters are added to the previous harbour layout, one gets a converging breakwater layout with inner and outer breakwaters, as shown in Figure 4.4. Important in this type of configuration is the relative location of the inner and outer breakwaters. The two openings are best orientated against the predominant wave direction, to reduce the penetration of waves in the harbour basin (Burcharth, 1993; Goda, 2000). The outer breakwaters minimize the eddy currents and related sedimentation in the outer basin, while the inner breakwaters are constructed for the protection of the main harbour basin (Liu & Burcharth, 1999). In the case of the outer breakwaters, a large overtopping discharge q may be acceptable because these breakwaters don't need any access or space. However, transmission of wave energy still needs to be limited (PIANC, 2016).

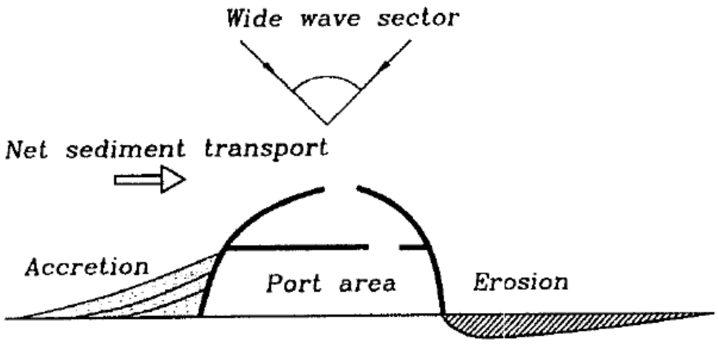


Figure 4.4: Converging breakwater layout with inner and outer breakwaters (Burcharth, 1993)

This type of harbour configuration has the same advantages as the normal configuration of converging breakwaters. Due to the inner breakwaters, the harbour basin can be possible even more sheltered from wave agitation than the previous configuration. An extra disadvantage of this configuration is the increased difficulty to expand the harbour basin in the future. An example of this configuration is the harbour of Hanstholm (Denmark), shown in Figure 4.5.

Figure 4.5: Harbour of Hanstholm (Denmark) (Google Maps)

4.2 Coastline parallel attached breakwater

In this harbour configuration, the main breakwater is connected with the coastline and tries to reach the requested bathymetric line quickly, as shown in Figure 4.6. By following the bathymetric line, a constant depth is achieved along the length of the breakwater, which will reduce the costs. The single main breakwater protects the harbour from the predominant waves occurring from a small, one-sided wave sector (Liu & Burcharth, 1999; Mangor, Drønen, Kærgaard, & Kristensen, 2017).

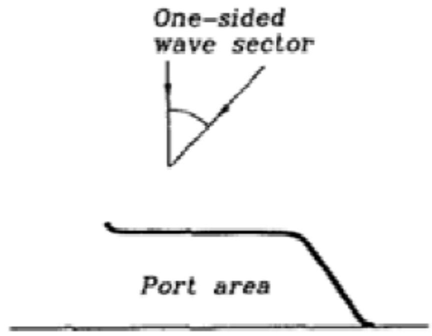


Figure 4.6: Coastline parallel attached breakwater (Burcharth, 1993)

This configuration can be used in case of a natural bay. In this case the configuration takes advantage of the natural features and the costs can significantly be further reduced (PIANC, 2019). This type of breakwater layout provides easy access from land for the construction, operation and maintenance of the breakwater (PIANC, 2016; US Army Corps of Engineers, 2002). However, this configuration can only be used at locations with no significant sediment transport and can have a negative effect on the water quality or sediment movement along the coastline due to return currents which can induce siltation of the harbour basin. The latter will also be discussed in paragraph 5.4 (US Army Corps of Engineers, 2002; Mangor, Drønen, Kærgaard, & Kristensen, 2017). Another disadvantage is the low expansion capability of this configuration. Figure 4.7, the harbour of Sines (Portugal), is an example of this harbour configuration.

Figure 4.7: Harbour of Sines (Portugal) (Google Maps)

4.2.1 Coastline parallel attached breakwater with a secondary breakwater

The coastline parallel attached breakwater configuration can be extended with a secondary breakwater, which is usually orthogonal to the isobathymetric lines (Figure 4.8). This secondary breakwater will close the harbour basin and increase the wave-sector which can be protected (Liu & Burcharth, 1999; Goda, 2000; Mangor, Drønen, Kærgaard, & Kristensen, 2017). This wave-sector can even be more enlarged by extending the main breakwater. This configuration has the same pro's and cons as the previous one. The harbour of Santa Maria di Leuca (Italy), shown in Figure 4.9, is an example of this configuration.

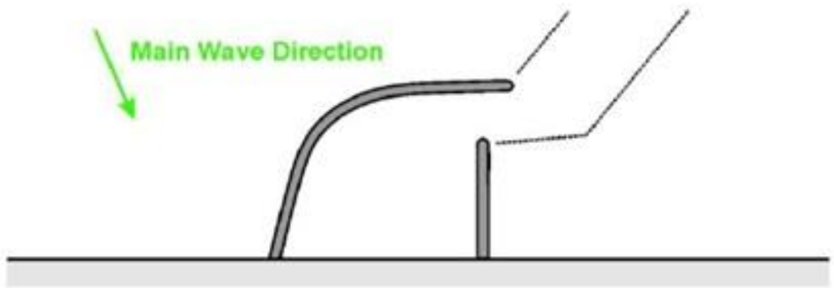


Figure 4.8: Coastline parallel attached breakwater extended with a secondary breakwater (Mangor, Drønen, Kærgaard, & Kristensen, 2017)

Figure 4.9: Harbour of Santa Maria di Leuca (Italy) (Google Maps)

4.3 Detached breakwater

In this layout a nonshore-connected nearshore breakwater is constructed (Figure 4.10). Usually the detached breakwater is constructed parallel to the coastline and the harbour basin is closed by converging breakwaters. The predominant wave direction is perpendicular to the breakwater. This type of breakwater is mostly constructed as rubble mound breakwaters (US Army Corps of Engineers, 2002).

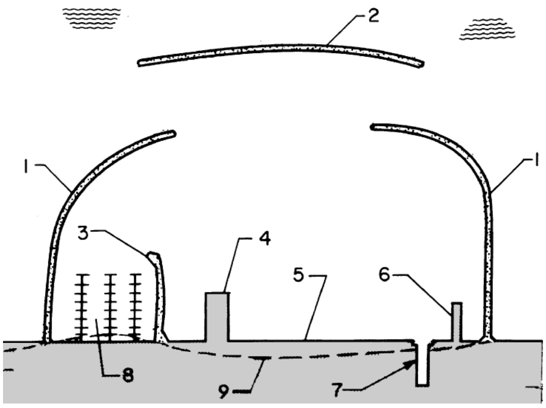


Figure 4.10: Detached breakwater configuration (Memos, 2018)

This type of configuration has several advantages. Due to the two openings of harbour basin at each side of the detached breakwater, one can be used as entrance and the other one as exit which provides an easy vessel circulation inside the harbour. However, this has a significant disadvantage: the vessels have to perform a double change of course when entering the harbour basin which can give manoeuvring problems. Another advantage of this configuration is the possibility to control the longshore sediment transport (Mangor, Drønen, Kærgaard, & Kristensen, 2017). Due to this, their exist a good water exchange inside the harbour basin which ensure the water quality. The detached breakwater can follow the bathymetric lines, as is the case with parallel attached breakwaters and also in this configuration use can be made of the natural features such as a lagoon (PIANC, 2019). Although this configuration has to be designed taking into account the multiple disadvantages such as the tombolo effect behind the breakwater, the high reflection of a vertical detached breakwaters in front of it, the relatively small wave-sectors, the reduced expansion capability and the difficult access for construction and maintenance (Goda, 2000; Mangor, Drønen, Kærgaard, & Kristensen, 2017).

The harbour of La Spezia (Italy), Figure 4.11, is an example of the detached breakwater layout.

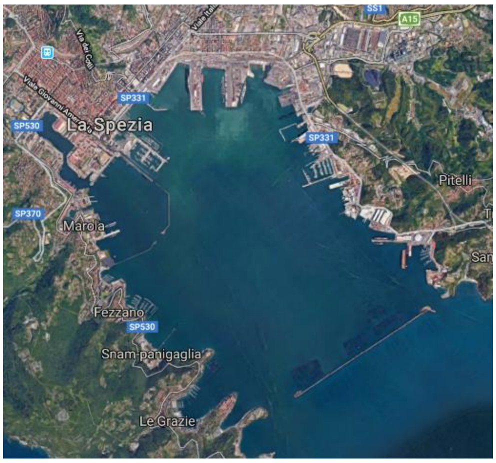


Figure 4.11: Harbour of La Spezia (Italy) (Google Maps)

4.4 Channel or river harbour

In this harbour configuration two parallel breakwaters are constructed perpendicular to the coastline at the inlet of a river (Figure 4.12). The aim of the this configuration is to protect the inlet to an inland harbour; to maintain the approach channel water depth, to protect against siltation and to improve the manoeuvring conditions for the entering vessels (Liu & Burcharth, 1999; PIANC, 2019). To favour the longshore sediment transport or to reduce the amount of maintenance dredging, the designer can opt for a sediment bypass system (Sawaragi, 1995; Mangor, Drønen, Kærgaard, & Kristensen, 2017).

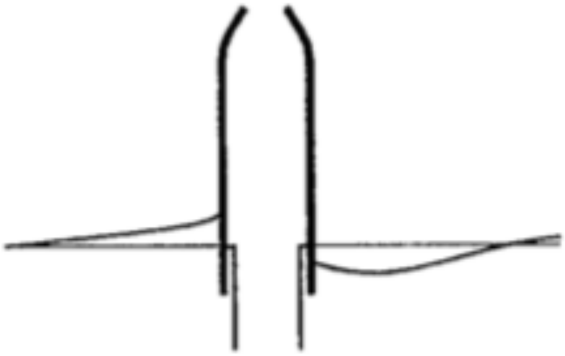


Figure 4.12: River harbour configuration (Burcharth, 1993)

An example of this typical configuration is shown in Figure 4.13

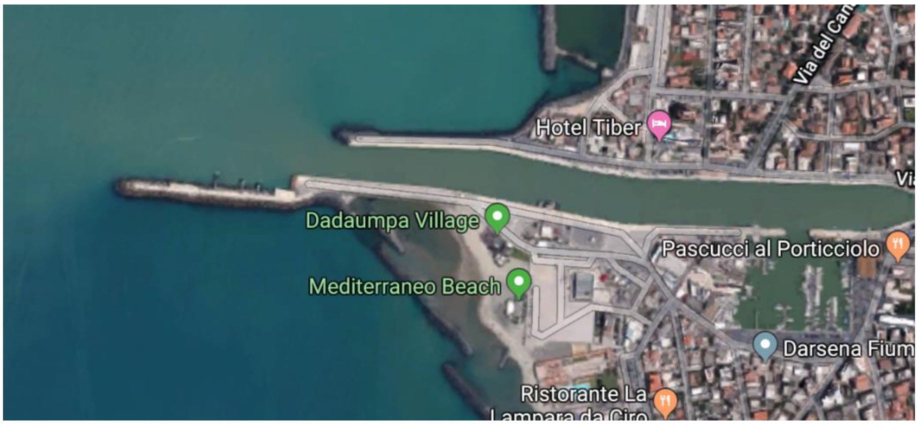
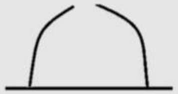
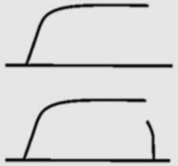
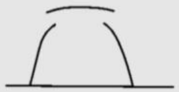
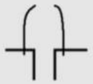


Figure 4.13: Inlet of the inland harbour of Fiumicino (Italy) (Google Maps)

4.5 Conclusion

In this chapter the typical existing harbour configurations with their advantages and disadvantages were discussed. A summary of the discussed harbour configurations and their design characteristics is displayed in Table 4.1. For a correct design of the harbour layout, during the preliminary design engineers and port planners need to examine and consider the different design aspects involved. In chapter 5, the most relevant design aspects which need to be considered during the preliminary phase of a harbour layout design, are identified and explained in detail.

Table 4.1: Summary of harbour configurations

TYPE	Port location	Layout	Navigation	Wave protection	Morphological aspect	Bathymetry	Cost
Converging breakwater 	Open coastline Artificial	Overlap possible: main breakwater faces direction of greatest wave exposure	Large turning area	Wide wave sector	Possibility to control longshore sediment transport (longshore currents) Minimize coastal impact	Steep sloping bathymetry will cause large depths	Large length Large depth, higher cost
Converging breakwater: inner and outer breakwater 	Open coastline Artificial	Outer breakwaters to minimize eddies and related sedimentation in outer basin Inner breakwater for protection of port area	Large turning area	Wide wave sector Inner basin more protected	Possibility to control longshore sediment transport (longshore currents) Minimize coastal impact Less sedimentation in outer basin	Steep sloping bathymetry will cause large depths	Large length Difficult to expand Large depth, higher cost
Coastline parallel breakwater 	Natural bay Artificial	Main breakwater attached with coastline in a way to rapidly reach the requested bathymetric line Allow access from land for construction, operation, and maintenance	Extension of breakwater to improve manoeuvre conditions Sometimes large turn is required to enter harbour	Configurations for large and small wave sectors (optional secondary breakwater)	Only with no significant sediment transport or littoral drift May have an adverse impact on water quality or sediment movement along the coast Approach channel siltation	Can follow bathymetry	Less expensive due to reduced depth Less expensive if combined with natural protection (bay) Low expansion capability
Detached breakwater 	Natural bay Lagoon Artificial	Detached and parallel to coastline Converging breakwaters to close harbour basin	Easy vessel circulation due to double entrance Double change of route	Predominant wave crests approach parallel to the coast Reduced wave penetration Relatively small wave sectors	Possibility to control longshore and cross-shore sediment transport and good water exchange Tombolo effect	Detached breakwater can follow bathymetry	Easy to enlarge Less expensive if combined with natural protection (bay or lagoon)
Channel/River breakwater 	River inlet	Two parallel breakwaters normal to coastline	Improved manoeuvring conditions		Protection of inlet to inland port to maintain approach channel water depths Possible sediment bypass system		Additional cost for sediment bypass systems

5 DESIGN ASPECTS

During the literature review the most important design aspects were identified, which need to be considered for the preliminary phase of a harbour layout design. Following design aspects will be discussed in more detail in the following paragraphs:

- Port location
- Navigation
- Metocean data
- Morphological aspects
- Bathymetry
- Geotechnical aspect
- Preliminary cost

Based on these design aspects a methodology will be identified later on. Often, the data needed to consider the design aspects are not applicable or available to the specific harbour location but is of a more general nature. In this case, adequate assumptions must be made (Ligteringen, 2017). Figure 5.1 gives an overview of the design aspects and their sub aspects.

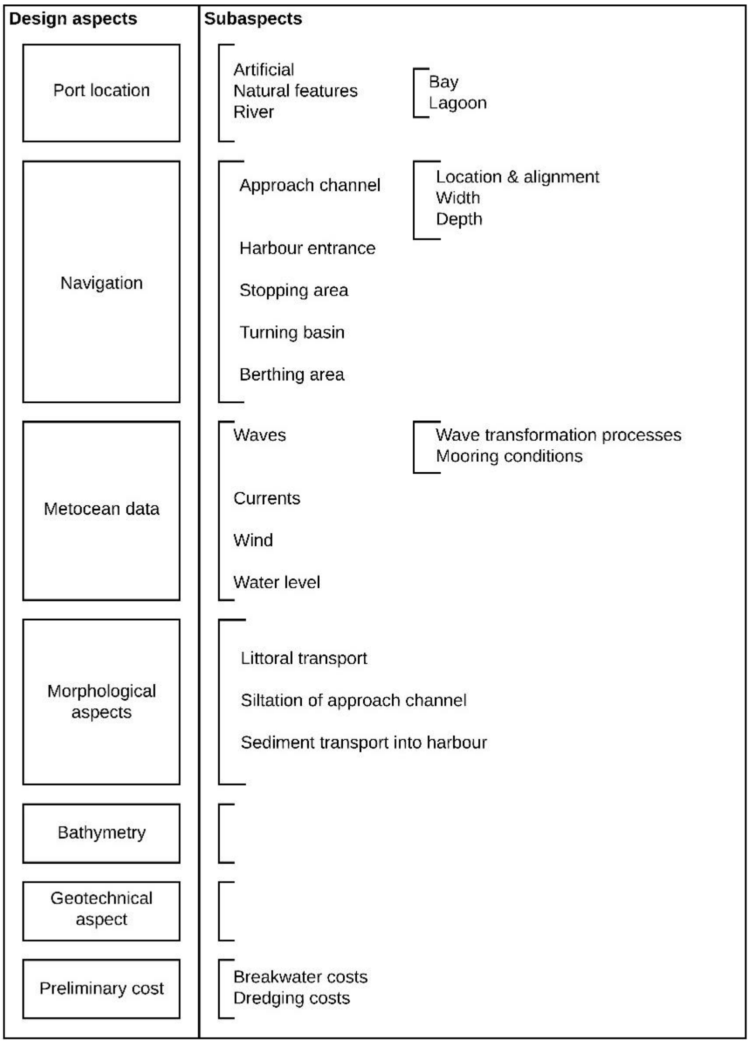


Figure 5.1: Design aspects

5.1 Port location

The first important design aspect is the port location or, as Memos (2018) and Tsinker (1997) call it, the port category. Harbours can be classified on the basis of their geographical location relative to the shoreline or coast. Figure 5.2 shows several port classifications. The location of the harbour can have some geographical features which can ensure already limited sheltering from ambient metocean conditions including wind, waves and currents. Such features can be a natural bay or a lagoon. These locations reduce the need for artificial protection structures as breakwaters and thus significantly reduce the construction cost. In case of an artificial harbour, the harbour is constructed along the coastline by excavations and earth backfilling. An artificial harbour also needs adequate material sources to construct extensive breakwater protecting the water area. It is thus clear that different locations ask for different configurations of the breakwaters layout and the port location can already give a first indication of the optimal harbour configuration (Tsinker, 1997; US Army Corps of Engineers, 2002; Memos, 2018; PIANC, 2019).

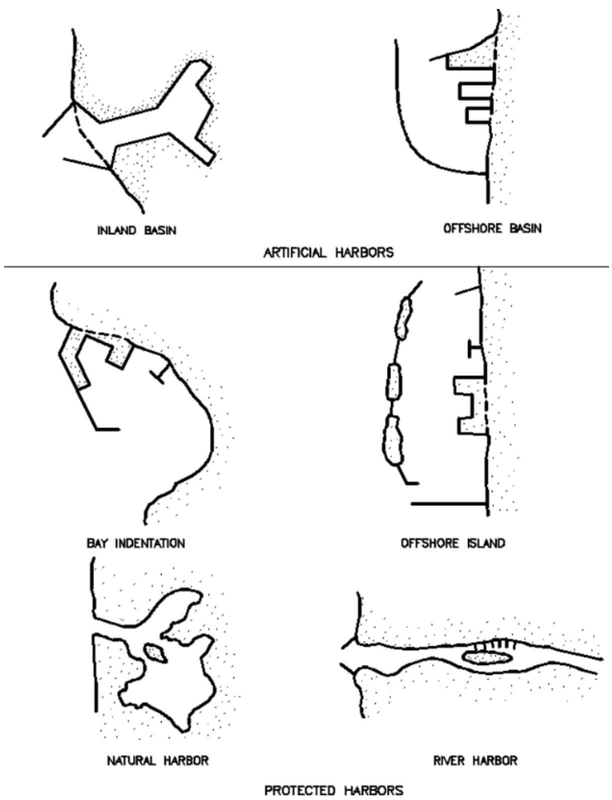


Figure 5.2: Port location (US Army Corps of Engineers, 2002)

A special case of port category is a river port. In this case the harbour is not located on the coastline, but more inland and the river mouth acts as an entrance of the port. As was discussed in previous chapter, this port category needs breakwaters to protect the river inlet. However, in what follows there will be more focus on the artificial type and the natural bay type of harbours.

The choice of harbour category is therefore highly influenced by the presence of land, backfilling material, soil characteristics, water depth, environmental circumstances and more (PIANC, 2019; US Army Corps of Engineers, 2002).

5.2 Metocean data

The Metocean data consists of the wave, wind and current conditions at the harbour location. As mentioned in previous paragraph, these conditions affect the manoeuvring of the vessel outside the harbour and the wave penetration into the harbour that must be limited and thus be examined. In the case of too large wave heights, and thus too much wave energy entering the harbour, the vessels cannot operate or be (un-)loaded inside the harbour. This situation leads to downtime, time the harbour can not operate.

5.2.1 Wave conditions

The wave condition in and near the harbour is one of the most important aspect in the preliminary design of the harbour. The designer must have access to data consisting of detailed knowledge of the wave activity which include the wave height H and the frequency of occurrence of a specific wave height, the return period RP. These data can be found at global wave databases, hindcast models, country-specific databases and national meteorological institutions, or by performing local measurements in later design stages (PIANC, 2019).

For the design of the coastal structures, such as breakwaters, both normal and extreme events are needed. This means wave events with different return periods need to be evaluated. For the design of the harbour layout and for evaluating the impact on harbour activities, such as loading and unloading, seasonal and annual event data is needed. In this case, wave conditions to be considered are therefore associated with smaller return periods (Ligteringen, 2017; PIANC, 2019; Verhagen & van den Bos, 2017; Sorensen, 2006).

When wave conditions in front of the harbour entrance are known, for instance due to the presence of measurement stations, they can be directly used to compute the wave heights inside the harbour basin. In other, and more general cases, the wave conditions are only available in deep water, further away from the coastline.

Due to the effect of bathymetry and currents, the wave will transform when it approaches the coastline. These processes are called the wave transformation processes (US Army Corps of Engineers, 2002; Goda, 2000). Following processes will be further considered during the elaboration of the tool:

- Wave shoaling
- Wave refraction
- Wave breaking
- Wave diffraction
- Wave reflection
- Overtopping and transmission

These processes will be estimated by the use of simplified methods, explained in the next chapter. It is worth underlying that it is certainly possible to compute the metocean conditions outside and inside the harbour using advanced numerical wave models or physical models. However, those tools are more complex and usually applied when different alternatives of harbour layout are already proposed during preliminary phases.

For the sake of completeness, the remaining processes are listed below, these processes will also have an influence on the wave propagation (US Army Corps of Engineers, 2002).

- Dissipation due to friction
- Dissipation due to percolation
- Additional growth due to wind
- Wave-current interaction
- Wave-wave interaction

However, they will not be used in the further elaboration of this thesis.

5.2.1.1 Mooring conditions

Wave characteristics will change when approaching from deep water towards the harbour entrance due to previous mentioned wave processes. Upon entering the harbour basin, the waves will be diffracted, reflected and transmitted by overtopping. The former processes determine the wave conditions inside the harbour basin. These wave conditions will induce vessel movements and will influence the manoeuvrability and operating of the vessels. A severe wave climate will ensure that no vessel operations can take place inside the harbour, such as loading and unloading. The limit wave heights are indicated by the mooring conditions.

Two possible situations can be distinguished upon determining the mooring conditions. In the first situation the vessels have to stop their operations such as loading and unloading but they can remain moored at the quay. This is called the operational limit condition. In the second situation, the wave climate is even more severe so that the vessels can't stay moored and have to leave the harbour to fly to open sea, this is the limit state condition (PIANC, 2019; Ligteringen, 2017). Puertos del Estado (2007) indicates that the operational limit condition has also to be used in the case of navigating and manoeuvring, stopping and turning, through the harbour basin.

These mooring conditions are determined by the design limits of structures, cargo packaging and cargo handling machinery, mooring equipment, vessel manoeuvrability and the tugs available. Further, these limit conditions depend on the size and type of vessel as it is possible that a large vessel does not feel a certain wave, while a smaller vessel can be forcibly thrown through the same waves (PIANC, 2019; Ligteringen, 2017; Puertos del Estado, 2007; Sawaragi, 1995; ASCE, 2012).

As is the case for dimensioning the manoeuvring areas, many different guidelines exist for the determination of the mooring conditions. Some of these guidelines express the mooring conditions with means of the wave height H in meters at the berthed vessel while others use the vessel movements in meters of the berthed vessel. However, the implementation of the vessel movements in the preliminary design are more complex and in what follows only the mooring conditions expressed as wave heights will be used.

PIANC (2019) mentions the mooring conditions in Table 5.1, depending on the type of vessel.

Table 5.1: Mooring conditions (PIANC, 2019)

Description	Limiting wave height at berths H_s [m]
Vessel berthing	
Forces longitudinal to the quay	2.0
Forces transverse to the quay	1.5
Loading and unloading operation stoppage	
(Operational conditions)	
Forces longitudinal to the quay	
Oil tankers	
< 30.000 DWT	1.5
30.000 DWT – 200.000 DWT	2.0
> 200.000 DWT	2.5
Bulk carriers	
Loading	1.5
Unloading	1.0
Liquid Gas Carriers	
< 60.000 m³	1.2
> 60.000 m³	1.5
General cargo merchant ships, Deep sea fishing	1.0
boats and refrigerated vessels	
Container ships, Roll-on/Roll-off (RoRo) ships	0.5
and ferries	0.5
Liners and cruise ships	0.6
Fishing boats	
Forces transverse to the quay	
Oil tankers	1.0
< 30.000 DWT	1.2
30.000 DWT – 200.000 DWT	1.5
> 200.000 DWT	
Bulk carriers	1.0
Loading	0.8
Unloading	
Liquid Gas Carriers	0.8
< 60.000 m³	1.0
> 60.000 m³	0.8
General cargo merchant ships, Deep sea fishing	
boats and refrigerated vessels	0.3
Container ships, RoRo ships and ferries	0.3
Liners and cruise ships	0.4
Fishing boats	
Vessel at quay (Limit state conditions)	
Oil tankers and Liquid Gas Carriers	
Actions longitudinal to the quay	3.0
Actions transverse to the quay	2.0
Liners and Cruise ships	
Actions longitudinal to the quay	1.0
Actions transverse to the quay	0.7
Recreational boats	0.4
Actions longitudinal to the quay	0.4

Actions transverse to the quay	0.2
Other types of vessels	Limitations imposed by the design loads

Ligteringen (2017) suggests using the values in Table 5.2. He also mentions that it is very important to asses the wave conditions inside the harbour basin in the preliminary design of the harbour, as it is very difficult to alter this when the breakwaters and harbour structures are constructed. The limiting wave height H_s for a container vessel is more demanding due to the fact that the unloading and loading equipment needs a higher amount of precision. In case of a RoRo vessel, the ramp is connected to the quay wall, which means it is very sensitive to the wave climate. In the design of the breakwater it is also necessary to look at the wave period of the penetrating waves. For a sea swell with a wave period equal to 12 s to 16 s it is more difficult to provide the appropriate sheltering than for a wind wave with a period between 6 to 8 s (Ligteringen, 2017).

Table 5.2: Operational conditions (Ligteringen, 2017)

Type of vessel	**Limiting wave height at berths H_s [m]**	
	0° (head or stern)	**45° - 90° (beam)**
General cargo	1.0	0.8
Container, RoRo vessel	0.5	-
Dry bulk (30.000 – 100.000); loading	1.5	1.0
Dry bulk (30.000 – 100.000); unloading	1.0	0.8 – 1.0
Tankers 30.000 DWT	1.5	-
Tankers 30.000 – 200.000 DWT	1.5 – 2.5	1.0 – 1.2
Tankers > 200.000 DWT	2.5 – 3.0	1.0 – 1.5

Wave heights exceeding the operational limit values of Table 5.2 will stop the vessel operations inside the harbour. When the wave heights even more increase, up to the limit state conditions, the ships will have to leave the harbour, which can be the case in older harbours while in newer harbours wave disturbance is less significant due to locks or in case of a upriver harbour. New constructed harbours can't afford this lose of productivity, downtime, and these limit state conditions are computed as a balance between the construction cost of the breakwaters and the downtime costs. For a harbour basin, a 1/10-year sea event can suffice as for a more offshore berth a once-per-year wave event can be opted (Ligteringen, 2017; PIANC, 2019).

ASCE (2012) developed two criteria based on Table 5.3. The table is made for vessels with an overall vessel length L_v between 12 and 61 m and for the wave conditions taking place in a small vessel harbour which would result in considerable damage to the vessels, the harbour construction or pose a threat to the safety of persons. These criteria are made for a "Good" wave climate, in case someone wants to know the criteria for respectively "Excellent" and "Moderate" wave climate, these values have to be multiplied by 0.75 resp. 1.25. These values are chosen in the way that a "Moderate" wave condition has 125% higher waves than a "Good" wave condition and an "Excellent" wave climate is 75% of good. The original criteria were presented in English units (ft) and are assessed for three different return period: a 50-year wave event, a 1-year wave event and a weekly wave event. The wave direction θ is relative to the head sea, so a head sea is 0° while a beam sea would be 90°, everything between these two is denoted as an oblique sea (ASCE, 2012).

Table 5.3: Generalized Harbour Tranquillity Goals (ASCE, 2012)

Provisionally Recommended Criteria for a "Good" Wave Climate in Small Craft Harbors			
Direction θ and peak period T_p of design wave	**Wave event exceeded once in 50 years ($RP = 50\ yr$)**	**Wave event exceeded once a year ($RP = 1\ yr$)**	**Wave event exceeded once each week ($RP = 1/52\ yr$)**
Head sea T_p < 2 s	These conditions are not likely to occur during this event	Less than 0.31 m	Less than 0.31 m
Head sea 2 s < T_p < 6 s	Less than 0.61 m	Less than 0.31 m	Less than 0.15 m
Head sea T_p > 6 s	Less than 0.61 m	Less than 0.31 m	Less than 0.15 m
Oblique sea	Less than (0.61 – 0.38 sinθ) m	Less than (0.31 – 0.15 sinθ) m	Less than (0.15 – 0.08 sinθ) m
Beam sea T_p < 2 s	These conditions are not likely to occur during this event	Less than 0.31 m	Less than 0.31 m
Beam sea 2 s < T_p < 6 s	Less than 0.23 m	Less than 0.15 m	Less than 0.08 m
Beam sea T_p > 6 s	Less than 0.23 m	Less than 0.15 m	Less than 0.08 m

It can be seen that the criteria are most limiting for a peak wave period T_p between 2 s and 6 s, which is the general range of natural roll and heave period for small vessels. With means of Table 5.3, the first criteria, based on the allowable wave height, can be generated (ASCE, 2012):

$$H_{allowable} = 0.3048\,(A - B \sin\theta) \tag{5.1}$$

with the wave direction θ relative to a head sea and the values for the constants A and B in Table 5.4. It should be mentioned that the formula was original expressed in feet.

It is clear that this criterion cannot be used in a general case, only in the case of small vessels, as there is only two parameters defining the allowable wave height $H_{allowable}$ which are the wave direction θ and the return period RP. The previous formula can be plotted for the different return periods and for the different wave climates, as shown in Figure 5.3. By doing this the tolerances can be shown for the three conditions: Good, Moderate and Excellent.

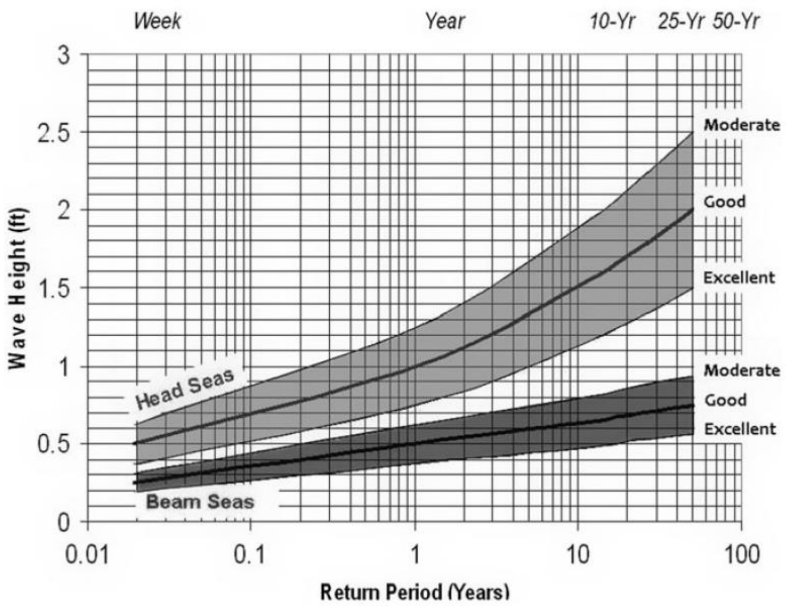


Figure 5.3 Tolerance for mooring criterion (ASCE, 2012)

A second criterion comes in place if the peak wave period T_p exceeds the value of 6 s, this second criteria is the horizontal movement of the vessel at the berth.

$$E_{goal} = 0.3048\,(C - D\sin\theta) \tag{5.2}$$

with the allowable movement E_{goal}, the wave direction θ relative to a head sea and the values for the constants C and D can be found in Table 5.4. This second criterion is needed due to the higher occurring loads in the anchoring and mooring equipment applied to a berthed vessel.

Table 5.4: Constants for mooring conditions criteria adapted from (ASCE, 2012)

Return Period RP	**A [-]**	**B [-]**	**C [-]**	**D [-]**
Weekly events	0.5	0.25	1.5	0.75
Yearly events	1.0	0.5	2.0	1.0
50-year events	2.0	1.25	4.0	2.0

To compute the horizontal movement of a vessel, one can take 65% of the total movement of a water particle in a wave at the free surface in case of wave periods T_p between 5 s and 20 s according to Hiraishi,et al. (1997) and ASCE (2012):

$$E_{hor} = 0.65\,H\,\frac{\cosh\left(\frac{2\pi d}{L}\right)}{\sinh\left(\frac{2\pi d}{L}\right)} \tag{5.3}$$

with the horizontal movement E_{hor}, the wave height H, the water depth d and the wavelength L. However, this last criterium will not be used in remaining part as this covers the ship motions.

MLIT (2009) suggests the use of following very general guidelines, shown in Table 5.5, which are only based on the size of the vessel. Because these values only depend on the size of the ship, they are rather general and should be used with caution.

Table 5.5: Operational limiting wave heights (MLIT (Ministry of Land, Infrastructure, Transport and Tourism, Japan), 2009)

Vessel type	Operational limiting wave height H_s [m]
Small vessel	0.3
Medium/large vessel	0.5
Very large vessel	0.7 – 1.5

It should be pointed out that the impact of the wave conditions on a vessel is also depending on the wave period T_p (Goda, 2000). A swell wave with a larger wave period and a wind wave with a smaller period will have a different effect, even if both periods come with the same wave height. For this reason, one can opt to transform the mooring conditions from the limiting wave height to the allowable vessel's movements. However, it is more difficult to asses these movements in a preliminary design phase as this is a very complicated hydrodynamic problem. It is thus important that during the detailed design of the harbour both the wave transformation processes, and the vessel movements are modelled in a numerical and physical way.

5.2.2 Current conditions

Currents will have an effect on the vessel's manoeuvring and on the sedimentation in and near the harbour, both will be discussed more in detail further on. Large crosscurrents in the approach channel will hinder the vessel from entering the harbour, so that the entrance has to be widened or reoriented (PIANC, 2014; Ligteringen, 2017). Most guidelines take this into account in the determination of the approach channel width. Large current velocities can also affect the hydraulic stability of the breakwater armour units and filter layers (Mangor, Drønen, Kærgaard, & Kristensen, 2017).

These currents can be created by waves, tides or wind. Waves approaching the coastline under an angle will induce a longshore current which is one of the main driving forces of sediment transport along the coastline (Goda, 2000). In case of a bad design of the harbour entrance, return currents can occur which will induce harbour siltation, as shown in Figure 5.4 and will be discussed further on.

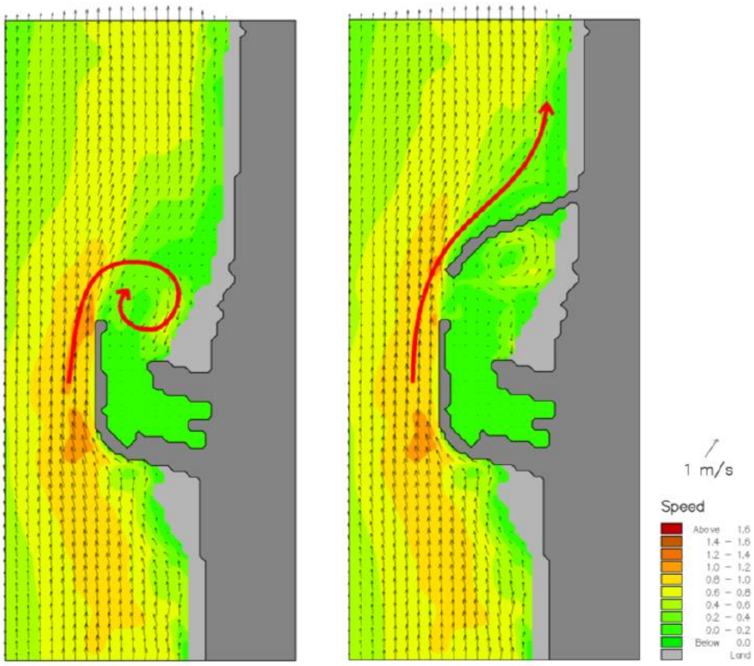


Figure 5.4: Current patterns around ports. Left: one parallel attached breakwater. Right: converging breakwaters (Mangor, Drønen, Kærgaard, & Kristensen, 2017)

Information about the currents can be found on nautical charts, tidal stream atlases, pilot books or even numerical modelling (PIANC, 2019).

5.2.3 Wind conditions

Wind has a dual effect regarding the design of harbours (PIANC, 2014). Firstly, wind will generate waves far away from the harbour and secondly the wind will have a direct impact on the navigation of the vessels. The latter should be taken into account during the design of the manoeuvring areas, as is done by most of the recommended guidelines in the previous paragraph. A cross wind in the approach channel will cause the vessel to drift sideways or induces an angle, both increase the width required for manoeuvring (PIANC, 2014).

Wind will generate waves far away, but in the case of large harbours with the possibility of having a large fetch length, wind can create locally generated waves which cannot be neglected and have to be examined during the design. However locally generated waves will have shorter periods which means they will have generally a lower impact on the vessels (PIANC, 2019).

Wind data is mostly found in wind roses which show the yearly distribution of wind directions and can be used to identify the main wind direction, Figure 5.5. These wind roses can mostly be found at local meteorological organisations (PIANC, 2019; Thoresen C. A., 2014).

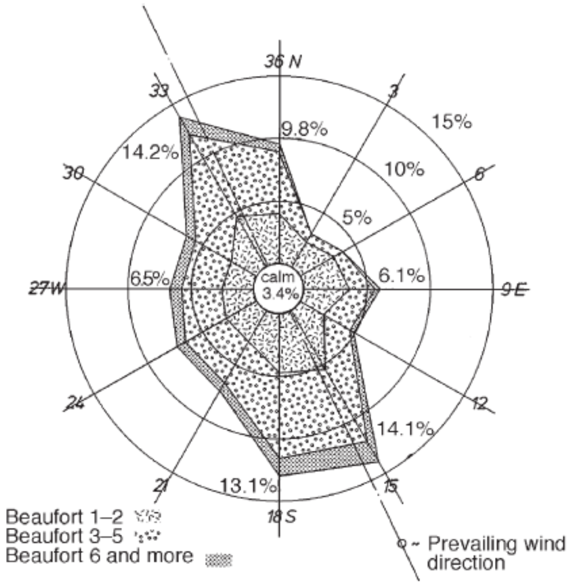


Figure 5.5: Wind rose (Thoresen C. A., 2014)

5.2.4 Water level

The design of the harbour depth is always related to the water level. The water level is not a constant and variates due to the following processes (PIANC, 2014):

- Astronomical tides
- Meteorological effects
- Wave set-up
- Harbour oscillations or seiches
- Seasonal variations
- Long-term variations (Climate change)

These factors all affect the design water level, related to a specific reference plane, the reference level or datum level. The standard reference level in a harbour design can be Chart Datum, Navy Chart Datum, Mean Sea Level (MSL), Lowest Astronomical Tide (LAT) or others. It is important to pay attention to the reference level used during the project as they differ from each other (PIANC, 2014).

The astronomical tides can be found by long-term continuous measurements of a location near the harbour site, these data are available in worldwide or national databases. When such data is not available, local measurements can be performed for a period of at least one or two months (PIANC, 2019).

The water level may be influenced by meteorological effects. These effects can not be predicted for the long-term and are considered by the means of statistical analysis (Verhagen & van den Bos, 2017).

During the development of the tool, a constant design water level is assumed.

5.3 Navigation

A ship needs sufficient space to navigate safely to and through the harbour. For this reason, manoeuvring areas need to be designed. These are areas with one or more of the following purposes: stopping the ship, turning the ship and/or gaining speed (Puertos del Estado, 2007).

A ship sailing from open sea towards the sheltered area of a harbour needs to sail at a minimum speed, which is sufficient to provide controlled navigation as a function of the metocean conditions outside and inside the harbour. After entering the harbour, the ship needs to berth at one of the harbour terminals by reducing its speed to almost zero. In many cases the ship has to turn to get aligned with the harbour quays or berths. On departure, the processes are similar, this time the ship has to accelerate to leave the harbour under favourable conditions (Puertos del Estado, 2007; PIANC, 2014).

The manoeuvring areas and their functions are listed below and are shown in Figure 5.6:

- Approach channel: connecting open sea with the inner harbour and the turning basin
- Harbour entrance: providing a safe access to the harbour
- Stopping area: provide space for the stopping manoeuvre
- Turning basin: provide space for turning manoeuvre of vessels
- Berthing area: providing space to berth

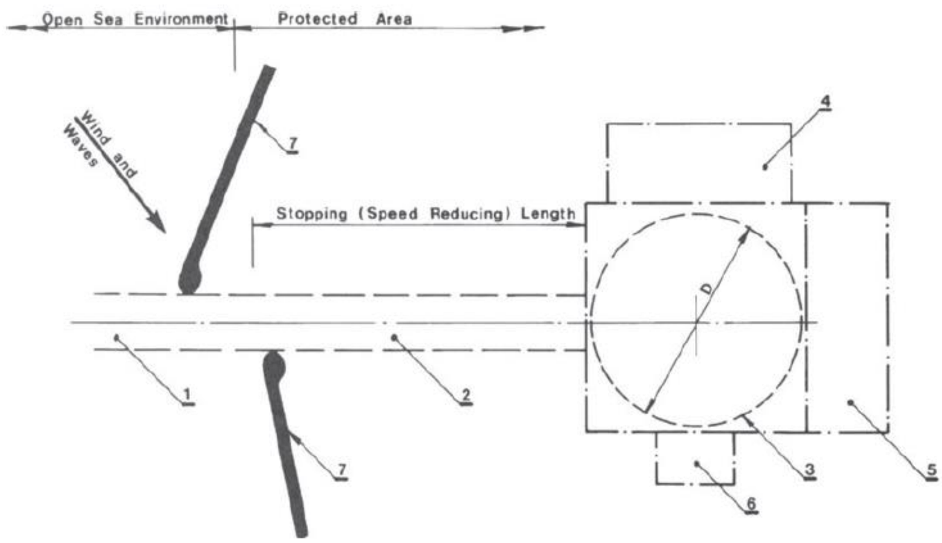


Figure 5.6: Harbour water area: 1) outer approach channel; 2) inner approach channel; 3) Turning basin; 4) Anchorage area; 5) Berthing area; 6) Special purpose area; 7) Breakwaters (Tsinker, 1997)

The dimensions of these areas mainly depend on the design vessel, namely its manoeuvring behaviour under the influence of the metocean conditions and its size (PIANC, 2014; PIANC, 2019; Thoresen C. A., 2014; Ligteringen, 2017; Puertos del Estado, 2007).

Because the layout of a harbour is largely depending on its water area and the alignment of the approach channel, the manoeuvring areas have a great influence on the outline of the harbour breakwaters. The dimensions of these manoeuvring areas are of high importance because of two reasons: firstly because the breakwaters contribute to a large part of the initial costs of the harbour construction and, secondly, because it is difficult to adapt the breakwater layout once the harbour is built (PIANC, 2014). In what follows the different manoeuvring areas will be discussed in detail.

5.3.1 Approach channel

The approach channel is considered to be any part of waterway that connects the moorings of a harbour and the open sea (PIANC, 2014). A distinction can be made between two different types. The outer channel is located in open water and is exposed to the open sea which can cause significant vessel movements. Contrary, the inner channel is more sheltered from wave agitation, due to the protective ability of one or more breakwaters.

The approach channel usually ends in one of the other manoeuvring areas mentioned above, which allows stopping, turning and mooring. Four different design parameters determine the layout of the approach channel: location, alignment, width and depth. Numerous guidelines regarding the design can be found in the literature (PIANC, 2014; Ligteringen, 2017; Tsinker, 1997; NAVFAC, 1982; Velsink, 1994; Puertos del Estado, 2007; Agerschou, et al., 2004).

5.3.1.1 Location and alignment of channel

The location of the approach channel determines the metocean conditions met by the vessels while sailing through the channel, these conditions affect the navigation of the vessels. It also determines the amount of dredging which is needed to acquire the proper depth. The latter is very important as this influences a large part of the initial costs and the maintenance costs.

For the alignment of the approach channel, Ligteringen (2017) provided several recommendations to improve the manoeuvrability of the sailing vessels. The approach channel should be ideally aligned with the prevailing wave and wind direction. The purpose of this is to make the waves appear from “aft” of the ship instead of “quartering” or “beam”, to reduce to risk of unstable vessel movements. However, the alignment of the harbour entrance should at the same time limit the penetration of wave energy. These two demands induce a small angle between the direction of the main wave and the axis of the approach channel, as shown in the Figure 5.7.

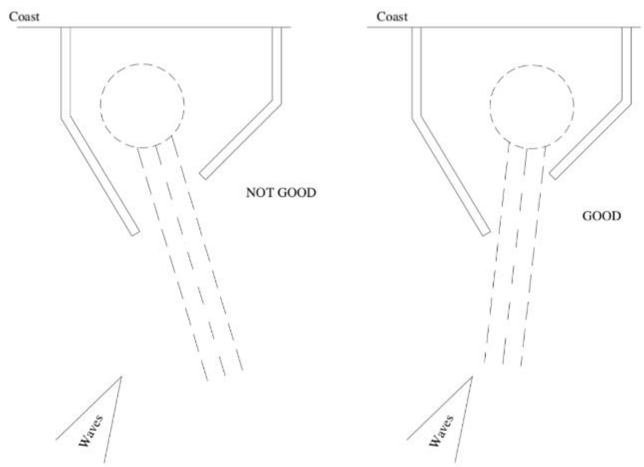


Figure 5.7: Orientation of the approach channel with respect to the wave direction (Ligteringen, 2017)

According to PIANC (2014) it is also recommended that the approach channel is aligned in such a way that the vessel does not sail directly towards the quays. If the channel is perpendicular to the ship berths, it should be aligned on one side of the quay, in that case the ship has to turn to reach the berths. This recommendation of PIANC (2014) reduces the risk of vessels coming into contact with the quay in case of losing control during manoeuvring.

It is important that when the harbour and the approach channel are sheltered by breakwaters, these components do not form a narrow “sleeve”. In a good design, there is enough space behind the breakwater heads for three reasons. At first, vessels navigating in a channel do not like a rigid construction near the channel borders. Secondly, when there is a lateral current along the harbour entrance, vessels need enough space in order to cross this current into more sheltered waters and at last the open water area behind the breakwaters contributes to the diffraction effects and thus reduces the penetration of the wave energy. In Figure 5.8 the breakwater length is not changed, while the more open layout is much safer and there are more opportunities to expand in the future (Ligteringen, 2017).

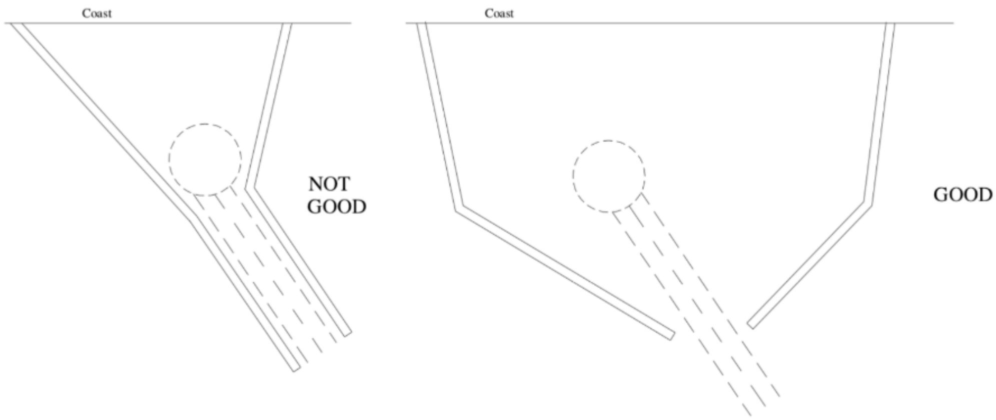


Figure 5.8: Breakwater alignment (Ligteringen, 2017)

In all cases, curves in the approach channel near the harbour entrance or directly behind it should absolutely be avoided, ships need straight courses without the difficulties of manoeuvring through a bend (Tsinker, 1997). In general, the amount of bends should be minimized. Sometimes the design conditions ask for different channel stretches, and bends are unavoidable, than the bend should not be too sharp. Before entering the harbour, a straight channel is needed, to optimize the manoeuvring conditions. The length of this channel is determined by the metocean conditions. Tsinker (1997) recommends using single curves instead of a sequence of smaller curves. Figure 5.9 shows the different parameters defining the alignment of the approach channel.

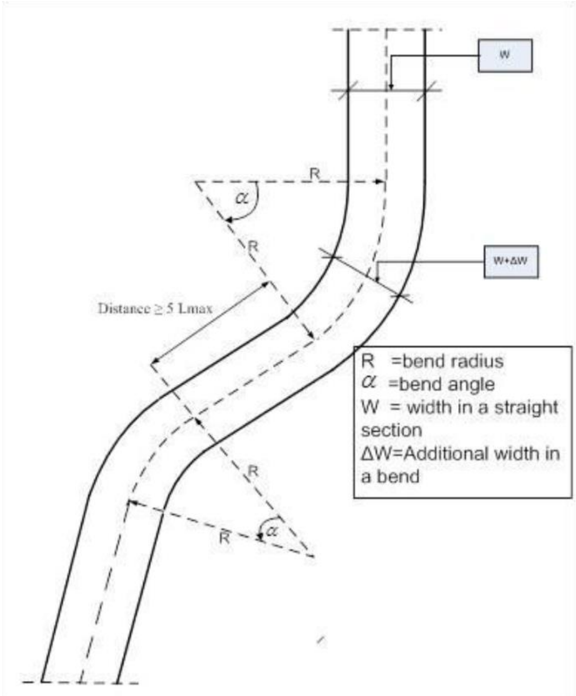


Figure 5.9: Approach channel bend configuration (PIANC, 2014)

NAVFAC (1982) has developed some methods regarding the radius of the bends in the approach channel. These methods are based on the angle of the bend and on the length of the vessel:

1. Angle of deflection

$$\begin{array}{cc} \alpha_b < 25° & R_b = 3L_v \\ 25° < \alpha_b < 35° & R_b = 5L_v \\ \alpha_b > 35° & R_b = 10L_v \end{array} \quad (5.4)$$

2. Vessel length

$$\begin{array}{cc} L_v < 150m & R_{min} \approx 1200m \\ L_v = 150m & R_{min} \approx 2100m \\ 150m < L_v < 210m & R_{min} \approx 1200m - 3000m \end{array} \quad (5.5)$$

with α_b the bend angle, R_b the bend radius, L_v the design vessel's overall length and R_{min} the minimum bend radius. The manoeuvrability of the design vessel influences the radius of the bends, which in turn depends on the angle of the deflection, the vessels speed, the channel characteristics, the Aids to Navigation (AtoN) and the demands placed on the pilot (Ligteringen, 2017). Figure 5.10 shows the turning radius as a function of the rudder angle and the water depth in which the vessel manoeuvres.

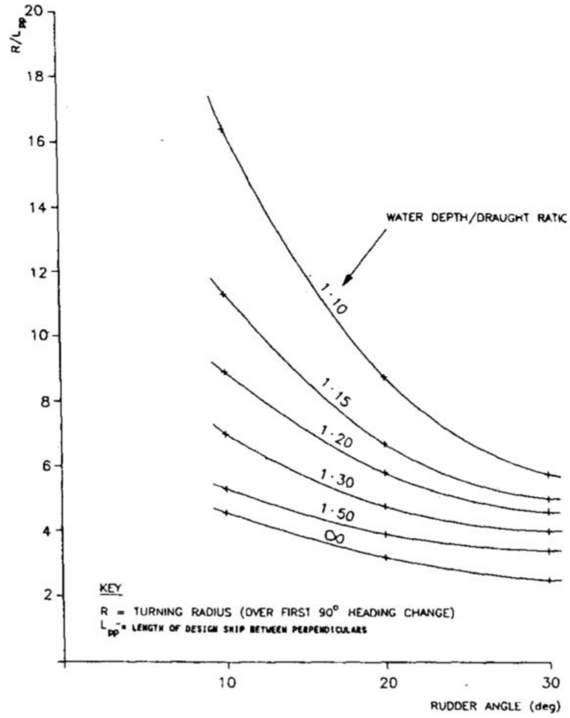


Figure 5.10: Radius as a function of rudder angle and water depth (Ligteringen, 2017)

According to Velsink (1994) the distance between successive bends should be more than 10 times the vessel length L_v and in exceptional cases more than 5 times, but with sufficient local widening of the approach channel.

5.3.1.2 Channel width

A vessel sailing through the approach channel in calm water makes a sinusoidal track and covers a 'basic width' which is larger than the vessel's beam B_v, this is shown in Figure 5.11 (PIANC, 2014; Ligteringen, 2017). The reason for this is both the vessel's operator's reaction speed in interpreting the visual clues indicating the vessel's position in the channel, and that of the vessel when responding to the rudder and main engine.

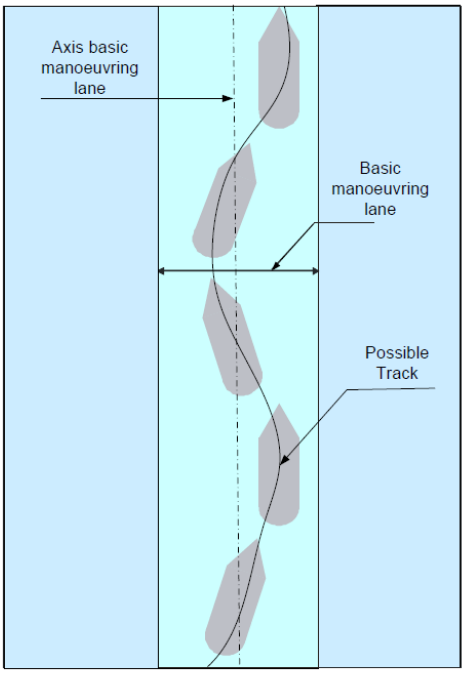


Figure 5.11: Basic manoeuvring width (PIANC, 2014)

The influence of the metocean conditions, wind, current and waves, on the vessel demand extra width to ensure safe navigation. Certain margins are necessary depending on the aids to navigation, the seabed, the amount of visibility and others (PIANC, 2014; Ligteringen, 2017). In order to determine these extra widths, one can consult different sources, each proposing their own values according to their guidelines. These widths are shown in the tables below and can be used in following formulas to determine the design width of the approach channel.

For the width of a straight one-way approach channel (PIANC, 2014):

$$W = W_{BM} + \sum W_i + W_{BR} + W_{BG} \tag{5.6}$$

and for a two-way approach channel (PIANC, 2014):

$$W = 2W_{BM} + 2\sum W_i + W_{BR} + W_{BG} + \sum W_p \tag{5.7}$$

where W_{BM} is the width of the basic manoeuvring lane, $\sum W_i$ the additional widths, W_{BR} and W_{BG} are the bank clearance on the 'red' and 'green' sides of the approach channel and $\sum W_p$ the passing distance. Figure 5.12 shows the different channel width factors.

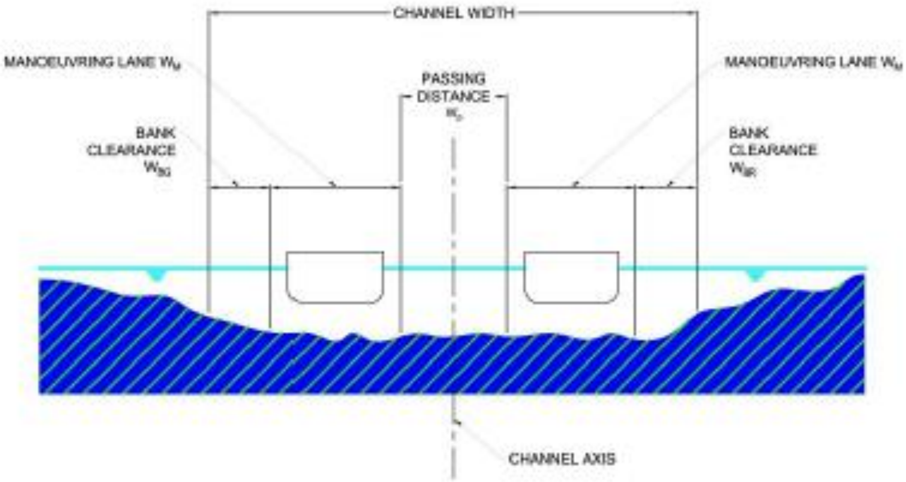


Figure 5.12: Channel width (PIANC, 2014)

Table 5.6 lists the basic manoeuvring lane widths W_{BM} for vessels with good, moderate and poor ship manoeuvring characteristics according to PIANC (2014). The manoeuvrability of the vessel governs the ability to maintain a straight course. In general, the manoeuvrability of tankers and bulk carriers is seen poor, containerships, car carriers, RoRo vessels, liquid natural gas (LNG) and liquid petroleum gas (LPG) vessels moderate, while twin-propeller ships, ferries and cruise vessels have in general a good manoeuvrability.

Table 5.6: Ship Manoeuvrability (PIANC, 2014)

Ship Manoeuvrability	**Good**	**Moderate**	**Poor**
Basic Manoeuvring Lane W_{BM}	1.3 B_v	1.5 B_v	1.8 B_v

Table 5.7 shows the additional widths W_i which need to be taken in account regarding environmental and other navigation effects on the manoeuvring of the vessel, according to the PIANC (2014). These extra widths are in function of the speed of the vessel and the exposure of the approach channel to waves.

Table 5.7: Additional widths W_i in outer and inner approach channel (PIANC, 2014)

Additional Width W_i	**Vessel Speed**	**Outer Channel (open water)**	**Inner Channel (protected water)**
Vessel speed V_s ([kn], with respect to the water)			
$V_s \geq 12\ kn$	Fast	$0.1\ B_v$	
$8\ kn \leq V_s \leq 12\ kn$	Mod	0.0	
$5\ kn \leq V_s \leq 8\ kn$	Slow	0.0	
Prevailing cross wind V_{cw} [kn]			
- mild	Fast	$0.1\ B_v$	
$V_{cw} < 15\ kn$	Mod	$0.2\ B_v$	
(< Beaufort 4)	Slow	$0.3\ B_v$	
- moderate	Fast	$0.3\ B_v$	
$15\ kts \leq V\_cw \leq 33\ kn$	Mod	$0.4\ B_v$	

(Beaufort 4 – Beaufort 7)	Slow	$0.6\ B_v$	
- strong $33\ kn \leq V_{cw} < 48\ kn$ (Beaufort 7 – Beaufort 9)	Fast Mod Slow	$0.5\ B_v$ $0.7\ B_v$ $1.1\ B_v$	
Prevailing cross-current $\boldsymbol{V_{cc}}$ [kn]			
- Negligible $V_{cc} < 0.2\ kn$	All	0.0	0.0
- Low $0.2\ kn \leq V_{cc} < 0.5\ kn$	Fast Mod Slow	$0.2\ B_v$ $0.25\ B_v$ $0.3\ B_v$	$0.1\ B_v$ $0.2\ B_v$ $0.3\ B_v$
- Moderate $0.5\ kn \leq V_{cc} < 1.5\ kn$	Fast Mod Slow	$0.5\ B_v$ $0.7\ B_v$ $1.0\ B_v$	$0.4\ B_v$ $0.6\ B_v$ $0.8\ B_v$
- Strong $1.5\ kn \leq V_{cc} < 2.0\ kn$	Fast Mod Slow	$1.0\ B_v$ $1.2\ B_v$ $1.6\ B_v$	– – –
Prevailing longitudinal current $\boldsymbol{V_{lc}}$ [kn]			
- Low $V_{lc} < 1.5\ kn$	All	0.0	
- Moderate $1.5\ kn \leq V_{lc} < 3\ kn$	Fast Mod Slow	0.0 $0.1\ B_v$ $0.2\ B_v$	
- Strong $V_{lc} \geq 3\ kn$	Fast Mod Slow	$0.1\ B_v$ $0.2\ B_v$ $0.4\ B_v$	
Beam and stern quartering wave heights $\boldsymbol{H_s}$ [m]			
- $H_s \leq 1\ m$	All	0.0	0.0
- $1\ m < H_s \leq 3\ m$	All	$\sim 0.5\ B_v$	–
- $H_s \geq 3\ m$	All	$\sim 1.0\ B_v$	–
Aids to Navigation (AtoN)			
- Excellent		0.0	
- Good		$0.2\ B_v$	
- Moderate		$0.4\ B_v$	
Bottom surface			
- If depth $d \geq 1.5\ T_v$		0.0	
- if depth $d < 1.5\ T_v$ then			
- smooth and soft		$0.1\ B_v$	
- rough and hard		$0.2\ B_v$	
Depth of water $\boldsymbol{d}$			
$d \geq 1.5\ T_v$		0.0	0.0
$1.5\ T_v > d \geq 1.25\ T_v$		$0.1\ B_v$	$0.2\ B_v$
$d < 1.25\ T_v$		$0.2\ B_v$	$0.4\ B_v$

Table 5.8 and Table 5.9 show respectively the additional widths for the bank clearance and the passing distance in a two-way traffic lane.

Table 5.8: Additional width for bank clearance W_{BR} and W_{BG} (PIANC, 2014)

Width for bank clearance (W_{BR} and/or W_{BG})	Vessel Speed	Outer channel (open water)	Inner channel (protected water)
Gentle underwater channel slope (1:10 or less steep)	Fast Moderate Slow	$0.2\ B_v$ $0.1\ B_v$ 0.0	$0.2\ B_v$ $0.1\ B_v$ 0.0
Sloping channel edges and shoals	Fast Moderate Slow	$0.7\ B_v$ $0.5\ B_v$ $0.3\ B_v$	$0.7\ B_v$ $0.5\ B_v$ $0.3\ B_v$
Steep and hard embankments, structures	Fast Moderate Slow	$1.3\ B_v$ $1.0\ B_v$ $0.5\ B_v$	$1.3\ B_v$ $1.0\ B_v$ $0.5\ B_v$

Table 5.9: Additional width for passing distance in a two-way traffic lane W_p (PIANC, 2014)

Width for passing distance W_p	Outer Channel (open water)	Inner Channel (protected water)
Vessel speed V_s [kn] - fast: $V_s \geq 12\ kn$ - moderate: $8\ kn \leq V_s < 12\ kn$ - slow: $5\ kn \leq V_s < 8\ kn$	 $2.0\ B_v$ $1.6\ B_v$ $1.2\ B_v$	 $1.8\ B_v$ $1.4\ B_v$ $1.0\ B_v$

Ligteringen (2017) chooses a slightly different approach. In his guideline, he uses the same bank clearance at both sides of the vessel and came up with following formula for a straight one-way approach channel:

$$W = W_{BM} + \sum W_i + 2W_B \tag{5.8}$$

For a two-way approach channel the separation distance between two manoeuvring lanes is added:

$$W = 2\left(W_{BM} + \sum W_i + W_B\right) + W_p \tag{5.9}$$

where W_{BM} represents the width of the basic manoeuvring lane, $\sum W_i$ is sum of the additional widths, W_B the bank and $\sum W_P$ the passing distance. The basic width is based on the draught of the vessel relative to the depth of the approach channel, instead of the type of the vessel as in the PIANC (2014). He also suggests slightly different values for the additional widths W_i, shown in Table 5.10.

Table 5.10: Additional widths W_i (Ligteringen, 2017)

Width component	Condition	Width [m]
Basic width ($\boldsymbol{W_{BM}}$)	$1.25\ T_v < d < 1.5\ T_v$ $d < 1.25\ T_v$	$1.6\ B_v$ $1.7\ B_v$
Additional width ($\boldsymbol{W_i}$)		
Prevailing cross-winds	$15 - 33\ kn$ $33 - 48\ kn$	$0.4\ B_v$ $0.8\ B_v$
Prevailing cross-current	$0.2 - 0.5\ kn$ $0.5 - 1.5\ kn$ $1.5 - 2.0\ kn$	$0.2\ B_v$ $0.7\ B_v$ $1.0\ B_v$
Prevailing longitudinal current	$1.5 - 3\ kn$ $> 3\ kn$	$0.1\ B_v$ $0.2\ B_v$
Prevailing wave height	$1 - 3\ m$ $> 3\ m$	$1.0\ B_v$ $2.2\ B_v$
Aids to navigation	Vessel Traffic System (VTS) Good	0.0 $0.1\ B_v$
Seabed characteristics	Soft Hard	$0.1\ B_v$ $0.2\ B_v$
Cargo hazard	Medium High	$0.5\ B_v$ $1.0\ B_v$
Separation distance ($\boldsymbol{W_p}$)	$8 - 12\ kn$ $5 - 8\ kn$	$1.6\ B_v$ $1.2\ B_v$
Bank clearance ($\boldsymbol{W_B}$)	Sloping edge Steep, hard embankment	$0.5\ B_v$ $1.0\ B_v$

H. Agerschou et al. (2004) use the same approach as the PIANC (2014), but they suggest slightly different values, as shown in Table 5.11.

Table 5.11: Additional widths W_i (Agerschou, et al., 2004)

Width $\boldsymbol{W_i}$	Vessel Speed	Outer Channel (open water)	Inner Channel (protected water)
Vessel speed $\boldsymbol{V_s}$ (kn, with respect to the water)			
$V_s \geq 12\ kn$	Fast	$0.1\ B_v$	
$8\ kn \leq V_s \leq 12\ kn$	Mod	0.0	
$5\ kn \leq V_s \leq kn$	Slow	0.0	
Prevailing cross wind $\boldsymbol{V_{cw}}$ [kn]			
- mild $V_{cw} < 15\ kn$ (< Beaufort 4)	All	0.0	0.0
- moderate $15\ kn \leq V_{cw} \leq 33\ kn$ (Beaufort 4 – Beaufort 7)	Fast Mod Slow	$0.3\ B_v$ $0.4\ B_v$ $0.5\ B_v$	– $0.4\ B_v$ $0.5\ B_v$
- severe $33\ kn \leq V_{cw} < 48\ kn$ (Beaufort 7 – Beaufort 9)	Fast Mod Slow	$0.6\ B_v$ $0.8\ B_v$ $1.0\ B_v$	– $0.8\ B_v$ $1.0\ B_v$

Prevailing cross-current $\boldsymbol{V_{cc}}$ [kn]			
- Negligible $V_{cc} < 0.2\ kn$	All	0.0	0.0
- Low $0.2\ kn \leq V_{cc} < 0.5\ kn$	Fast Mod Slow	$0.1\ B_v$ $0.2\ B_v$ $0.3\ B_v$	$0.1\ B_v$ $0.2\ B_v$ $0.3\ B_v$
- Moderate $0.5\ kn \leq V_{cc} < 1.5\ kn$	Fast Mod Slow	$0.5\ B_v$ $0.7\ B_v$ $1.0\ B_v$	– $0.5\ B_v$ $0.8\ B_v$
- Strong $1.5\ kn \leq V_{cc} < 2.0\ kn$	Fast Mod Slow	$0.7\ B_v$ $1.0\ B_v$ $1.3\ B_v$	– – –
Prevailing longitudinal current $\boldsymbol{V_{lc}}$ [kn]			
- Low $V_{lc} < 1.5\ kn$	All	0.0	0.0
- Moderate $1.5\ kn \leq V_{lc} < 3\ kn$	Fast Mod Slow	$0.0\ B_v$ $0.1\ B_v$ $0.2\ B_v$	– $0.1\ B_v$ $0.2\ B_v$
- Strong $V_{lc} \geq 3\ kn$	Fast Mod Slow	$0.1\ B_v$ $0.2\ B_v$ $0.4\ B_v$	– $0.2\ B_v$ $0.4\ B_v$
Significant wave heights $\boldsymbol{H_s}$ and length $\boldsymbol{L}$ [m]			
- $H_s \leq 1\ m\ and\ L \leq L_v$	All	0.0	0.0
- $1\ m < H_s \leq 3\ m\ and\ L = L_v$	Fast Mod Slow	$\sim 2.0\ B_v$ $\sim 1.0\ B_v$ $\sim 0.5\ B_v$	– – –
- $H_s \geq 3\ m\ and\ L > L_v$	Fast Mod Slow	$\sim 3.0\ B_v$ $\sim 2.2\ B_v$ $\sim 1.5\ B_v$	– – –
Aids to Navigation (AtoN)			
- Excellent with shore traffic control		0.0	
- Good		$0.1\ B_v$	
- Moderate with infrequent poor visibility		$0.2\ B_v$	
- Moderate with frequent poor visibility		$> 0.5\ B_v$	
Bottom surface			
- If depth $d \geq 1.5\ T_v$		0.0	
- if depth $d < 1.5\ T_v$ then			
- smooth and soft		$0.1\ B_v$	
- smooth or sloping and hard		$0.1\ B_v$	
- rough and hard		$0.2\ B_v$	
Depth of water $\boldsymbol{d}$			
$d \geq 1.5\ T_v$		0.0	0.0
$1.5\ T_v > d \geq 1.25\ T_v$		$0.1\ B_v$	$0.2\ B_v$

$d < 1.25\ T_v$		$0.2\ B_v$	$0.4\ B_v$
High cargo hazards Low Medium High		 0.0 $\sim 0.5\ B_v$ $\sim 1.0\ B_v$	 0.0 $0.4\ B_v$ $0.8\ B_v$

In the bends the approach channel should have an additional width according to following formula (PIANC, 2014):

$$\Delta W \geq \frac{L_v^2}{\mathrm{a}R_b} \tag{5.10}$$

Where ΔW represents the additional width, L_v is the vessel length, R_b is the radius of the bend and a is a factor depending on the type of vessel; a is equal to 8 in case of a normal vessel and equal to 4.5 in case of vessels with a larger displacement (block coefficient$C_B \geq 8$). Larger displacement vessels are for instance tankers or bulk carriers.

5.3.1.3 Channel depth

The last design parameter of the approach channel is the depth. The depth is important for the alignment of the approach channel considering the local bathymetry and for the capital and maintenance dredging which are a large part of the cost as already mentioned. The vertical design of the approach channel is shown in Figure 5.13 and is determined by water level, ship and bottom factors.

The water level must be considered including its variability related to a predetermined reference level. The ship factors include the vessel draughts, manoeuvrability margins and safety factors associated with ships capable of lowering a given point of their hull to a level which is below the keel under static or dynamic conditions in seawater. Bottom related factors are aspects related to the variability of the seabed, such as bathymetric inaccuracies, sedimentation and dredging tolerances.

This approach is used during the detailed design (DD) stage of the harbour. It is very important that this depth is available in the harbour basin and approach channel at all times (PIANC, 2014).

However, in the preliminary phase of the design of a harbour one can use the Concept Design (CD) method suggested by the PIANC (2014). The recommendations in Table 5.12 can be used in an early design phase and is intended to be fast in execution without the use of a large amount of input data.

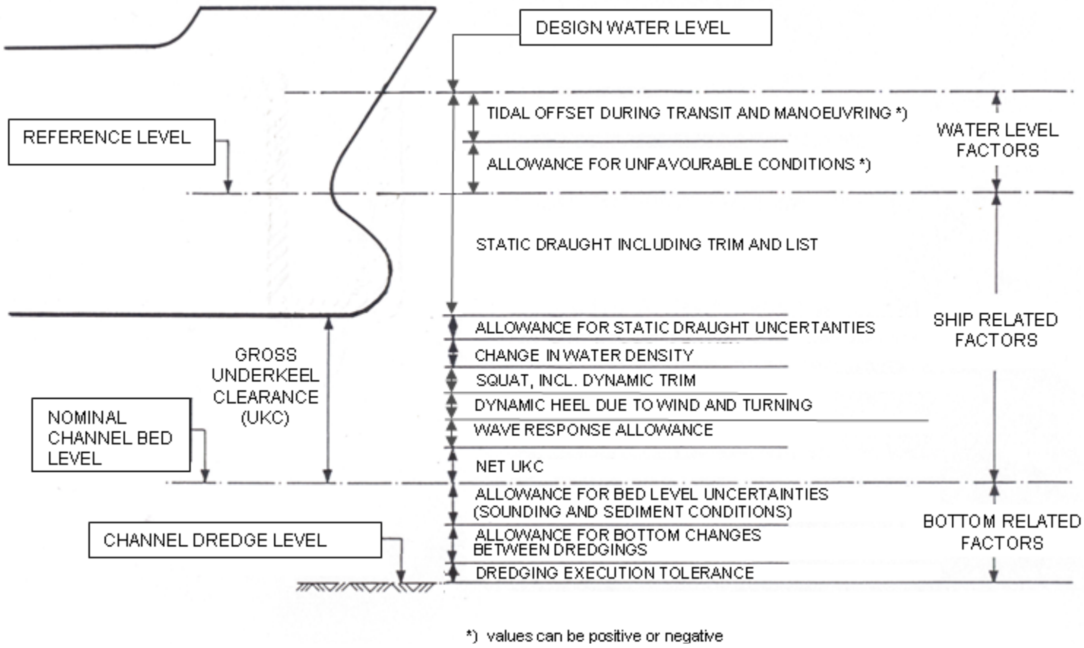


Figure 5.13: Channel depth factors (PIANC, 2014)

Table 5.12: Channel depth components and air draught estimates for Concept Design (CD) (PIANC, 2014)

<table>
<tr><th>Description</th><th>Vessel Speed</th><th>Wave Conditions</th><th>Channel Bottom</th><th>Inner Channel</th><th>Outer Channel</th></tr>
<tr><td colspan="6">Ship Related Factor F_s</td></tr>
<tr><td rowspan="11">Depth d</td><td>$\leq 10\ kn$</td><td rowspan="3">None</td><td></td><td>$1.10\ T_v$</td><td></td></tr>
<tr><td>$10 - 15\ kn$</td><td></td><td>$1.12\ T_v$</td><td></td></tr>
<tr><td>$> 15\ kn$</td><td></td><td>$1.15\ T_v$</td><td></td></tr>
<tr><td rowspan="3">All</td><td>Low swell ($H_s < 1\ m$)</td><td></td><td></td><td>$1.15\ T_v\ to\ 1.2\ T_v$</td></tr>
<tr><td>Moderate swell ($1\ m < H_s < 2\ m$)</td><td></td><td></td><td>$1.2\ T_v\ to\ 1.3\ T_v$</td></tr>
<tr><td>Heavy swell ($H_s > 2\ m$)</td><td></td><td></td><td>$1.3\ T_v\ to\ 1.4\ T_v$</td></tr>
<tr><td colspan="5">Add for Channel Bottom Type</td></tr>
<tr><td rowspan="3">All</td><td rowspan="3">All</td><td>Mud</td><td>None</td><td>None</td></tr>
<tr><td>Sand/clay</td><td>$0.4\ m$</td><td>$0.5\ m$</td></tr>
<tr><td>Rock/coral</td><td>$0.6\ m$</td><td>$1.0\ m$</td></tr>
</table>

5.3.2 Harbour entrance

The harbour entrance is the zone to enter and exit the harbour basin. Through the harbour entrance the vessel will sail into a more sheltered water area, protected by breakwater structures.

Thoresen (2014) suggests locating the harbour entrance on the lee side of the harbour, if possible. If not possible, and it has to be positioned on the windward side of the harbour, sufficient overlap of the breakwaters is needed to allow the vessel can pass through restricted access and to rotate freely with the wind before being hit aside by the waves. Because of this overlapping, the harbour basin will be more sheltered from wave agitation.

The width of the harbour entrance is determined by the density of the vessel traffic, the size of the design vessel and the metocean conditions in front of the harbour entrance. In the transition area, from open sea towards sheltered water, a strong current gradient can occur which will result in a drift angle of the vessels and an additional width requirement. As Memos (2018) and Tsinker (1997) stated, the harbour entrance width and orientation should combine two opposing criteria. For comfortable navigation, the entrance should be as wide as possible. This will also prevent the development of strong currents and ensure circulation and flushing of the harbour water. On the opposite side, the narrower and more sheltered the entrance, the lower the level of wave energy penetrating the harbour entrance, which results in more favourable manoeuvring conditions for reaching the calm of the water surface in the harbour basin.

According to PIANC (2014), the width of the harbour entrance must be equal to or wider than the design vessels overall length L_v, in order to avoid the possibility of ships stranding across the entrance during an accident. Minikin (1963), Thoresen (1988) and Liu & Burcharth (1999) suggest that generally the harbour entrance width should be as wide as 0.7 to 1.0 times the length of the design vessel L_v. While Quinn (1972) relates the width of the entrance to the size of the harbour and the vessel using it. Quinn (1972) stated that for small harbours a width of 90 m should suffice, 120-150 m for medium harbours and 150-240 m for large harbours. Smirnov et al. (1979) recommend following formula:

$$B_E = B_v + \left(\frac{V_\alpha^n}{V_{min}}\right) L_v + V_{min}\ t_y\ sin\ \beta_y + \Delta B_E \tag{5.11}$$

where B_E represents the harbour entrance width, B_v is the beam of the design vessel, $V_\alpha^n = V_c^n + V_w^n$ the normal component of speed drifting out of course under the action of metocean conditions, V_{min} is the minimum speed of the vessel entering the harbour, L_v is the overall length of the design vessel, t_y is the time of ship yawing(s) (in general equal to 60 s), β_y is the yawing angle between 3° and 10° depending on the metocean conditions and ΔB_E the width allowance to avoid the vessel from colliding. The parameter ΔB_E depends on the protection of the harbour entrance and is equal to B_v in case of a well protected entrance and $2B_v$ in the case of a less protected entrance.

In case of a harbour for small vessels, a marina, the recommendations of ASCE (2012) can be used. The basic width is a function of both the ship's manoeuvrability and the sea state, so for pure steerage considerations. For ideal conditions a width of 1.3 B_v should be sufficient, for moderate conditions 1.5 B_v suffice and for poor conditions 1.8 B_v is needed. Additional to this width, additional width adjustments need to be taken into account according to the values in Table 5.13.

Table 5.13: Additional widths W_i (ASCE, 2012)

Parameter	Condition	Limit	Additional Adjustment
Vessel speed V_s	Mild	$< 8\ kn$	0.0
	Moderate	$8 - 12\ kn$	0.0
	Poor	$> 12\ kn$	$0.1\ B_v$
Crosswinds V_{cw}	Mild	$< 15\ kn$	0.0
	Moderate	$15 - 33\ kn$	$0.4\ B_v$
	Poor	$> 33\ kn$	$0.8\ B_v$
Crosscurrents V_{cc}	Mild	$0.2 - 0.5\ kn$	$0.2\ B_v$
	Moderate	$0.5 - 1.5\ kn$	$0.7\ B_v$
	Poor	$> 1.5\ kn$	$1.0\ B_v$
Longitudinal currents V_{lc}	Mild	$< 1.5\ kn$	0
	Moderate	$1.5 - 3.0\ kn$	$0.1\ B_v$
	Poor	$> 3.0\ kn$	$0.2\ B_v$
Wave height H_s	Mild	$< 1\ m$	0
	Moderate	$1 - 3\ m$	$1.0\ B_v$
	Poor	$> 3\ m$	$2.2\ B_v$

In case of passing vessels, a minimum passing distance of 1.5 B_v between the manoeuvring lanes is recommended by ASCE (2012). The recommended minimal width is thus about 5 B_v to 6 B_v for two-way traffic, which is roughly 25 to 30 m for small crafts. In case of larger number of vessels in the marina, this recommended width will increase. In case the marina can berths more than 200 vessels, a width of 1 to 3 m should be added. For one-way traffic, a harbour entrance width of 3 B_v should suffice (ASCE, 2012).

The harbour entrance should ideally be oriented away from the dominating storm events and more towards the everyday wind and waves. Generally, it should be as far away from the coastline as possible. It is also important that the harbour entrance is designed in such a way that the vessel does not have to make any difficult manoeuvres upon entering the harbour. A vessel can start his manoeuvres after passing through the harbour entrance into more sheltered water area (Ligteringen, 2017).

Tsinker (1997) recommends to design the harbour entrance and approach channel so the angle between the channel axis and shoreline α_2 in Figure 5.14 does not exceed 30°. It is also preferable to reduce the angle α_1, formed by the breakwater heads and the approach channel axis, as much as possible to ensure safe navigation and avoid vessels to collide with the breakwaters (ASCE, 2012; Tsinker, 1997).

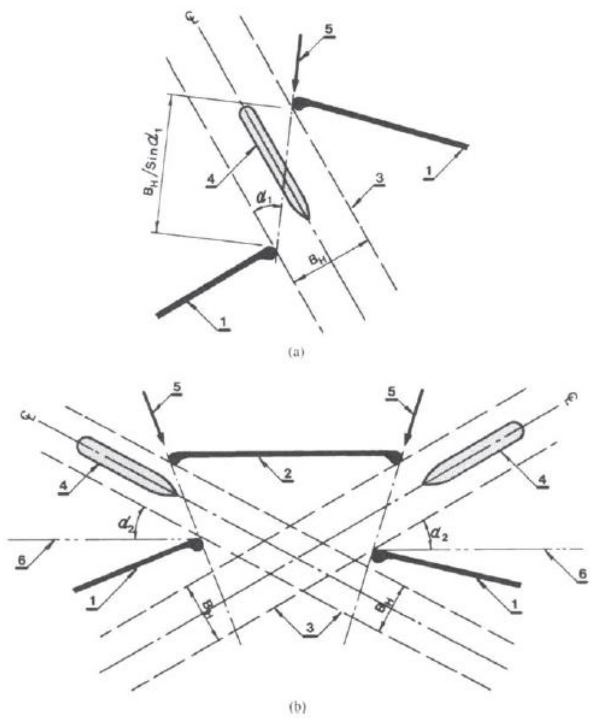


Figure 5.14: Harbour entrance orientation: a) entrance of two shore-connected breakwaters; b) entrance of a detached breakwater configuration (Tsinker, 1997)

The width of the harbour entrance is also important for the diffraction calculations, which will be discussed further on.

5.3.3 Stopping area

To provide safe manoeuvring of the vessel upon entering and navigating in the harbour basin, a stopping area should be designed. This stopping area should be long enough so the vessel can stop safely at a reasonable speed. A vessel needs to reduce its speed to continue with berthing and anchoring manoeuvres.

According to Memos (2018) vessels will enter a harbour at a speed of 8 to 11 kn and will reduce speed over a length of 2 to 3 times to vessel's overall length L_v, however larger vessels will need more space to execute this manoeuvre. It is also important to consider the manoeuvring equipment as well as the type of propeller during the design of the stopping area. If the propeller is of variable pitch, the stopping length can be reduced to 1.5 times to vessel length L_v. Liu & Burcharth (1999) suggest to use a much longer stopping length of around 7 to 8 times the design vessel length L_v.

Tsinker (1997) suggests to use following formula to determine a preliminary vessel stopping length L_{st} (Dzhunkovski, Kasperson, Smirnov, & Sidorova, 1964):

$$L_{st} = \frac{0.27 V_0^3 \Delta}{P} \tag{5.12}$$

where V_0 is the speed of the vessel, Δ the displacement of the vessel and P the installed power. It should be mentioned that the installed power P used in previous formula is in horsepower hp, while this is mostly given in kilowatt kw.

The stopping length of a vessel usually depends on the vessel displacement, the travelling speed, the installed power of the engines and the metocean conditions (Velsink, 1994). The previously mentioned formula is just a rough recommendation, no general guidelines exist for determining the exact stopping length.

As a general guideline, Tsinker (1997) recommends to take the stopping length equal to 3.5 to 8 times the overall length of the design vessel L_v. Table 5.14, presented by Tsinker (1997), shows the importance of speed during the stopping manoeuvre. Performing a stopping manoeuvre starting from full speed will give a much larger stopping length than doing this starting from 4 kn. This table shows also the influence of the vessel displacement Δ, a larger displacement will give a larger stopping length.

Table 5.14: Stopping length; with full astern power from full speed and from 4 kn (Tsinker, 1997; Dickson, 1969)

Deadweight Tonnage	Stopping distance [m]: engines going full speed astern		
	Vessel's length L_v [m]	From full speed	From 4 kn
18,000	70	1,600	240
50,000	230	2,400	250
110,000	265	2,600	350
210,000	330	4,000	520

According to Velsink (1994), the stopping length L_{st} is influenced by: the size of the vessel and its relation to the installed power, the speed at which the vessel enters the harbour and the type of stopping procedure. The ratio of the installed power P to the displacement of the vessel Δ is inversely proportional to the size of the vessel. Thus, the installed power P used for reducing speed decreases with increasing ship size, as shown in Figure 5.15. The upper and lower bound, respectively $L_{st,u}$ and $L_{st,l}$ derived from this figure can be interpolated and the following formulas are computed:

$$L_{st,u} = L_v \left(4.2853 \ln\left(\frac{\Delta}{P}\right) + 6.0034\right) \tag{5.13}$$

$$L_{st,l} = L_v \left(4.6664 \ln\left(\frac{\Delta}{P}\right) + 3.5177\right) \tag{5.14}$$

where L_v is the design vessel's overall length, Δ the vessel displacement and P is the installed power in hp.

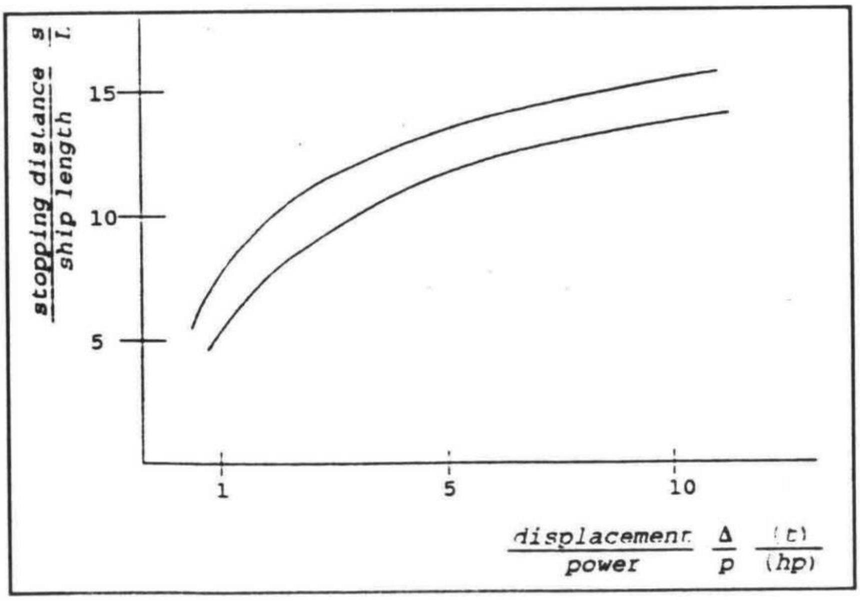


Figure 5.15: Stopping length of vessels (Velsink, 1994; IAPH, 1981)

Velsink (1994) also mentions that the stopping length L_{st} will increase if the speed upon entering the harbour increases. In general, two different stopping procedures exist: the crash stop and the fully controlled stop. In case of the crash stop the engines are set full astern when entering the harbour, which results in a minimal stopping length. However, due to the turbulence generated around the rudder of the vessel, the vessel will lose course control. For small to medium vessels this should be no problem but for bigger vessels it is a problem as the vessel becomes uncontrollable. During the fully controlled procedure, the vessel continues navigating at a minimum speed which is needed to maintain sufficient course control. Tugs are attached to the vessel and the vessel will stop under its own installed power but tugs assist the vessel to keep course (Velsink, 1994).

PIANC (2014) indicates that there should be no problems regarding the design of the stopping area in case of small to medium size vessels. The length of these manoeuvring areas is generally limited and can be generally integrated in the design of the harbour basin. Their manoeuvrability is usually good during entering the harbour and they are able to navigate and stop under their own power. This changes for larger vessels, due to the larger mass the vessels have much longer stopping lengths and lack course control during this stopping manoeuvre. Usually large vessels get assisted by tugs to keep course control. The design approach of PIANC (2014) also accounts for the time required for tugs to attach to the vessel and to manoeuvre the vessel in position, as shown in Figure 5.16. The PIANC (2014) suggests a stopping length L_{st} of 1.5 to 2 times the overall length of the design vessel L_v at an initial speed V_0 of 4 kn. The distance over which tugs are attached to the vessel should be added to this.

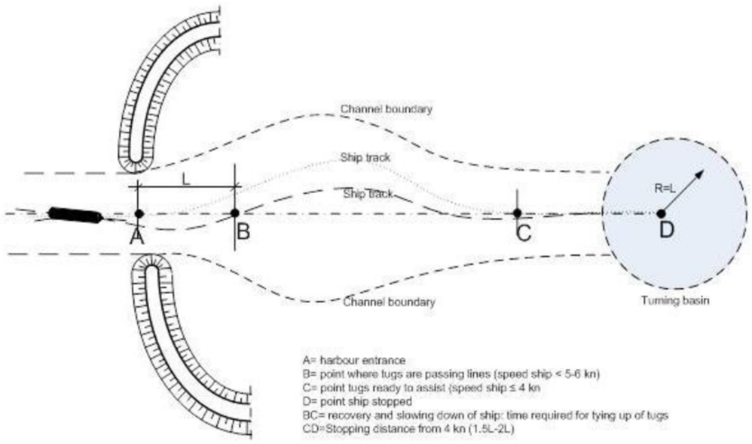


Figure 5.16: Stopping procedure and channel dimensions (PIANC, 2014)

According to Puertos del Estado (2007) three different paths can be followed upon stopping a vessel:

a) Stopping in a straight line
b) Stopping in a circle
c) Stopping in a mixed path

These paths are respectively shown in Figure 5.17, Figure 5.18 and Figure 5.19. In the case of stopping in a straight line, a straight alignment is specified equal to or larger than the stopping length L_{st} and increased by a safety factor of 2, together with a width equal to the length of the design vessel L_v (Figure 5.17). If the vessel is stopped by assistance of tugs, a larger width should be provided to allow the tugboats to manoeuvre safely. An exception can be made for effective tugs, no additional widths are then required.

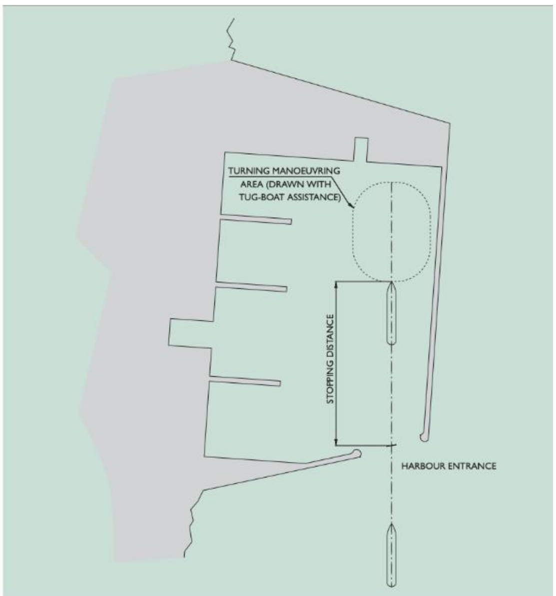


Figure 5.17: Stopping in a straight line (Puertos del Estado, 2007)

In case of stopping in a circle, one should use the worst complete circle to design the stopping area (Figure 5.18). Due to this advice, this type of stopping area will require more space. An over-dimensioned stopping area, in case of stopping in a circle, is necessary to ensure that the end of the stopping manoeuvre is directed to the inside of the circle and avoid the need for extra space. Due to this, stopping in a circle is mostly unrecommendable because of the higher costs.

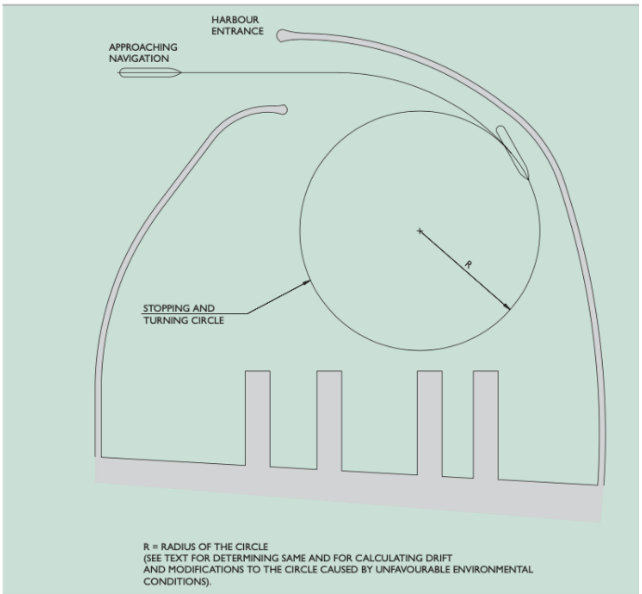


Figure 5.18: Stopping in a circle (Puertos del Estado, 2007)

A last option is stopping in a mixed path, by combining curved and straight stretches (Figure 5.19). This case is actually a combination of the two previous cases. It’s important to end in a straight stretch to ease manoeuvring. In case of stopping in a straight line and stopping in mixed paths, a turning area is required after stopping, which will be discussed in the next paragraph.

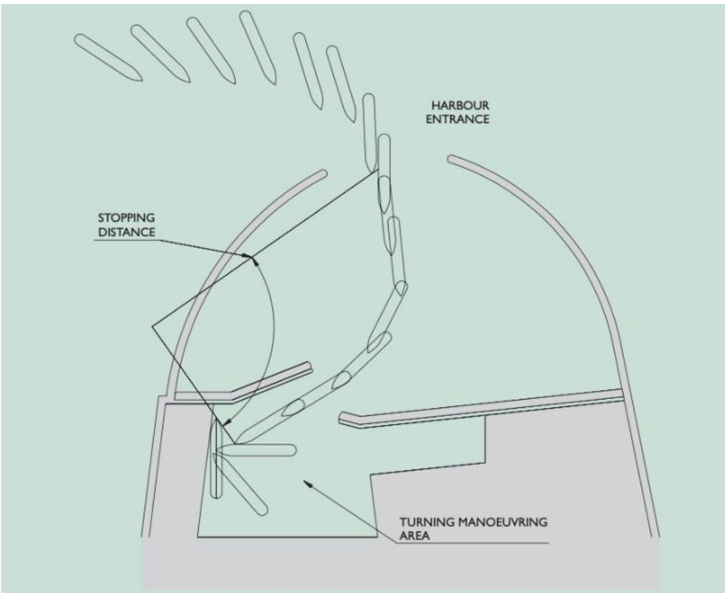


Figure 5.19: Stopping in a mixed path (Puertos del Estado, 2007)

5.3.4 Turning basin

After entering the harbour basin, the vessel should execute some manoeuvres in order to position itself correctly for the mooring position, which is determined in advance. The area which is designed for this purpose is called the turning basin. The dimension of this area is mainly determined by the size of the design vessel. Further, one should consider the manoeuvring conditions, which are determined by the metocean conditions and the manoeuvring equipment of the vessels including the type of propellers, rudder and tugs. In case of a large harbour, multiple turning basins can be designed and these can be located in the outer basin or in the main harbour (PIANC, 2014; Ligteringen, 2017; Memos, 2018; Puertos del Estado, 2007).

The PIANC (2014) and Ligteringen (2017) recommend to use a turning basin diameter of minimum two times the overall length of the design vessel L_v during the Concept Design (CD) of the harbour. However, the designer can opt for a larger turning diameter when the environmental circumstances are unfavourable. The turning diameter can be enlarged up to 3 times the vessel's length L_v in case of small harbours or in case no tugboats are used. In the case of river harbours with large currents or strong winds, the turning diameter should be larger to allow vessels to drift when turning. PIANC (2014) suggests to delay the decision of reducing the turning diameter from a minimum of 2 L_v to the Detailed Design phase.

Lui & Burcharth (1999) suggest the same minimum value of 2 times the design vessel length L_v, in case with tugboats. However, without tugboats they suggest increasing the turning basin diameter up to 4 times L_v.

Memos (2018) recommends other methods to use during the design of the turning basin. In case of adverse manoeuvring conditions, no use of tugboats and ships equipped with only one rudder, a turning basin diameter of 4 times the design vessel's length L_v will be required. Manoeuvring in favourable circumstances with the possibility to use modern navigation system may reduce the diameter to 3 times L_v. The following is meant by favourable conditions; wind speed less than 10 m/s, current speed less than 0.5 m/s and wave height less than 1.5 or 2 m according to type of tugboats. Memos (2018) also mentions the possibility to use an ellipse as turning area with 3 L_v and 2 L_v as axes, this may be sufficient to meet the navigation requirements. In case the vessel can make use of the aid of tugs to keep course control or in case a second rudder or a bow thruster is installed, the turning basin diameter may be reduced to 2 times L_v. The diameter may be even further reduced if the vessel can use bow and stern anchors or wrapping dolphins to an absolute minimum of 1.2 L_v. Memos (2018) also suggests to add an additional safety zone between the turning area and rigid constructions or/and ship berths of 1.5 L_v which should be larger than 30 m.

Puertos del Estado (2007) uses a different approach which leads to other recommendations for the cases with or without tugs and with or without dropped anchor. The radius of the turning basin is denoted as R_{sr} in case no tugboats are used for assistance and its value depends on if the anchor is dropped or not.

a) Without an anchor dropped following formula can be used:

$$R_{sr} = R\,tan\,30° + K\,L_v + 0.35\,L_v \tag{5.15}$$

where R is the radius of the vessel's path going ahead or astern, K the distance from the pivot centre to the ship's bow or stern as a fraction of the length of the vessel, L_v the design vessel's length and a safety margin of 0.35 times the design vessel's length L_v. These parameters are also indicated in Figure 5.20.

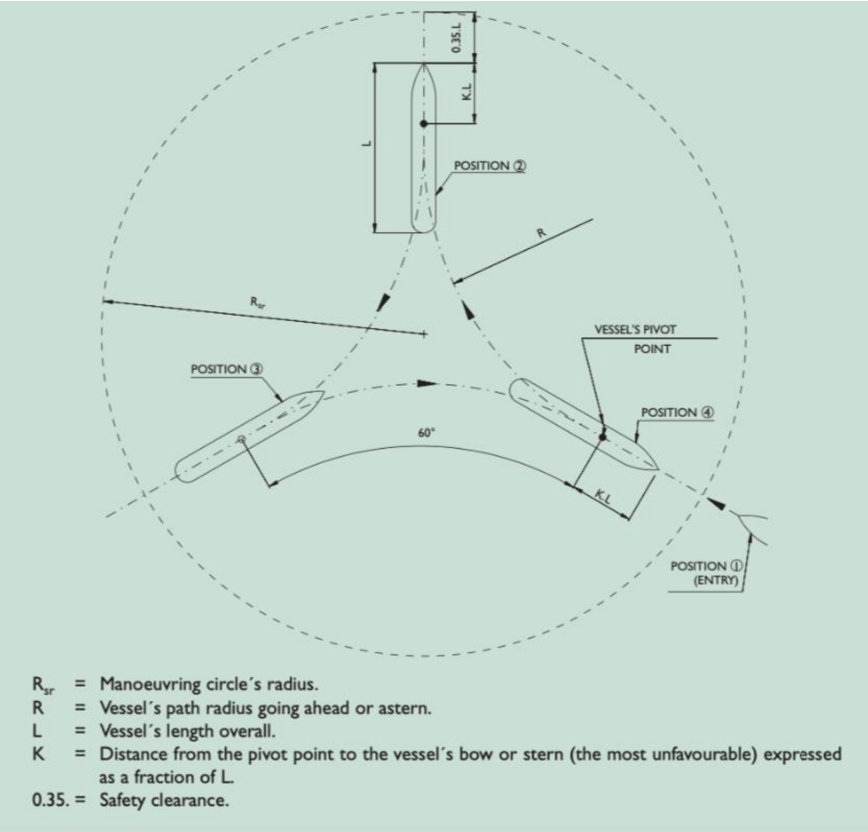


Figure 5.20: Turning area without tugboats or dropped anchors (Puertos del Estado, 2007)

The minimum radius of the path can be chosen with the aid of Table 5.15 in case of lack detailed values.

Table 5.15: Minimum radius of vessel's path (Puertos del Estado, 2007)

Water depth	Minimum radius of vessel's path
≥ 5.0 T_v	3.0 L_v
1.5 T_v	3.5 L_v
≤1.2 T_v	5.0 L_v

In case the ratio water depth over vessel's draught d/T_v is smaller than 1.20, K will be equal to a value of 0.5, this is for vessel's with a large displacement and with full underwater body shapes thus in the case of container ships, oil tanker, bulk carriers, etc.. If this ratio is larger than 1.20, K will take a value of $2/3$. K will be equal to 1.00 for recreational boats, which have a much finer underwater body shape.

b) With an anchor dropped. In this case the dropped anchor serves as the centre of the turning manoeuvre and the radius of the manoeuvring circle is equal to the design vessel's length L_v. To provide enough space to manoeuvre, a turning basin radius of 1.5 L_v increased with an additional safety margin of 0.2 L_v is used in the design. (Figure 5.21)

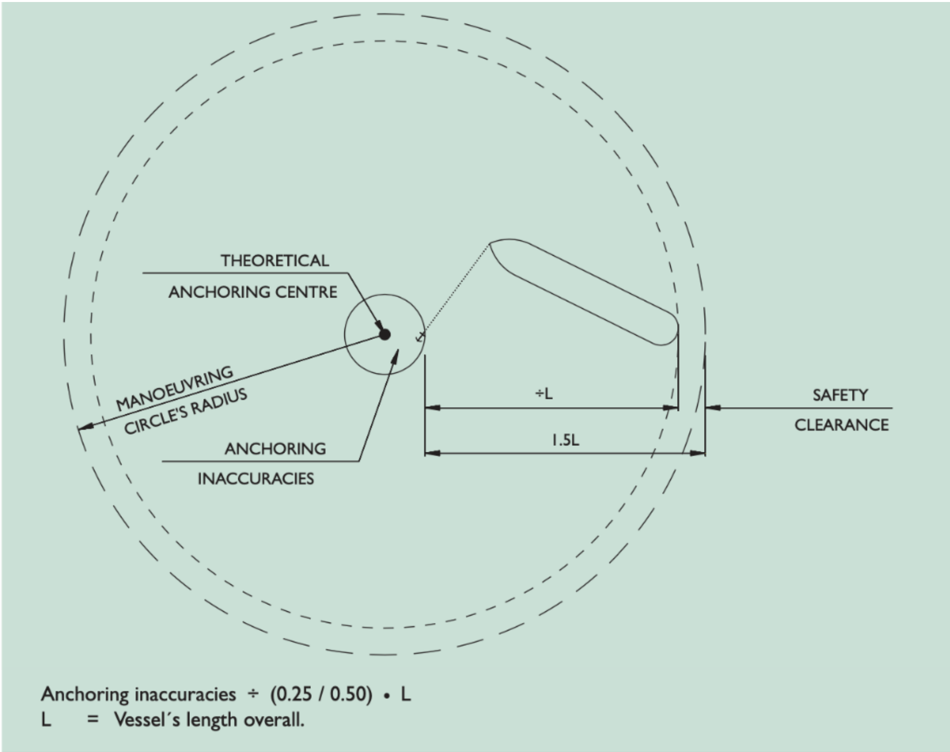


Figure 5.21: Turning area without tugboats but with dropped anchor (Puertos del Estado, 2007)

In case tugboats assist the design vessel to perform the turning manoeuvres, Puertos del Estado (2007) take a completely different approach than recommended by other references. Instead of one manoeuvring circle they suggest to design a central rectangle with dimensions 2 B_G and 2 L_G, as shown in Figure 5.22. To this central area, 4 quart circles are added with radius R_{cr}. The suggested dimensions have to satisfy following criteria:

$$B_G \geq 0.10\ L_v \tag{5.16}$$

$$L_G \geq 0.35\ L_v \tag{5.17}$$

$$R_{cr} \geq 0.80\ L_v \tag{5.18}$$

These requirements are determined in the case of favourable conditions, in the case of unfavourable conditions additional space should be designed.

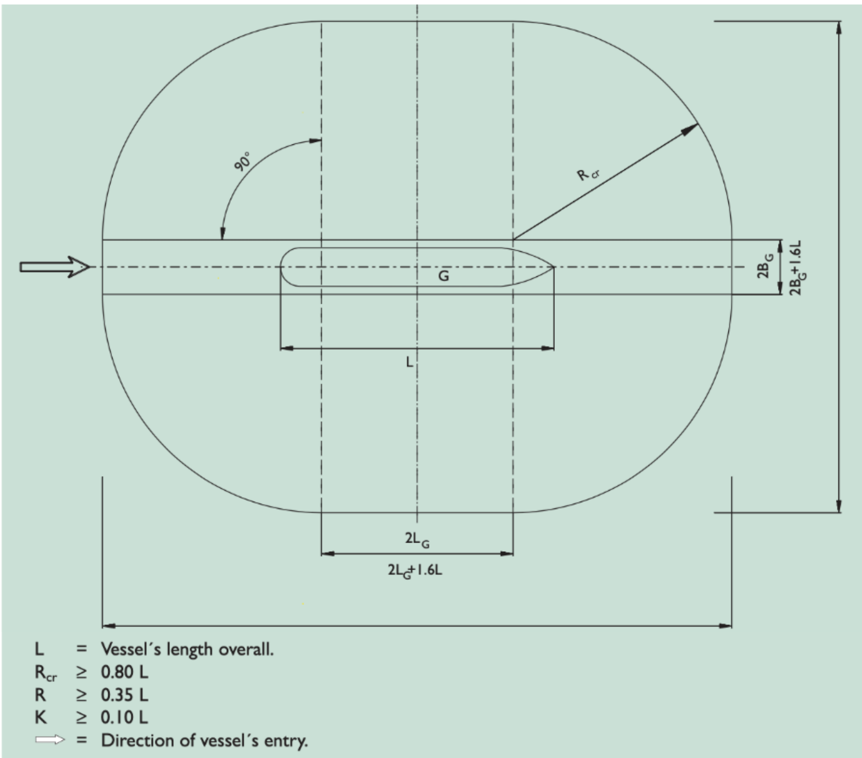


Figure 5.22: Turning area with tugboats (Puertos del Estado, 2007)

The dimensions of the turning basin can also be determined by the type of the design vessel, as Velsink (1994) recommends. In case of container ships, which have in general poor manoeuvring capability, he suggests using a turning basin diameter between 6 and 8 times the length of the design vessel L_v. For large oil and dry bulk carries, which navigate at a speed between 15 and 17 kn, he recommends using 3 to 4 times the design vessel length L_v and for LNG carriers 2 to 2.5 times L_v.

It is important to mention that all of the previous manoeuvring areas should be optimised by using ship manoeuvring simulations during a more detailed design phase. During this simulations, the navigation of the design vessel can be assessed for difficult conditions such as high wave heights, strong cross winds, strong currents or a combination of these (PIANC, 2014).

5.3.5 Berthing area

After manoeuvring into and through the harbour basin the vessel should be berthed at the ship berths. The length of the necessary berths, the quay length L_q, is determined by the number of vessel berths that is required and the length of the design vessel L_v. Between adjacent vessels an additional space of 0.1 L_v has to be taken into account, as shown in Figure 5.23. PIANC (2019) and Ligteringen (2017) suggest following formula to calculate the quay length L_q.

$$L_q = \begin{cases} L_v + 2\,B_{gap} & for\; n = 1 \\ 1.1\,n\,(L_v + B_{gap}) + B_{gap} & for\; n > 1 \end{cases} \tag{5.19}$$

where L_v is the design vessel length, n the number of ship berths and B_{gap} the gap between ships for mooring ropes. PIANC (2019) suggests to use a berthing gap B_{gap} around 15 m to 30 m. Thoresen (2014) and Liu & Burcharth (1999) recommend a similar approach.

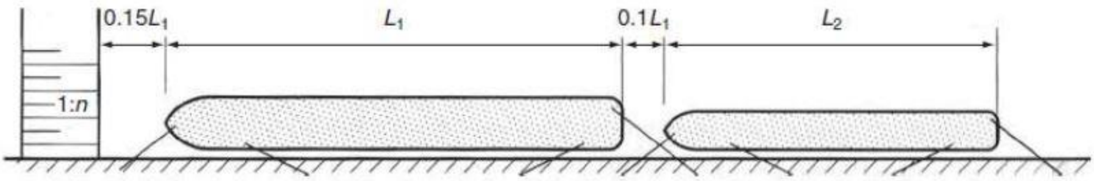


Figure 5.23: Quay length (PIANC, 2019; Thoresen C. A., 2014)

Behind the quays operational land is required to process to cargo of the vessels in the harbour. UNCTAD (1985) mentions the use of 50 m² operational land per meter quay in the past, which is rather small. Nowadays, 100 to 200 m² operational land is designed per meter quay. As a general guideline a minimum of two times the design vessel's length L_v can be taken into account to determine the width of the quays. Figure 5.24 shows some typical berths layouts. One can also account for the metocean conditions by designing slanted berths as in the last figure.

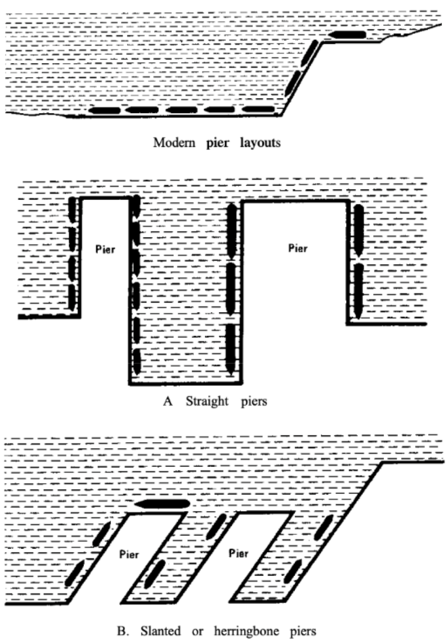


Figure 5.24: Berthing area layout (UNCTAD, 1985)

5.4 Morphological aspects

The construction of breakwaters can drastically influence the sediment transport in and around the harbour basin. Accumulation and erosion of sediment adjacent to the harbour can and mostly will have an influence on the community and ecosystems (Verhagen & van den Bos, 2017). It is possible to distinguish three different processes affecting the harbour layout (Ligteringen, 2017):

1. Littoral transport: breakwaters can disturb a balanced situation of the sediment transport along the coastline inducing accretion and erosion.
2. Siltation of the approach channel
3. Sediment transport into the harbour area

5.4.1 Littoral transport

Oblique breaking waves in the surf zone will produce, due to the induced longshore currents and turbulence, a transport of non-cohesive beach materials along and parallel to the coastline, termed littoral transport (Mangor, Drønen, Kærgaard, & Kristensen, 2017; US Army Corps of Engineers, 2002; Sorensen, 2006). Figure 5.25 is a plan view scheme explaining the littoral transport process. Sediment transport can also originate from direct wave action, wind and tide induced currents or direct wind. However, these processes will have a reduced influence on the longshore transport.

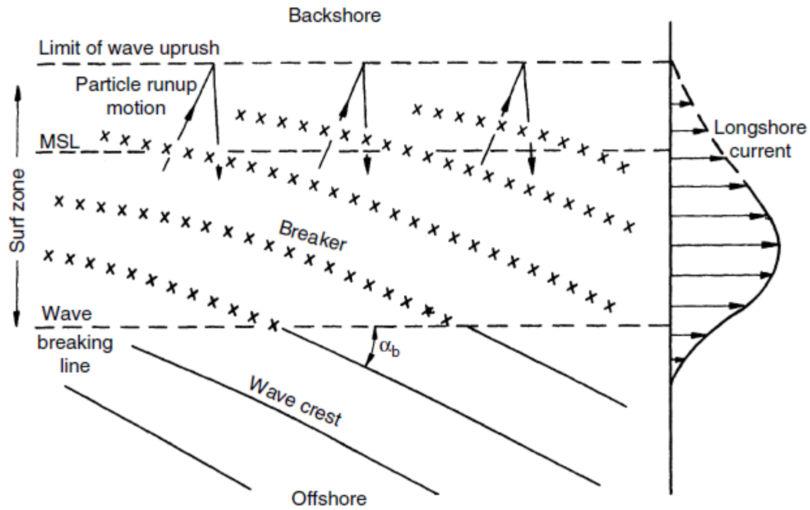


Figure 5.25: Wave-generated longshore current (Sorensen, 2006)

After some time, an equilibrium can exist along the coastline which is influenced by the transport capacity of the different processes and the amount of sediment material available to the processes. This equilibrium is most of the time a “dynamic” equilibrium (Sorensen, 2006). This means that the coastline is fairly constant over a large period of time, while it changes instantaneous due to short term oscillations.

This balanced situation can be disturbed by constructing breakwaters as a part of a harbour project. Due to this disturbance, large accretion or erosion can take place at neighbouring sites and this process needs a careful assessment to avoid severe problems and high costs.

After construction upstream accumulation and downstream erosion will occur in the most elementary case as show in Figure 5.26. The upstream accumulation of sediment material can introduce a need for maintenance dredging, while the downstream erosion could ask for coastline protection structures to avoid the loss of beaches (Mangor, Drønen, Kærgaard, & Kristensen, 2017; Sorensen, 2006; US Army Corps of Engineers, 2002).

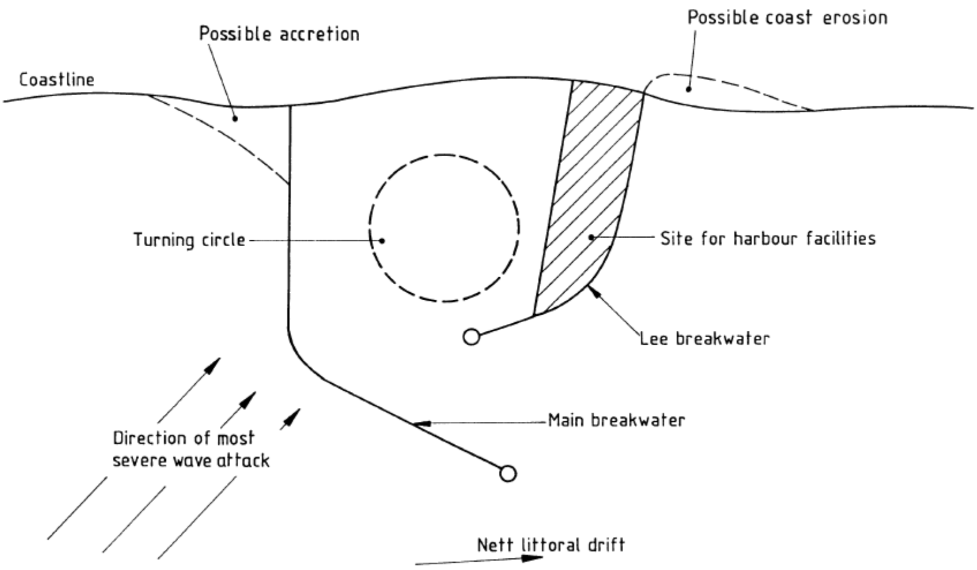


Figure 5.26: Harbour with net littoral drift (BS 6349, 2000)

5.4.2 Siltation of the approach channel

The increased channel depth and reduced current velocities can cause the deposition of finer sediments, which causes the approach channel to silt up. This process is an important design aspect in case of harbours situated in areas with finer sediment material, estuaries or in case of a natural river that needs to be dredged to give larger vessels access to the harbour. However, the siltation of the approach channel is a very complex process and should be assessed by the means of computer software (Ligteringen, 2017).

The possible effects of siltation, such as maintenance dredging of the approach channel, can be taken into account in the preliminary phase of the harbour layout design. Unlike the littoral sediment transport, little can be done regarding the design of the harbour layout to avoid this process, especially with an existing harbour. In the case of a new harbour, this may lead to reconsidering the port location (Ligteringen, 2017).

5.4.3 Sediment transport into the harbour

As was the case with the siltation of the approach channel, sediment transport into the harbour is often the case for finer sediment material entering through the harbour entrance and settling in the harbour basin. According to Ligteringen (2017) three different processes can be distinguished leading to this type of sediment transport; (i) tidal filling of the harbour, (ii) density currents due to salt concentration differences or temperature differences and (iii) eddy formation behind the breakwater, as shown in Figure 5.27. Mostly, these three processes occur at the same moment and again lead to a need for maintenance dredging.

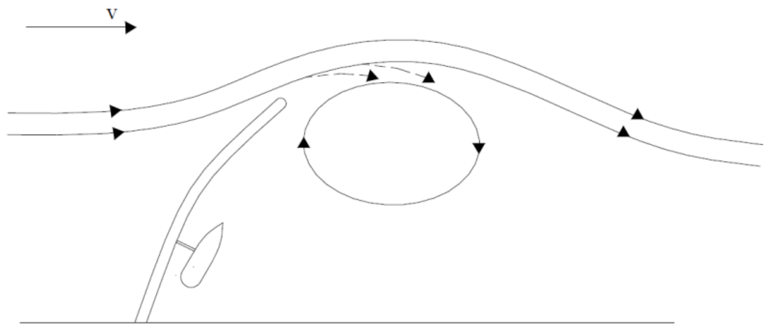


Figure 5.27: Sediment exchange between main current and eddy current behind breakwater (Ligteringen, 2017)

5.4.4 Measures to limit the effect of sedimentations

To limit these effects of sedimentation, in and around the harbour, several authors provide different suggestions to improve the design of the harbour layout. During the design of the harbour layout the general ideas are to reduce the erosion, to try to build up beaches along the coastline using the longshore transportation of sediment and to extend the sediment transport into deeper water, where they are less effective. In general, this means minimalizing the coastal impact of the harbour (BS 6349, 2000; Memos, 2018; Mangor, Drønen, Kærgaard, & Kristensen, 2017). Implementing these general ideas in a preliminary phase of the harbour design is often not that simple. Mostly, the morphological aspects are assessed in a more detailed phase of the design, with the help of physical and numerical modelling.

According to Ligteringen (2017) the length of the breakwaters can be defined by the width of the surf zone. The width of the surf zone, or breaking zone, depends on the deep-water wave height and the storm frequency. A small storm frequency will need long breakwaters, but this will reduce the sedimentation, a storm with a higher frequency will need short breakwaters and a lot of maintenance dredging. During the preliminary design phase of the harbour layout the annual wave conditions can be used to this extend. However, in a more detailed design phase the optimal capital construction cost, maintenance and capital dredging cost has to be computed.

A second main design aspect is the storage capacity of the breakwaters. During the design of the breakwater layout, the designer can opt for a reservoir which has to be filled before the sands accumulates along the breakwater length and the littoral transport continues along the coastline (Figure 5.28). Several aspects have to be looked at in the elaboration of this type solution. It is a trade-off between the construction cost of the (longer) breakwaters, the maintenance dredging cost and the depth of the approach channel (Ligteringen, 2017; Agerschou, et al., 2004).

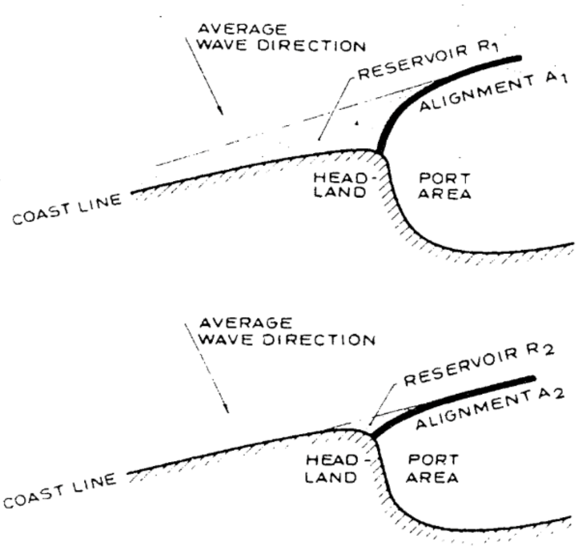


Figure 5.28: Storage reservoir of breakwaters (Agerschou, et al., 2004)

In case there is significant sediment transport on both sides of the harbour, it needs two breakwaters stretching to adequate depths. This depth is needed to ensure no sediment material settles in the approach channel or in the harbour itself. In case the longshore sediment transport is only significant on one side of the harbour basin, one main breakwater can be enough. However, the main breakwater, for one sided or two-sided transport, is positioned according to the direction most sediment transport comes from and it must be sufficiently large to prevent sediment from settling in the approach channel. When a second breakwater is needed, it is constructed in order no sediment material can enter the harbour basin, as shown in Figure 5.29 (Ligteringen, 2017).

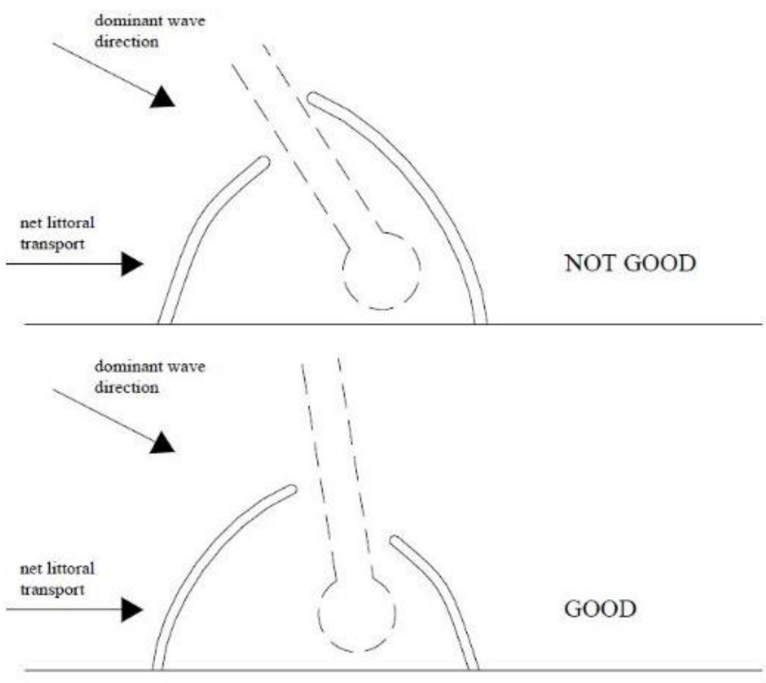


Figure 5.29: Layout of breakwaters in relation with the longshore sediment transport (Ligteringen, 2017)

In some harbour designs, artificial sediment bypassing systems are considered to remove the sand accumulated at the upstream part of the harbour and pump it to outlets past the harbour, at the downstream side. By doing this, the natural net littoral transport is simulated (Mangor, Drønen, Kærgaard, & Kristensen, 2017; Memos, 2018; Verhagen & van den Bos, 2017). Several configurations are possible for this solution, as shown in Figure 5.30.

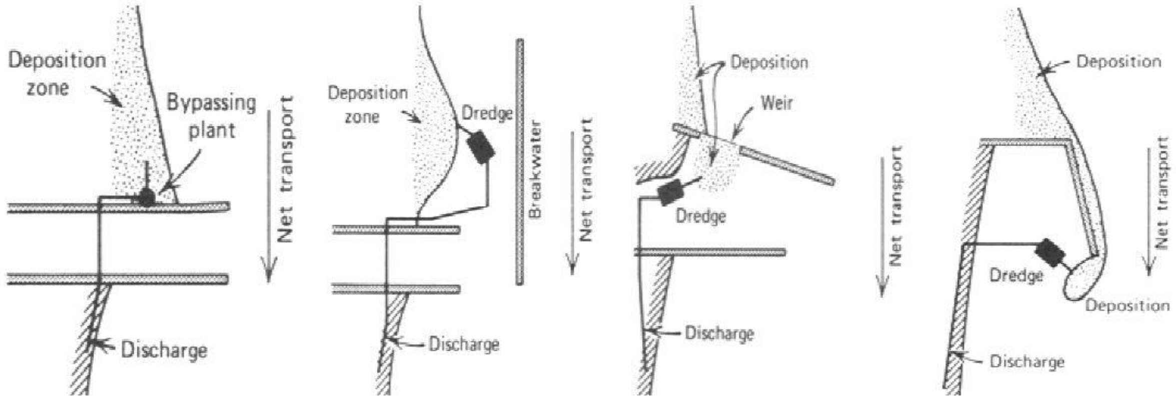


Figure 5.30: Sediment bypass systems (Sorensen, 2006)

Two other solutions are to fill the eroding beach with sediment material dredged during maintenance dredging or to provide a buffer of sediment material during the construction of the harbour as a buffer against future sediment transport (Verhagen & van den Bos, 2017). According to Verhagen & van den Bos (2017), a detached offshore breakwater parallel to the coastline could be a possible solution to control the longshore and cross-shore sediment transport adjacent to the harbour. However, this will induce a tombolo effect and have negative consequences for the navigation of the vessels.

As already mentioned, one of the popular design solutions is to extend the sediment transport to larger water depths, and thus to extend the breakwater(s). In deeper water the driving force of longshore littoral transport is smaller. However, Sawaragi (1995) mentions that this type of solution is not effective in case the suspended load transport is predominant. In the case longshore transport is the main process driving the sediment transport, the bed load coming into the harbour entrance even increases sometimes because of the changing adjacent bathymetry.

In case of sandy shores, and thus larger sediment size, extending the breakwater(s) or constructing groynes can be effective, as shown in Figure 5.31. In such cases, the bed load transport is predominant. If the suspended load is predominant, which can be the case during storms or fine sands, constructing a detached breakwater can be as efficient as prolonging the breakwater(s) (Figure 5.31) (Sawaragi, 1995).

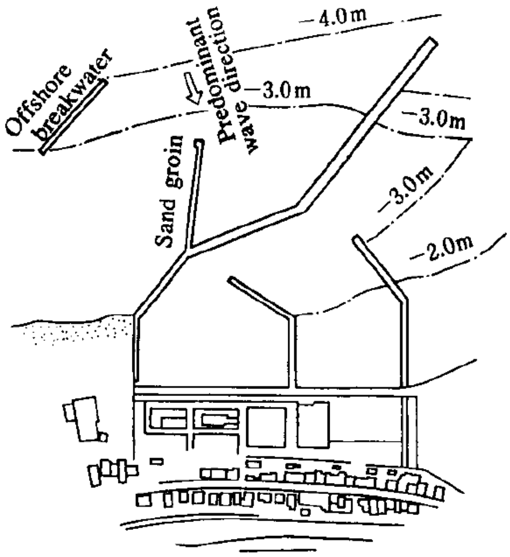


Figure 5.31: Design guidelines for sediment transport (Sawaragi, 1995)

To minimise the effect of the harbour layout on the longshore sediment transport, Mangor et al. (2017) suggest to design a layout that guides the currents beyond the harbour entrance in a streamlined way. This will reduce the creation of eddy currents, which reduces sedimentations and increases natural sediment transport past the harbour structures. According to Mangor et al. (2017), the breakwater ends should be almost parallel to the shore with an optimal angle of 140° between them, as shown in Figure 5.32.

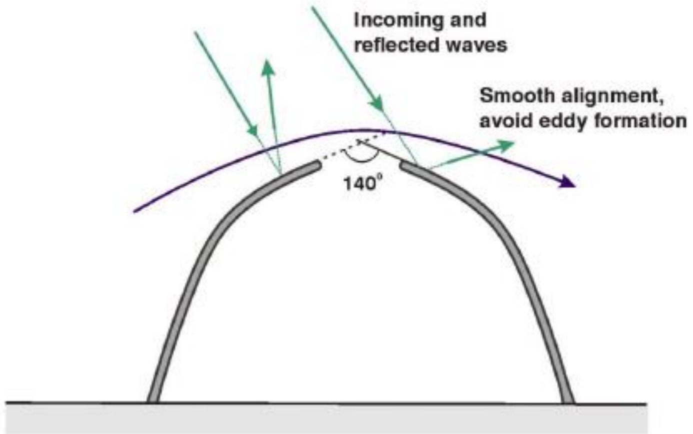


Figure 5.32: Optimal harbour entrance layout (Mangor, Drønen, Kærgaard, & Kristensen, 2017)

Additionally, a narrow entrance can be provided which also reduces the entering of materials (Mangor, Drønen, Kærgaard, & Kristensen, 2017). However, this can have a serious impact on the vessel manoeuvring. The connection of the breakwaters with the shoreline should also be smooth, such that the sediment transport can evolve streamlined past the breakwaters.

Several problems can rise in the case of one main parallel attached breakwater, as was shown Figure 5.4. In this case eddy currents can form inside the harbour entrance, causing siltation of the approach channel and harbour entrance. While a converging layout, which is more streamlined, could lead the sediments past the harbour entrance and give the possibility to control the littoral transport. The left side of Figure 5.33 shows different breakwater configurations intended for coastal protection, not as protection of a harbour basin. However, the concept remains the same in case of a harbour layout, smoothening the transition between the coastline and the breakwater structure inducing a streamlined sediment transport and a minimal coastal impact. The right side of Figure 5.33 shows the current patterns for the different configurations.

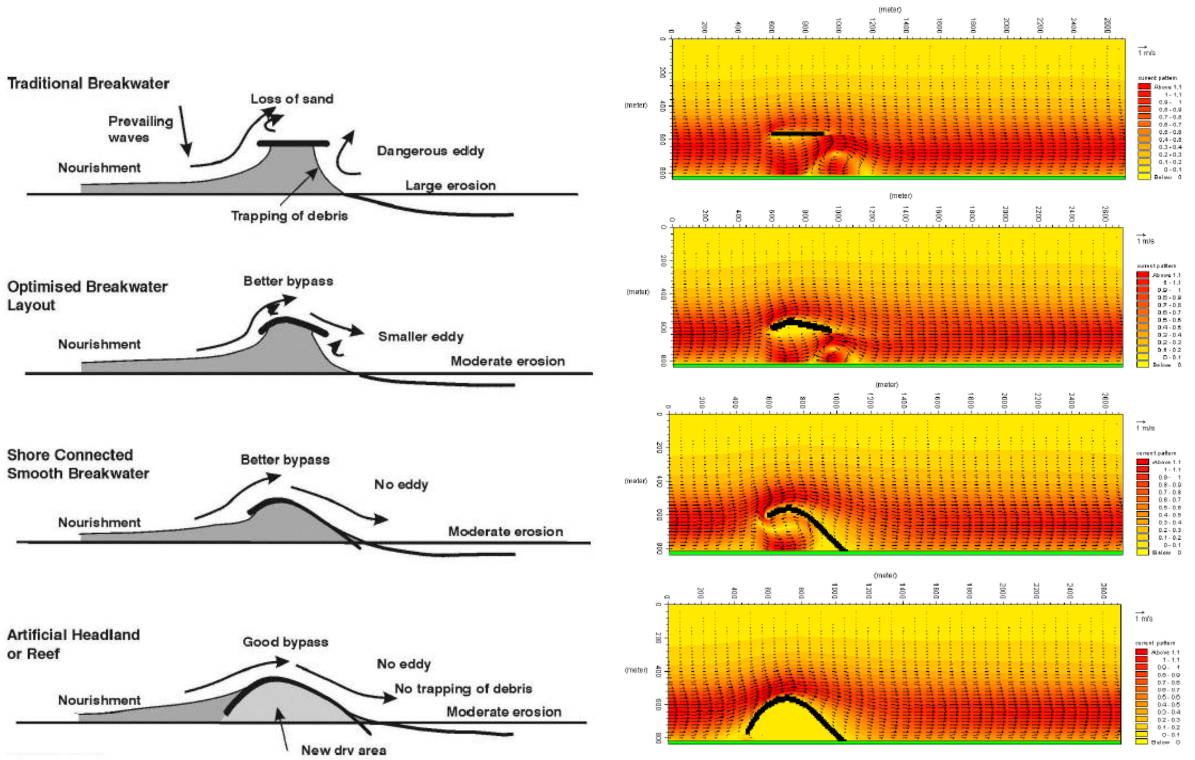


Figure 5.33: Optimisation of coastal breakwaters (left) and Current patterns (right) (Mangor, Drønen, Kærgaard, & Kristensen, 2017)

The provided advices, suggestions and recommendations will also be included in the tool in order to define the optimal layout of the breakwaters. However, no quantification of the sediment transport process will be implemented.

5.5 Bathymetry

The bathymetry in and adjacent to the harbour location is fundamental to known, as this has a large effect on the wave propagation processes as shoaling, refraction and wave breaking and on the port category as discussed in paragraph 5.1.

It is also recommended to reduce the depth along the breakwater structures and to follow bathymetry contour lines, this to avoid excessive construction material, especially in the case of rubble mound breakwaters (BS 6349, 2000; Agerschou, et al., 2004; PIANC, 2014). According to Massie (1986), the volume of material needed for a rubble mound breakwater increases quadratically with the water depth.

The depth will also influence the type of breakwaters constructed to protect the harbour. For small to medium water depths the choice of a rubble mound breakwater is generally advantageous from an economic point of view, while in larger depths or when rock material are not available, vertical caisson breakwaters are more optimal. The reason for this is that the volume of construction material for a rubble mound breakwater increases faster with the depth than it does in case of a vertical breakwater. In general, a vertical breakwater is more costly, except in the case of large depths with small wave loading (PIANC, 2016).

The bathymetry also determines the amount of dredging needed to provide vessel navigation in and outside the harbour basin, which means it will have a significant impact on the costs.

During the preliminary design phase earlier studies, nautical charts or bathymetric dataset, nowadays freely available and accessible, can be used to determine useful sites with already adequate water depths along a coastline. Afterwards, a more in-depth survey can take place (PIANC, 2019).

5.6 Geotechnical aspect

The geotechnical characteristics of the subsoil are important for the stability of the breakwater structures and the planning of dredging and reclamation operations. During the choice of the harbour location, it is possible to avoid locations with rocky and/or hard soils to reduce dredging costs. Locations with very soft clay layers or loose sand are also regarded as poor harbour location, due to their weak geotechnical characteristics. Therefore, it is better to avoid subsoils with significant settlement, liquefaction or sliding (PIANC, 2014).

The geotechnical characteristics will also influence the choice of breakwater type. Rubble mound breakwaters are in general less sensitive to differential settlement of soft subsoils, compared to vertical caisson breakwaters which need sufficient replacement of soft subsoils or other foundation improvements. The latter is the case with vertical composite breakwaters. The loading of small to medium rubble mound breakwaters is also relatively low, in some cases it is possible to construct these breakwaters without any replacement of weak subsoil needed (PIANC, 2016).

The necessary geotechnical data can be acquired by the use of regional geological maps, borehole investigations, side scan sonar and geophysical surveys. However, these detailed local measurements will usually be executed in a later, more detailed, design stage (BS 6349, 2000; PIANC, 2019).

In the further elaboration of the present tool, this design aspect is not considered, and it is assumed that the soil conditions under the breakwater are adequate to construct both rubble mound and vertical breakwaters. Nevertheless, it is an important design aspect that should not be forgotten during the detailed design of the harbour.

5.7 Preliminary cost

The cost of the breakwaters is in fact related to all previous mentioned design aspects and should than also be taken into account when looking at each design aspect separately. During the design one can optimise the harbour layout by trying to reach an extreme value for one of the chosen cost criteria.

Many different parameters can be listed here, such as the breakwater location, the crest free board, the breakwater depth, the breakwater type, the breakwater construction costs, the dredging and backfilling costs, the armour unit, the severity of the metocean conditions, the length, the direct and indirect damage, sedimentation, environmental factors and many more (Massie, 1986; ASCE, 2012).

However, not all of the parameters mentioned can be linked to or assessed during the preliminary layout design of the harbour. The most significant parameter in computing the breakwater cost is the depth of the water, taking into account the tides and storm surges. As the depth indicates the optimal breakwater type, this is another important cost factor in the harbour layout design. The breakwater location can impose poor foundation conditions or the need to remove bedrock, which both result in additional costs. Significant longshore sediment transport can introduce a need for frequent maintenance dredging or a sediment bypassing system, with each its impact on the project cost. Another import parameter is the wave climate, severe conditions impose more stable breakwater constructions.

The total cost of the harbour project consists of the construction material costs, the dredging cost, the mobilisation and installation of construction equipment and more. During the development of the tool the total cost of the harbour layout will be used to evaluate the different configurations.

5.8 Conclusion

Several design aspects need to be considered during the preliminary design of the harbour layout. The design aspects discussed in this chapter will now be used in chapter 6 to develop a methodology and tool, which can be used to generate and evaluate multiple harbour configurations during the preliminary phase of a harbour layout design.

6 DESCRIPTION OF METHODOLOGY AND TOOL

The current chapter provides a description of the developed tool with the design methodology implemented. The tool is a combination of an Excel spreadsheet and a MATLAB script and will generate and evaluate multiple harbour configurations in a relative short time period.

The first step is to enter the required input in the tool, such as the harbour requirements, the bathymetry data, the metocean data offshore, the design vessel, etc.. Based on these input parameters some first calculations are done to determine the metocean data in front of the harbour, the manoeuvring areas dimensions and the ship berths dimensions. With this data, an initial harbour layout can be proposed by the tool. After these first steps, the tool plots this configuration together with multiple alternative configurations. These first steps are schematised in Figure 6.1. In this figure, blue boxes represent the set of input parameters, the yellow ones represent the calculation process and in red are displayed the output.

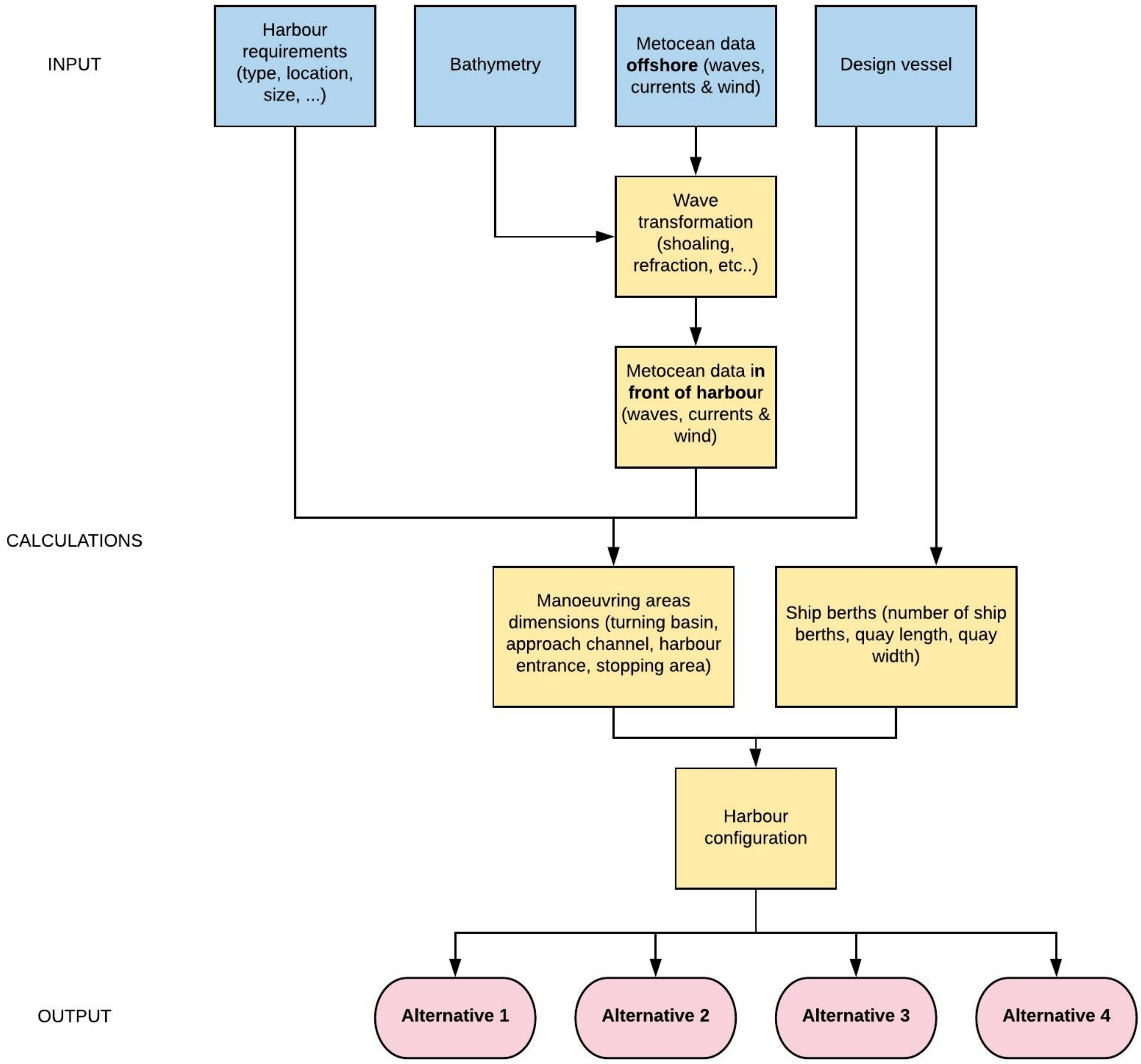


Figure 6.1: Scheme of tool (part 1)

For each alternative harbour layout, the tool computes the wave agitation inside the harbour basin with the means of diffraction, reflection and transmission calculations. Further, the tool provides a preliminary cost estimation based on the dredging needs and the breakwater costs. The different harbour alternatives are then compared by the tool with a comparison of three criteria: the preliminary costs, the navigational aspect and the wave agitation by means of a decision matrix. These steps are schematised in Figure 6.2 for one specific alternative harbour layout and the same scheme is then repeated for every other alternative.

For one alternative harbour configuration, selected by the user, the tool can perform an analysis of the variation in wind/wave direction and the variation of geometrical harbour features. The effect of these variations on the preliminary cost estimation, the wave agitation inside the harbour and the navigational aspect is then examined.

In what follows, the input, the different formulas, decisions, chosen methodologies and output of the tool will be discussed in detail.

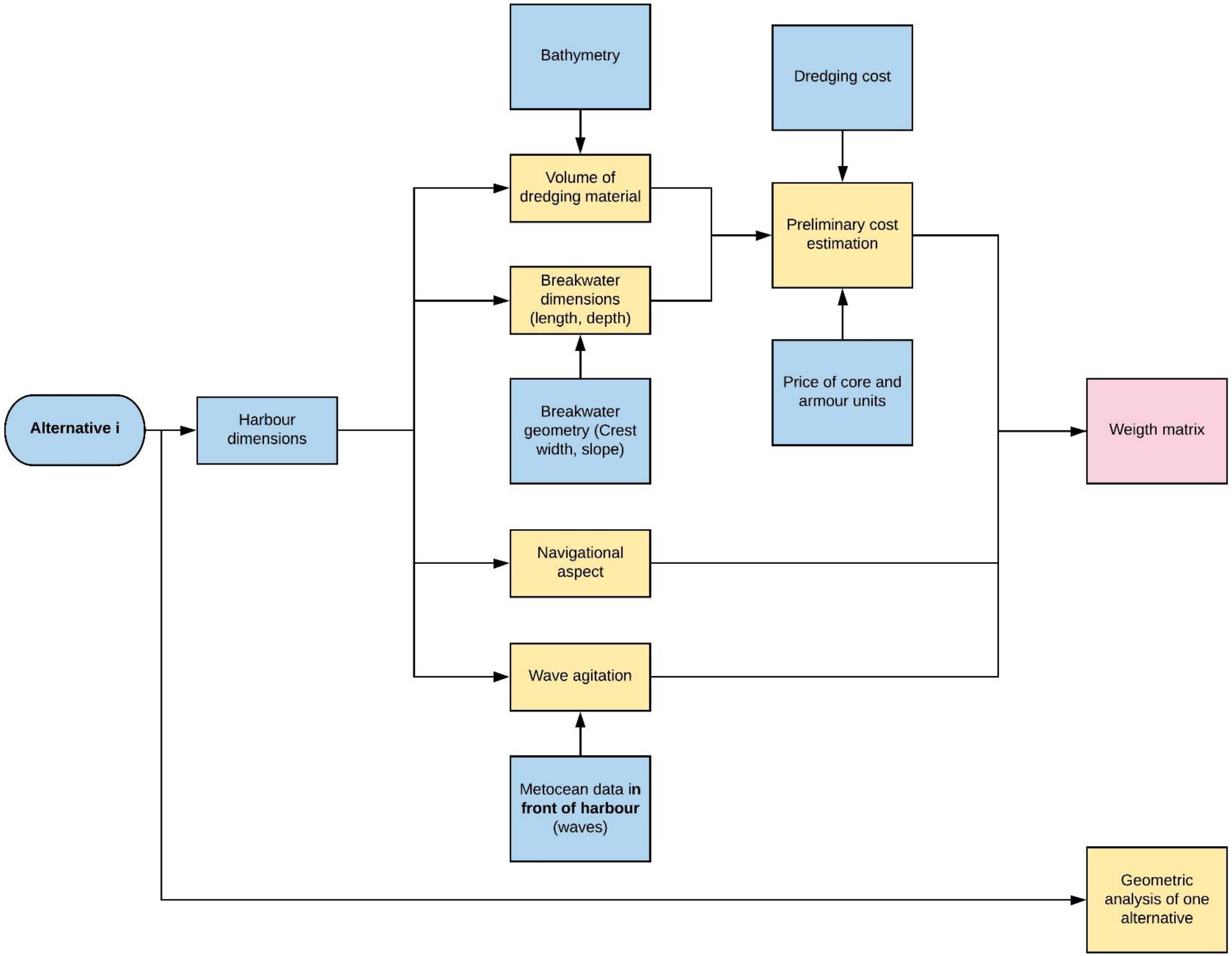


Figure 6.2: Scheme of tool (part 2)

6.1 Input of the tool

In this paragraph the different input parameters of the tool, which need to be entered in the Excel spreadsheet, are explained. An example of every input parameter is also given.

6.1.1 Harbour requirements

As a first input, the tool asks for the general harbour requirements. Here, the type of the harbour has to be selected. For this, a choice can be made from the harbour types in Table 6.1. The harbour type is linked to the selected design vessel, discussed further in paragraph 6.1.2.

Table 6.1: Harbour types

Harbour types
Container terminals
General cargo and multipurpose terminals
RoRo and ferry terminals
Liquid bulk terminals
Dry bulk terminals
Fishery port
Marina

Further, the port location has to be inserted. As was discussed in paragraph 5.1, the port location can be either an open coastline or a natural protected harbour such as a bay, a lagoon, or a river harbour. An evaluation of the port location must therefore be carried out before using the tool. Based on this evaluation the tool can already provide a preliminary recommendation regarding the harbour configuration, as a natural protected harbour requires less breakwaters. The size of the harbour also needs to be determined, this is needed for the determination of the manoeuvring areas dimensions and the mooring conditions.

It is assumed that the determination of the amount of ship berths n is already done prior to the engineering tool and has to be entered as an input parameter. Further, it is asked if there is a need of breakwater structures with berthing facilities as shown in Figure 6.3. This, together with the depth, will determine the type of breakwater structure. At last, it is requested if there are tugboats assisting the vessels inside the harbour basin.

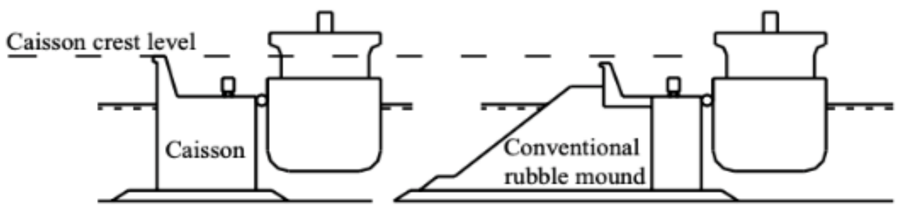


Figure 6.3: Examples of berthing facilities behind breakwaters (PIANC, 2016)

An example of these input parameters is shown in Figure 6.4. The grey cells must be filled in.

Harbour requirements		
Type of Harbour	Container Terminals	
Port Location	Open coastline	
Size of Harbour	Small	
Amount of ship berths	3	[-]
Breakwaters with berthing facilities	No	
Tugboats	Without tugs	

Figure 6.4: Example of the harbour information input parameters

6.1.2 Design vessel

The design vessel is needed to determine the dimensions of the manoeuvring areas and the mooring and limit state conditions. It is assumed that a design vessel is selected before using the tool. The type, standard dimensions (beam B_v, overall length L_v and draught T_v) and displacement Δ of the design vessel are requested by the tool. To determine the manoeuvring areas dimensions in detail, the tool needs to know the number of rudders, the possible presence of a thruster, the possible ability to use bow and stern anchors and the installed power P of the engines. If these last parameters are not known during the preliminary design, following assumptions are proposed: (i) only one rudder, (ii) no thruster, and (iii) no ability to use bow and stern anchors. By using previous assumptions, larger manoeuvring areas will be computed which ensures a more conservative approach.

If the installed power P of the design vessel is not known following recommendations of PIANC (2015) can be used for some vessel types:

- RoRo: 10,000 kw to 20,000 kw
- Cruise: 70,000 kw to 120,000 kw
- Container ship:

$$P = 2\,800\,B_v - 60\,000 \tag{6.1}$$

where P is the installed power and B_v the design vessel's beam.

The last input parameter of the design vessel is the speed of the vessel sailing through the approach channel V_s, which is needed for the determination of the approach channel dimensions. It should be mentioned that this speed is not the maximum speed of the design vessel.

Figure 6.5 is an example of the input parameters for the design vessel in the tool.

Design Vessel				
Type		Oil Tanker		
Beam	B_v		10,00	[m]
Length	L_v		65,00	[m]
Draught	T_v		4,50	[m]
Displacement	Δ		1.200,00	[tonnes]
Rudder			1	[-]
Thruster		No		
Ability to use bow and stern anchors		No		
Installed power	P		No data available	[kw]
Installed power	P		1.000,00	[kw]
Vessel speed	V_s		4,00	[kn]

Figure 6.5: Example of the design vessel input parameters

6.1.3 Metocean data

The input of the metocean data is divided into the input of the design wave conditions, the input of the conditions in the approach channel and the input of the water level.

6.1.3.1 Design wave

To design any coastal structure a design wave is needed, usually represented by the significant wave height H_s based on statistical analysis of long-term wave event data. However, this wave data set will often be insufficient large and extrapolation of this data set to a longer time frame is needed. It is assumed that the extreme wave analysis is performed before using the tool.

The main goal of this extreme wave analysis is to determine the wave height $H_{s,0}$ at a deep-water location with a specific return period RP. The return period RP is the average number of years in which the wave height $H_{s,0}$ is expected to occur or even exceeded once. This extreme wave analysis can be done with methods proposed by Goda (2000), Liu & Burcharth (1999), Sorensen (2006) or others.

The predicted offshore design wave height, or significant wave height $H_{s,0}$, can than be used to determine the wave height in front of the harbour H_s, by estimating the wave transformation processes which will be discussed in the following paragraphs. Based on the latter, the design of the harbour layout will be preformed. However, if known, it is also possible to enter the significant wave height H_s in front of the harbour entrance directly into the tool. In this case no calculations of the wave transformation processes are needed.

The tool requests the characteristics of the deep-water wave, offshore of the harbour. These are the deep-water significant wave height $H_{s,0}$, the peak wave period $T_{p,0}$, the mean direction of the deep-water wave θ_0 and the size of the wave sector as shown in Figure 6.6.

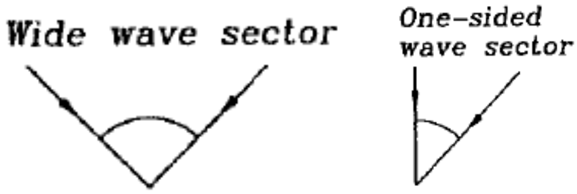


Figure 6.6: Size of wave sector adapted from (Liu & Burcharth, 1999)

If the wind speed U is known, it can be used to determine the directional spreading parameter s_{max} . This can be done in two ways, first by using following formula (Goda, 2000):

$$s_{max} = 11.5 \left(\frac{2\pi f_p U}{g}\right)^{-2.5} \tag{6.2}$$

where s_{max} represents the spreading parameter, U the wind speed, f_p the peak frequency equal to $1/T_p$ and g the gravitational acceleration. The second possibility is to use the graph in Figure 6.7 with means of the deep-water wave steepness, which is equal to:

$$s_0 = H_{s,0}/L_0 \tag{6.3}$$

with $H_{s,0}$ the deep-water significant wave height and L_0 the deep-water wavelength.

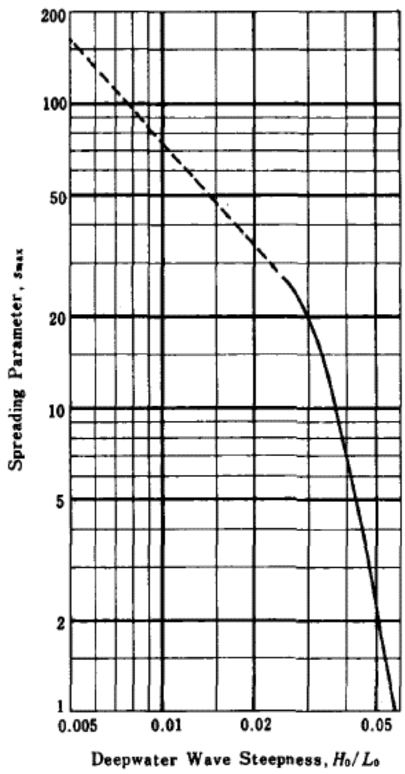


Figure 6.7: Relationship between spreading parameter and deep-water wave steepness (Goda, 2000)

If known, the directional spreading parameter s_{max} can also be entered as an input parameter.

The spectral mean wave period $T_{m-1,0}$ is determined by following simple formula and used in the overtopping calculations.

$$T_{m-1,0} = 0.9T_p \tag{6.4}$$

All wave parameters are requested for two different return periods RP. For the determination of the mooring conditions a return period RP of 1 year will be used, this means yearly storm events which will hinder the operations inside the harbour. For the breakwater design a return period RP of 50 years will be used.

Figure 6.8 shows an example of the input parameters requested for the design wave conditions. As shown, three different significant wave heights H_s are mentioned in the "*In front of harbour entrance*"-section. The first one is the possible input parameter if known, the second one is the significant wave height H_s in front of the harbour computed by the tool and the third one is the wave height H_s which will be used in the remaining calculations of the tool. The same can be mentioned for the directional spreading parameter s_{max}.

Design wave					
	Offshore of port (Deep water wave)				
	Return period	R	1	50	[years]
	Deep water significant wave height	$H_{s,0}$	1,00	4,00	[m]
	Peak wave period	$T_{p,0}$	15,00	12,00	[s]
	Depth	d_0	500,00	500,00	[m]
	Direction of deep water wave	θ_0	10,00	10,00	[°N]
	Wave sector		Wide wave sector	Wide wave sector	
	Wind speed	U			[m/s]
	Directional spreading parameter	s_{max}	No wind data	No wind data	[-]
	Deep-water wave steepness	H_0/L_0	0,003	0,018	[-]
	Directional spreading parameter	s_{max}	49,00	49,00	[-]
	Directional spreading parameter	s_{max}	49,00	49,00	[-]
	In front of harbour entrance				
	Return period	R	1	50	[years]
	Significant wave height	H_s			[m]
	Significant wave height	H_s	1,28	3,21	[m]
	Significant wave height	H_s	1,28	3,21	[m]
	Peak wave period	T_p	8,00	11,50	[s]
	Wave period	$T_{m-1,0}$	7,20	10,35	[s]
	Depth	d	5,53	10,00	[m]
	Direction	θ	10,00	10,00	[°N]
	Wave sector		Wide wave sector	Wide wave sector	
	Directional spreading parameter	s_{max}	75,00	75,00	[-]

Figure 6.8: Example of the input of the design wave conditions

6.1.3.2 Conditions in approach channel

To determine the width and depth of the approach channel, a detailed description of the approach channel conditions is needed. First, the metocean conditions inside the approach channel are requested by the tool, these consist of the crosswind speed V_{cw}, the cross current V_{cc} and the longitudinal current V_{lc}. Based on these metocean conditions, the user is asked to make a judgment whether the manoeuvring conditions are favourable, unfavourable or moderated. Puertos del Estado (2007) provides following criteria for favourable manoeuvring conditions:

$$\begin{gathered} V_{cw} \leq 10.00\ m/s\ (20\ kn) \\ V_{cc} \leq 0.50\ m/s\ (1\ kn) \\ H_s \leq 3.00\ m \end{gathered} \tag{6.5}$$

In addition to the metocean data, more input parameters are requested by the tool for the determination of the approach channel dimensions. For the level of Aids to Navigation (AtoN) and associated systems, a selection needs to be made out of (i) Excellent with shore traffic control, (ii) Good, (iii) Moderate with infrequent poor visibility and (iv) Moderate with frequent visibility. An indication for the terms 'Excellent', 'Good' and 'Moderate' can be found in Table 6.2 (PIANC, 2014).

Table 6.2: Indication of Aids to Navigation (AtoN) (PIANC, 2014)

Terms	Aids to Navigation (AtoN) and associated systems
Excellent	Channel: • Paired lighted buoys with radar reflectors • Lighted leading lines • Vessel Traffic Service (VTS), where applicable With the availability of • Pilots • Differential global navigation satellite positioning systems (DGPS) • Electronic chart display and information system (ECDIS)
Good	Channel: • Paired lighted buoys with radar reflectors • Lighted leading lines With the availability of • Pilots • Differential global navigation satellite positioning systems (DGPS)
Moderate	Anything less than the facilities mentioned above

Next, the bottom type and surface are requested. For the bottom type, a choice can be made out of mud, sand, clay or coral. The bottom surface has an effect on the channel width in shallow waterways (d < 1.5 T_v). Smooth and soft bottoms, such as silt and mud, have an influence on the manoeuvrability and propulsion of the vessel. Hard materials, rock and coral, will induce larger damage due to grounding than soft materials (PIANC, 2014).

The next input parameter is the cargo hazard level. The International Maritime Organization (IMO) and national regulations prescribe the cargo hazards based on the toxicity, explosive potential, pollution potential, combustion potential and the corrosive potential. LNG, LPG and certain classes of chemicals are high cargo hazards.

Further, the encounter traffic density is requested. With 0 to 1 vessel per hour, the traffic density is light. It becomes moderate with 1 to 3 vessels per hour and is heavy when more than 3 vessels sail through the approach channel per hour.

At last, to determine the bank clearance as shown in Table 5.8, the bank characteristics are requested. Here, a choice can be made between a steep bank and a sloping edge.

An example of the input parameters of the approach channel conditions is shown in Figure 6.9.

Conditions Approach Channel				
	Crosswinds	V_{cw}	5,00	[kn]
	Crosswinds	V_{cw}	5,00	[kn]
	Crosscurrents	V_{cc}	0,50	[kn]
	Crosscurrents	V_{cc}	0,50	[kn]
	Longitudinal currents	V_{lc}	2,00	[kn]
	Longitudinal currents	V_{lc}	2,00	[kn]
	Manoeuvring conditions		Favorable	[-]
	Aids to navigation		Excellent with shore traffic control	[-]
	Bottom type		Sand	[-]
	Bottom surface		Smooth and soft	[-]
	Cargo hazard level		Low	
	Encounter traffic density		Light	
	Bank		Sloping edge	[-]

Figure 6.9: Example of the input of the approach channel conditions

One may notice that the crosswinds, crosscurrents and longitudinal currents have both an input cell and a white cell. This is due to the fact that these parameters can be calculated based on the input wind speed and design wave explained in paragraph 6.1.3.1. These calculations will be explained in detail in paragraph 6.2.1.

6.1.3.3 Water level

As was explained in paragraph 5.2.4 the harbour layout needs a design water level and a specific reference level. These parameters can be entered under the water level input. It is assumed that the user has performed an extreme analysis regarding the water level and has computed already a design water level.

An example of the water level in put in shown in Figure 6.10.

Water level					
	Reference level		LAT		
	Return period	R	1	50	[years]
	Design water level		2,00	2,00	[m]

Figure 6.10: Example of the input of the water level

6.1.4 Sediment transport

The quantification and assessment of the entire sediment transport in and near the harbour basin, is too complex to implement in a simple engineering tool. So, it is expected that before using the tool, a well-considered estimation of the sediment transport has already been made. The tool requests if the amount of sediment transport is significant or not significant near the harbour location and whether this transport is one or two sided. These two input parameters are used to give a preliminary indication of the harbour configuration, as was explained in paragraph 5.4.

Figure 6.11 gives an example of the input parameters of the sediment transport.

Sediment transport		
	Amount	Not Significant
	Sides	One sided

Figure 6.11: Example of the input of the sediment transport

6.1.5 Bathymetry

For the bathymetry a very simplified input is required. Here, only the beach slope m is requested, which is defined before using the tool, and the assumption is made that the depth contour lines are evenly spaced and parallel to the shoreline. The tool then makes a judgment of the beach slope, if m is smaller than 50 the beach is called steep, otherwise it is a smooth slope.

Figure 6.12 shows the input parameter for the bathymetry.

Bathymetry					
	Slope	m		100,00	[-]
	Slope angle	β		0,57	[°]
			Smooth		
			Uniform		

Figure 6.12: Example of the input of the bathymetry parameter

6.1.6 Breakwater requirements

Regarding the preliminary cost estimation and overtopping calculations, more information is required about the breakwater geometry. Although there is usually no detailed design of the cross section of the breakwater structures during the preliminary design phase of a harbour layout. So, here the tool uses a very simplified breakwater design consisting of two layers, the core and the armour layer. For these two layers the cost of the units is requested.

Further the crest width, slope and allowable overtopping discharge are required as input parameters, these are needed for the overtopping and transmission calculations.

An example of the preliminary breakwater design input parameter is shown in Figure 6.13.

Breakwater requirements			
Cost of core rocks	C_{core}	20,00	[€/m^3]
Armour unit		Rock	[-]
Armour unit cost	C_{unit}	30,00	[€/m^3]
Transport cost	C_{trans}	20,00	[€/tonnes]
Crest width	B_{crest}	7,00	[m]
Slope	cotan(α)	1,50	[-]
Slope angle	α	33,69	[°]
Overtopping discharge	q	5,00	[l/s/m]
Cost of Vertical breakwater	C_v	110.000,00	[€/m]

Figure 6.13: Example of the input of the breakwater design parameters

Further, the tool asks the limiting depth of a rubble mound breakwater. If the bathymetry exceeds this depth, a vertical breakwater is proposed for the locations with a deep-water depth in the harbour layout.

6.1.7 Dredging cost

Finally, the dredging costs are requested as an input. This requires a general dredging and backfilling cost, regardless of the material being dredged, expressed in euros per cubic metre. Further the mobilisation, demobilisation and installation cost of one dredger is required as input parameter.

Figure 6.14 shows an example of the input parameters of the dredging cost.

Dredging cost			
Dredging cost	C_{dredge}	6,00	[€/m^3]
Backfilling with dredged material	C_{back}	6,00	[€/m^3]
Mobilisation/demobilisation of one dredger	C_{mob}	1.500.000,00	[€]

Figure 6.14: Example of the input of the dredging cost parameters

6.2 Calculations

In this paragraph the implemented calculations and methods are discussed. First, the necessary calculations are performed to determine an initial harbour layout by the Excel spreadsheet. Based on the proposed dimensions, the MATLAB script computes three more alternative configurations and determines the wave agitation and the preliminary cost of the harbour layouts. After doing this, the MATLAB script sends the output of the calculations back to the Excel spreadsheet, where the computations are evaluated.

6.2.1 Metocean conditions

To determine the dimensions of the harbour and to compute the wave agitation inside the harbour basin some calculations need to be performed regarding the wave, wind and current conditions outside the harbour basin.

6.2.1.1 Wavelength

For some of the upcoming calculations, the wavelength will be needed. The wavelength is computed by means of the linear (airy) wave or small-amplitude theory (US Army Corps of Engineers, 2002). This theory gives an acceptable approximation of the wave characteristics during the preliminary design phase. Formula (6.6) gives the wavelength according the linear wave theory and should be solved iteratively.

$$L = \frac{gT^2}{2\pi} \tanh\left(\frac{2\pi d}{L}\right) \tag{6.6}$$

where L represents the wavelength, T is the wave period, g the gravitational acceleration and d the water depth.

Based on the wavelength the relative water depth d/L can be derived, which can be used for the classification of the water wave as shown in Table 6.3. Here, k represents the wave number and is equal to $2\pi/L$.

Table 6.3: Classification of water waves (US Army Corps of Engineers, 2002)

Classification	d/L [-]	kd [-]	$tanh\,(kd)$ [-]
Deep water	1/2 to ∞	π to ∞	≈1
Transitional water	1/20 to 1/2	π/10 to π	$\tanh\,(kd)$
Shallow water	0 to 1/20	0 to π/10	$\approx kd$

6.2.1.2 Wave transformations of approaching waves

In most cases, wave conditions are known in deeper water, but the location of the harbour is often near the coastline. Due to wave propagation towards the shore, the wave characteristics will change by the effect of bathymetry and currents (US Army Corps of Engineers, 2002). In what follows, some of the wave transformation processes included in the tool will be explained.

In general, a wave transformation process will transform the incident wave height H_{inc} into a disturbed wave height H_d, the ratio of the these two wave heights is equal to the disturbance coefficient K_d:

$$K_d = \frac{H_d}{H_{inc}} \tag{6.7}$$

6.2.1.2.1 Wave shoaling

Wave shoaling occurs when a wave propagates from deep water into shallower water, which will change the wave height. To estimate the wave shoaling process a method, based on the linear wave theory, proposed by CEM (2002) and Sorensen (2006) is followed.

Assume a gradual sloping bathymetry and wave crests parallel to the straight bottom contours. Due to the gradually decreasing depth, the wavelength and celerity according to the theory of small amplitude waves will also decrease, the waves 'feel' the sea bottom and the wave height H will start to increase while the wavelength L will decrease. The former means that the waves will shorten and the wave steepness s will increase, which is shown in Figure 6.15.

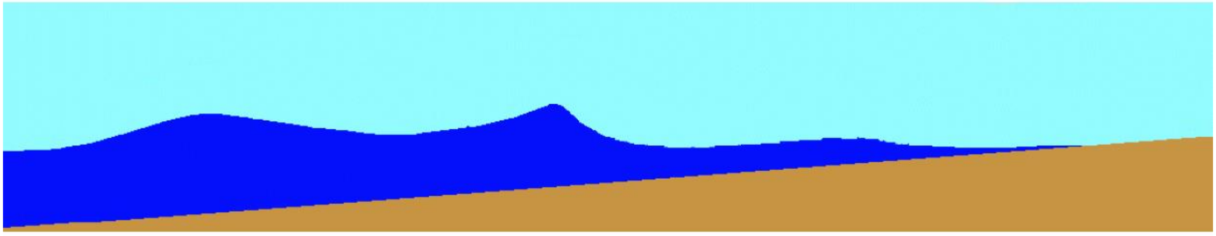


Figure 6.15: Wave shoaling (Mangor, Drønen, Kærgaard, & Kristensen, 2017)

To derive an expression for the shoaling coefficient K_s one has to state the conservation of energy, which means the energy entering the area is equal to the energy leaving the area (US Army Corps of Engineers, 2002; Sorensen, 2006):

$$\bar{P}_0 = \bar{P}_1 \tag{6.8}$$

where $\bar{P}_0$ is the amount of energy entering the area (in deep water) and $\bar{P}_1$ is the amount of energy leaving the area at a water depth d_1. Expression (6.8) can be expressed as the following

$$\bar{E}_0 n_0 C_0 = \bar{E}_1 n_1 C_1 \tag{6.9}$$

where $\bar{E}$ is the average energy per unit surface area, n the ratio of wave group to phase celerity and C the wave celerity. Formula (6.9) can be in turn rewritten by using of following expression

$$\bar{E} = \frac{E}{L} = \frac{\rho g H^2}{8} \tag{6.10}$$

where E is the total energy per unit crest width, L the wavelength, ρ the water density, g the acceleration of gravity and H is the wave height.

$$\frac{1}{8}\rho g H_0^2 n_0 C_0 = \frac{1}{8}\rho g H_1^2 n_1 C_1 \tag{6.11}$$

By removing the identical parameters at the left- and right-hand side of formula (6.11) the shoaling coefficient K_s is derived:

$$K_s = \frac{H_s}{H_0} = \sqrt{\frac{n_0 C_0}{n_1 C_1}} = \sqrt{\frac{C_{g,0}}{C_{g,1}}} \tag{6.12}$$

Following formulas are used for the computation of the group velocity in deep water and in front of the harbour, based on the linear (Airy) wave theory (US Army Corps of Engineers, 2002):

$$C_{g,0} = n_0 C_0 = \frac{1}{2} C_0 = \frac{gT}{4\pi} \tag{6.13}$$

$$C_{g,1} = n_1 C_1 = \frac{1}{2}\left(1 + \frac{\frac{4\pi d_1}{L}}{\sinh\left(\frac{4\pi d_1}{L}\right)}\right)\frac{gT}{2\pi}\tanh\left(\frac{2\pi d_1}{L}\right) \tag{6.14}$$

6.2.1.2.2 Wave refraction

In case of wave refraction, the waves will approach the shore under an angle, as shown in Figure 6.16 and Figure 6.17. As was the case with wave shoaling, a method proposed by CEM (2002) and Sorensen (2006) is followed.

Assuming a gradual sloping bathymetry with straight bottom contours parallel to the shoreline, there will be a speed difference between different points of one wave crest. This is caused by the fact that the group velocity will decrease in decreasing depth in case of shallow water waves. The depth of point A in Figure 6.17 is smaller than in point B, so the group velocity C_A will be smaller. Due to this phenomenon point B moves faster than point A and the wave crest will come more parallel to the coastline.

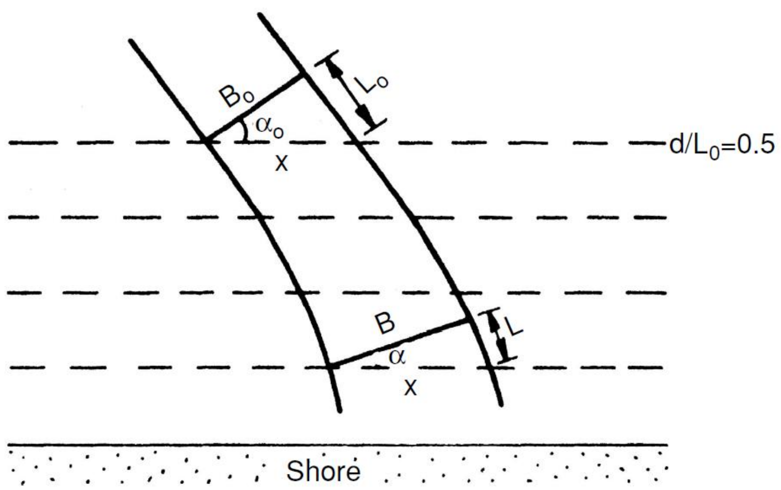


Figure 6.16: Wave refraction, with straight parallel bottom contours (Sorensen, 2006)

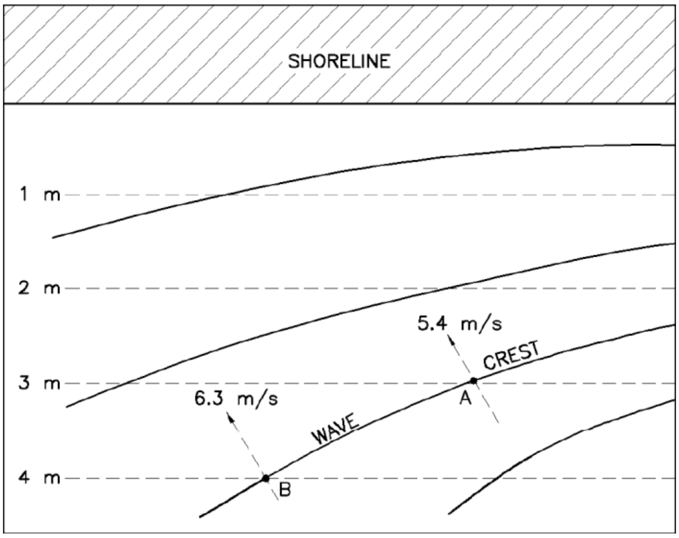


Figure 6.17: Straight shore with all depth contours evenly spaced and parallel to the shoreline (US Army Corps of Engineers, 2002)

The derivation of the refraction coefficient K_r is similar to the previous derivation. Again the conservation of energy can be stated, however this time the width of the areas can not be neglected (US Army Corps of Engineers, 2002; Sorensen, 2006):

$$\bar{P}_0 b_0 = \bar{P}_1 b_1 \tag{6.15}$$

where $\bar{P}_0$ is the amount of energy entering the area (in deep water), $\bar{P}_1$ is the amount of energy leaving the area at a water depth d_1 and b is the width of the areas. Now, the same derivation as with the shoaling coefficient can be preformed.

$$\bar{E}_0 n_0 C_0 b_0 = \bar{E}_1 n_1 C_1 b_1 \tag{6.16}$$

$$\frac{1}{8}\rho g H_0^2 n_0 C_0 b_0 = \frac{1}{8}\rho g H_1^2 n_1 C_1 b_1 \tag{6.17}$$

$$K_{sr} = K_s\, K_r = \frac{H_{sr}}{H_0} = \sqrt{\frac{n_0 C_0}{n_1 C_1}}\sqrt{\frac{b_0}{b_1}} \tag{6.18}$$

The refraction coefficient K_r can be rewritten considering Snell's law (Sorensen, 2006):

$$\frac{\sin\alpha_0}{\sin\alpha_1} = \frac{C_0}{C_1} \tag{6.19}$$

$$\sin\alpha_1 = \frac{C_0}{C_1}\sin\alpha_0 \tag{6.20}$$

$$\frac{\cos\alpha_0}{\cos\alpha_1} = \frac{b_0}{b_1} \tag{6.21}$$

where α is the angle between the wave crest and the straight bottom contour. Formula (6.20) can be used to derive the wave propagation angle α_1 at the harbour entrance. Combining formulas (6.18) and (6.21) gives the refraction coefficient K_r:

$$K_r = \sqrt{\frac{b_0}{b_1}} = \sqrt{\frac{\cos\alpha_0}{\cos\alpha_1}} \tag{6.22}$$

6.2.1.2.3 Wave breaking

When a wave approaches the coastline, the wave height will increase together with the steepness due to wave shoaling. However, the wave height H is restricted by the water depth d and the wavelength L, this means that for certain water depth d and wave period T there exist a limit wave height. For a certain wave steepness s, the wave will break, and its energy will dissipate. The maximum wave height is also called the breaking wave height H_{br}. There exists a distinction in determining the breaking wave height H_{br} in deep water on one hand and in shallow and transitional water on the other hand. For a deep-water wave, where the relative water depth d/L is larger than ½, the breaking wave height H_{br} depends only on the wavelength L_0, using the wave steepness $s = H/L$. The breaker criteria in deep water is expressed as a limit for the wave steepness s and can be found in Table 6.4.

Table 6.4: Breaker criteria in deep water expressed as a limit of wave steepness s

Classification	Wave steepness s [-]
Open sea	0.02 – 0.03
Coast	0.05 – 0.07
Maximum value	1/7 = 0.142

For shallow and transitional water, the breaking wave height H_{br} depends not only on the wavelength L, but also on the water depth d due to the effects of the shoaling and refraction processes. During this processes the wave height will increase while the water depth decreases, thus according to the linear Airy wave theory, the wavelength will also decreases (US Army Corps of Engineers, 2002).

$$L = T\sqrt{gd} \tag{6.23}$$

where L represents the wavelength, T the wave period, g the gravitational constant and d the water depth. Which means the wave steepness H/L will increase until the limit steepness after which the wave will break. The breaker criteria in this case is expressed by the breaker index γ_b:

$$\gamma_b = \frac{H_{br}}{d_{br}} \tag{6.24}$$

with the breaking wave height H_{br} and the depth at breaking d_{br}. Following expression is used to determine the breaker index γ_b (Kamphuis, 2000):

$$\gamma_b = 0.56e^{3.5m} \tag{6.25}$$

where m is the beach slope.

6.2.1.2.4 Estimation of breaking zone location

Sawaragi (1995), Ligteringen (2017) and Memos (2018) suggest to extend the breakwaters past the breaking zone because the sediment transport is smaller in deeper water and to reduce the load-impact of waves. In order to do this, the location of the breaking zone needs to be estimated. It should be stressed that this is very difficult, and the following formulas are merely a rough approximation. For the estimation of the breaking zone location, a method proposed by Kamphuis (2000) is followed.

Again, it is assumed that the shallow water region in front of the shoreline has straight depth contours, evenly spaced and parallel to the shoreline. The slope of the sea bottom is defined as m. As mentioned in previous paragraphs, a wave will grow approaching the coastline to the breaking height and then break. The breaker index γ_b can be calculated by formula (6.25) and with means of formula (6.24) the breaking wave height H_{br} can be expressed as a function of the water depth d:

$$H_{br} = \gamma_b\, d \tag{6.26}$$

With means of formulas (6.12) and (6.22) the shoaling and refraction coefficient K_{sr} can be determined for each water depth and the resulting wave height H_{sr} can be calculated along a line extending from the coastline to deeper water:

$$H_{sr} = H_0 K_{sr} \tag{6.27}$$

where H_0 is the deep-water wave. Both the breaking wave height H_{br} and the shoaling and refraction resulting wave height H_{sr} can now be plotted as a function of the water depth d, as shown in Figure 6.18. At a certain water depth d_{br}, the dashed line of refraction and shoaling crosses the wave breaking line, and the wave will break. So, the breaking zone will start approximately at the breaking water depth d_{br} and will extend to the left of the graph.

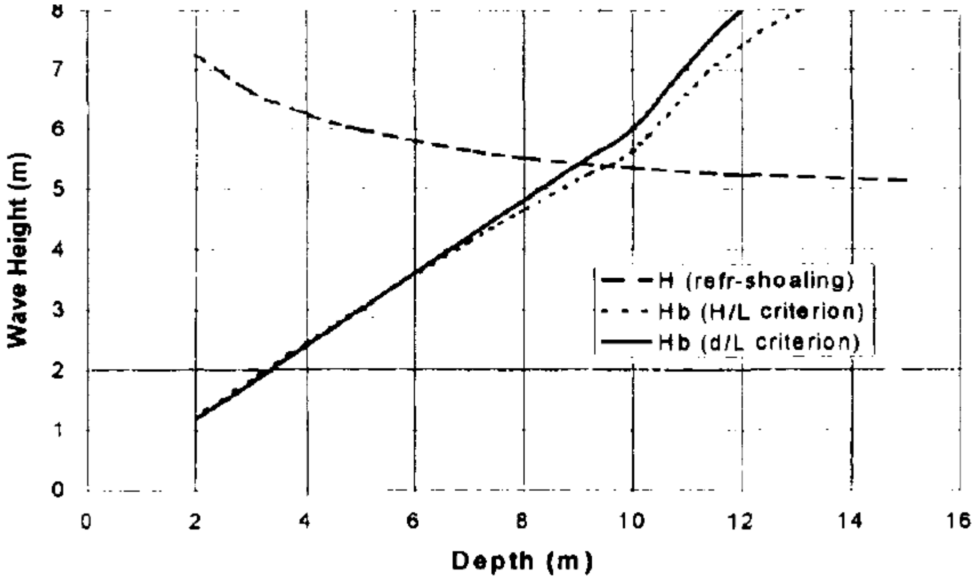


Figure 6.18: Example of the calculation of wave breaking, shoaling and refraction (Kamphuis, 2000)

To use the previous observations in the tool, the following is done. The shoaling coefficient K_s can be directly calculated from the water depth d and wavelength L:

$$K_s = \sqrt{\frac{1}{\left(1 + \frac{4\pi d/L}{\sin(4\pi d/L)}\right)\tanh(2\pi d/L)}} \tag{6.28}$$

where d is the water depth and L the wavelength is approximated by following formula to reduce the complexity in the tool (US Army Corps of Engineers, 2002):

$$L = \frac{gT^2}{2\pi}\sqrt{\tanh\left(\frac{4\pi^2}{T^2}\frac{d}{g}\right)} \tag{6.29}$$

where T is the wave period, g is the gravitational acceleration and d the water depth. The wave height at breaking can be expressed as the intersection of formulas (6.26) and (6.27):

$$H_{br} = H_0 K_s K_r \tag{6.30}$$

By combining formulas (6.22), (6.26), (6.28) and (6.30) a formula for the breaker depth d_{br} can be found:

$$\gamma_b d_{br} = H_0 \sqrt{\frac{1}{\left(1 + \frac{4\pi d_{br}/L}{\sin(4\pi d_{br}/L)}\right) \tanh(2\pi d_{br}/L)}} \sqrt{\frac{\cos \alpha_0}{\cos \alpha_1}} \tag{6.31}$$

By iteration of previous formula, the breaking water depth d_{br} can be found and with help of the bottom slope m the distance from the shoreline to the breaking zone can be found:

$$x_{br} = m\, d_{br} \tag{6.32}$$

However, this method is a rough approximation of the real distance to the breaking zone and a more detailed calculation and/or investigation should be preformed in a more detailed design phase. Therefore, if the breaking zone distance x_{br} is known, it can be entered as an input parameter in the tool and will be used in the further elaboration of the harbour configurations instead of the rough estimation.

6.2.1.3 Calculation of the longshore current by breaking waves

In paragraph 6.1.3.2 it was mentioned that the current inside the approach channel should be entered as an input parameter. However, it is possible to compute this based on the characteristics of the breaking waves if the current conditions near the harbour entrance are not known. The longshore current is given by Komar (1979):

$$V_c = 1.17\sqrt{g\, H_{br}} \sin \beta_b \ \cos \beta_b \tag{6.33}$$

where g is the gravitational acceleration, H_{br} represents the wave height on breaking and β_b the direction of propagation of the waves on the breaker line. The latter is the angle between the wave direction and the normal to the breaker line and can be computed by means of formula (6.20) based on the wave angle α_0 of the deep-water wave. If the approach channel orientation θ_{ch} is known, the longitudinal and cross component, respectively V_{lc} and V_{cc}, can be derived.

However, the longshore current is not the only sources of currents along the harbour entrance. As was mentioned in paragraph 5.2.2, the tides and wind near the harbour will also induce currents.

6.2.1.4 Calculation of the cross wind

As was mentioned in paragraph 6.1.3.2 the wind velocity U can be entered in the tool as an input parameter. If this is the case, the crosswind velocity V_{cw} can directly be calculated by means of the wind orientation θ_w and approach channel orientation θ_{ch}. The prevailing wind orientation θ_w is assumed to be equal to the wave direction θ. With means of these parameters the component of the wind speed U perpendicular to the vessels sailing axis can be computed which is the crosswind velocity V_{cw}.

6.2.2 Harbour dimensions

The harbour dimensions are determined by the dimensions of the manoeuvring areas, the length of the ship berths and the alignment of the breakwaters. The required harbour dimensions computed by the tool are shown in Figure 6.19.

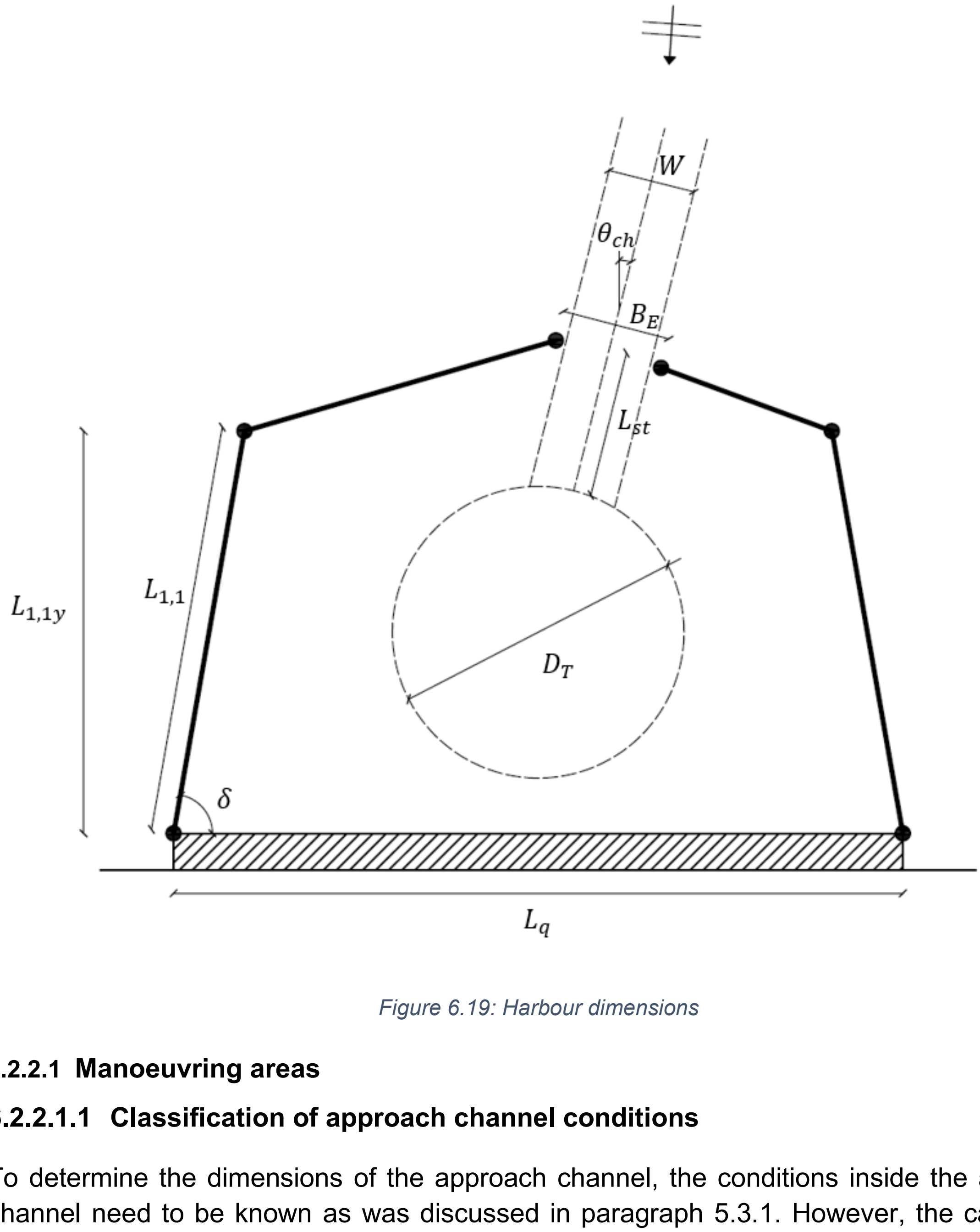


Figure 6.19: Harbour dimensions

6.2.2.1 Manoeuvring areas

6.2.2.1.1 Classification of approach channel conditions

To determine the dimensions of the approach channel, the conditions inside the approach channel need to be known as was discussed in paragraph 5.3.1. However, the calculation method differs from guideline and each classifies the approach channel conditions differently. Table 6.5 and Table 6.6 give the classification of the approach channel conditions according to PIANC (2014), Agerschou et al. (2004) and ASCE (2012). Based on these classifications the manoeuvring area dimensions are determined.

Table 6.5: Classification of conditions in approach channel according to PIANC (2014) and Agerschou, et al. (2004)

Component	Criteria	Classification
Vessel speed V_s	$V_s \geq 12\ kn$	Fast
	$8\ kn \leq V_s < 12\ kn$	Moderate
	$5\ kn \leq V_s < 8\ kn$	Slow
Prevailing cross wind V_{cw}	$V_{cw} < 15\ kn$	Mild
	$15\ kn \leq V_{cw} < 33\ kn$	Moderate
	$33\ kn \leq V_{cw} < 48\ kn$	Strong
Prevailing cross-current V_{cc}	$V_{cc} < 0.2\ kn$	Negligible
	$0.2\ kn \leq V_{cc} < 0.5\ kn$	Low
	$0.5\ kn \leq V_{cc} < 1.5\ kn$	Moderate
	$1.5\ kn \leq V_{cc} < 2.0\ kn$	Strong
Prevailing longitudinal current V_{lc}	$V_{lc} < 1.5\ kn$	Low
	$1.5\ kn \leq V_{lc} < 3\ kn$	Moderate
	$V_{lc} \geq 3\ kn$	Strong

Table 6.6: Classification of conditions in approach channel according to ASCE (2012)

Component	Criteria	Classification
Vessel speed V_s	$V_s < 8\ kn$	Mild
	$8\ kn \leq V_s < 12\ kn$	Moderate
	$12\ kn \leq V_s$	Poor
Prevailing cross wind V_{cw}	$V_{cw} < 15\ kn$	Mild
	$15\ kn \leq V_{cw} < 33\ kn$	Moderate
	$33\ kn \leq V_{cw}$	Poor
Prevailing cross-current V_{cc}	$0.2\ kn \leq V_{cc} < 0.5\ kn$	Mild
	$0.5\ kn \leq V_{cc} < 1.5\ kn$	Moderate
	$1.5\ kn \leq V_{cc}$	Poor
Prevailing longitudinal current V_{lc}	$V_{lc} < 1.5\ kn$	Mild
	$1.5\ kn \leq V_{lc} < 3\ kn$	Moderate
	$V_{lc} \geq 3\ kn$	Poor
Wave height H_s	$H_s < 1\ m$	Mild
	$1\ m \leq H_s < 3\ m$	Moderate
	$H_s > 3\ m$	Poor

6.2.2.1.2 Manoeuvring areas dimensions

In paragraph 5.3 the dimensions of the different manoeuvring areas were discussed according the existing guidelines. All these approaches were implemented in the tool. This means that for each manoeuvring area multiple dimensions will be calculated. The computed dimensions for one specific manoeuvring area are then compared in a table inside the tool, as shown in Table 6.7 for the approach channel width.

Table 6.7: Comparison of the approach channel width according to the different methods

	One-way traffic	Two-way traffic	
PIANC n° 121 - 2014			
Outer channel (open water)	47,00	100,00	[m]
Inner channel (protected water)	42,00	88,00	[m]
Ligteringen (2017)	46,00	92,00	[m]
Agershou (2004)			
Outer channel (open water)	36,00	66,00	[m]
Inner channel (protected water)	42,00	78,00	[m]
Average	42,60	84,80	[m]
Conservative	47,00	100,00	[m]
Economical	36,00	66,00	[m]

For each manoeuvring area dimension the average, maximum and minimum of the calculated values is determined as shown in Table 6.7. The user must now take a decision between the average of the values, a conservative approach or an economical approach. These calculations are performed for all dimensions. Further, the user has to decide whether the approach channel needs to be designed for one-way or two-way traffic.

Table 6.8, Table 6.9 and Table 6.10 show the comparison of the other manoeuvring areas. Here, in case a guideline provides a range of two values for the specific manoeuvring area dimension, the two boundaries are also taken into account. For the determination of the average value, the average of the two boundary values is then used.

Table 6.8: Comparison of the harbour entrance width according to the different methods

PIANC n° 121 - 2014	113,22		[m]
G. Tsinker (1997)			
Minikin (1963) and Thorensen (1988)	79,25	113,22	[m]
Quinn (1972)	120,00	150,00	[m]
Smirnov (1979)	235,92		[m]
ASCE (2012)			
Two-way traffic	115,64		[m]
Minimum	98,00	117,60	[m]
Average	133,97		[m]
Conservative	235,92		[m]
Economical	79,25		[m]

Table 6.9: Comparison of the stopping length according to the different methods

PIANC n° 121 - 2014	169,83	226,44	[m]
G. Tsinker (1997)	11,26		[m]
	396,27	905,76	[m]
C. D. Memos (2018)	226,44	339,66	[m]
H. Velsink (1994)	172,04	471,95	[m]
Average	293,09		[m]
Conservative	905,76		[m]
Economical	11,26		[m]

Table 6.10: Comparison of the turning basin diameter according to the different methods

PIANC n°121 - 2014	339,66 [m]
Ligteringen (2017)	339,66 [m]
C. D. Memos (2018)	509,49 [m]
ROM (2007)	846,15 [m]

H. Velsink (1994)	
Average	508,74 [m]
Conservative	846,15 [m]
Economical	339,66 [m]

When selecting the desired approach (average, conservative or economical), the user should pay attention as these values can differ strongly due to the large differences in the methods described by the guidelines.

The tool also computes recommendations regarding the alignment of the approach channel, as discussed in paragraph 5.3.1.1. These include the distance between curves, the curve radius and the widening in the bends. Here, it is also requested to the user to enter the deviation of the approach channel $\Delta\theta_{ch}$ from the wave direction θ.

Based on paragraph 5.3.1.3, the tool computes the required depth of the harbour basin d_{inner} and approach channel d_{outer}.

6.2.2.2 Ship berths

The minimum required quay length L_q is calculated as was explained in paragraph 5.3.5 with means of formula (5.19). The land area required for terminal operations, A_q, is also calculated taking 100 m^2 and 200 m^2 operational land per unit length of quay as extreme values into account. Further, the user has to enter the quay width B_q as an input parameter.

6.2.2.3 Breakwater dimensions and alignment

To determine the breakwater dimensions, the user has to decide whether he wants the dimensions based on the turning basin diameter D_T or on the distance of the breaking zone from the coastline x_{br}, as indicated in Figure 6.20. An additional safety zone of 1.5 L_v is added to these dimensions, as Memos (2018) suggests. The length perpendicular to the coastline of the first part of the main breakwater is then equal to:

$$L_{1,1\,y} = \begin{cases} D_T + 1,5\,L_v & if\ D_T \\ x_{br} + 1,5\,L_v & if\ x_{br} \end{cases} \tag{6.34}$$

This dimension is also shown on Figure 6.19.

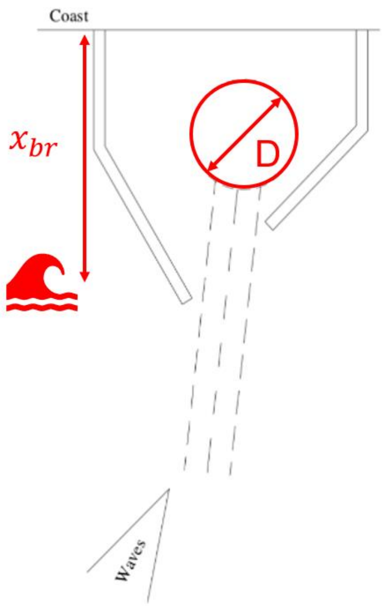


Figure 6.20: Dimensions of breakwater based on breaking zone location or turning basin, adapted from (Ligteringen, 2017)

Usually, the breakwater follows the depth contour lines of the bathymetry to reduce the depth variation along the breakwater. This is not possible in the tool, as the tool only has the slope m as an input and assumes a uniformly increase of the bathymetry and should be examined in a more detailed design phase.

To favour the sediment transport along the coastline, a smooth alignment of the breakwaters has to be provided as was explained in paragraph 5.4.4. The tool provides the possibility to enter the angle between the breakwater and the coastline δ, shown in Figure 6.21, as an input parameter.

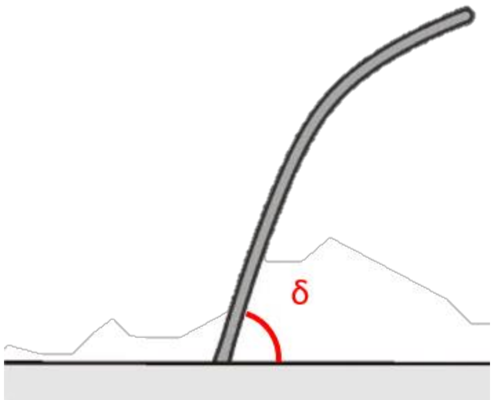


Figure 6.21: angle between breakwater and coastline, adapted from (Mangor, Drønen, Kærgaard, & Kristensen, 2017)

This angle can than be used to determine the length of the first part of the main breakwater:

$$L_{1,1} = \frac{L_{1,1y}}{\sin \delta} \quad (6.35)$$

Based on the manoeuvring area dimensions, the ship berths and the breakwater alignment, the tool will suggest an initial harbour layout. An example of an initial harbour layout is shown in Figure 6.22, including the manoeuvring areas and the dominant wave direction.

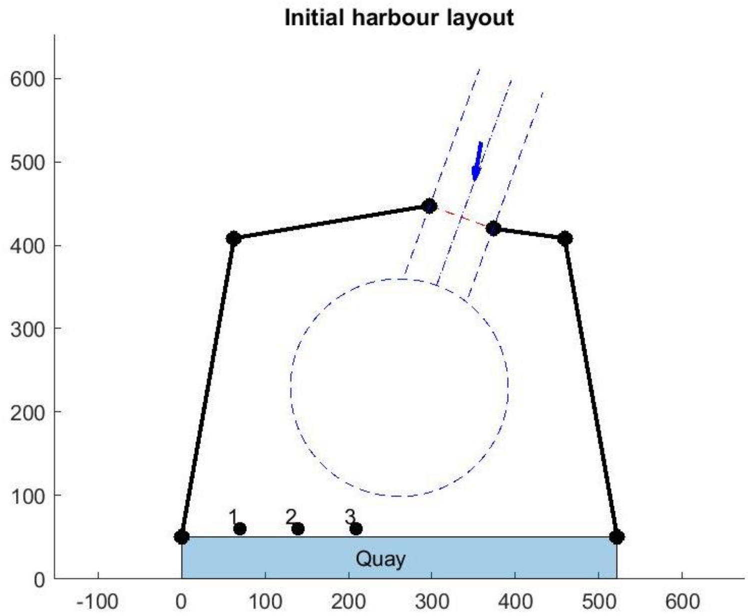


Figure 6.22: Example of an initial harbour configuration

6.2.3 Mooring conditions

The wave heights inside the harbour and at the berths have to be limited as was explained in 5.2.1.1. The tool determines the mooring and limit state conditions according to the different mentioned methods. To do this, it uses the design vessel type as an input. Some values are missing, or a range is listed in the provided tables of paragraph 5.2.1.1.

To solve these two problems, the following assumptions were made: (i) if a value for 45°-90°(beam) is missing, the value for 0° (head or stern) is assumed, (ii) in the case the wave heights are given as a range, an average of the two values is taken. Further, the volume of a ship is roughly estimated by:

$$V_v = L_v \, B_v \, T_v \quad (6.36)$$

with L_v the overall length of the vessel, B_v the beam of the vessel and T_v the draught of the vessel. For the approach provided by ASCE (2012), the tool will ask additional input parameters like the requested return period RP and the wave angle relative to the head sea.

The different mooring and limit state conditions are calculated by the tool and are put in a table, as in Table 6.11. Again, as was the case with the determination of the manoeuvring areas, the user has to decide if he wants the average of all values, the most conservative value or the most economical value to be used. When choosing an approach, the user should take a look at the computed values, because they can be quite different due to the difference in calculation methods.

Table 6.11: Comparison of the mooring conditions according to different approaches

		Limiting wave heights H_s		
Operational conditions		Forces longitudinal to quay	Forces transverse to quay	
Ligteringen (2017)	H_s	1,50	1,50	[m]
PIANC (2019)/ROM (2007)	H_s	1,50	1,00	[m]
ASCE (2012)	H_s	0,85	0,85	[m]
MLIT (2009)	H_s	0,30	0,30	[m]
Average	H_s	1,04	0,91	[m]
Conservative	H_s	0,30	0,30	[m]
Economical	H_s	1,50	1,50	[m]
Limit state conditions				
PIANC (2019)/ROM (2007)	H_s	3,00	2,00	[m]

6.2.4 Wave agitation inside harbour basin

6.2.4.1 Wave diffraction

When a wave faces a structure or obstacle, such as a breakwater, the wave crest will pivot around the side of the structure and propagate into a shadow zone. This phenomenon is called diffraction and it is shown in Figure 6.23. If the wave travels forward, there is a lateral transfer of wave energy along the crest, perpendicular to the wave movement direction. This energy transfer will take place from points with larger wave heights towards points with smaller wave heights (Goda, 2000; US Army Corps of Engineers, 2002).

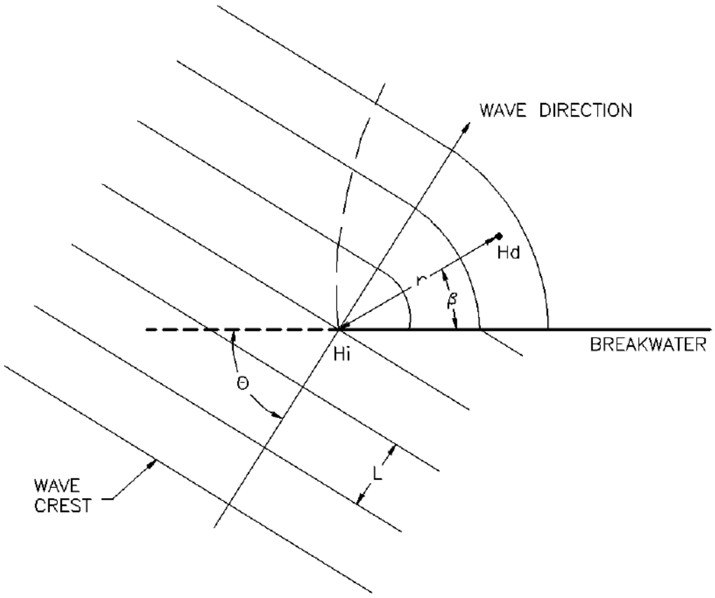


Figure 6.23: Wave diffraction around breakwater (US Army Corps of Engineers, 2002)

Diffraction will influence the wave heights which are present in the harbour. Waves travelling past the end of one or more breakwaters will undergo diffraction and the wave crest will disperse into the shadow zone in the sheltered area of the breakwaters. The wave heights and the alignment of the crests have changed considerably.

So, first the waves will undergo shoaling and refraction upon arriving at the harbour entrance, from this point the dominant wave transformation process will be diffraction. Two different cases can be distinguished: diffraction of waves passing a single breakwater (Figure 6.23) and of waves passing through a gap as in Figure 6.24.

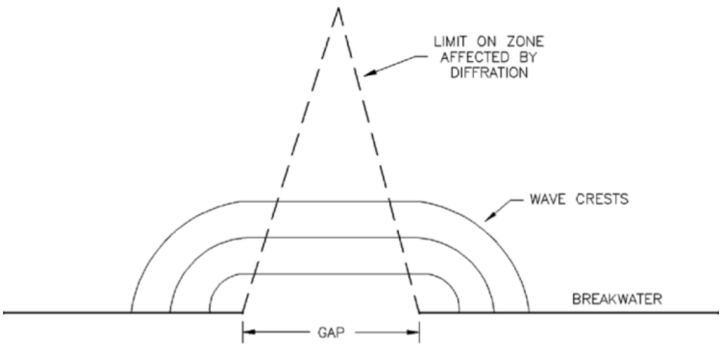


Figure 6.24: Diffraction through a gap between breakwaters (US Army Corps of Engineers, 2002)

To implement the diffraction calculation in the tool, a method proposed by Goda (2000) will be used. According to Goda (2000), diffracted wave heights should be calculated as follows:

$$(K_d)_{eff} = \left[\frac{1}{m_0}\int_0^{\infty}\int_{\theta_{min}}^{\theta_{max}} S(f,\theta)K_d^2(f,\theta)d\theta df\right]^{1/2} \tag{6.37}$$

where $(K_d)_{eff}$ is the diffraction coefficient of irregular waves, $K_d(f,\theta)$ is the diffraction coefficient of a regular wave component with frequency f and wave direction θ, $S(f,\theta)$ the frequency spectrum and m_0 the integral of the directional spectrum. One could also use diffraction diagrams generated with previous equation. It is worth mentioning that these conventional diagrams can give incorrect outcomes when used in an actual case since they are based on random waves. However, they can give a quick, first useful insight in the real situation during the preliminary design of the harbour layout. In a more detailed design phase, the diffraction phenomenon should be examined with means of numerical and physical modelling.

Goda (2000) provided several diffraction diagrams in the irregular case waves are propagating perpendicular to the breakwaters for a semi-infinite and for an opening between two breakwaters. Figure 6.25 and Figure 6.26 show respectively diffraction diagrams for a semi-infinite breakwater and for a breakwater opening with a ratio $B/L = 1.0$, both with a spreading parameter s_{max} equal to 10. In Figure 6.25 the dashed line is the period ratio and the solid line is the height ratio, or the diffraction coefficient K_d, while in Figure 6.26 the left side represents the period ratio and the right side the wave height ratio.

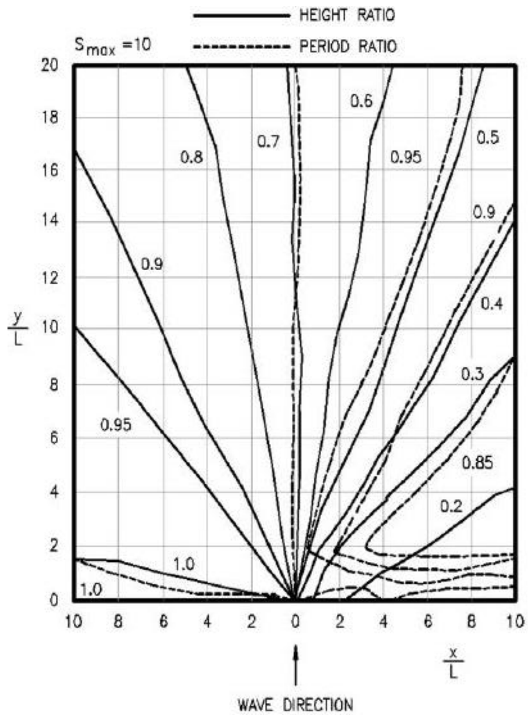


Figure 6.25: Diffraction diagram for a semi-infinite breakwater, with a spreading parameter s_{max} equal to 10 (Goda, 2000; US Army Corps of Engineers, 2002)

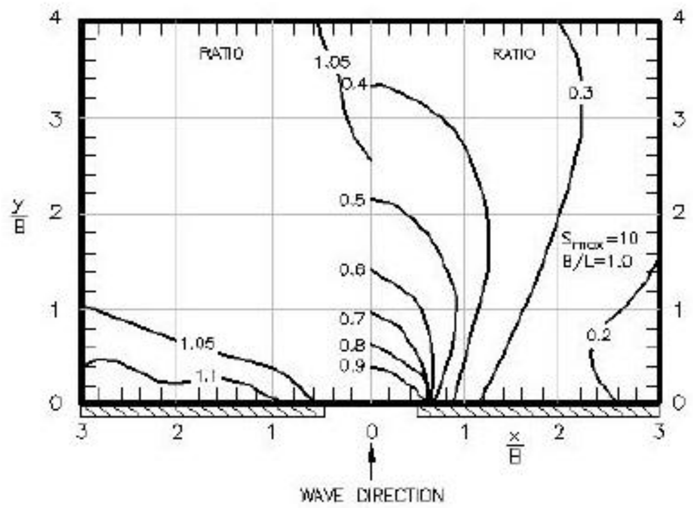


Figure 6.26: Diffraction diagram for an opening between breakwaters with a relative entrance width B/L equal to 1.0 and a spreading parameter s_{max} equal to 10 (Goda, 2000; US Army Corps of Engineers, 2002)

All diffraction diagrams for a semi-infinite breakwater and for a harbour entrance width equal to 1,2,4 and 8 times the wavelength can be found in Annex A with a spreading parameter s_{max} equal to 10 or 75. The lines of the diffraction coefficients K_d on these diffraction diagrams were read by a MATLAB script and a matrix with the x- and y- coordinate together with the K_d-coefficient was formed.

An important comment on these diffraction diagrams is that the coordinates are normalized by either the opening width B_E in case of a gap or the wavelength L in case of a semi-infinite breakwater.

Goda (2000) recommends following values for the spreading parameter s_{max} for engineering applications, in case no detailed surveys are available.

(i)	Wind waves:	s_{max} = 10,	
(ii)	Swell with short decay distance:	s_{max} = 25,	
	(with relatively large wave steepness)		(6.38)
(iii)	Swell with long decay distance:	s_{max} = 75.	
	(with relatively small wave steepness)		

In most cases, waves will approach the harbour at an angle and not perpendicular to the breakwater axis or opening axis. According to Goda (2000), this problem in the case of a semi-infinite breakwater can be solved by pivoting the axis of the breakwater in the graph, preserving the wave direction and the coordinate axes at their initial position. However, this method gives an incorrect result when the angle between the wave direction and the normal of the breakwater is larger than 45°. For diffraction through a harbour entrance, the line linking the deepest penetrated points of each contour line deviates lightly toward the normal of the breakwaters as shown in Figure 6.27. The amount of deviation depends on the wave direction, the relative harbour entrance B/L and the spreading parameter s_{max}.

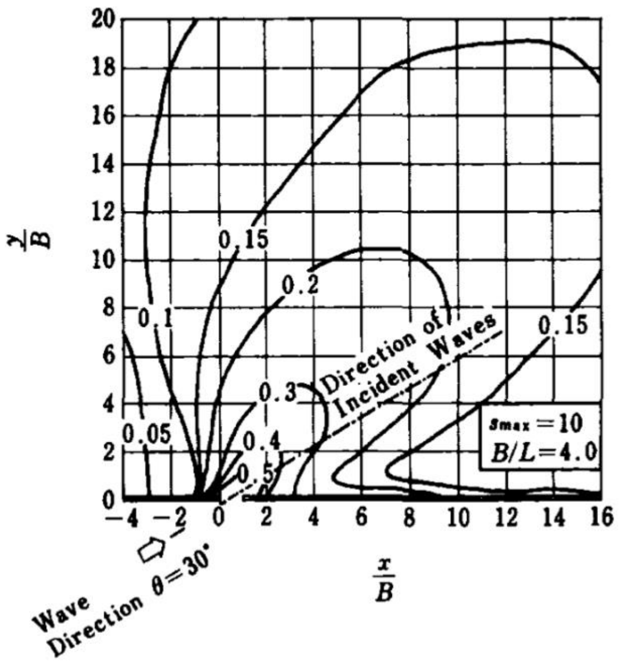


Figure 6.27: Diffraction diagram in case of oblique incidence (Goda, 2000)

Goda (2000) analysed several diffraction diagrams for different incident wave angles and computed the deviation angle for these diffracted waves, shown in Table 6.12. In the case of oblique incidence, the diffracted waves should be shifted by the deviation angle $\Delta\theta$.

Table 6.12: Deviation angle of diffracted waves through a breakwater opening for obliquely incident waves (Goda, 2000)

s_{max} [-]	B/L [-]	Deviation angle $\Delta\theta$ [°]			
		$\theta = 15°$	$\theta = 30°$	$\theta = 45°$	$\theta = 60°$
10	1.0	37	28	20	11
	2.0	31	23	17	10
	4.0	26	19	15	10
75	1.0	26	15	10	6
	2.0	21	11	7	4
	4.0	15	6	4	2

Instead of the actual width of the harbour entrance one should use the apparent width of the harbour entrance, looking from the shifted wave direction (Figure 6.28) (Goda, 2000; Rijksinstituut voor Kust en Zee, 2004; US Army Corps of Engineers, 2002).

$$B_a = B_E \cos(\theta + \Delta\theta) \tag{6.39}$$

where B_a represents the apparent width of the harbour entrance, B_E is the actual harbour entrance width, θ the wave direction and $\Delta\theta$ the deviation angle for oblique waves.

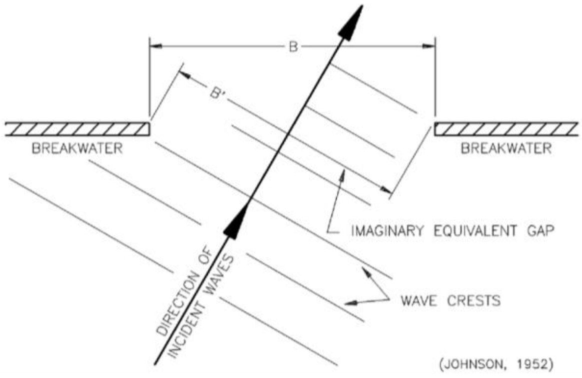


Figure 6.28: Apparent width of harbour entrance (US Army Corps of Engineers, 2002)

This apparent width is used to determine the ratio B_a/L and to select the appropriate diffraction diagram to use. By multiplying the coordinates of the diffraction diagram with the harbour entrance width or wavelength, depending on the type of diffraction diagram, the diffracted wave heights can be plotted inside the harbour layout. An example of this method will be given in the next paragraph, together with the reflection of waves.

6.2.4.2 Wave reflection

Waves which propagate towards and into a structure, such as a breakwater, will be reflected. The amount of wave energy that is reflected strongly depends on the type of structure. The amount of wave height reflected can be calculated by the reflection coefficient:

$$C_r = \frac{H_{ref}}{H_{inc}} \tag{6.40}$$

with the reflected wave height H_{ref} and the incident wave height H_{inc}. A reflection coefficient C_r equal to 1 means total wave reflection and can occur in case of vertical walls. Table 6.13 gives typical values in different situations.

Table 6.13: Values of reflection coefficients C_r (Goda, 2000)

Structural type	Reflection coefficient C_r [-]
Vertical wall with crown above water	0.7 – 1.0
Vertical wall with submerged crown	0.5 – 0.7
Slope of rubble stones (slope of 1 on 2 to 3)	0.3 – 0.6
Slope of energy dissipating concrete blocks	0.3 – 0.5
Vertical structure of energy dissipating type	0.3 – 0.8
Natural beach	0.05 – 0.2

The amount of reflected wave energy is important during the evaluation of the wave agitation inside the harbour. Too much wave reflection can induce unfavourable conditions for vessel manoeuvring and vessel operation at the berths.

As with the diffraction of waves, the problem of the distribution of reflected waves can be solved graphically, by using the so-called "mirror-image method" proposed by Goda (2000). First, one has to draw the selected diffraction diagram over the harbour layout considering the harbour entrance and the conditions of the incident waves; the spreading parameter s_{max}, the apparent harbour opening B_a and the relative harbour entrance width B_a/L. This is done as an example for the harbour layout in Figure 6.29. Now the reflection can be taken into account by mirroring the harbour layout about the reflective sides in the figure, this is shown in Figure 6.30. The amount of wave energy that is reflected by a line can be taken into account by including the reflection coefficient C_r. The resulting wave height at each point is than equal to

$$H_s = \sqrt{H_{dif}^2 + H_{ref}^2} \tag{6.41}$$

where H_s is the significant wave height at a certain point, H_{dif} the diffracted wave height and H_{ref} is the reflected wave height. This formula uses the principle of summation of energy components. In the tool, only the quay wall will be taken into account as a reflective side with a reflection coefficient C_r equal to 1.00.

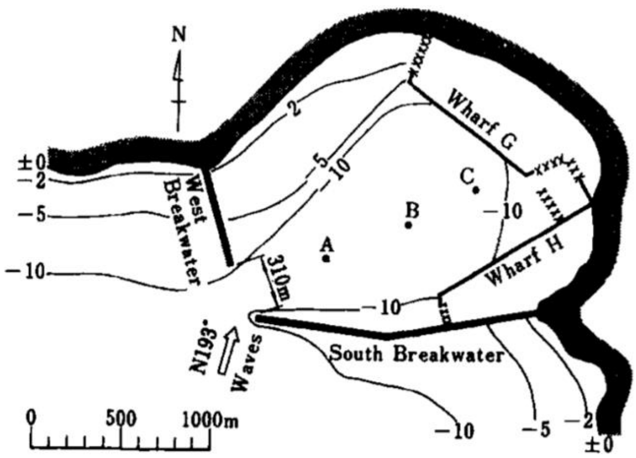


Figure 6.29: Harbour geometry for application of the mirror-image method (Goda, 2000)

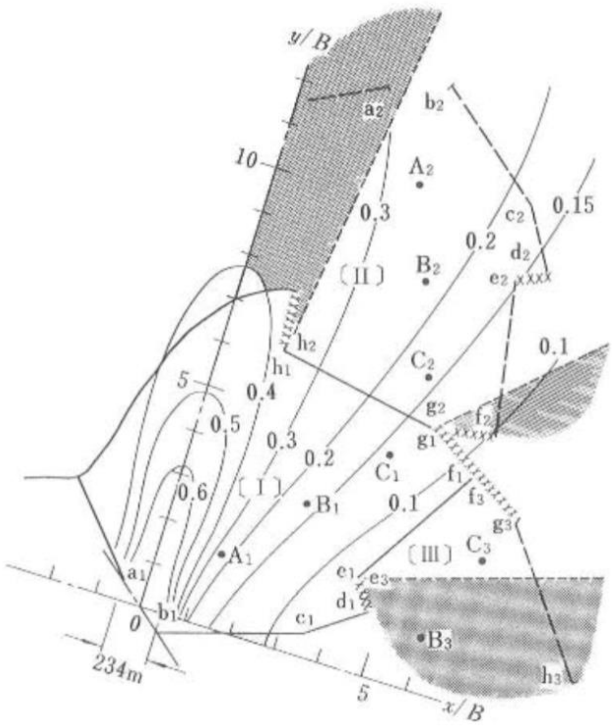


Figure 6.30: Mirror-imaged diffraction diagram applied to a particular harbour geometry (Goda, 2000)

In the tool this method is implemented as the following. First, the lines of all the diffraction diagrams were computed into matrices with the coordinates and diffraction coefficients. For a certain harbour layout, the right diffraction diagram can then be selected based on the directional spreading parameter s_{max}, the relative apparent opening B_a/L and the amount of breakwaters (semi-infinite breakwater or gap).

When the diffraction diagram is selected, the x- and y-coordinates are multiplied with either the apparent entrance width B_a, in case of a gap, or the wavelength L in case of a semi-infinite breakwater. The diffraction diagram is then rotated and shifted towards the right position in the harbour layout, so the diffraction coefficient contour lines are drawn in the harbour basin.

This means a rotation according to the wave direction and deviation angle $\theta + \Delta\theta$ and shifted towards the harbour entrance. An example is shown in Figure 6.31.

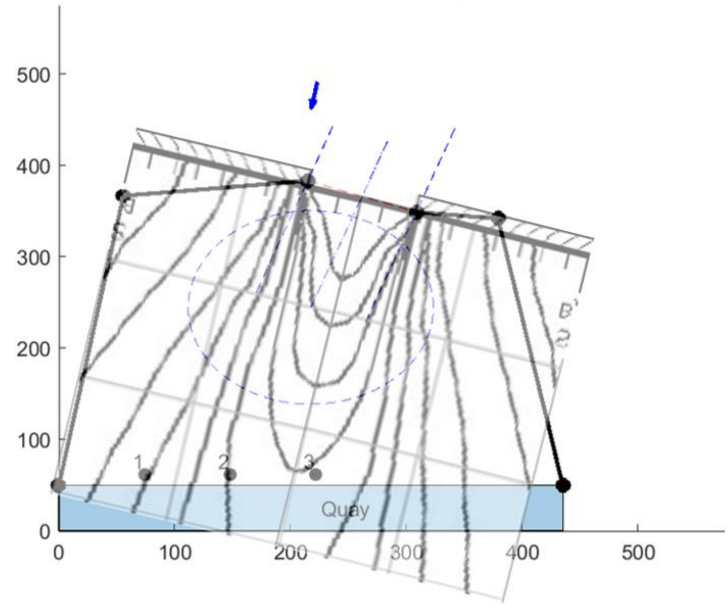


Figure 6.31: Rotated and shifted diffraction diagram

When this is done, the MATLAB script will interpolate the data between the diffraction coefficient lines, as shown in Figure 6.32.

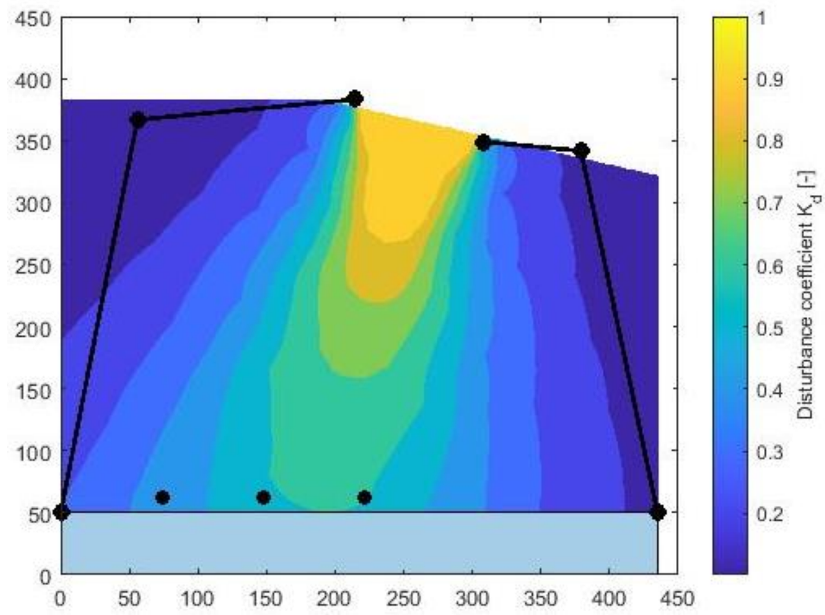


Figure 6.32: Diffraction coefficient inside the harbour

Now, for the reflection the mirror-image method is used. First, the harbour layout is mirrored about the quay wall and the diffraction diagram is drawn in the mirrored layout as shown in Figure 6.33. The reflected wave heights inside the mirrored layout and the diffracted wave heights inside the original layout are than added through the principle of summation of energy components as in formula (6.41). At last, the data points outside the harbour basin are removed.

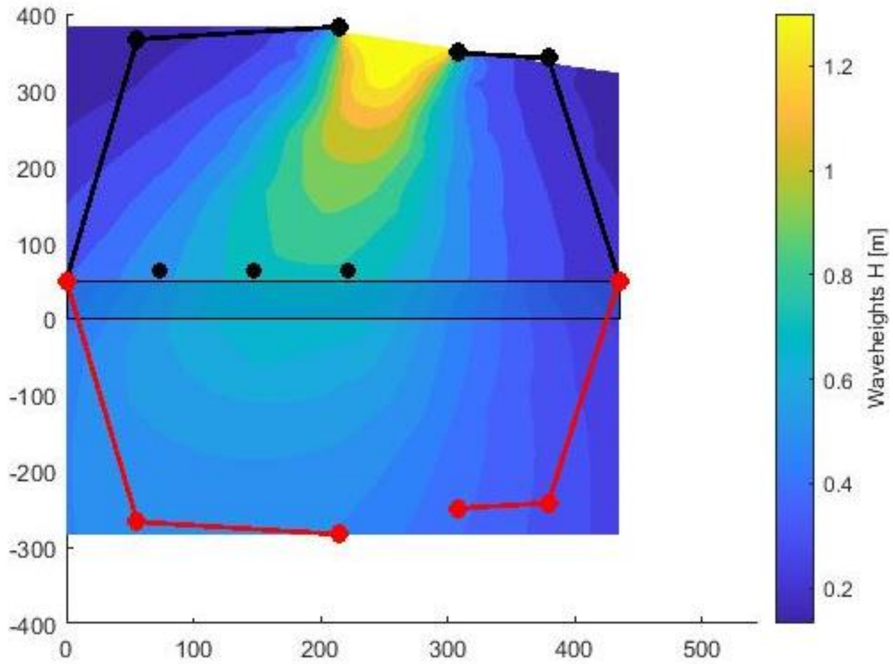


Figure 6.33: Mirrored harbour layout with diffraction lines

6.2.4.2.1 Measures to limit wave agitation

During the design of the harbour entrance and approach channel, the designer should also asses the orientation of the breakwaters relative to the main wave direction and with this the induced reflection of wave energy, shown in Figure 6.34. Wave reflection can cause severe problems for the vessel navigation inside the approach channel, the summation of the energy of the reflected and original wave height can give extreme wave conditions. This is certainly a problem in the case of vertical caisson breakwaters. During the detailed design of the harbour layout, physical and numerical models can be assessed to ensure the vessel manoeuvrability and safety and to refine the breakwater layout. Choosing for energy dissipating structures can also solve this problem, examples are gently rock slopes or perforated caissons (Goda, 2000; McBride, Smallman, & Allsop, 1996).

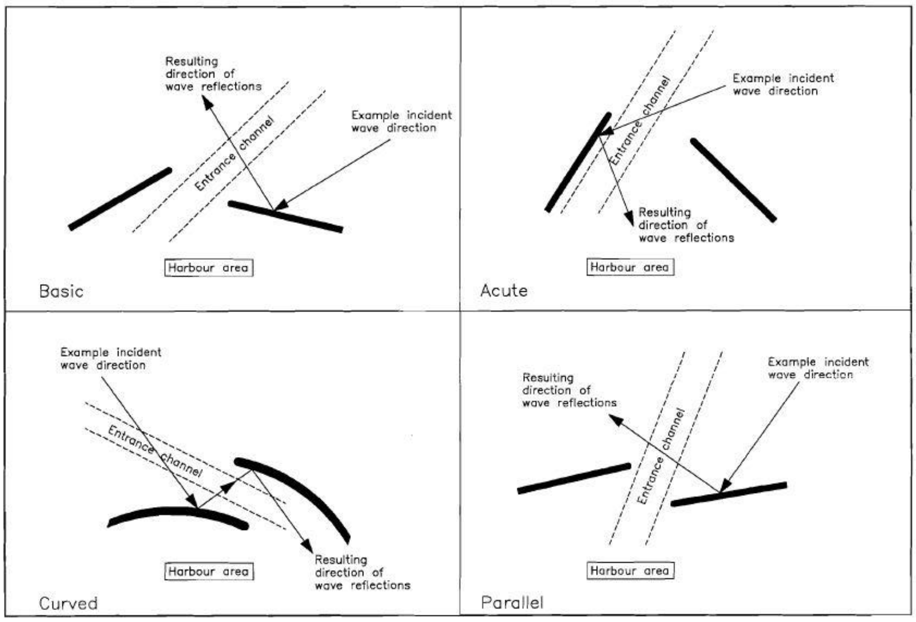


Figure 6.34: Reflection of waves due to breakwaters near the approach channel (McBride, Smallman, & Allsop, 1996)

In order to limit the wave energy reflection inside the harbour basin, the designer could opt for a layout in which the parts from which the open sea can be seen through the opening between the breakwaters, a wave dissipating structure is designed or left as a beach. This is suggested by Goda (2000) and is shown in Figure 6.35. Secondary or inner breakwaters ensure the sheltering of the harbour to this extend. The overlap of inner and outer breakwaters against the direction of the predominant wave is in this case important, as was the case with the configurations in paragraphs 4.1.1 and 4.2.1. This kind of solution is often used in marinas.

Another way to reduce reflection is to avoid rectangular shapes and design a broad interior of the harbour basin, by doing the former the penetrating and reflecting waves will disperse (Goda, 2000). An additional advantage to having a broad interior inside the harbour basin, is the possible space for future expansions.

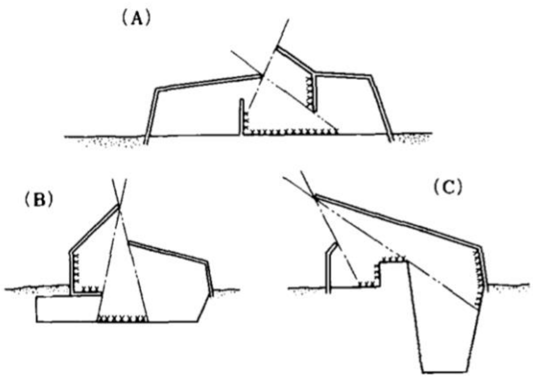


Figure 6.35: Areas facing the open sea (Goda, 2000)

6.2.4.3 Wave transmission

When waves approach and reach a breakwater, part of the wave energy will dissipate, another part will be reflected, and a last part of the wave energy may transmit past or through the breakwater. Transmission through the breakwater may be the case for a permeable rubble mound breakwater. However, most frequent occurring transmission of wave energy is through overtopping, as shown in Figure 6.36 (van der Meer, et al., 2018; US Army Corps of Engineers, 2002).

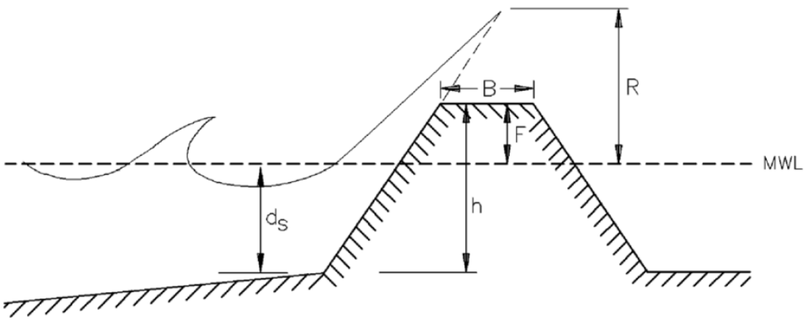


Figure 6.36: Schematic breakwater profile with overtopping (US Army Corps of Engineers, 2002)

The amount of wave energy transmitted through wave overtopping is highly related with the crest freeboard R_c. If the crest freeboard R_c is smaller than the wave run up R_u, overtopping occurs. The required crest freeboard R_c can be calculated by setting an overtopping criterion, which is the allowable overtopping discharge q. This overtopping discharge q can be entered as an input parameter in the tool. For the computation of the amount of wave energy transmitted over the breakwaters, formulas suggested by EurOtop (2018) are used.

Following general formula accounts for the design and safety assessments of a rubble mound breakwater according to EurOtop (2018):

$$\frac{q}{\sqrt{g\, H_{m0}^3}} = 0.1035 \exp\left[-\left(1.35 \frac{R_c}{H_{m0}\, \gamma_f \gamma_\beta \gamma_*}\right)^{1.35}\right] \tag{6.42}$$

where q represents the mean overtopping discharge, g the gravitational constant, H_{m0} the significant wave height from spectral analysis, R_c the crest freeboard, γ_f an influence factor for permeability and roughness, γ_β an influence factor for oblique wave attack and γ^* an influence factor for non-breaking waves for a storm wall on a slope or promenade. The influence factor for permeability and roughness γ_f for different types of armour layer can be found in Table 6.14. In the tool, an influence factor γ_f equal to 0.55 will be used for the overtopping calculations.

Table 6.14: Influence factor for permeability and roughness for different types of armour units (van der Meer, et al., 2018)

Type of armour layer	γ_f [-]
Smooth impermeable surface	1.00
Rocks (1 layer, impermeable core)	0.60
Rocks (1 layer, permeable core)	0.45
Rocks (2 layers, impermeable core)	0.55
Rocks (2 layers, permeable core)	0.40
Cubes (1 layer, flat positioning)	0.49
Cubes (2 layers, random positioning)	0.47
Antifers	0.50
HARO's	0.47
Tetrapods	0.38
Dolosse	0.43
AccropodeTM I	0.46
Xbloc®; CORE-LOC®; Accropode™ II	0.44
Cubipods one layer	0.49
Cubipods two layers	0.47

For the influence factor for oblique wave attack γ_β following formula can be used:

$$\gamma_\beta = \begin{cases} 1 - 0.0063\,|\beta| & 0° < |\beta| < 80° \\ 0.496 & |\beta| > 80° \end{cases} \tag{6.43}$$

where β is the angle of wave which is the angle between the line normal to the breakwater and the wave approach direction. For a vertical breakwater following formula can be used for the design and safety assessment (van der Meer, et al., 2018):

$$\frac{q}{\sqrt{g\,H_{m0}^3}} = 0.054\exp\left[-\left(2.12\frac{R_c}{H_{m0}}\right)^{1.3}\right] \tag{6.44}$$

As was the case with the previous mentioned wave processes, a disturbance coefficient, the transmission coefficient K_t can be found, which is the ratio between the transmitted wave height H_t and the incident wave height H_{inc} and which is related to the crest freeboard R_c found in formulas (6.42) and (6.44). EurOtop (2018) and Goda (2000) suggest using following formulas, respectively for a rubble mound breakwater and a vertical breakwater.

$$K_t = -0.4\frac{R_c}{H_{m0}} + 0.64\left(\frac{B_{crest}}{H_{m0}}\right)^{-0.31}\left(1 - \exp(-0.5\xi_{op})\right) \tag{6.45}$$

$$K_t = 0.45 - 0.3 \frac{R_c}{H_{m0}} \tag{6.46}$$

with the crest freeboard R_c, the significant wave height from spectral analysis H_{m0}, the crest width B_{crest} and the breaker parameter ξ_{op} which can be calculated as follows:

$$\xi_{op} = \frac{\tan \alpha}{\sqrt{\frac{H_{m0}}{L_{0p}}}} \tag{6.47}$$

where L_{0p} is the wavelength based on the spectral wave peak period T_p.

The transmitted wave height can then be computed as

$$H_t = K_t * H_{inc} \tag{6.48}$$

with H_{inc} the incident wave height.

6.2.4.4 Total wave agitation

The total wave height at any location can again be computed by using the principle of summation of energy components:

$$H_s = \sqrt{H_{dif}^2 + H_{ref}^2 + H_t^2} \tag{6.49}$$

with the diffracted wave height H_{dif}, the reflected wave height H_{ref} and the transmitted wave height H_t. With means of formula (6.49) the total wave agitation inside the harbour basin can be computed.

6.2.5 Preliminary cost estimation

For the preliminary cost estimation, two different components are taken into account: the dredging cost and the breakwater cost. As stated in paragraph 6.1.7 the cost to dredge per m³ is an input parameter. The required depth of the approach channel and harbour basin is calculated in the manoeuvring areas section. Based on this depth, the bathymetry slope m and the proposed harbour layout, the volume and cost to dredge can be calculated. The left side of Figure 6.37 shows a harbour configuration with the original bathymetry, after dredging of the harbour basin and approach channel, the right side is obtained. Together with the cost for the mobilisation and demobilisation of one dredger, this gives a rough estimation of the total dredging cost.

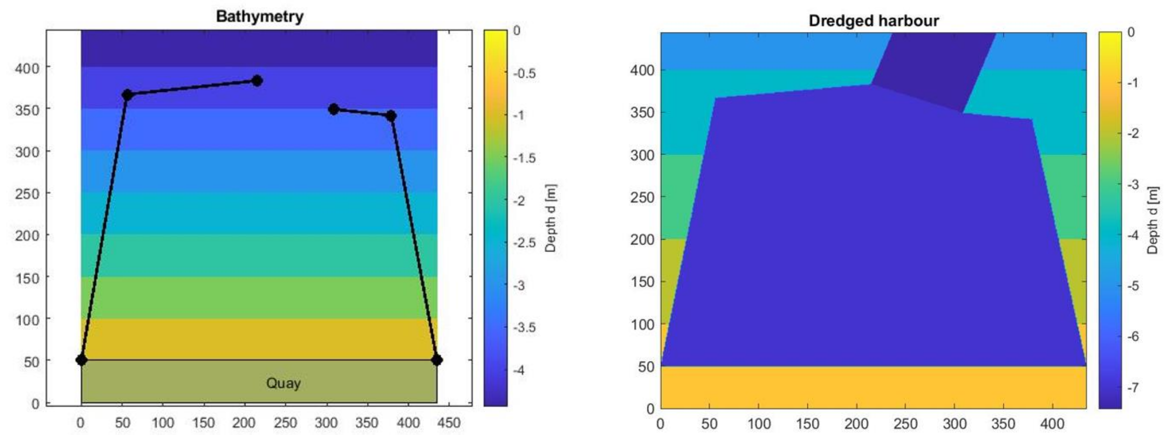


Figure 6.37: Harbour configuration with original bathymetry (left) and dredged harbour (right)

By means of the harbour layout, the total length of the breakwater structures can be calculated together with the depth along the breakwaters. Based on the input parameters which are the crest width B_{crest} and breakwater slope α, the computed crest freeboard R_c and the depth along the breakwaters, the volume of core material and the volume of armour units can be determined. Here, an armour layer thickness of 2 m is assumed, as this has not yet been specified in the breakwater design. The volume of respectively the core layer and armour layer can then be multiplied by the cost per cubic meter of the core rocks and armour units to calculate the total cost of the breakwaters. It is assumed that the geotechnical characteristics of the soil below the breakwaters are adequate and no measurements need to be taken to strengthen these. However, the geotechnical characteristics should be examined in a more detailed design phase.

The cost of the dredging and the breakwaters is then added in order to obtain the preliminary cost of the harbour layout. This will probably be an underestimation as not all costs are involved, and several assumptions are made. However, this can already give a first insight into the possible costs of the harbour layout.

6.3 Output of the tool

6.3.1 Harbour configurations

Using the computed manoeuvring area dimensions, the location of the breaking zone and the decision what the breakwater dimensions should be based on, the tool will suggest four different harbour layouts. An initial harbour layout is based on the Excel spreadsheet output. Further, two configurations with each a different approach channel orientation and a configuration with only one breakwater are generated as possible alternatives.

An example is shown in Figure 6.38. These harbour configurations are a simplified representation of the real harbour layout, with straight breakwaters stretching from the quay wall to deeper water. The tool will save every figure it creates in a separate folder.

For each of these harbour configurations, the wave agitation inside the harbour basin and the preliminary cost estimation will be computed for the same metocean conditions. With means of these parameters, the configurations will be compared through a decision matrix. These four configurations need to be optimised and eventually reduced to one final harbour configuration, during more detailed design stages.

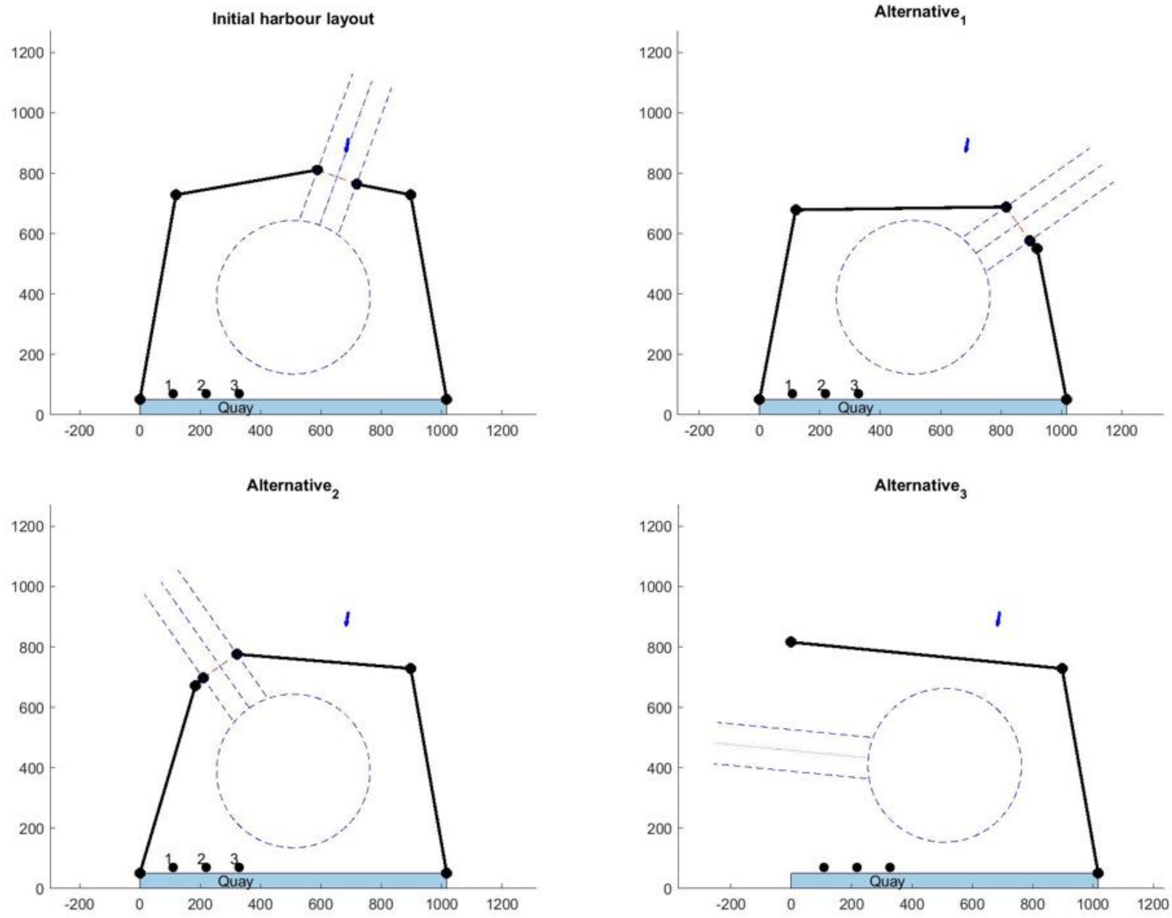


Figure 6.38: Harbour layout alternatives

6.3.2 Wave agitation in harbour basin

The tool will determine the wave agitation inside the harbour basin based on the methods for the computation of diffraction, reflection and transmission discussed in paragraph 6.2.4. For these calculations the significant wave height H_s at the harbour entrance will be used. Further, the disturbance coefficients K_d inside the harbour are derived using formula (6.7). An example of both the wave heights and disturbance coefficients inside a harbour is shown in Figure 6.39.

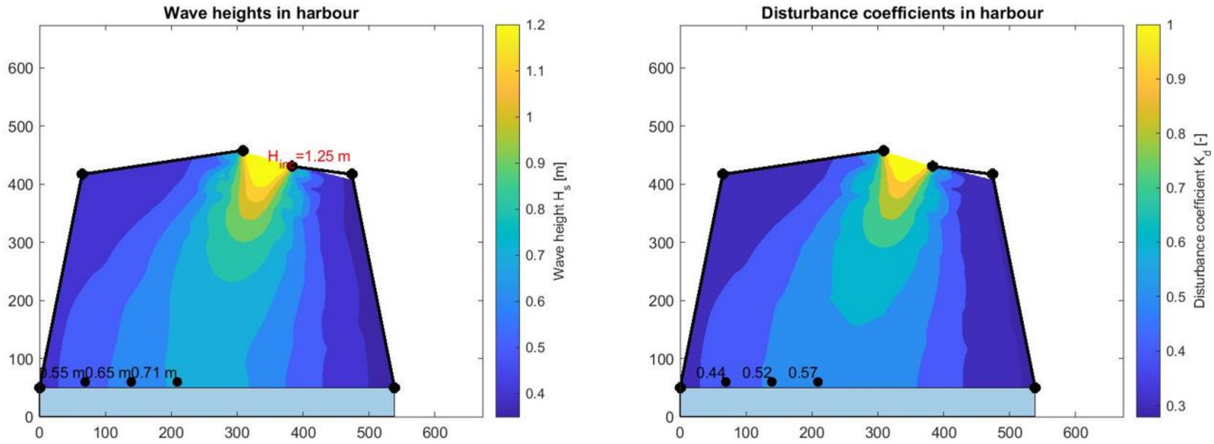


Figure 6.39: The wave heights in the harbour basin (left) and the disturbance coefficient K_d in the harbour basin (right)

6.3.3 Preliminary cost estimation

For each of the alternative harbour configuration the preliminary costs of the breakwater structures and dredging are estimated, as was mentioned in paragraph 6.2.5. It should be mentioned that this cost estimation is a very rough approximation and intended to form a first impression about the possible costs of the harbour layout and to compare the different alternatives.

6.3.4 Decision matrix

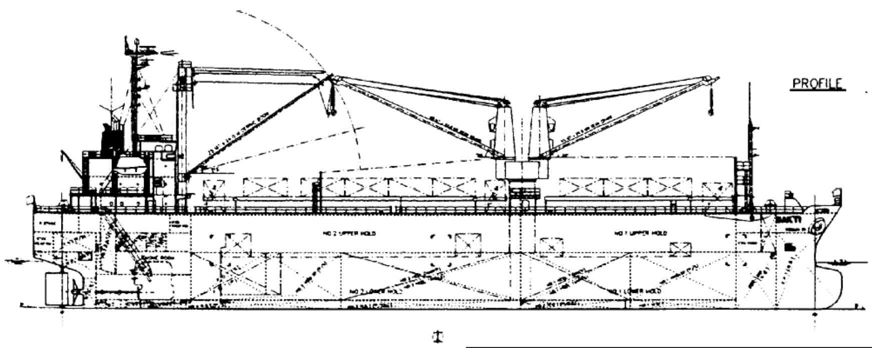
Once all computations have been completed, the results for each alternative configuration are summarised in a table. Here, the preliminary costs, the wave agitation inside the harbour and the navigational aspect are compared. The navigational aspect consists of the deviation of the approach channel from the wave direction and the harbour entrance width. The user has to allocate weights to the three criteria. Each harbour configuration then receives a ranking for each of the mentioned criteria. The ranking is then multiplied with the weights and a final score is computed. An example of the decision matrix and computations will be shown in the chapter 7.

7 EXAMPLE OF THE APPLICATION OF THE TOOL

In this chapter, an example of an application of the tool will be discussed. It demonstrates the capabilities, practicality and what can be achieved with the tool. The main purpose of the tool is to provide a handy engineer instrument that helps with the preliminary design of a harbour layout. With the help of the tool it should be possible to generate and evaluate several alternative harbour configurations in a short period of time. These alternative configurations can then be optimised in a more detailed design stage.

7.1 Input parameters

First, the necessary input parameters are entered in the Excel spreadsheet. For this example, the general cargo ship '*Sakti'* is chosen, selected from Ports and Terminals (Ligteringen, 2017). The vessel and its with the dimensions are shown in Figure 7.1. These dimensions are entered in the Excel spreadsheet. Only one rudder, no thruster and no ability to use bow and stern anchors are entered as input parameters. The vessel will sail at a speed of 4.00 kn through the approach channel and has an installed power P of 10 000 kw.



Length Over All:	113.22 m
Length Between Perpendiculars:	105.40 m
Breadth:	19.60 m
Draught:	7.29 m
Deadweight:	8,739 MT
Maximum speed:	13.30 knots

Figure 7.1: Design vessel: general cargo ship 'Sakti' (Ligteringen, 2017)

The type of harbour is "*General Cargo and Multipurpose Terminals*" and the port is situated at an open coastline. Further, 3 ship berths are assumed, the breakwater structures will not facilitate berths and there will no use of tugboats.

It is assumed only offshore wave data is available and the wave conditions shown in Table 7.1 will be used.

Table 7.1: Design wave conditions

Design wave					
	Offshore of port (Deep water wave)				
	Return period	RP	1	50	[years]
	Deep water significant wave height	$H_{s,0}$	1,00	4,00	[m]
	Peak wave period	$T_{p,0}$	8,00	12,00	[s]
	Depth	d_0	500,00	500,00	[m]
	Direction of deep water wave	θ_0	10,00	10,00	[°N]
	Wave sector		Wide wave sector	Wide wave sector	
	Wind speed	U	14,81	14,81	[m/s]
	Directional spreading parameter	s_{max}	7,51	20,70	[-]
	Deep-water wave steepness	H_0/L_0	0,010	0,018	[-]
	Directional spreading parameter	s_{max}	75,00	35,00	[-]
	Directional spreading parameter	s_{max}	7,51	20,70	[-]

The tool computes the deep-water wave steepness H_0/L_0 and the directional spreading parameter s_{max} using Figure 6.7. s_{max} is equal to 75.00 for wave conditions associated to a return period RP equal to 1 year and equal to 35 for a return period RP equal to 50 years.

For the wind and current conditions in the approach channel, the values shown in Table 7.2 are used. Here, the crosswind V_{cw} is computed with means of the wind speed U and the orientation of the approach channel θ_{ch}. It is assumed that the approach channel must deviate 10° from the wave direction, which gives an approach channel orientation θ_{ch} of 20 °N.

Further, the tool has computed an estimation of the longshore current velocity induced by breaking waves. According to the tool, the longshore current has a component V_{lc} parallel to the approach channel equal to 0.16 kn and a component V_{cc} perpendicular to the channel equal to 0.44 kn. However, the currents near the harbour entrance are not only created by the breaking waves and previous values are an underestimation. So, higher values are considered, as shown in Table 7.2.

Table 7.2: Conditions in approach channel

Input parameter	Input value
Crosswinds V_{cw}	5.00 kn
Crosscurrents V_{cc}	0.50 kn
Longitudinal currents V_{lc}	2.00 kn

According to Eq. (6.5) these conditions are favourable for manoeuvring. The Aids to Navigation (AtoN) are assumed excellent with shore traffic control. Sand is selected as bottom type, which is smooth and soft. Furthermore, a general cargo harbour has a low hazard level, light encounter traffic density and the approach channel has a sloping edge.

It is assumed that the sediment transport is not significant and two sided. For the beach slope m, a value of 100 is considered.

For the preliminary requirements of the rubble mound breakwater the input parameters in Table 7.3 are used.

Table 7.3: Breakwater requirements

Input parameter	Input value
Armour unit	Rock
Cost of armour units C_{unit}	20.00 €/m³
Cost of core rocks C_{core}	30.00 €/m³
Crest width B_{crest}	7.00 m
Slope $cotan\ \alpha$	1.50
Allowable overtopping discharge q	5.00 l/s/m

At last, the dredging cost parameters are entered as shown in Table 7.4.

Table 7.4: Dredging cost

Input parameter	Input value
Dredging cost C_{dredge}	6.00 €/m³
Cost of core rocks C_{back}	6.00 €/m³
Mobilisation of one dredger C_{mob}	1 500 000.00 €

7.2 Output of the Excel spreadsheet

Based on the input parameters, the tool will immediately give preliminary advice for the harbour layout as shown in Figure 7.2.

Port location	Open coastline
Completely artificial port	

Sediment transport	Not Significant and Two sided
Parallel attached not possible	

Metocean data	
Wave sector	Wide wave sector
Add secondary breakwater, overlap breakwaters, extend main breakwater and/or add inner breakwaters	
Main breakwater parallel to predominant wave crests	

Bathymetry	Uniform	Smooth
Try to rapidly reach and follow constant bathymetric line		
Consider using coastline parallel attached breakwater or detached breakwater		

Figure 7.2: Preliminary advice of tool regarding the harbour configuration

The Excel also performs some initial calculations. The wavelength for respectively 1-year and 50 years RP, is equal to 55.51 m and 108.10 m. The distance of the breaking zone from the coastline x_{br} is therefore computed and is approximately equal to 200 m. This value can be used in determining the breakwater alignment further on.

Next, the Excel spreadsheet computes the dimensions of the different manoeuvring areas according to the different approaches mentioned. The tool then provides the comparison of the different approaches as shown in Figure 7.3.

Comparing approach channel widths according to different approaches	One-way traffic	Two-way traffic	
PIANC n° 121 - 2014			
Outer channel (open water)	72,52	156,80	[m]
Inner channel (protected water)	66,64	141,12	[m]
Ligteringen (2017)	90,16	180,32	[m]
Agershou (2004)			
Outer channel (open water)	70,56	129,36	[m]
Inner channel (protected water)	74,48	137,20	[m]
Average	74,87	148,96	[m]
Conservative	90,16	180,32	[m]
Economical	66,64	129,36	[m]

Comparing harbour entrance widths according to different approaches			
PIANC n° 121 - 2014	113,22		[m]
G. Tsinker (1997)			
Minikin (1963) and Thorensen (1988)	79,25	113,22	[m]
Quinn (1972)	90,00	90,00	[m]
Smirnov (1979)	235,92		[m]
ASCE (2012)			
Two-way traffic	135,24		[m]
Minimum	98,00	117,60	[m]
Average	129,74		[m]
Conservative	235,92		[m]
Economical	79,25		[m]

Comparing stopping lengths according to different approaches			
PIANC n° 121 - 2014	169,83	226,44	[m]
G. Tsinker (1997)	11,26		[m]
	396,27	905,76	[m]
C. D. Memos (2018)	226,44	339,66	[m]
H. Velsink (1994)	172,04	471,95	[m]
Average	293,09		[m]
Conservative	905,76		[m]
Economical	11,26		[m]

Comparing turning basins according to different approaches			
PIANC n°121 - 2014	339,66		[m]
Ligteringen (2017)	339,66		[m]
C. D. Memos (2018)	509,49		[m]
ROM (2007)	846,15		[m]
H. Velsink (1994)			[m]
Average	508,74 [m]		
Conservative	846,15 [m]		
Economical	339,66 [m]		

Figure 7.3: Comparison of approaches for manoeuvring areas

Now, a choice has to be made if in the further design a conservative approach, economical approach or the average of the values needs to be used. Further, the user must enter whether the approach channel is one-way or two-way. In this example, the average values and a two-way channel will be used. The tool provides an overview of the manoeuvring area dimensions as shown in Figure 7.4.

Approach channel: dimensions		Average	
One-way traffic	W_{one}	75	[m]
Two-way traffic	W_{two}	149	[m]
Two-way traffic			
Harbour entrance	B_E	149	[m]
Stopping length	L_{st}	293	[m]
Turning basin diameter	D	509	[m]

Figure 7.4: Overview of manoeuvring area dimensions

The tool has also computed the required water depth and Under Keel Clearance (UKC). In the approach channel an UKC of 1.13 m and a depth d of 8.42 m is required while this is respectively 2.19 m and 9.48 m in the outer approach channel.

The next and final decisions to be made regarding the initial harbour configuration are which parameter should define the breakwater dimensions, the angle between the breakwaters and the coastline δ and the width of the quay B_q. The first can be the distance of the breaking zone from the coastline x_{br} or the turning basin diameter D_T. For this example, the turning basin diameter D_T will be used as the defining dimension. To provide a smooth transition between the coastline and breakwaters an angle δ equal to 80° will be used. Together with a safety zone of 1.5 L_v, a length of 689.04 m is acquired for the length of the first part of the main breakwater $L_{1,1}$. Further, a value of 50 m will be entered for the quay width B_q. Based on these dimensions the MATLAB script will plot later on an initial harbour layout.

The tool computes the mooring conditions according to the different approaches and shows a comparison of them, as shown in Figure 7.5. It should be noticed that the values proposed by the MLIT (2009) are much lower than the other proposed values. This is because they rely only on the size of the ship and do not look at the type of the design vessel, as was shown in Table 5.5. Further, their exist no guidelines regarding the limit state conditions for a general cargo type vessel.

Comparing mooring conditions according to different references				
		Limiting wave heights H_s		
Operational conditions		Forces longitudinal to quay	Forces transverse to quay	
Ligteringen (2017)	H_s	1,00	0,80	[m]
PIANC (2019)/ROM (2007)	H_s	1,00	0,80	[m]
ASCE (2012)	H_s	0,85	0,85	[m]
MLIT (2009)	H_s	0,50	0,50	[m]
Average	H_s	0,84	0,74	[m]
Conservative	H_s	0,50	0,50	[m]
Economical	H_s	1,00	0,85	[m]
Limit state conditions				
PIANC (2019)/ROM (2007)	H_s			[m]

Figure 7.5: Comparison of mooring conditions

As was the case with the manoeuvring area dimensions, the user has the decide which approach the tool has to follow: a conservative approach, an economical approach or the average of the values. During this example the economical approach will be used for the mooring conditions, which are equal to 1.00 m for waves longitudinal to the quay and 0.85 m for waves transverse to the quay.

At last, the Excel spreadsheet provides a summary of the parameters needed for the diffraction calculations as shown in Figure 7.6.

Input for diffraction calculation in Matlab			
	Wavelength	L	55,51 [m]
	Harbour entrance	B_E	148,96 [m]
	Ratio entrance width - wavelength	B/L	2,68 [-]
	Directional spreading parameter	s_{max}	75,00 [-]

Figure 7.6: Summary of diffraction calculation parameters

Now, the Excel spreadsheet needs to be saved and closed in order to proceed with the MATLAB script.

7.3 Output of the MATLAB script

With the output parameters of the Excel spreadsheet, the MATLAB script plots an initial harbour layout and two more alternatives by changing the approach channel's orientation and one alternative with only one breakwater. These four harbour configurations are shown in Figure 7.7.

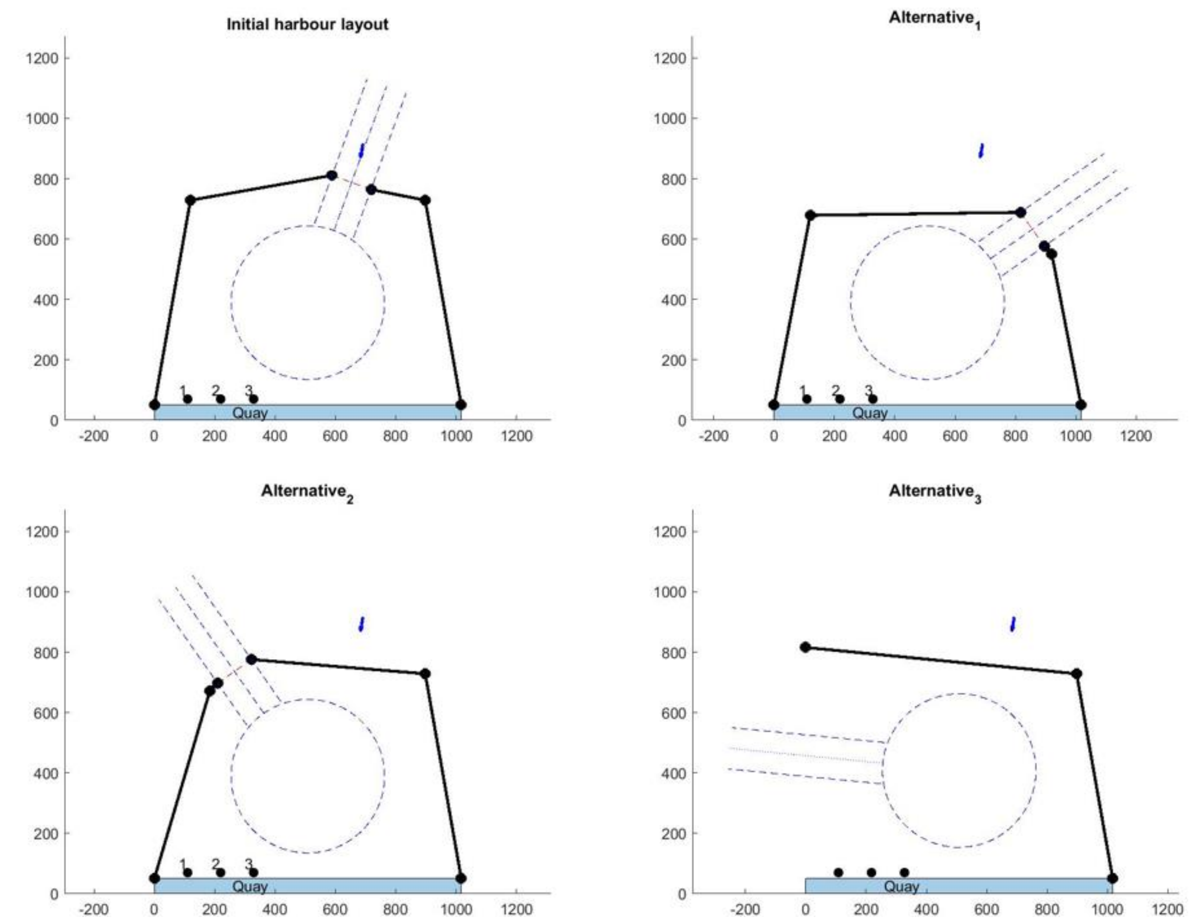


Figure 7.7: Initial harbour layout and three alternatives

Based on the wave transformation estimation methods explained in 6.2.1.2, the tool computes the significant incident wave height $H_{s,inc}$ at the harbour entrance and proceeds with the diffraction, reflection and transmission calculations. After doing this, the tool shows the wave agitation inside the harbour configurations with two figures each time: the wave heights H and the disturbance coefficients K_d inside the harbour basin. The wave agitation for the initial harbour configuration is shown in Figure 7.8. The wave agitation figures of the three other configurations can be found in Annex B.

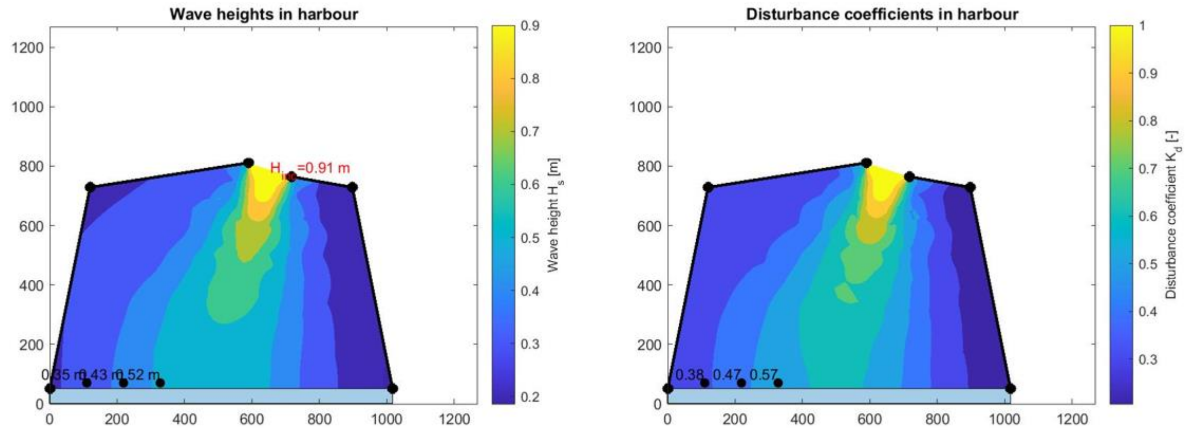


Figure 7.8: Wave agitation inside harbour basin; the wave heights in the harbour H_s (left) and disturbance coefficients in the harbour K_d (right)

These figures give already a primary insight of the wave agitation inside the harbour basin due to the diffraction, reflection and transmission of wave energy. In a more detailed design phase, the designer will assess numerical and physical models to obtain a more accurate impression of these phenomena.

Further, the MATLAB script computes the dredging needs and the breakwater cost based on the harbour layout. For this, it will use the harbour configurations and bathymetry surrounding the harbour basin. At the end, the MATLAB script sends these computations together with the precise harbour configurations and the wave agitation inside the configurations back to the Excel spreadsheet where the different configurations are then compared and evaluated.

7.4 Evaluation of the configurations in the Excel spreadsheet

In the Excel spreadsheet the four configurations are compared in three different tables, each covering one aspect as shown in Table 7.5, Table 7.6 and Table 7.7. Based on these summarizing tables the user can make some primary observations.

Table 7.5 considers the preliminary cost estimation, based on the breakwater construction cost and the dredging cost. The third alternative has the lowest breakwater cost, which makes sense because it only has one single main breakwater. Although the difference with the first alternative is not so big. The first alternative has a kind of compact layout, reducing its breakwater and dredging costs. Due to this, it will have the most optimal preliminary cost. The initial layout is more spacious and therefore more expensive.

Table 7.5: Cost comparison of harbour alternatives

Preliminary cost estimation	Cost breakwater material [€]	Dredging cost [€]	Preliminary cost [€]
Initial layout	5.457.388	18.779.162	24.236.550
Alternative 1	4.555.565	18.158.686	22.714.250
Alternative 2	5.025.366	18.168.722	23.194.089
Alternative 3	4.243.698	19.654.512	23.898.210

Table 7.6 shows the average wave agitation at the berths inside the different harbour configurations. In the initial harbour configuration, the approach channel deviates slightly from the mean wave direction, which allows a lot of wave energy to enter the harbour basin resulting in relatively large wave agitation. In the first and second alternatives, the approach channel is orientated more away from the mean wave direction. However, there is still a high amount of wave agitation at the berths in the second alternative. The problem of this alternative is the location of the ship berths. These are located directly in line with the harbour entrance, causing large wave heights at the berths. To solve this, the designer may opt to shift the ship berths more to the right of the harbour basin, where there is a larger amount of wave shelter. The main breakwater in the third alternative is almost perpendicular to the main wave direction and extending far enough to reduce the wave agitation significantly.

Table 7.6: Wave agitation comparison of harbour alternatives

Wave agitation	Wave agitation at berths (Average) [m]	Wave disturbance coefficient at berths [-]
Initial layout	0,43	0,47
Alternative 1	0,25	0,28
Alternative 2	0,52	0,58
Alternative 3	0,24	0,26

Finally, the navigational aspect of the four harbour configurations are assessed as shown in Table 7.7. For this aspect, the deviation of the approach channel from the main wave direction and the harbour entrance width are considered. In this example, the tool has computed the first three configurations with the same entrance width, so no comparison can be made there. However, the orientation of the approach channel in the configurations does differ. In the initial layout, the deviation is only 10° which favours the manoeuvring of the vessels when entering the harbour basin. The deviation of both the first and second alternative is 45° which already makes it more difficult to enter the harbour basin. In the last alternative, the deviation is more than 90°, which means the waves are coming in “beam” of the vessels when entering the harbour, causing unfavourable manoeuvring conditions.

Table 7.7: Navigation comparison of harbour alternatives

Navigational aspect	Deviation approach channel of wave direction $\Delta\theta_{ch}$ [°]	Entrance width B [m]
Initial layout	10,00	148,96
Alternative 1	45,00	148,96
Alternative 2	45,00	148,96
Alternative 3	94,45	-

The four configurations can now be evaluated for each of the aspects discussed above. To do this, the user first has to allocate weights to the three criteria: cost, wave agitation and navigation. After doing this, the tool will rank the alternatives and give them scores for each of the criteria, as shown in Figure 7.9. These scores are then multiplied with the criteria weights to give a final score to each of the four configurations. This way the tool can indicate which configuration scores best for each aspect and which is the most optimal configuration.

For this example, the cost aspect has a weight equal to 5, the wave agitation a weight equal to 3 and the navigational aspect has a weight equal to 1.

	COST	WAVE AGITATION	NAVIGATION	Final score
Initial layout	1	3	7	7,00
Alternative 1	7	4	3	16,67
Alternative 2	4	1	3	8,67
Alternative 3	3	7	1	12,33
Criteria weight	5	3	1	

Figure 7.9: Decision matrix

These scores can be graphically shown, as shown in Figure 7.10, to give a clear and quick overview of the total scores.

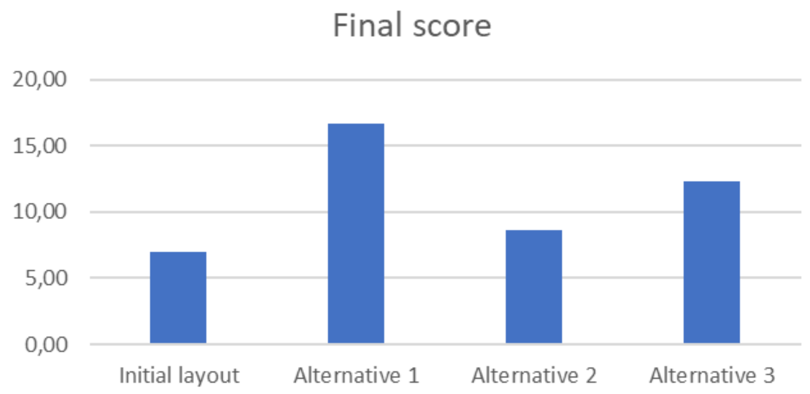


Figure 7.10: Final score graph

According to the evaluation performed by the tool, the first alternative is the most optimal one, considering the different criteria weights and scores.

The scores of the different alternatives for the three aspects can also be put on a radar chart as shown in Figure 7.11. By each spoke, one of the three aspects is represented. The length of the spoke is proportional to the magnitude of the scores. Then, a line is drawn for each alternative connecting the scores for each aspect. This graph can be used to see which alternatives are similar and which ones excel. However, in this chart the weights of the three criteria are not taken into account.

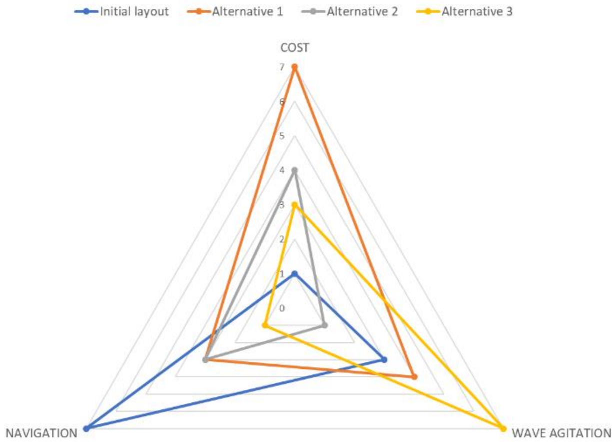


Figure 7.11: Radar chart of aspects

7.5 Layout optimization

After the tool has computed the wave agitation and the preliminary cost of four alternative harbour configurations, it will request which of these four configurations the user wants to be used for a further sensitivity geometrical analysis.

In this analysis the tool investigates the following:

- the effect of a variation of the wave direction on the wave agitation;
- the effect of a variation of the geometry for the same incident wave condition, including:
    - rotating the approach channel;
    - rotating the main breakwater;
    - extending the main breakwater;
    - widening the approach channel and harbour entrance width.

For the variation of the geometry, the effect on the preliminary cost, wave agitation and navigational aspect is examined.

7.5.1 Effect of different wave directions

For the first analysis, the wave direction is changed from 10 °N to 35 °N, an increase of 25°. Due to this variation in direction, the incident significant wave height H_s will change and reduce to 0.8 m.

Figure 7.12 shows the wave agitation for a wave direction of 10 °N and a wave direction of 35 °N. Due to the variation in wave direction, the wave penetration is more orientated towards the ship berths which means the disturbance coefficients K_d at the berths will increase. However, the incident wave height H_s is reduced, from 0.91 m to 0.80 m, due to which the wave heights at the berths will not significantly change. The increased disturbance coefficient K_d at the berths indicates that significantly higher wave agitation will be present in case more severe storm events are coming from this wave direction.

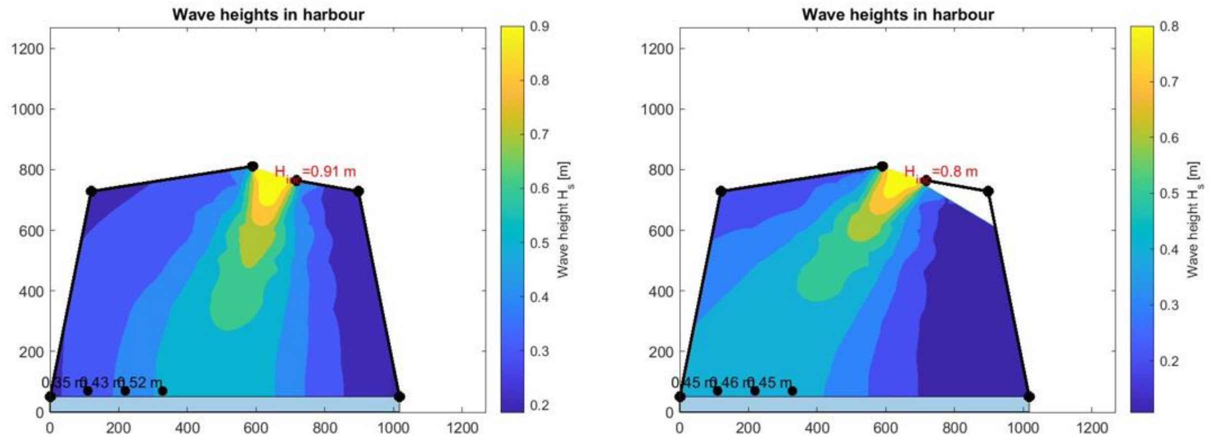


Figure 7.12: Wave heights for a wave direction of 10 °N (left) and wave heights for a wave direction of 35 °N (right)

Table 7.8 shows the analysis of the variation of wind/wave direction, which only includes the wave agitation as the preliminary costs remains the same. The navigation of the vessels will be slightly worse, because the waves are coming less "aft" of the vessel.

It can be seen that the wave heights at the berths is slightly reduced, from 0.47 m to 0.45 m, a decrease of 4.23%. However, the wave disturbance coefficients at the berths is increased, 0.43 to 0.57, which is already a considerable increment.

Table 7.8: Analysis of a variation of wind direction

	Wave agitation at berths (Average) [m]	Wave disturbance coefficient at berths [-]
Original	0,47	0,43
Changed wind direction	0,45	0,57
	-4,23%	+31,54%

7.5.2 Geometrical optimization

Further, the geometric optimization is examined by the tool. As a first geometric modification, the approach channel is rotated 20° more clockwise as shown in Figure 7.13. Due to this modification the deviation angle between the approach channel and the main wave direction $\Delta\theta_{ch}$ is increased to 30°.

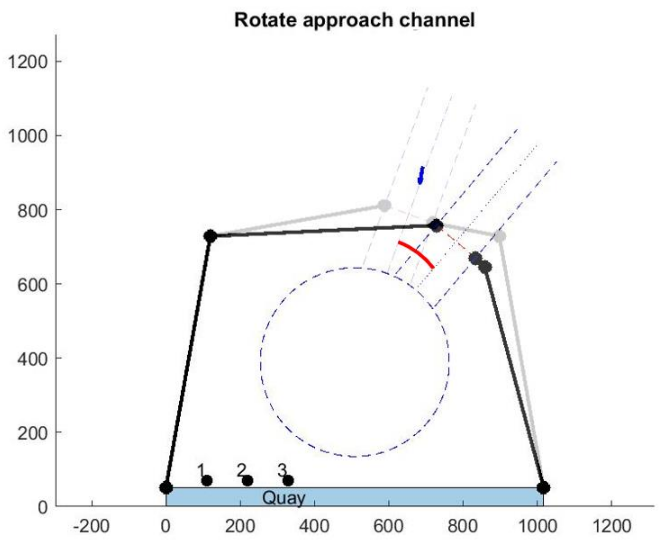


Figure 7.13: Rotated approach channel

The wave agitation in the original and modified configuration are shown in Figure 7.14.

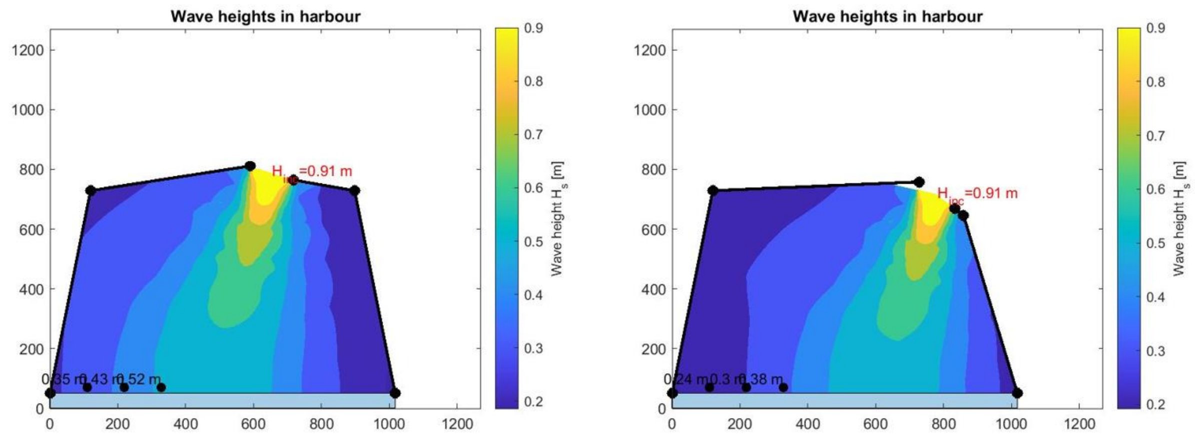


Figure 7.14: Wave agitation in original configuration (left) and configuration with rotated approach channel (right)

As can be seen in the figure, the wave agitation at the berths is slightly reduced in the modified configuration, because the approach channel orientation deviates more from the wave direction, reducing the apparent harbour entrance width B_a. This will result in less wave energy penetrating the harbour basin. In addition, the harbour entrance, and thus the wave penetration, is located more to the right side of the harbour basin, causing less wave energy to reach the berths. However, the increase in deviation angle $\Delta\theta_{ch}$ will lead to waves coming in quartering or beam of the vessel and worse manoeuvring conditions. Looking at the modified harbour layout, one can notice that the modified configuration is more compact, reducing the breakwater and dredging costs.

Secondly, the main breakwater is rotated over 10° and is shown in Figure 7.15.

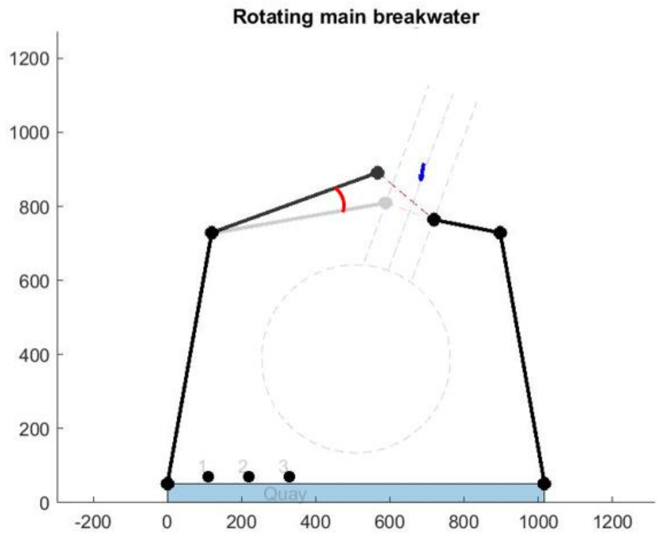


Figure 7.15: Rotated main breakwater

The wave agitation of this modified configuration, for the same incident wave condition, is shown in Figure 7.16. Due to rotation of the main breakwater, the harbour entrance, and thus the apparent harbour entrance width B_a, will enlarge which will lead to a higher penetration of wave energy to the inside the harbour basin. As can be seen, the wave heights at the berths have increased slightly. Although increasing the harbour entrance width will ease the manoeuvring of the vessels upon entering the harbour, rotating the main breakwater towards deeper water will increase the depth along the breakwater and increase the breakwater cost. Additionally, the harbour basin is enlarged, increasing the dredging costs slightly.

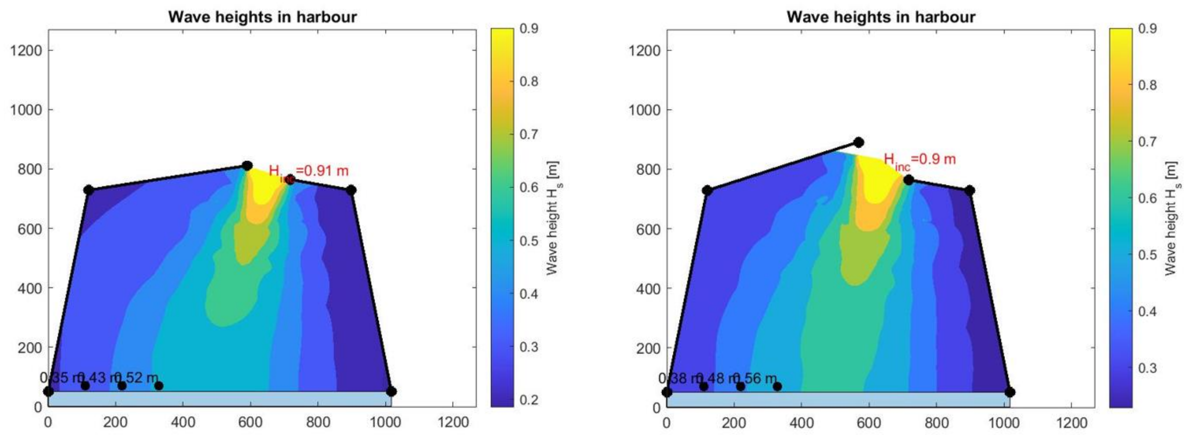


Figure 7.16: Wave agitation in original configuration (left) and configuration with rotated main breakwater (right)

Next, the main breakwater is extended by 20% of the length of the second part, namely 95 m. The extended main breakwater is shown in Figure 7.17.

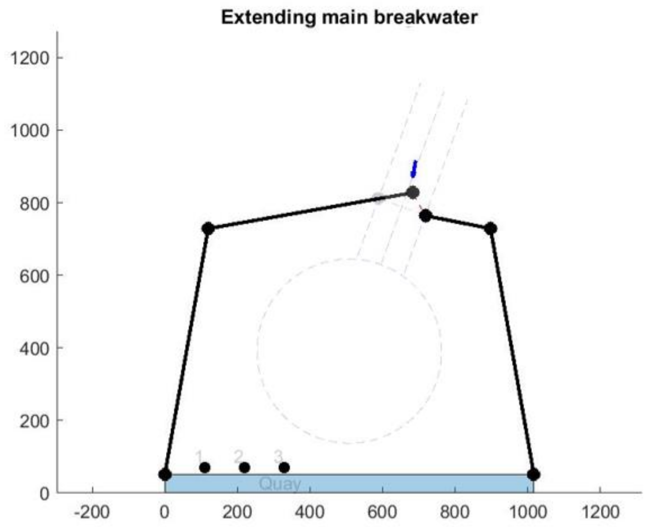


Figure 7.17: Extended main breakwater

By extending the main breakwater, the harbour entrance is narrowed, and less wave energy can enter the harbour basin as shown in Figure 7.18. The wave heights at the berths will reduce to almost half of the original wave heights. However, this narrowed harbour entrance, which is less than half the original width, makes it impossible for vessels to manoeuvre safely when entering the harbour. The risk of collision with the breakwater heads will increase drastically. It is worth underlying that extending the main breakwater will increase the total structure costs.

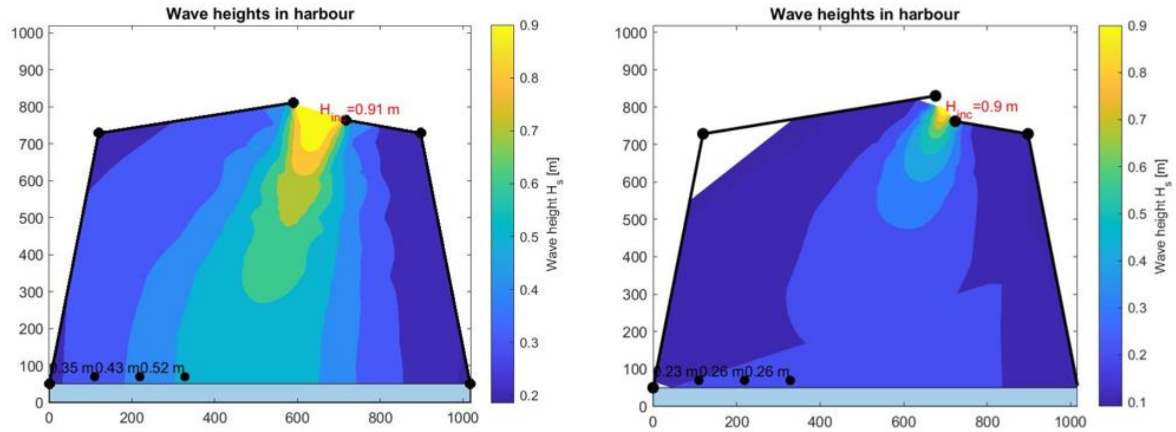


Figure 7.18: Wave agitation in original configuration (left) and configuration with extended main breakwater (right)

Finally, the approach channel and harbour entrance width are widened by 25% compared to the original configuration, which gives a width of 185 m. This modification is shown in Figure 7.19.

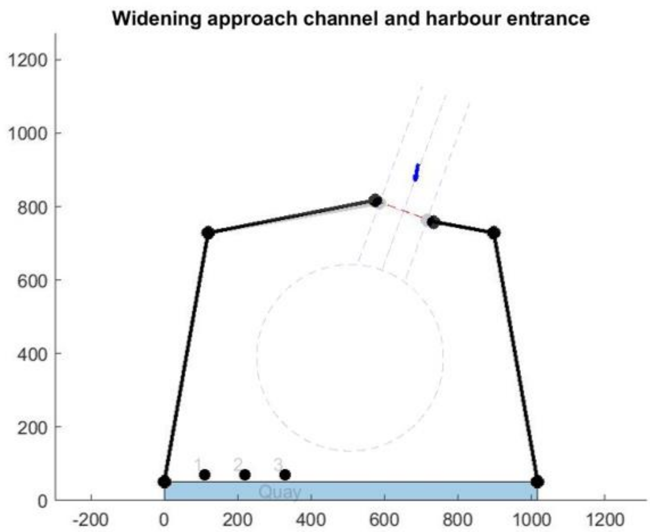


Figure 7.19: Widened approach channel and harbour entrance

By widening the harbour entrance, more wave energy can enter the harbour basin, increasing the wave agitation as shown in Figure 7.20. The wave heights will be slightly higher in the modified configuration. Although widening the harbour entrance and approach channel will favour the manoeuvring of the entering vessels and slightly reduce the breakwater costs as less material is needed.

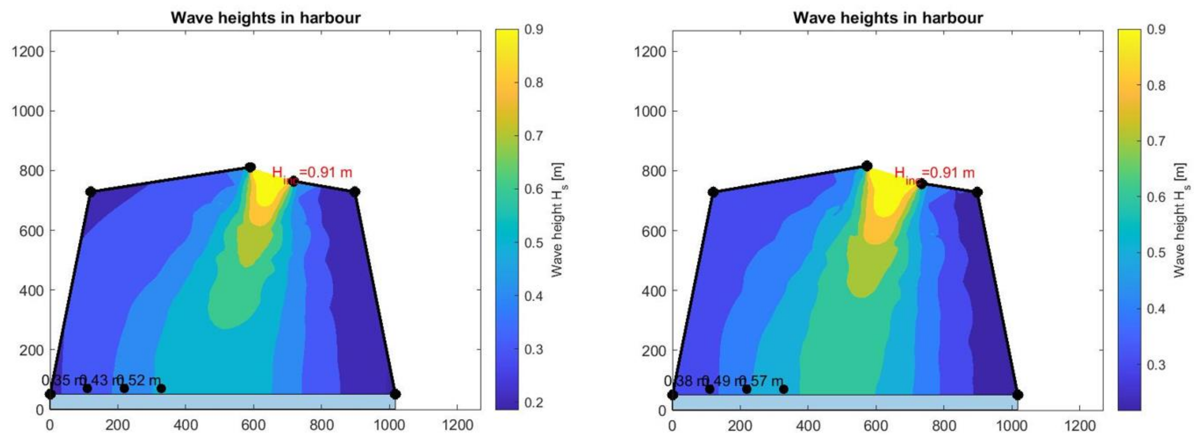


Figure 7.20: Wave agitation in original configuration (left) and configuration with widened approach channel (right)

The tool summarizes the geometric analysis in three different tables, one for each aspect as displayed in Table 7.9, Table 7.10 and Table 7.11. These tables give a clear overview of how the different geometric modifications affect the costs, the wave agitation and the navigational aspect.

Table 7.9 considers the effects of the geometrical variations on the preliminary costs. Here can be seen that rotating the approach channel, which in this case made the harbour basin more compact, will have the largest reduction of the breakwater costs and the total preliminary costs. Extending the main breakwater will of course lead to the biggest cost increase

Table 7.9: Geometric analysis of preliminary cost aspect

	Cost breakwater material [€]	Dredging cost [€]	Preliminary cost [€]
Original	5.457.388	18.779.162	24.236.550
Rotating approach channel	5.031.516	18.231.262	23.262.778
	-7,80%	-2,92%	-4,02%
Rotating main breakwater	5.647.138	18.824.735	24.471.873
	+3,48%	+0,24%	+0,97%
Extending main breakwater	5.939.620	18.785.202	24.724.822
	+8,84%	+0,03%	+2,01%
Widening approach channel	5.374.350	18.782.788	24.157.138
	-1,52%	+0,02%	-0,33%

Next, the tool considers the effects of the geometrical variations on the wave agitation inside the harbour basin, as shown in Table 7.10. As was mentioned above, extending the main breakwater, and thus narrowing the entrance, will significantly reduce the wave agitation at the ship berths. Widening the entrance and rotating the main breakwater will increase the wave agitation slightly and rotating the approach channel more away from the main wave direction will reduce it.

Table 7.10: Geometric analysis of wave agitation aspect

	Wave agitation at berths (Average) [m]	Wave disturbance coefficient at berths [-]
Original	0,47	0,43
Rotating approach channel	0,31	0,34
	-35,21%	-22,31%
Rotating main breakwater	0,48	0,52
	+0,61%	+20,77%
Extending main breakwater	0,25	0,28
	-47,18%	-36,15%
Widening approach channel	0,48	0,53
	+1,41%	+22,31%

Finally, the effects on the navigational aspect is considered in Table 7.11. From this point of view, widening the harbour entrance is very beneficial as this does not affect the approach channel orientation and favours the vessel manoeuvring by making it easier to enter the harbour basin. Taking in account the low increase in wave agitation, this optimization is definitely worth considering during the design of the harbour layout.

Furthermore, rotating the approach channel, rotating the main breakwater and extending the main breakwater affect the navigation of vessels negatively. Although rotating the main breakwater will widen the harbour entrance slightly.

Table 7.11: Geometric analysis of navigational aspect

	Deviation approach channel of wave direction $\Delta\theta_{ch}$ [°]	Entrance width B [m]
Original	10,00	148,96
Rotating approach channel	30 +200,00%	137,20 -7,89%
Rotating main breakwater	30 +201,93%	196,78 +32,10%
Extending main breakwater	51 +411,13%	72,45 -51,36%
Widening approach channel	10 0,00%	171,50 +15,13%

7.6 Conclusion

The aim of this chapter was to show how handy the tool could be for port and harbour engineers during the preliminary design of the harbour layout. As shown, the tool can generate and evaluate several alternative harbour configurations in a relative short period of time. The tool can provide a rough estimation of the wave agitation inside the harbour basin and the preliminary costs of the harbour. It should be mentioned that these computations are a rough approximation of the real situation, however they can already give a primary and quick insight in the design of the harbour layout.

Further, the tool performs a quick geometric analysis of the variation of wave conditions and geometric features. This geometric analysis can be used to investigate the effect of several modifications on the preliminary costs, the navigation and the wave agitation.

8 CONCLUSIONS

8.1 General conclusions

The preliminary design process of a harbour layout is a complex practice due to the complicated conditions and various design aspects involved. Due to the numerous guidelines and recommendations found in literature, there exist not one consistent methodology which can be followed in a general preliminary port planning process.

When looking at existing harbours, one can distinguish some typical breakwater layouts: converging breakwaters with or without inner breakwaters, a coastline parallel attached breakwater with or without a secondary breakwater, a detached breakwater and a river harbour. These harbour layouts each have their own benefits and possible applications.

Port location, metocean data, navigation, morphological aspects, bathymetry, geotechnics and preliminary cost estimation are all design aspects the port or coastal engineer needs to consider during the initial design phase. These design aspects have their own sub aspects and some aspects require more attention than others.

These design aspects were used to develop a methodology for the preliminary design of a harbour layout. By following this methodology in a general case, multiple harbour configurations can be found. This methodology was implemented in a handy engineering tool. The tool could be used during the primary design stages and will propose several harbour configurations in a short time span. After proposing the configurations, it evaluates and compares them based on three main aspects: the preliminary cost, the wave agitation inside the harbour and the navigation aspects.

During the development of the tool some problems were encountered. Several times a choice had to be made, which method was going to be implemented in the tool, for example with the estimation of each wave transformation, the dimensions of the manoeuvring areas, the mooring conditions, the evaluation of the different configurations, etc.. Some calculations were also too complex to implement in a simple engineering tool, such as the quantification of the sediment transport and the mooring conditions in terms of vessel movements. During the computation of the wave agitation inside large harbour basins a problem arose with the diffraction and reflection calculations. Due to the limited data provided by the diffraction diagrams for large distances, the diffraction lines can not be completely drawn inside the harbour basin and the total wave heights at some locations can not be computed. Also, the computation of the wave agitation is at some locations unclear due to the interpolation of the MATLAB script.

However, the tool is intended to be used in a preliminary design phase and can already give a good view of the possible wave heights in the harbour basin of different candidate harbour layouts.

8.2 Future work

The work conducted by this master thesis presents some ideas regarding the future of the preliminary harbour design process. Assumptions were made for some of the design aspects, such as the geotechnical characteristics of the soil below the breakwater structures and the design water level. These design aspects could be examined more in detail.

The tool could possible be improved on certain aspects. Instead of a simple bathymetry, a more detailed bathymetry could be used as an input. However, this would require few modifications in the tool as the bathymetry is computed as a matrix inside the tool. In the case of a more detailed bathymetry, it needs to be converted to a matrix and imported into the tool.

The calculation of the diffraction, reflection, transmission and other wave processes could be more detailed. However, this will increase the complexity and the computation time of the tool. At the moment, the tool gives a fast but preliminary view of the different processes which could be used for the preliminary design. In a more detailed design stage, the designer should make use of numerical and physical modelling of the harbour layout to acquire more detailed insight in the design.

Further, no calculations are made for the quantification of the sediment transport by the tool due to its complexity. However, to acquire a better overview and evaluation of the configurations this can be of great use during the preliminary design of the harbour layout. The tool could be also improved to use more detailed data regarding tides to compute a detailed design water level and the flow fields around the harbour layout.

Although upgrading the tool with the previously mentioned improvements, it might drastically increase the complexity and computation time of the tool, which can reduce the comfort and usability of the tool in the preliminary design phase.

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ANNEX A

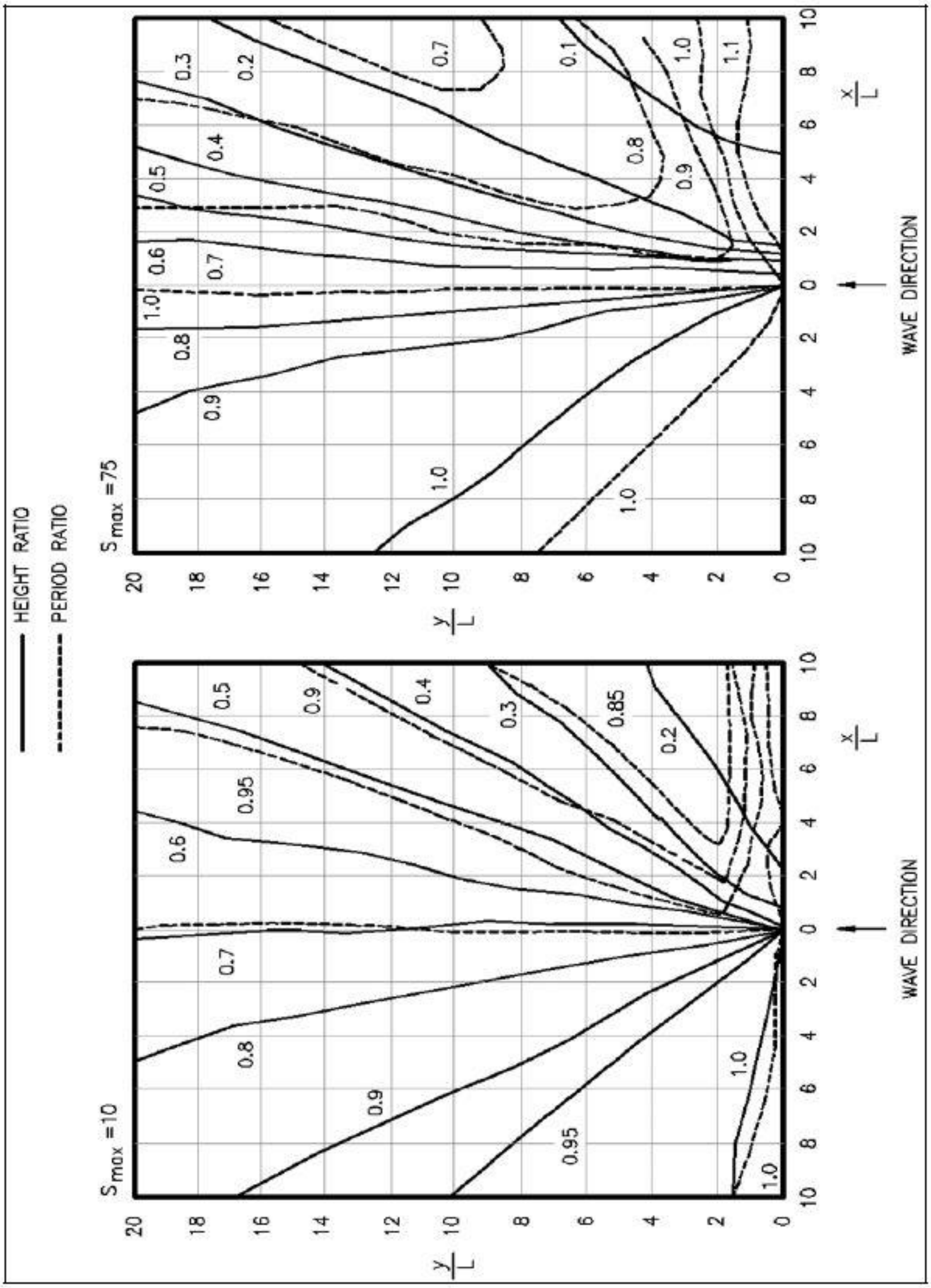

Figure A. 1: Diffraction diagrams of a semi-infinite breakwater for normal incidence (Goda, 2000; US Army Corps of Engineers, 2002)

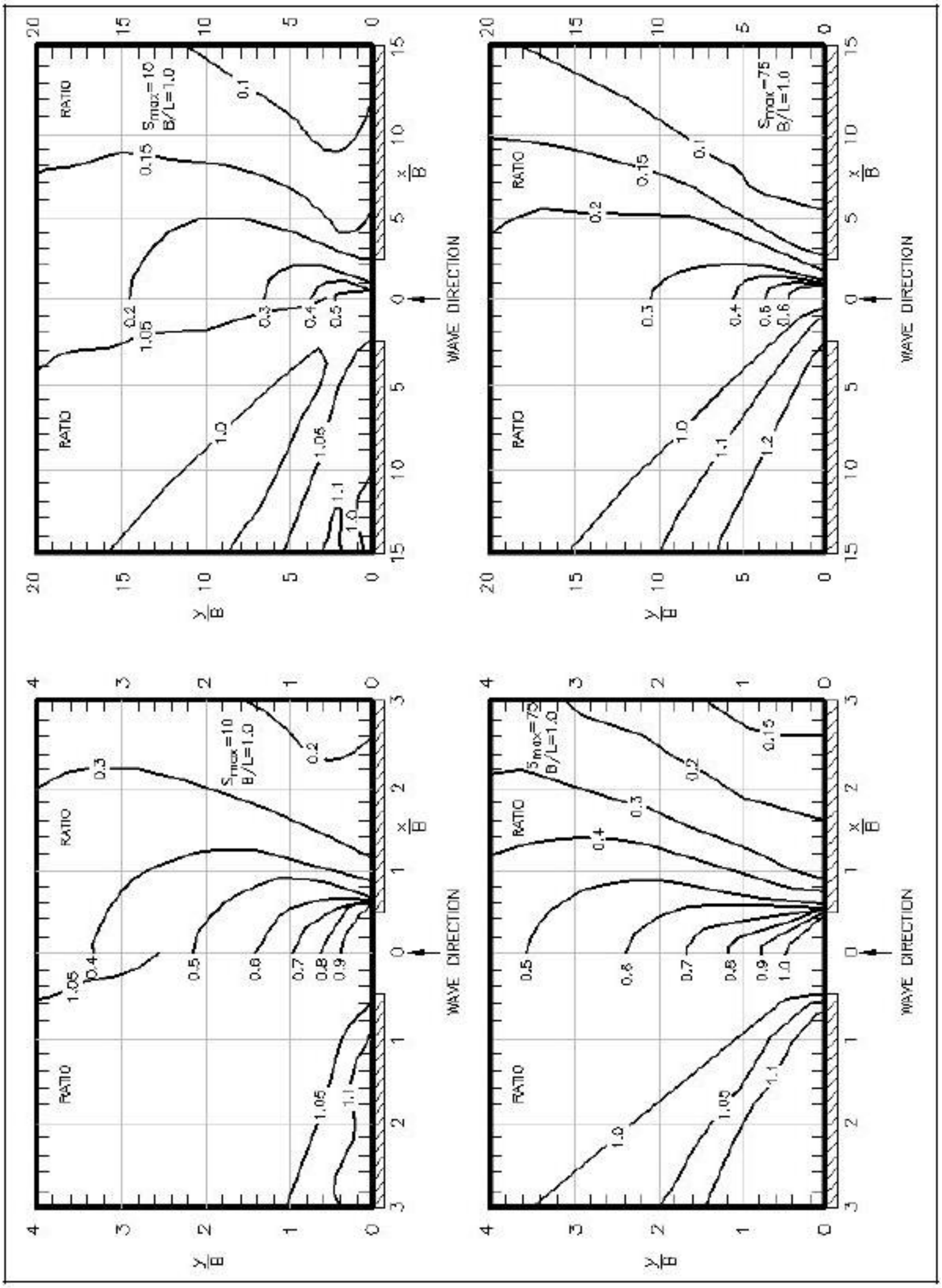

Figure A. 2: Diffraction diagram of a breakwater opening with B/L equal to 1.0 for normal incidence (Goda, 2000; US Army Corps of Engineers, 2002)

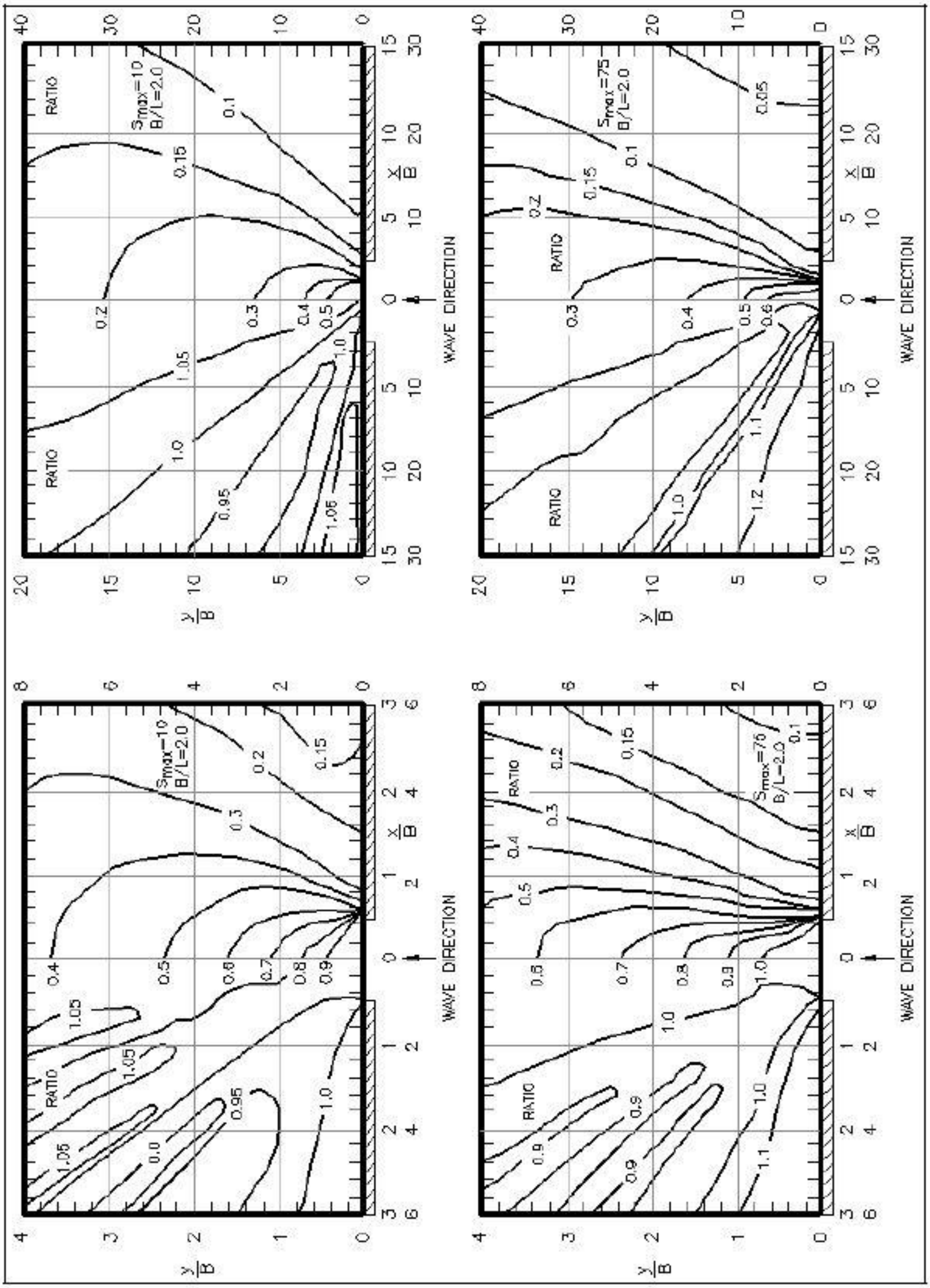

Figure A. 3: Diffraction diagram of a breakwater opening with B/L equal to 2.0 for normal incidence (Goda, 2000; US Army Corps of Engineers, 2002)

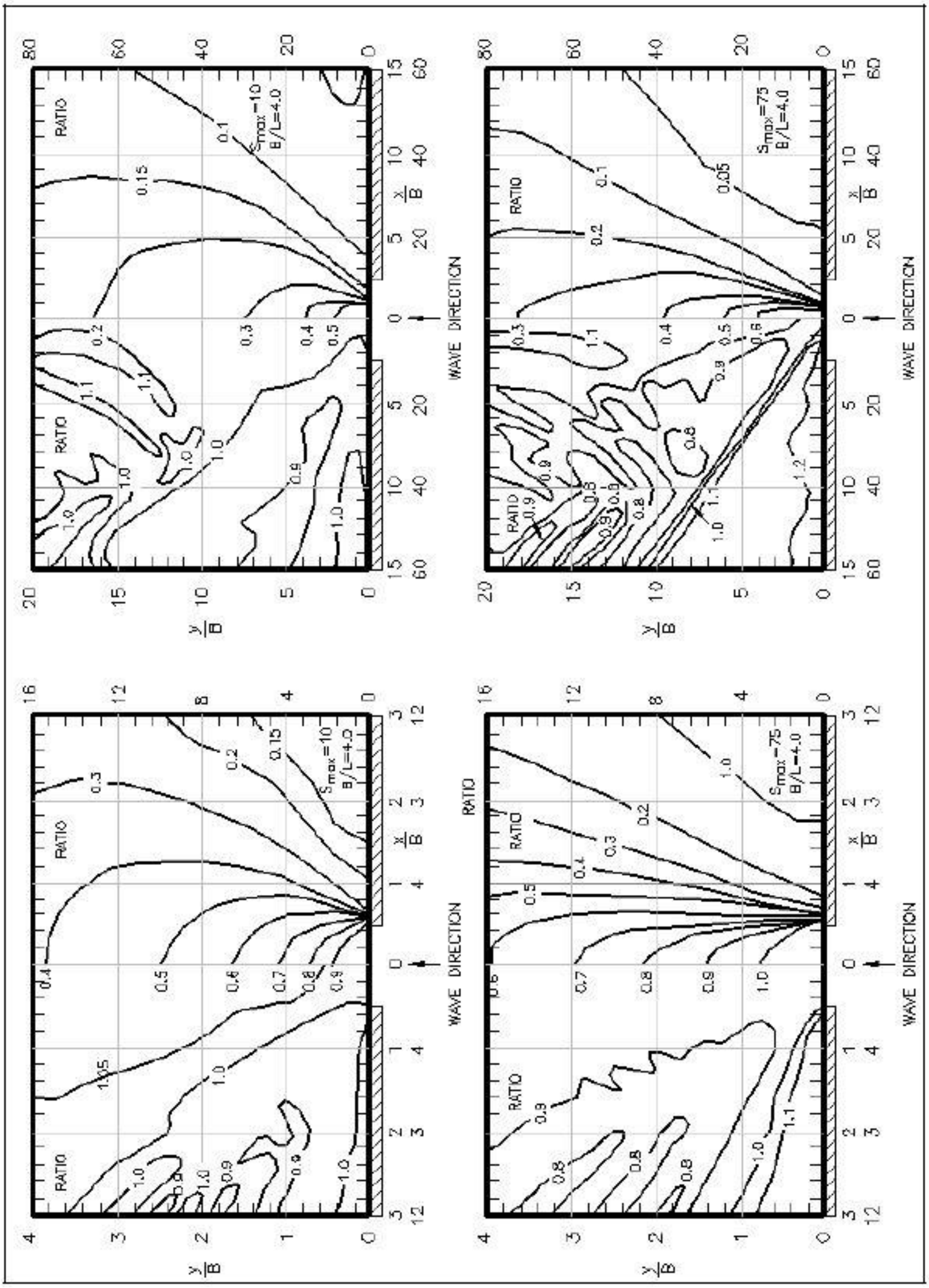

Figure A. 4: Diffraction diagram of a breakwater opening with B/L equal to 4.0 for normal incidence (Goda, 2000; US Army Corps of Engineers, 2002)

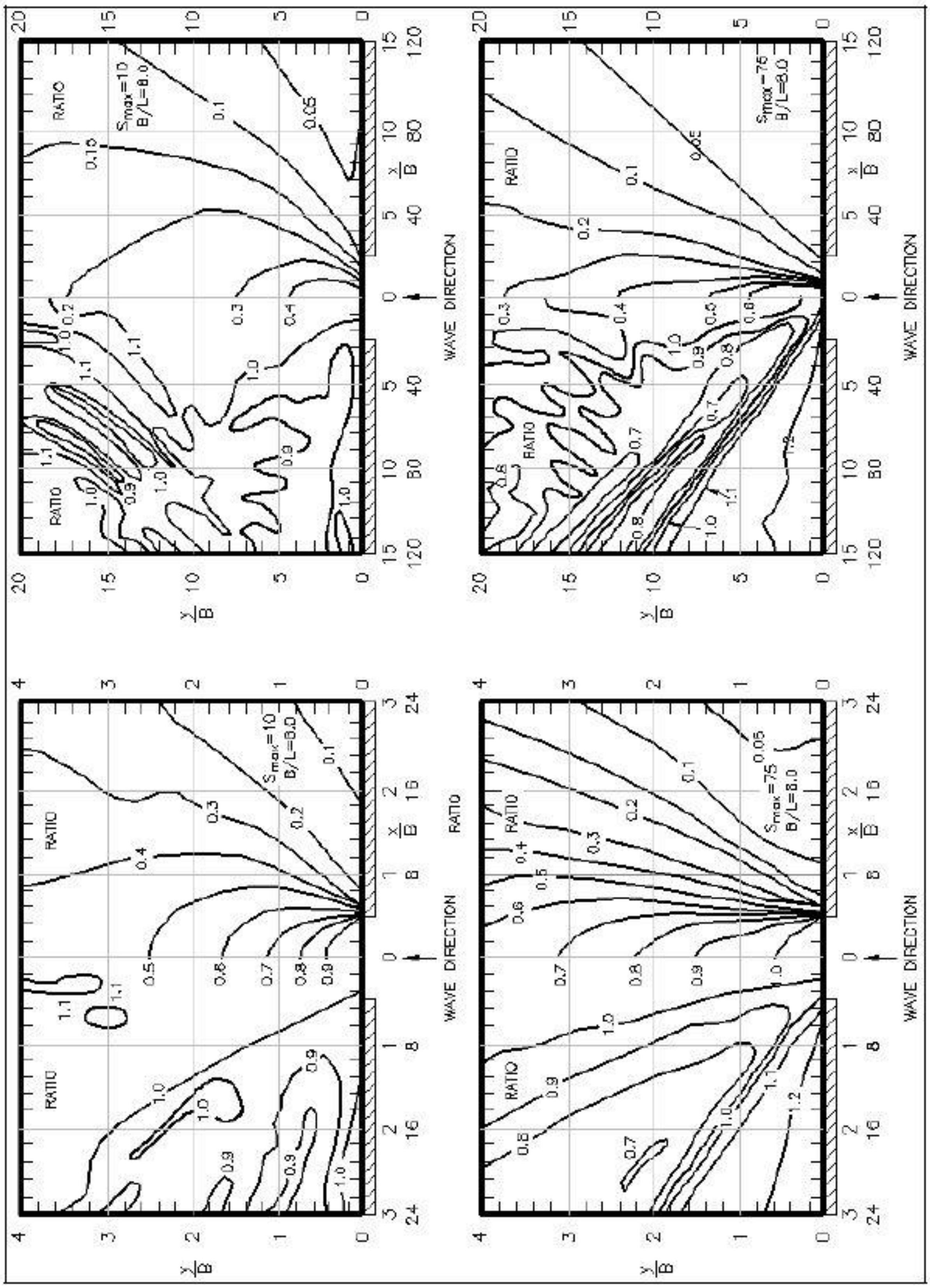

Figure A. 5: Diffraction diagram of a breakwater opening with B/L equal to 8.0 for normal incidence (Goda, 2000; US Army Corps of Engineers, 2002)

ANNEX B

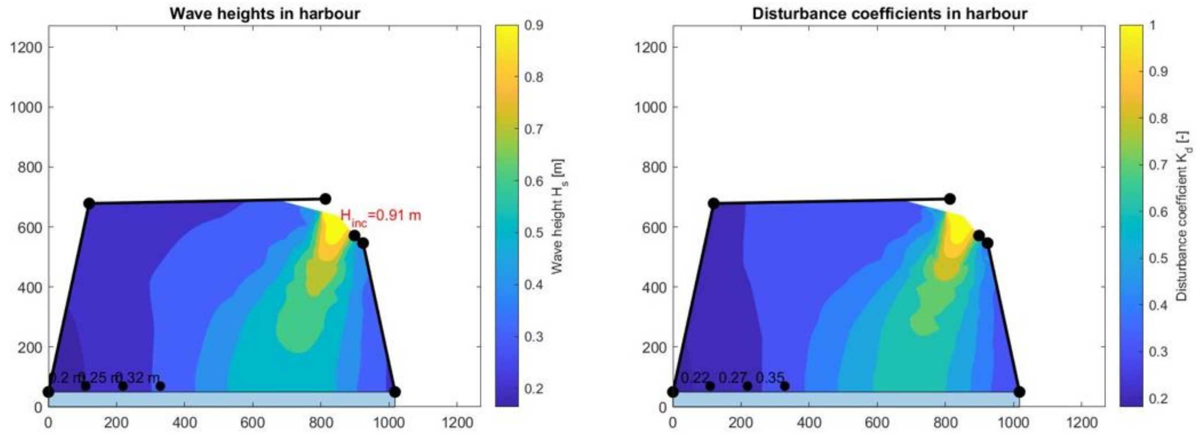

Figure B. 1: Wave agitation in first alternative

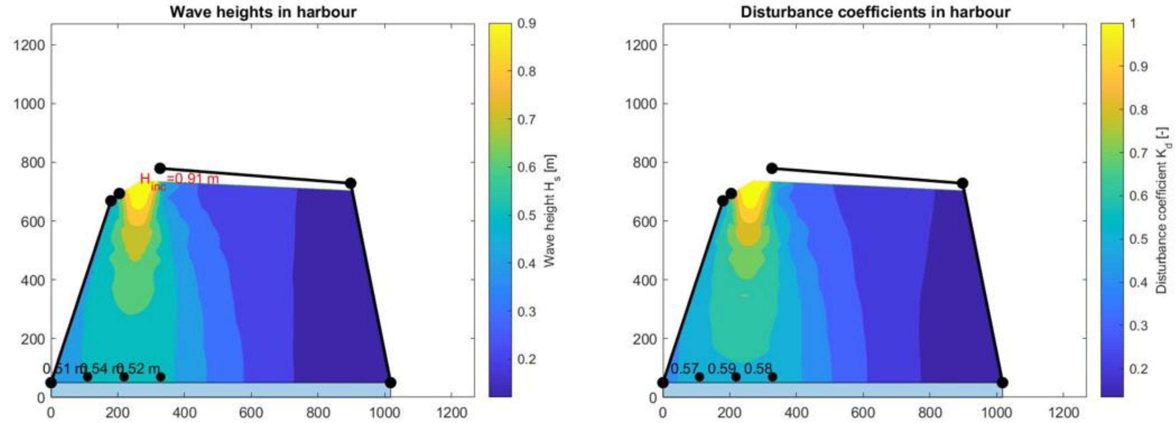

Figure B. 2: Wave agitation in second alternative

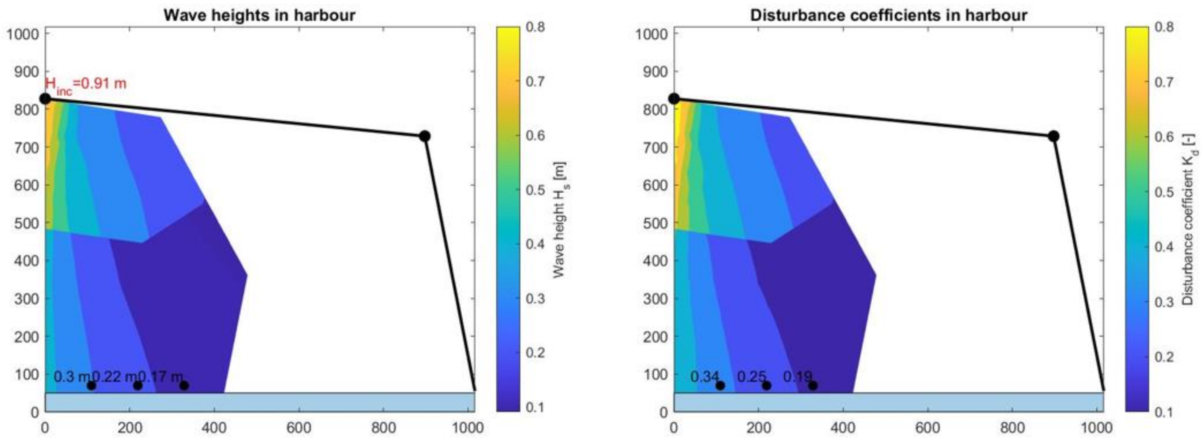

Figure B. 3: Wave agitation in third alternative

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Development of a methodology for a preliminary design of port and harbour layout

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Master's dissertation submitted in order to obtain the academic degree of
Master of Science in Civil Engineering

Academic year 2019-2020

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