New concept
LNG marine terminal

Hydrodynamics of a LNG carrier
Behind a detached Breakwater

Thesis report

Wieke Wuisman

Delft, February 2005
Preface

The final section of the Masters study program at Delft University of Technology consists of an individual thesis on a subject related to the student’s specialization. This report is an overview of the results of an investigation on the thesis subject: New concept LNG Marine terminal; Hydrodynamics of a LNG carrier behind a detached breakwater. The research objective is the development of a simulation model that predicts the response of a moored LNG carrier in an exposed LNG marine terminal, as a function of the breakwater configuration and coast.

During this investigation a graduation committee, consisting of the following persons, supervised all stages of the thesis work, for which I am very grateful; Prof.ir. H. Ligteringen (head of the committee), Dr.ir. A.J.H.M. Reniers, Ir. L. Groenewegen and Ir. W. van der Molen.

Especially Wim van der Molen I would like to thank for his daily supervision and support during the past months. Due to his clear explanations on the subject and our cooperation, this thesis has been a very constructive and educational process.

I also would like to thank Prof.dr.ir. J.A. Pinkster for the DELFRAC simulation results.

Besides the graduation committee I would like to thank all those who have donated their time, energy and have given me advice, my family, and friends.

Delft, 24th of February 2005

Wieke Wuisman
Abstract

As the worldwide gas market continues to grow and environmental concerns with respect to in-port unloading of gas have increased, there has been a boom of interest in new liquefied natural gas (LNG) import terminals in the past five years. For these terminals, which are more and more located in areas with hostile sea conditions, dedicated provisions are required to create sufficient shelter for the carriers. Proposals have been made to construct a marginal low crested breakwater parallel to the coast protecting a ship moored at a jetty close to the shore. For an optimal economic design of such an LNG marine terminal, the dimensions and orientation of the detached breakwater have to be optimized as a function of the weather related downtime of the moored LNG carrier. Doing so requires adequate simulation tools. However, for the combination of wave and ship motion, a link between an efficient wave simulation tool and a program for ship response calculations is not available at present.

The research on ship behaviour has resulted in the development of various so-called six degrees of freedom (SDF) computer programs. These programs solve the equations of motion of a moored vessel for all six degrees of freedom. As a consequence of the non-linear characteristics of the mooring system the equation of motion is solved in the time domain. The wave force time series are calculated from a homogeneous wave field of irregular, long-crested waves. In case of an open jetty configuration these assumptions are valid. However, considering a carrier behind a detached breakwater, the wave field is not homogeneous, but the wave height varies over the ship length. Consequently the influence of the detached breakwater on the ship motions must be considered. In addition, the reflection of the waves at the coast also has to be taken into account.

This thesis describes a methodology to predict the hydrodynamics of a moored LNG carrier behind a detached breakwater. A rapid assessment tool has been developed in order to assess the optimum breakwater dimensions in the preliminary design stage of an LNG marine terminal. In particular the effects of the breakwater dimensions on the hydrodynamic behaviour of the moored LNG carrier are considered. The computational approach for the calculation of ship motions from a given offshore wave field is described. In addition results are presented for different terminal layouts.
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### List of symbols

**Roman symbols**

- $a$ Added inertia matrix
- $a$ [-] Coefficient of reflection coefficient formula
- $b$ Damping matrix
- $b$ [-] Coefficient of reflection coefficient formula
- $c$ [-] Permeability parameter
- $c$ Hydrostatic resoring coefficient
- $g$ [m/s²] Acceleration of gravity
- $h$ [m] Water depth
- $i$ [-] Imaginary unit $= \sqrt{-1}$
- $i$ [-] i-th frequency component
- $j$ [-] j-th directional component
- $k$ [-] Mode of motion [1..6]
- $j$ [-] Mode of motion coupled with mode of motion $k$
- $k$ [m⁻¹] Wave number
- $m$ Added mass matrix
- $m_0$ Zeroth moment spectrum
- $m_1$ First moment spectrum
- $r$ [m] Radius
- $s_{wp}$ [1/s²] Wavwe steepness
- $t$ [s] Time
- $v^2$ [m²] Spectral width
- $x_j$ [m] Displacement or rotation in the j-mode in ship-bound coordinate system
- $x_o$ [m] earth-fixed x coordinate
- $x_s$ [m] ship-bound x coordinate
- $y_o$ [m] earth-fixed y coordinate
- $y_s$ [m] ship-bound y coordinate
- $y_{refl}$ [m] Mirrored y-coordinate with respect to coast

- $A$ [-] Scaling parameter
- $A$ [m²/s] Complex amplitude velocity potential
- $B$ [m] Breadth of ship
- $B$ [m] Cross-sectional area of breakwater
- $bG$ [m] Distance sectional buoyancy point – CoG
- $C$ [-] Fresnel integral
- $D$ [m] Depth of ship
\( D_{50} \) Nominal diameter
\( F \) [kN] Force
\( F_{\text{total}} \) [kN] Total wave force
\( F^{(1)} \) [kN] First order wave force
\( F^{(2)} \) [kN] Second order wave force
\( F_{\text{drift}} \) [kN] Wave drift force
\( F_{FK} \) [kN] Froude-Krilov force
\( F \) [-] Complex x,y dependent function of velocity potential
\( H \) [m] Wave height
\( H_s \) [m] Significant wave height
\( H_d \) [m] Diffracted wave height
\( H_i \) [m] Incoming wave height
\( H_r \) [m] Reflected wave height
\( I \) Moment of inertia
\( K_d \) [-] Diffraction coefficient
\( KG \) [-] Distance keel – CoG
\( K_r \) [-] Reflection coefficient
\( K_t \) [-] Transmission coefficient
\( LGB \) [m] Longitudinal centre of gravity relative to station 10
\( L_{pp} \) [m] Length between perpendiculars
\( M \) Inertia matrix
\( M \) [-] \( N^o \) of frequency intervals
\( N \) [-] \( N^o \) of directional intervals
\( P \) [-] Left breakwater-end [earth-fixed coordinate system]
\( P \) [-] In-phase part of second order transfer function
\( Q \) [-] Right breakwater-end [earth-fixed coordinate system]
\( Q \) [-] Out-of-phase part of second order transfer function
\( oG \) [m] Height CoG above free surface
\( R \) Retardation function
\( R_c \) [m] Crest freeboard relative to SWL
\( S \) [m\(^2\)] Submerged body surface
\( S \) [-] Fresnel integral
\( S_{GS} \) Gaussian variance density
\( S_{JS} \) Jonswap variance density
\( S_{PM} \) Pierson-Moskowitz variance density
\( T \) [m] Draft of ship
\( T \) [s] Wave period
\( T_z \) [s] Mean zero wave-crossing wave period
\( T_p \) [s] Peak wave period
\( T_m \) [s] Wave period transmitted waves
\( X_j \) [m] Displacement or rotation in the j-mode in earth-fixed coordinate system
Greek symbols

\(\alpha\) \hspace{1cm} [\text{rad}] \hspace{1cm} \text{Beach slope}

\(\chi\) \hspace{1cm} [\text{m}^2/\text{s}] \hspace{1cm} \text{Normalised velocity potential}

\(\varepsilon\) \hspace{1cm} [\text{rad}] \hspace{1cm} \text{Random phase angle}

\(\phi\) \hspace{1cm} [\text{m}^2/\text{s}] \hspace{1cm} \text{Velocity potential}

\(\phi^{(1)}\) \hspace{1cm} [\text{m}^2/\text{s}] \hspace{1cm} \text{First order velocity potential}

\(\phi^{(2)}\) \hspace{1cm} [\text{m}^2/\text{s}] \hspace{1cm} \text{Second order velocity potential}

\(\varphi\) \hspace{1cm} [\text{rad}] \hspace{1cm} \text{Phase angle}

\(\varphi_{ij,0}\) \hspace{1cm} [\text{rad}] \hspace{1cm} \text{Phase angle incoming wave component ij}

\(\varphi_{ij,D}\) \hspace{1cm} [\text{rad}] \hspace{1cm} \text{Phase angle diffracted wave component ij}

\(\varphi_{ij,R}\) \hspace{1cm} [\text{rad}] \hspace{1cm} \text{Phase angle reflected wave component ij}

\(\varphi_{ij,T}\) \hspace{1cm} [\text{rad}] \hspace{1cm} \text{Phase angle transmitted wave component ij}

\(\gamma\) \hspace{1cm} [-] \hspace{1cm} \text{Peak enhancement factor Jonswap spectrum}

\(\lambda\) \hspace{1cm} [\text{m}] \hspace{1cm} \text{Wave length}

\(\mu_s\) \hspace{1cm} [\text{deg}] \hspace{1cm} \text{Ship-bound rotation}

\(\mu_{Fk}\) \hspace{1cm} [\text{kN}] \hspace{1cm} \text{Mean of wave force}

\(\theta_o\) \hspace{1cm} [\text{deg}] \hspace{1cm} \text{Incident wave direction, earth-fixed coordinate system}

\(\theta_b\) \hspace{1cm} [\text{rad}] \hspace{1cm} \text{Breakwater angle with respect to} \ x_o - \text{axis}

\(\theta_{ij,D}\) \hspace{1cm} [\text{rad}] \hspace{1cm} \text{Wave direction of diffracted wave component ij, polar coordinate system}

\(\theta_{ij,R}\) \hspace{1cm} [\text{rad}] \hspace{1cm} \text{Wave direction of diffracted wave component ij, polar coordinate system}

\(\theta_{ij,T}\) \hspace{1cm} [\text{rad}] \hspace{1cm} \text{Wave direction of transmitted wave component ij, polar coordinate system}

\(\rho\) \hspace{1cm} [\text{kg/m}^3] \hspace{1cm} \text{Density of water}

\(\sigma\) \hspace{1cm} [-] \hspace{1cm} \text{Numerical parameter Jonswap spectrum}

\(\sigma_{Fk}\) \hspace{1cm} [\text{kN}] \hspace{1cm} \text{Standard deviation of wave force}

\(\tau\) \hspace{1cm} [\text{s}] \hspace{1cm} \text{Time lag}

\(\omega\) \hspace{1cm} [\text{rad/s}] \hspace{1cm} \text{Wave angular frequency}

\(\omega_p\) \hspace{1cm} [\text{rad/s}] \hspace{1cm} \text{Peak wave angular frequency}

\(\bar{\omega}\) \hspace{1cm} [\text{rad/s}] \hspace{1cm} \text{Mean frequency}

\(\omega^*\) \hspace{1cm} [\text{rad/s}] \hspace{1cm} \text{Wave angular frequency, bound second order waves}

\(\xi\) \hspace{1cm} [-] \hspace{1cm} \text{Irribarren parameter}

\(\eta\) \hspace{1cm} [\text{m}^2/\text{s}] \hspace{1cm} \text{Normalised velocity potential which is independt of the vessel movement}

\(\psi\) \hspace{1cm} [\text{m}] \hspace{1cm} \text{Wave surface elevation}

\(\zeta\) \hspace{1cm} [\text{m}] \hspace{1cm} \text{First order surface elevation}

\(\zeta^{(1)}\) \hspace{1cm} [\text{m}] \hspace{1cm} \text{First order surface elevation}
\( \zeta^{(2)} \) [m] Second order surface elevation
\( \zeta_{a,j,0} \) [m] Off-shore wave amplitude of wave component \( ij \)
\( \zeta_{i,j,D} \) [m] Diffracted water surface elevation at point \( (x_o,y_o) \) of wave component \( ij \)
\( \zeta_{i,j,R} \) [m] Reflected water surface elevation at point \( (x_o,y_o) \) of wave component \( ij \)
\( \zeta_{i,j,T} \) [m] Transmitted water surface elevation at point \( (x_o,y_o) \) of wave component \( ij \)
\( \dot{\zeta}_{ik} \) [m/s] Equivalent velocity of the water particles in direction \( k \)
\( \ddot{\zeta}_{ik} \) [m/s²] Equivalent acceleration of the water particles in direction \( k \)

\( \Delta x \) [m] Small partition in \( x \) direction
\( \Delta y \) [m] Small partition in \( y \) direction
\( \Delta x_p \) [m] Longitudinal partition of ship length
\( \Delta \omega \) [rad/s] Small frequency interval partition
\( \Delta \theta \) [°] Small directional partition

\( \nabla \) Volume displacement

**Abbreviations**

- AP: Aft perpendicular
- AQWA: Atkins Quantitative Wave Analysis
- BAS: Beweging Afgemeerde Schepen
- CoG: Centre of Gravity
- FFT: Fast Fourier Transform
- FK: Froude-Krilov
- FP: Forward perpendicular
- FTF: First order Transfer Function
- JONSWAP: JOint North Sea WAve Project
- LAO: Length Over All
- LNG: Liquid Natural Gas
- LNGC: LNG Carrier
- MARIN: MAritime Research Institute Netherlands
- MatLab: Matrix Laboratory
- MSL: Mean Sea Water Level
- PM: Pierson-Moskowitz
- QTF: Quadratic Transfer Function
- RAO: Response Amplitude Operator
- SDF: Six Degrees of Freedom
<table>
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<th>Abbreviation</th>
<th>Description</th>
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<tr>
<td>SWL</td>
<td>Sea Water Level</td>
</tr>
<tr>
<td>TERMSIM</td>
<td>TERminal SIMulation</td>
</tr>
<tr>
<td>WSP</td>
<td>Wave Simulation Program</td>
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1 Introduction

1.1 LNG transport

Liquefied Natural Gas (LNG) is the liquid form of natural gas. It is normal gas, cooled to approximately -160°C, in order to store it as a boiling liquid in insulated tanks. From the point of view of the major electric utilities, electricity generation is increasingly dependent on gas as a flexible contributor to the merit order, particularly given the environmental desire to reduce dependence on nuclear and oil, and to phase out coal. Furthermore, LNG is one of the cleanest and most efficient forms of energy available. LNG appears to be a feasible alternative for meeting the increasing demand for gas.

Figure 1-1 gives the worldwide LNG trade development from 1970 to 2003 [ref 1]. The total capacity of the world’s liquefaction plants in 2003 was 155 billion m³ per year. This was a 10% increase over the past year according to 2002.

As the worldwide gas market continues to grow in order to supply domestic/industrial users and in many cases new power generation projects, there has been a recent renewal of interest in new LNG import terminals. Historically, the LNG terminals have been located in sheltered sites, without significant influence of swell and wind induced waves, or at exposed sites with relatively calm sea conditions. The availability and discovery of gas fields in areas, where (naturally) sheltered sites are not often available, ask for the development of new concepts for sites in increasingly hostile marine environments.
1.2 New concept

In LNG terminals, exposed to the open sea, dedicated provisions are required to create sufficient shelter for LNG carriers. Proposals have been made to construct a marginal low crested detached breakwater parallel to the coast to protect a ship moored at a jetty closer to the shore. Figure 1-2 presents a schematization of this new concept. The figure is not drawn to scale.

![Diagram of LNG marine terminal concept](image)

*Figure 1-2: Schematisation of a LNG marine terminal concept [not to scale]*

Generally, due to the high cost of LNG ships, the cost of storage and the impact of a shortfall in delivery, any investment in port development to eliminate marine/weather related downtime was justified without (any) question. However, as terminal exposure and the cost of breakwater structures increase, these old assumptions need to be challenged in order to reduce capital expenditure and to increase the economic viability of a project.

The weather related downtime is defined as the ratio of period of delay of marine operations due to adverse weather conditions. It depends on the ship's response and thus on the near shore wave parameters. These wave parameters are influenced by the phenomena of wave diffraction and transmission, which on their turn depend on the breakwaters dimensions. Furthermore, the wave field at the jetty is influenced by reflection of the incoming wave field by the coast.

The relation between the different aspects is clarified in Figure 1-3 on the next page.
A more thorough knowledge of the influence of diffraction, transmission and reflection on the weather related downtime would most likely result in options for further optimization of the concept.

For an optimal economic design of the LNG (un)loading facility, the dimensions of the detached breakwater need to be optimised as a function of the weather related downtime of the moored LNG-carrier.
2 Scope of work

2.1 Problem context

Studies are carried out for a new concept for a LNG marine terminal, consisting of a detached breakwater protecting a LNG jetty. Due to the presence of the detached breakwater and a reflective coast, ocean waves experience changes in height, direction and phase. The wave transformation process comprises diffraction, transmission and reflection. For an optimal economic design of a LNG marine terminal, the dimensions of the detached breakwater have to be optimised as a function of the weather related downtime of the moored LNG-carrier. Doing so requires adequate simulation tools. A survey of the currently available simulation tools for predicting wave the wave field at a LNG jetty and the ship's response can be found in Appendix A. From this overview, the conclusion can be drawn that a link between an efficient wave simulation tool and a program for ship response calculations, for the preliminary design stage of a LNG marine terminal, is not available at present.

2.2 Research objective

The research objective is the development of a model that predicts the response of a moored LNG carrier in an exposed LNG marine terminal, as a function of the breakwater configuration and coast.

2.3 Problem approach

Based on the above-formulated objective, the procedure of research breaks in two parts:

Part A: Development of an efficient wave - and wave force simulation tool

Wave Simulation Program (WSP)
An efficient and simplified model to calculate the effects of site-specific (protective) structures (i.e. breakwater, coast) on the wave field, as a supplement to the ship motion simulation programs, is not available. Without such a wave field simulation tool, the influence of the breakwater dimensions on diffraction, transmission and reflection, and subsequently on the ship's response and the weather related downtime, cannot be predicted accurately.
For a better understanding in the dynamic behaviour of a moored LNG carrier behind a detached breakwater, an efficient wave field simulation program needs to be developed. This new simulation tool needs to provide more accurate input of wave characteristics along the ship’s hull for a convenient ship response simulation program and must be well applicable in the preliminary design stage of a LNG mooring facility.

**Figure 2-1: Wave Simulation Program**

**Wave force calculation**

As the near shore wave field parameters are strongly spatially varying, and the LNG carrier’s length is significant with respect to the wave length, the wave parameters and the resulting wave forces have to be introduced along the ship’s hull.

Therefore a solution for introducing the strongly varying wave parameters and the resulting wave forces along the ship’s hull, is required

**Figure 2-2: Wave force determination**
Part B: Investigation to the influence of site-specific features on the ship response.

Ship motion dependency to the breakwater configuration
Ocean wave conditions, breakwater configuration (especially the main parameters length and height), and coastal features have influences on the ship’s response, the mooring line and fender forces and eventually on the downtime of the moored LNG carrier. Influences of these site-specific features have to be examined in order to get more insight in the development of the new LNG marine terminal concept.

Figure 2-3: Optimization of the breakwater configuration

2.4 Report set-up
This report is set-up in the following way; the development of the WSP is based on theory of waves and wave forces given in literature. In chapter three (wave description), four (wave transformation phenomena), the theory, used during the development of the program, is described. The dynamics of a LNG carrier can be computed using the equation of motion. In order to get more insight in this topic, the theory of ship motions is discussed in chapter five. In chapter six the determination of the hydrodynamic loads is explained. The model set-up of the Wave Simulation Program (WSP) and the structure of the computer simulation program are presented in chapter seven. In chapter eight the program is validated. Applications of the Wave Simulation Program and the influence of diffraction, transmission and reflection on the ship response are evaluated in chapter nine. Finally, the conclusions and recommendations of this thesis project are presented in chapter ten.
3 Wave description

3.1 Spectra

In order to describe the wave climate at a specific location, it is usually considered to be stationary within periods of three hours. This implies that within such a period of time, the statistical properties of the wave climate are assumed to be constant. The wave characteristics within such durations are called sea states and can be described statistically by a variance density spectrum, commonly referred to as a wave spectrum.

A directional spectrum describes the distribution of wave energy in the frequency domain as well as in the direction (angle \( \theta \)). It is generally expressed as

\[
S(\omega, \theta) = S(\omega) D(\theta | \omega)
\]  

in which \( S(\omega, \theta) \) is the directional wave spectrum, \( S(\omega) \) denotes the frequency spectrum and \( D(\theta | \omega) \) is the directional spreading function, i.e. the frequency spectrum represents the absolute value of the wave energy density and the spreading function represents the relative magnitude of directional spreading of wave energy. The frequency distributions and directional spectra that have been used in the model can be found in Appendix B.

3.2 Surface elevation

Changing the spectrum from its continuous form to a discrete form transforms the spectra into wave time series. Doing so requires a small bandwidth \( \Delta \omega \) for the frequency spectrum and \( \Delta \theta \) for the directional spectrum. This is illustrated in Figure 3-1. It shows a 2-dimensional frequency spectrum. A series of bands with the same amount of wave energy (gray area) is created.

\[ \text{Figure 3-1: Frequency-directional wave component} \]
Irregular waves can be described in the time domain and in the spatial domain as a summation of regular waves with random phase, also known as a Fourier series:

\[
\zeta^{(1)}_{a,\theta}(t) = \sum_{i=1}^{M} \sum_{j=1}^{N} \zeta_{a,ij} \cdot e^{i(\omega_{ij}t - k_{ij}x \cos \theta_j - k_{ij}y \sin \theta_j + \epsilon_{ij})}
\]

(2)

in which the frequency range of the spectrum is divided into segments from \(i = 1\) to \(M\) and the directional range is divided in parts form \(j = 1\) to \(N\), and:

- \(\zeta^{(1)}\) = First order surface elevation [m]
- \(\zeta_{a,ij}\) = Amplitude of wave \(ij\) [m]
- \(\omega_{ij}\) = Angular frequency of wave component \(ij\) [rad/s]
- \(k_{ij}\) = Wave number [m\(^{-1}\)]
- \(\theta_j\) = Wave direction [rad]
- \(t\) = Time [s]
- \(\epsilon_{ij}\) = Random phase angle of wave component \(ij\) [rad].

For each angular frequency and direction segment, a wave amplitude and a phase are required. The phase is to be chosen randomly, according to the random-phase/amplitude model. This phase angle is uniformly distributed in the range from 0 to \(2\pi\).

The amplitude can be derived from the discrete wave spectrum with use of the following equation:

\[
S(\omega_i, \theta_j) \cdot \Delta \omega_i \Delta \theta_j = \frac{1}{2} \zeta^{(1)2}_{a,ij}
\]

(3)

or

\[
\zeta^{(1)}_{a,ij} = \sqrt{2 \cdot S(\omega_i, \theta_j) \cdot \Delta \omega_i \Delta \theta_j}
\]

(4)

\(\Delta \omega_i\) = Small frequency interval partition [rad/s]
\(\Delta \theta_j\) = Small directional interval partition [°]

The wave number can be determined iteratively by evaluating the dispersion relationship:

\[
k_{ij} = \frac{\omega_{ij}^2}{g \tanh k_{ij} h}
\]

(5)

with:
- \(h\) = Water depth [m]
- \(g\) = Acceleration of gravity [m/s\(^2\)]
4 Wave transformation phenomena

4.1 Introduction

Ocean waves experience, as they hit the detached breakwater and coastline, changes in height and direction. The wave transformation process modeled here comprises diffraction, reflection and transmission. This section contains theoretical background on diffraction, reflection and transmission and their individual influence on the wave parameters, surface elevation and direction.

In this thesis the transformation processes are approached as linear problems. This implies that the following assumptions have to be made like in all other linear wave theories;

- Water is an ideal fluid; i.e. non-viscous and incompressible
- Waves are of small amplitude and can be described by linear wave theory
- Flow is irrotational and conforms to a potential function, which satisfies the Laplace-equation
- Water depth shoreward of the breakwater is constant, i.e. no refraction

Following the separate determination of the diffraction, reflection and transmission patterns, the wave characteristics at location \((x_0, y_0)\) at time \(t\) will be derived by the superpositioning of the three individual wave components:

\[
\zeta_{\omega,\theta, D}(x, y, t) = \sum_{i=1}^{M} \sum_{j=1}^{N} \zeta_{\omega, D}(x, y) \cdot e^{i(\phi_i + \epsilon_j)}
\]

\[
\zeta_{\omega,\theta, R}(x, y, t) = \sum_{i=1}^{M} \sum_{j=1}^{N} \zeta_{\omega, R}(x, y) \cdot e^{i(\phi_i + \epsilon_j)}
\]

\[
\zeta_{\omega,\theta, T}(x, y, t) = \sum_{i=1}^{M} \sum_{j=1}^{N} \zeta_{\omega, T}(x, y) \cdot e^{i(\phi_i + \epsilon_j)}
\]

\[
\Sigma \zeta_{\omega,\theta}(x, y, t) = \sum_{i=1}^{M} \sum_{j=1}^{N} \zeta_{\omega, \theta}(x, y) \cdot e^{i(\phi_i + \epsilon_j)}
\]

Of each wave component, and for the three phenomena, the following parameters have to be derived;

- \(\zeta(x,y)\) = Amplitude surface elevation at point \(x,y\) [m]
- \(\phi(x,y)\) = Phase at point \(x,y\) [rad/s]
- \(\theta\) = Direction at point \(x,y\) [rad]
4.2 Diffraction

4.2.1 Diffraction: definition of the phenomenon

When a wave train meets an obstacle such as a breakwater or an offshore platform it may be reflected backward, but the wave crests can also bend around the obstacle and thus penetrate into the lee zone of the obstacle. This phenomenon is called diffraction. The degree of diffraction depends on the ratio of the characteristic lateral dimension of the obstacle (i.e. the length of a detached breakwater) to the wavelength [ref.1].

Figure 4-1 shows a long-crested monochromatic wave approaching a semi-infinite breakwater in a region where the water depth is constant (i.e. no wave refraction or shoaling). The portion of the wave that passes the breakwater tip will diffract into the breakwater lee. The diffracted wave crests will essentially form concentric circular arcs with the wave height decreasing along the crest of each wave.

\[
K_d = \frac{H_d}{H_i}
\]

where \(H_d\) is the diffracted wave height at a point in the lee of the breakwater and \(H_i\) is the incident wave height at the breakwater tip. If \(r\) is the radial distance from the breakwater tip to the point where \(K_d\) is to be determined and \(\theta\) is the angle between the breakwater-end and this radial, then

\[
K_d = fcn(r/\lambda, \theta, \theta_b)
\]

Figure 4-1: Diffraction around a semi-infinite rigid breakwater
where $\theta_0$ defines the incident wave direction and $\lambda$ is the wave length. Consequently, for a given location in the lee of the breakwater, the diffraction coefficient is a function of the incident wave period and direction of approach. So, for a spectrum of incident waves, each frequency component in the wave spectrum would have a different diffraction coefficient for a given location in the breakwater's lee.

### 4.2.2 Diffraction solution of Penny and Price

The first to solve a general diffraction problem was Sommerfeld (1896) for the diffraction of light passing the edge of a semi-infinite screen. Penny and Price [ref. 2] showed that the same solution applies to the diffraction of linear surface waves on water of constant depth that propagate past the end of a semi-infinite, vertical-faced, rigid, impermeable barrier with negligible thickness.

Thus, the diffraction coefficients in the structure lee include the effects of the diffracted incident wave and the much smaller diffracted wave that reflects completely from the structure. Wiegel [ref. 3] made a summary of the Penny and Price findings and has listed the results in tables ($K_d = fcn(r/\lambda, \theta, \theta_0)$ for selected values of $r/\lambda$, $\theta$ and $\theta_0$). The results of the method of Penny and Price will be explained in this section. The complete derivation of the solution can be found in Appendix C.

**Surface elevation progressive waves**

Separation of variables is a convenient method that can be used when solving linear partial differential equations like the velocity potential (see Appendix C). Following this method, the water surface elevation can be expressed as

$$\zeta^{(1)} = \frac{\omega \cdot e^{i\omega t}}{g} \cdot e^{iky} \cdot \cosh kh \cdot F(x, y)$$  \hspace{1cm} (10)$$

in which,

- $\zeta^{(1)}$ = First order surface elevation [m]
- $i$ = complex number [-]
- $\omega$ = Angular frequency of wave component [rad/s]
- $A$ = Complex amplitude of the velocity potential with unit [m²/s].
- $g$ = Acceleration of gravity [m/s²].
- $k$ = Wave number [m⁻¹]
- $h$ = Water depth [m]
- $t$ = Time [s]
- $F$ = Complex function depending on $x$ and $y$
**Diffraction coefficient**

The diffraction coefficient $K_d$ is defined as the ratio of the wave height in the area affected by diffraction to the wave height in the area unaffected by diffraction. It is given by the modulus for the diffracted wave

$$K_d(x, y) = |F(x, y)|$$ \hspace{1cm} (11)

The phase difference is given by the argument of $F(x, y)$

$$\phi(x, y) = \arg(F(x, y))$$ \hspace{1cm} (12)

For the general case of waves approaching the breakwater under any angle $\theta_0$, which has to be used during the development of the Wave Simulation Program, Penny and Price derived an equation to express $F$ in polar coordinates, thus

$$K_d(r, \theta) = |F(r, \theta)|$$ \hspace{1cm} (13)

where $F(r, \theta)$ is still a complex function defined as

$$F(r, \theta) = E + iF$$ \hspace{1cm} (14)

The values of $E$ and $F$ can be determined using Fresnel integrals $C$ and $S$. Appendix D provides background on these integrals. Wiegel has worked out the solution. The equation of $E$ and $F$ become:

$$E = \left\{ \begin{array}{l} U_1 \cos[kr \cos(\theta - \theta_0)] + U_2 \cos[kr \cos(\theta + \theta_0)] \\ -W_1 \sin[kr \cos(\theta - \theta_0)] - W_2 \sin[kr \cos(\theta + \theta_0)] \end{array} \right\}$$ \hspace{1cm} (15)

$$F = \left\{ \begin{array}{l} W_1 \cos[kr \cos(\theta - \theta_0)] + W_2 \cos[kr \cos(\theta + \theta_0)] \\ +U_1 \sin[kr \cos(\theta - \theta_0)] + U_2 \sin[kr \cos(\theta + \theta_0)] \end{array} \right\}$$ \hspace{1cm} (16)

in which:

$$U_1 = \frac{1}{2}(1+C+S)$$
$$W_1 = \frac{1}{2}(S+C)$$
$$U_2 = \frac{1}{2}(1-C-S)$$
$$W_2 = \frac{1}{2}(S+C)$$

$C$ = Fresnel integral given by equation D.1 (see Appendix D)
$S$ = Fresnel integral given by equation D.2 (see Appendix D)
Wave transformation phenomena

- $k$ = Wave number [m$^{-1}$]
- $\theta_o$ = Incident wave direction [rad]
- $\theta$ = Angle with respect to breakwater-end [rad]
- $r$ = Radius with respect to breakwater-end [m]

### 4.2.3 Transformation of wave parameters by diffraction

**Amplitude**

The diffracted wave amplitude at location $(x_0, y_0)$ thus becomes:

$$
\zeta_{ij,D}(x, y) = \sum_{i=1}^{M} \sum_{j=1}^{N} K_d(x, y) \cdot \zeta_{a,ij,0}
$$

(17)

$\zeta_{ij,D}(x, y)$ = Diffracted water surface elevation at point $x, y$ [m]

- $K_d(x, y)$ = Diffraction coefficient at point $x, y$ [-]
- $\zeta_{a,ij,0}$ = Off-shore wave amplitude [m]

**Phase**

The phase of each monochromatic wave in the diffraction field will be represented by the summation of the argument of the real and imaginary part of the diffracted wave and the random phase angle:

$$
\phi_{ij,D}(x, y) = \arg\left(F(x, y)\right)
$$

(18)

**Direction**

The direction of the diffracted wave crests can be obtained by the argument of the phase-differences in both $x_0$ and $y_0$-direction.

$$
\theta_{ij,D}(x, y) = a \tan\left(\frac{\Delta \varphi_{ij,y}}{\Delta \varphi_{ij,x}}\right)
$$

(19)

with:

- $\Delta \varphi_{ij,y} = \varphi_{ij,x,y+\Delta y} - \varphi_{ij,x,y}$ \hspace{0.5cm} $\Delta y$ = very small y-directional interval
- $\Delta \varphi_{ij,x} = \varphi_{ij,x+\Delta x,y} - \varphi_{ij,x,y}$ \hspace{0.5cm} $\Delta x$ = very small x-directional interval
**Surface elevation**

The resulting surface elevation at point \((x_0,y_0)\) and time \(t\) is:

\[
\zeta_{a,b,D}(x,y,t) = \sum_{i=1}^{M} \sum_{j=1}^{N} \zeta_{i,j,D}(x,y) \cdot e^{i(\phi_{i,j} + \epsilon_{ij})}
\]

\(20\)
4.3 Reflection

4.3.1 Reflection: definition of the phenomenon

Water waves may be either partially or totally reflected from both natural and manmade barriers. When a wave interferes with a vertical, impermeable, rigid surface-piercing wall, essentially all of the wave energy will reflect from the wall. On the other hand, when a wave propagates over a small bottom slope, only a very small portion of the energy will be reflected. Consideration of wave reflection may often be as important as diffraction in the design of coastal structures or harbor development. Figure 4-3 illustrates a wave reflected by a barrier. It shows that the incident waves bend while they approach the coast. This indicates a decreasing water depth in coastal direction. Nevertheless, the water depth considered in the model is constant.

\[ K_r = \frac{H_r}{H_i} \]  

where \( H_r \) and \( H_i \) are respectively, the reflected and incident wave heights.

Reflection coefficients for most structures are usually estimated by means of laboratory model tests. Approximate values of reflection coefficients as reported in various sources are listed in Table E.1 of Appendix E.
Theoretical analysis found that, the reflection coefficient for a surface-piercing sloped plane not only depends on the slope angle, surface roughness, and porosity, but it also on the incident wave steepness $H/\lambda$. Consequently, for a given slope roughness and porosity, the wave reflection will depend on a parameter known as the surf similarity number or Iribarren number [ref. 1]:

$$\zeta = \frac{\tan \alpha}{(H/\lambda)^{\frac{1}{2}}}$$  \hspace{1cm} (22)

### 4.3.2 Reflection from beaches

**Sloping sand beach**

For a sloping sand beach the following formula can be used:

$$K_r = 0.1 \cdot \zeta^2$$  \hspace{1cm} [ref 1] (23)

**Rocky coast**

When the coast is rocky, it can be considered as a rigid structure. Therefore $K_r$ should be obtained by using a formula for rigid structures, in stead of a formula for sloping beaches.

Laboratory investigations (Seelig and Ahrens 1981; Seelig 1983; Allsop and Hettiarachchi 1988) [ref. 5] indicate that the reflection coefficients for most structure forms can be given by the following equation:

$$K_r = \frac{a \cdot \zeta^2}{b + \zeta^2}$$  \hspace{1cm} (24)

where the values of coefficients $a$ and $b$ depend primarily on the structure geometry and to a smaller extent on whether waves are monochromatic or irregular. The Iribarren number employs the structure slope and the wave height at the toe of the structure. Values of $a$ and $b$ are tabulated in Table E.2 of Appendix E. Generally the following values can be used for a rocky coast: $a = 0.6$ and $b = 6.6$. 


4.3.3 Transformation of wave parameters by reflection

**Amplitude**
The reflected surface elevation equals the mirrored diffracted surface elevation with respect to the reflecting barrier; multiplied by the reflection coefficient $K_r$, see Figure 4-3.

\[
\zeta_{ij,R}(x, y) = \sum_{i=1}^{M} \sum_{j=1}^{N} K_r \cdot \zeta_{ij,D}(x, y_{refl})
\]  

(25)

**Phase**
The phase of the reflected waves has the same value; as if they would propagate without reflection, i.e. the same phase as the incident non-reflecting waves, see

\[
\varphi_{ij,R}(x, y) = \varphi_{ij,D}(x, y_{refl})
\]  

(26)

**Direction**
The direction of the reflected wave crests equals the mirrored direction of the incoming wave crests, see Figure 4-4.

\[
\theta_{ij,R}(x, y) = 2\pi - \theta_{ij,D}(x, y_{refl})
\]  

(27)

---

**Figure 4-4: Reflected wave parameters**
Surface elevation

The resulting surface elevation at point \((x_0, y_0)\) and time \(t\) is:

\[
\zeta_{0,y,R}(x, y, t) = \sum_{i=1}^{M} \sum_{j=1}^{N} K_r \cdot \zeta_{ij,R}(x, y_{\text{refl}}) \cdot e^{i(\theta_j \cdot x + \phi_j)}
\]

(28)

in which:

- \(\zeta_{u,0,R}(x, y, t)\) = Reflected water surface elevation at point \(x, y\) [m]
- \(K_r\) = Reflection coefficient [-]
- \(\zeta_{ij,D}(x, y_{\text{refl}})\) = "Diffracted" surface elevation at point \(x, y_{\text{refl}}\) [m]
- \(y_{\text{refl}}\) = Mirrored y-coordinate with respect to the coast:
  \[y_{\text{refl}} = y + 2|y|\]
4.4 Transmission

4.4.1 Transmission: definition of the phenomenon

When waves interact with a structure, a portion of their energy will be dissipated, a portion will be reflected and, depending on the geometry of the structure, a portion of the energy may be transmitted past the structure. If the crest of the structure is submerged, the wave will simply transmit over the structure. However, if the crest of the structure is above the waterline, the wave may generate a flow of water over the structure, which, in turn, regenerates waves in the lee of the structure. Also, if the structure is sufficiently permeable, wave energy may transmit through the structure. When designing structures that protect the interior of a harbor from wave attack, as little wave transmission as possible should be allowed, when optimizing the cost versus performance of the structure.

The degree of wave transmission that occurs is commonly defined by a wave transmission coefficient

\[ K_t = \frac{H_t}{H_i} \]  

(29)

where \( H_t \) and \( H_i \) are respectively, the transmitted and incident wave heights. When considering irregular waves, the transmission coefficient might be defined as the ratio of the transmitted and incident significant wave heights or some other indication of the incident and transmitted wave energy levels. Figure 4-5 shows the parameters affecting wave transmission.
4.4.2 Transmission studies

Extensive studies with 2-D models have been performed in order to investigate wave transmission at structures. These studies resulted in various predictive formulae and are dependent on the following input data: configuration and properties, water level, and wave condition. These formulae have been empirically derived from data collected from different laboratories without the certainty of equal analysis procedures; while most predictive equations of wave transmission perform quite well for the limited conditions in which they were tested, the question rises whether they are generally applicable. Recently different comparison studies have been carried out, leading to recommendations for the most appropriate predictive formulae for wave transmission modeling.

This section gives an overview of the present formulae. Appendix F contains some evaluation studies on this subject.

**Empirical derived transmission formulae**

Many authors have investigated the effects of wave transmission. This has resulted in the diagram presented in Figure 4-6 [ref 6]. Note that the transmission coefficient can never be smaller than 0 or larger than 1. In practice, limits of about 0.1 and 0.9 are found.

![Figure 4-6: Influence Rc/Hs on transmission coefficient Kt](image)

The following transmission studies and formulae are all empirically derived by hydraulic model tests on permeable breakwaters. In chronological order:

**Van der Meer (1990)**

From this analysis a simple prediction formula has been derived in which the transmission coefficient decreases with relative crest board $R_c/H_s$. Crest width effect is not directly taken into account.

\[
K_r = 0.46 - 0.3 \cdot \left( \frac{R_c}{H_s} \right) \quad (30)
\]

in which:
- $R_c$ = Crest freeboard relative to SWL [m]
- $H_s$ = Incident significant wave height [m]

Daemen (1991)

Daemen [ref. 8] re-analysed the same data set as van der Meer, excluding the data of Ahrens (1987), because hydraulic response of reef structures and conventional breakwaters deviates to a high extent. Daemen introduced a different dimensionless freeboard, including the permeability of the armour layer:

\[
K_r = a \cdot \left( \frac{R_c}{D_{50}} \right) + b \quad (31)
\]

\[
a = 0.031 \left( \frac{H_{50}}{D_{50}} \right) - 0.24
\]

\[
b = -5.42 \cdot s_{op} + 0.0323 \cdot \frac{H_{50}}{D_{50}} - 0.0017 \cdot \left( \frac{B}{D_{50}} \right)^{1.84} + 0.51
\]

\[
1 < \frac{H_{50}}{D_{50}} < 6
\]

\[
0.01 < s_{op} < 0.05
\]

where:
- $s_{op}$ = Wave steepness $= 2\pi H_s/D_{50} T_p^2$
- $L_p$ = Local wave length [m]
- $D_{n50}$ = Nominal diameter armour stone
D’Angremond et al. (1996)

Further analysis has been performed by d’Angremond et al. [ref. 9]. Model test results have been filtered, deleting tests with:

- High steepness: $s_{op} \geq 0.6$
- Breaking waves: $H_s / h \geq 0.54$
- Highly submerged structures: $R_c / H_s < -2.5$
- Highly emerged structures: $R_c / H_s > 2.5$

Resulting in:

$$K_t = a \frac{R_c}{H_s} + b$$  \hspace{1cm} (32)

$$a = -0.4$$

$$b = \left( \frac{B}{H_s} \right)^{-0.31} \cdot \left( 1 - e^{-0.5 \xi} \right) \cdot c$$

where:

- $R_c$ = Crest freeboard relative to SWL [m]
- $B$ = Crest width [m]
- $c$ = Permeability parameter
- $\xi$ = Irribaren-parameter = $(\tan(\alpha)/H_s/\lambda)^{0.5}$

Seabrook and Hall (1998) [ref. 10]

$$K_t = 1 - \left( e^{-0.65 \left( \frac{R_c}{H_s} \right) - 1.09 \left( \frac{H_s}{B} \right)} + 0.0047 \left( \frac{B \cdot R_c}{\lambda \cdot D_{50}} \right) - 0.0067 \left( \frac{R_c \cdot H_s}{B \cdot D_{50}} \right) \right)$$  \hspace{1cm} (33)

$$0 \leq \frac{B \cdot R_c}{L \cdot D_{50}} \leq 7.08$$

$$0 \leq \frac{H_s \cdot R_c}{B \cdot D_{50}} \leq 2.14$$
where:
- \( R_c \) = Crest freeboard relative to SWL [m]
- \( B \) = Cross-sectional area of structure [m]
- \( \lambda \) = Wave length [m]
- \( D_{50} \) = Nominal diameter armour stone

Ahrens (2001)

**Dominant mode approach**

Dominant modes of wave transmission can be identified as a function of the relative freeboard. If the structure is submerged (i.e. \( R_c/H_{si} < 0 \)), the dominant mode will be wave transmission over the crest. For surface-piercing structures (i.e. \( 0 < R_c/H_{si} < \tau \)), the dominant mode will be by wave run-up and overtopping, where \( \tau \) is a specific threshold that defines high structure. If the structure is high, defined as \( R_c/H_{si} < \tau \), transmission is primarily through the breakwater.

Ahrens [ref. 11] developed a parameterization and prediction equation that comprehends each of the fundamental modes. The transmission over and through the structure are computed using empirically derived formulas are combined into an overall transmission coefficient, \( K_t \), using

\[
K_t = \sqrt{(K_{t\text{over}})^2 + (K_{t\text{thru}})^2}
\]  

(34)

From this approach, an empirical general predictive procedure that produces logical trends in the transmission coefficient over a broad range of configurations and conditions can be developed.

**Analytical derived transmission formulae**

In the article ‘Interaction between porous media and wave motion’, [ref 12], Chwang and Chan review the use of Darcy’s law for analyzing waves moving past a porous structure. It summarizes a large amount of literature on the analytical study of free-surface wave motion past porous structures. The ratio between the reflected and transmitted wave energy forms an important subject in these studies. Yu and Chwang provided plots showing the variation of reflection and transmission coefficients as a function of the dimensionless thickness of the structure.

Dalrymple et al., [ref 13], studied the reflection and transmission of a wave train at an oblique angle of wave incidence \( \theta \) by an infinitely long porous structure of thickness \( b \). This provides the basis for treating an incident directional spectrum.
4.4.3 Spectral change by wave transmission

Due to transmission at the breakwater the wave spectrum is changed not only with respect to the total energy, but also with respect to the spectral shape. The loss of total energy results in the decrease of significant wave height, while the spectral shape change results in lower mean wave periods. Within the scope of this subject only little research is available:

Lower mean wave period
In 1998 Mai et al. introduces the ratio of mean period of the transmitted waves $T_{m,T}$ to the incident waves $T_m$ in his study on study on wave transmission at summer dikes. According to Mai, Ohle, Daemrich and Zimmerman (2002) [ref. 14] non-linear wave transformation over submerged breakwaters causes the change in wave spectrum, by the effect of transfer of energy from spectral peak to the higher harmonics. The incoming spectrum is changed into a double peak wave spectrum and subsequently causes the reduction of the mean wave period. However, the spectral peak remains nearly constant (van der Meer et al. 2000) and the influence of wave transmission on the spectral shape diminishes for relative high freeboards.

Overtopping
Overtopping generates so-called new waves with higher frequencies in the breakwater's lee. Because of the random character of the sea waves, and the discontinuous wave crests, the waves generated by overtopping are located at various positions along the breakwater, depending on the layout of a particular breakwater. According to Goda [ref. 15], analysis on such individual situations is not feasible. Therefore it is necessary to assume that transmitted waves propagate in a pattern similar to that of the incident waves.

4.4.4 Implementation in WSP

Due to lack of detailed research and literature, which is available on the subject of spectral shape change due to wave transmission, it is assumed that the spectral shape of the transmitted waves is similar to the spectral shape of the offshore wave field.

The Wave Simulation Model as described here, the user may choose either a constant value of the transmission coefficient $K_t$ or allow the model to calculate values based on wave and structure characteristics. It could be interesting to integrate the empirical and analytical derived transmission formulae; the empirical formulae provide more information about the overtopping over the structure and breakwater characteristics. On the other hand, the analytical solutions provide the possibility to include oblique wave incidence and dependency on wavelength.

As such integration requires more detailed study, only the empirical formulae have been included in the WSP. For each structure the user specifies geometric properties (crest height and width, and median rock size) and can select between calculation methods of van der

The evaluation studies of Wamsley and Ahrens (2003, ref 16), Wamsley, Hanson, Kraus (2002, ref 17) and Calabrese, Vicinanza, Buccino (2003, ref 18), provide guidance for selecting a calculation method for a given application. A survey of these evaluation studies is given in Appendix F.

4.4.5 Transformation of wave parameters by transmission

**Amplitude**

The transmitted wave amplitude at location \((x_0, y_0)\) becomes:

\[
\zeta_{ij,T}(x,y) = \sum_{i=1}^{M} \sum_{j=1}^{N} K_{ij}(x,y) \cdot \zeta_{a,ij,0}
\]  

(35)

in which:

- \(\zeta_{ij,T}(x,y)\) = Transmitted water surface elevation at point \((x,y)\) [m]
- \(K_{ij}(x,y)\) = Transmission coefficient [-]
- \(\zeta_{a,ij,0}\) = Off-shore wave amplitude [m]

**Phase**

Assuming that the phase of the incoming wave components will not change by wave transmission, the transmitted phase is defined similar to the incoming phase:

\[
\varphi_{ij,T} = \varphi_{ij,0}
\]  

(36)

\(\varphi_{ij,0}\) = Phase incoming wave component [rad/s]
\(\varphi_{ij,T}\) = Phase transmitted wave component [rad/s]

**Direction**

According to the assumption that transmitted waves propagate in a pattern similar to that of the incident waves, the direction will not be altered:

\[
\theta_{ij,T} = \theta_{ij,0}
\]  

(37)

\(\theta_{ij,0}\) = Direction incoming wave component [°]
\(\theta_{ij,T}\) = Direction transmitted wave component [°]

**Surface elevation**

The transmitted water surface elevation thus becomes:
\[ \zeta_{\alpha,\theta}(x, y, t) = \sum_{j=1}^{M} \sum_{i=1}^{N} \zeta_{ij} \cdot e^{i(\Phi_{ij} + \varepsilon_{ij})} \] 

(38)
5 LNG carrier dynamics

This chapter presents the theory of the response of a moored LNG carrier at a jetty.

5.1 Equations of motion

Ships floating on water have six degrees of freedom, i.e. surge (translation in x-direction), sway (translation in y-direction), heave (translation in z-direction), roll (rotation around x-axis), pitch (rotation around y-axis) and yaw (rotation around z-axis). The motions of a floating body under actions of winds and waves are analyzed by solving the equations of these six modes.

\[ \sum_{j=1}^{6} \left\{ \left( M_{kj} + d_{kj} \right) \ddot{x}_j + b_{kj} \dot{x}_j + C_{kj} x_j \right\} = F_k \quad k = 1, 2, \ldots, 6 \quad (39) \]
with:

\[ k = 1 \sim 6 \text{ six modes of motion respectively} \]
\[ j = \text{mode of motion coupled with mode of motion } k \]
\[ x_j = \text{motion in } j\text{-mode} \]
\[ M_{kj} = \text{Inertia matrix; the mass of the floating body in the direction of } k \text{ when the body makes a motion in the mode } j \]
\[ a_{ij} = \text{Added mass matrix; represents the coefficients of the component of fluid resistance proportional to the acceleration in the direction of } k \text{ when the floating body moves in the mode } j \]
\[ b_{ij} = \text{Wave damping matrix; coefficients of the component of fluid resistance proportional to the velocity} \]
\[ C_{ij} = \text{static restoring matrix} \]
\[ F_k = \text{External force in the } k\text{-mode} \]

The masses in these equations of motion consist of:

- solid masses or solid mass moments of inertia of the ship \((M_{kj})\) and
- “added” masses or “added” mass moments of inertia caused by the disturbed water, called hydrodynamic masses or mass moments of inertia \((a_{ij})\).

An oscillating ship generates waves, since energy will be radiated from the ship. The hydrodynamic damping terms \((b_{ij})\) account for this.

In the linear approach solving the equations of motion for every frequency and then integrating the results, can solve them in the frequency domain.

**Time domain**

However, in many cases the external forces \(F_k\) are not linear, and the mooring system shows non-linear behavior, which violates this linear assumption. Examples are non-linear viscous damping, forces and moments due to currents and wind, second order wave drift forces and non-linear characteristics of fender and mooring-line. If the system is non-linear, then the superposition principle - the foundation of the frequency domain approach - is no longer valid. Instead, the direct solution of the equations of motion as functions of time needs to be reverted. These equations of motion result from Newton’s second law.

The hydro mechanical reaction forces and moments, due to time varying ship motions, can be described using the classic formulation given by Cummins [ref 19]. In order to describe this non-linear behavior, of the impulse response theory can be used, which in it turn describes the fluid reactive forces. The equation of motion, to be solved in the time domain, is denoted as:
in which:

\[ k = 1 \sim 6 \text{ six modes of motion respectively; surge, sway, heave, roll, pitch, yaw} \]
\[ j = 1 \sim 6 \text{ mode of motion coupled with mode } k \]
\[ X_j = \text{motion in } j\text{-mode} \]
\[ M_{kj} = \text{Inertia matrix} \]
\[ m_{kj} = \text{Added mass matrix} \]
\[ R_{kj} = \text{matrix of retardation functions} \]
\[ C_{jk} = \text{static restoring matrix} \]
\[ F_{k} = \text{In time varying external force in the } k\text{-mode} \]

As it can be convenient to keep the left hand side of the equation motion linear, all the non-linear effects can be transferred to the opposite side, where they all form a part of the external force \( F(t) \).

The derivation of this equation is given in Appendix G. The only basic assumption in the approach is the separate treatment of the hydrodynamic reactive forces and all other external loads.

### 5.1.1 Solving the equations of motion by SDF simulation

In order to solve equation (40), a time-domain analysis has to be carried out, based on the input of time-varying external forces \( F_k(t) \). TERMSIM, developed by the Maritime Research Institute Netherlands (MARIN), is a simulation program, used to calculate vessel motions and forces in mooring lines and fenders. It prepares a time series of external forces for each mode of freedom by numerical simulation techniques based on the spectra of forces and moments exerted on the vessel by wind, waves and currents.

Usually many coefficients from equation (40) can be neglected. The equations of motion for a moored LNG-carrier to a jetty is given by the following set of equations, in which the mooring forces consist of both mooring line and fender (equations 41 – 47):

\[
m\ddot{X}_i + \sum_{k=1}^{6} a_{ik} \dot{X}_k + \sum_{k=1}^{6} \int_0^t R_{ik}(t-\tau) \dot{X}_k(\tau) \cdot d\tau + b_{i1} X_1 = F_{i}^{\text{wind}} + F_{i}^{\text{current}} + F_{i}^{\text{wave}} + F_{i}^{\text{moor}}
\]
in which:

\( t \sim 6 \) = Respectively surge, sway, heave, roll, pitch yaw (k-direction)

\( m \) = Mass displacement of the ship

\( I_{kk} \) = Moment of inertia in the k-mode

\( a_{kj} \) = Frequency dependent hydrodynamic mass coefficient

\( b_{kk} \) = Additional viscous damping coefficient

\( c_{kj} \) = Hydrostatic restoring coefficient

\( \tau \) = Time lag

\( F_{j}^{\text{wind}} \) = Wind force in k direction (k = 1,2,6)

\( F_{j}^{\text{current}} \) = Current force in k direction (k = 1,2,6)

\( F_{j}^{\text{wave}} \) = Wave drift force in k direction (k = 1,6)

\( F_{j}^{\text{moor}} \) = Mooring force due to fenders and mooring legs in k direction (k = 1,6)

The hydrodynamic loads in this thesis only consist of the wave forces; the forces due to current and wind loads are not considered.

Figure 5-2 presents the calculation procedure that has to be evaluated for each time-step. The limitations, in terms of the integration of site-specific wave parameters, of such a ship motion simulation program are discussed in section A-2 of Appendix A. The gray outlined area, namely the computer module which generates the input wave parameters and the eventual wave forces, has to be re-designed.
Figure 5-2: Flow diagram SDF-program TERMSIM
6 Hydrodynamic loads

6.1 Introduction

In a sea state, as described in chapter 3, different wave phenomena occur, resulting in different kinds of wave forces affecting the carrier's movements. Taking only the first order wave forces in consideration will not be sufficient, regarding a moored ship's movement. The second order wave forces also influence the ship response, especially the low-frequency drift forces, which may be close to the natural frequency of the ship. However, the contribution of the difference second order potential must be taken into account in order to complete the low-frequency wave forces. Thus the time-varying external wave forces are divided in three categories: first order wave forces, second order waves forces due to wave set-down and the low-frequency drift forces, which are related to the phenomena of wave grouping. This section describes the above-mentioned division and the derivation of the wave forces acting on the ship's hull. The time series of the first - and second order wave forces are prepared for each mode of freedom based on Strip Theory Method calculations. The wave drift force is determined using the Newman's approximations; the varying drift forces are calculated using mean drift force transfer functions in regular waves. These methods will be explained in section 6.3.2.

Figure 6-1: Wave phenomena and resulting wave forces

To start, the strip theory method will be discussed.
6.2 Strip Theory Method

The strip theory solves the three-dimensional problem of the hydro mechanical and exciting wave forces and moments on the ship by integrating the two-dimensional potential solutions over the ship’s length. The strip theory considers a ship to be made of a finite number of transverse two-dimensional slices, which are rigidly connected to each other. The total wave force acting on each cross-section is determined by evaluating the six forces, which originate from the six modes of motion. Integrating these total wave forces over the length of the floating body, leads to the total wave force acting on the vessels hull.

\[ F_k = \int_{L}^{F_k} \cdot dx, \quad k = 1, 2...6 \]

(48)

Each of the slices will have a shape which closely resembles the segment of the ship which it represents, and is treated hydro dynamically as if it is a segment of an infinitely long floating cylinder, see Figure 6-2.

Two different loads act on the floating body: the waves, produced by the oscillating ship (hydro mechanical loads) and the diffracted waves (wave loads). These are assumed to travel perpendicular to the middle line plane (parallel to the \( y_s, z_s \) plane) of the ship. This means that the fore and aft side of the body, do not produce waves in the \( x \)-direction. For the zero speed case, interactions between the cross sections are ignored as well.

The separate treatment of each cross section simplifies the coupling to a wave model. At each component of the cross section, the incident wave field is introduced as a summation of individual linear free surface waves. This implies that differentiation of amplitude, direction and phase-angle along the length of the ship is possible, causing a high applicability at terminals located in a strongly varying wave field. The wave parameters of the wave field behind the breakwater are introduced at each cross-section, at the "aft-side" of the section.
Hydrodynamic loads

6.2.1 Relative motion approach

According to the classical strip theory, the exciting wave moments for the six modes on a restrained cross-section of a ship in waves, are based on the relative motion principle. The exciting wave loads were found from the loads in undisturbed waves – the so-called Froude-Krilov forces or moments – completed with diffraction terms, accounting for the presence of the ship in these waves. The principle states that the force generated by diffraction of a floating body in waves equals an equivalent oscillation of the body in still water: the equivalent accelerations and velocities in the undisturbed wave are used to determine the additional wave loads due to diffraction of the waves. They are considered as potential mass and damping terms, as applied for the hydro mechanical loads.

The forces are obtained by evaluating the following equations:

Surge
\[ F'_{w1} = F'_{FK1} + m_{11} \cdot \dot{\zeta}_{w1} + n_{11} \cdot \ddot{\zeta}_{w1} \] (49)

Sway
\[ F'_{w2} = F'_{FK2} + m_{22} \cdot \dot{\zeta}_{w2} + n_{22} \cdot \ddot{\zeta}_{w2} \] (50)

Heave
\[ F'_{w3} = F'_{FK3} + m_{33} \cdot \dot{\zeta}_{w3} + n_{33} \cdot \ddot{\zeta}_{w3} \] (51)

Roll
\[ F'_{w4} = F'_{FK4} + m_{42} \cdot \dot{\zeta}_{w2} + n_{42} \cdot \ddot{\zeta}_{w2} + X'_{w2} \cdot \overline{O_G} \] (52)

Pitch
\[ F'_{w5} = -F'_{w1} \cdot \overline{bG} - X'_{w3} \cdot x_b \] (53)

Yaw
\[ F'_{w6} = F'_{w2} \cdot x_b \] (54)

where:
\[ \dot{\zeta}_{w1} \] = Equivalent velocity of the water particles in direction k [m/s]
\[ \ddot{\zeta}_{w1} \] = Equivalent acceleration of the water particles in direction k [m/s²]
\[ \overline{O_G} \] = Height centre of gravity above free surface [m]
\[ \overline{bG} \] = Distance between sectional buoyancy point and CoG [m]
\[ F_{FKj} \] = Froude-Krilov force [N]

In beam waves the strip theory method could lead to erroneous results. In the strip theory method, the floating body is assumed to be a cylinder in still water. Considering a restrained body, the total force is assumed to be the sum of the Froude-Krilov force \( F_{FK} \) and the force exerting on a cylinder in still water \( F(\zeta_{wd}) \), which is oscillating in the opposite direction of the particle motions. In case of heave these particle motions \( \zeta_{wd} \) are averaged over width of the carrier. For long waves this approach is plausible, whereas in short wave the velocities
Hydrodynamic loads

vary over the width of the ship. The same yields for the sway motion: particle motions are averaged over the height of the carrier. In practice the velocities will decrease with increasing depth.

In order to overcome these errors the radiated wave approach could be used. This method overcomes the above-mentioned problems for beam waves. However, due to the assumptions made in this approach, it leads to inaccurate results for oblique waves. In consideration of uniformity, the radiated approach is not included in the model.

Appendix H lists the detailed formulae of the Strip Theory Method, the relative motion approach and the radiated wave approach. It also gives an illustration of the ship-related notations used in the strip theory method. For further literature background reference is made to the theoretical manual of Seaway. [ref. 20]

6.3 Wave forces

Wave loads on a moored ship, as well as the ship response, can be split into several components. The first order wave forces have the same frequencies as the waves and the amplitudes are linearly proportional to the wave amplitudes. Forces with both higher and lower frequencies than the wave frequencies are second order wave forces, which are proportional to the square of the wave amplitudes. Low-frequency second order wave forces have frequencies that correspond to the wave group frequencies, present in an irregular wave field. The forces, consisting of a time varying and a non-zero mean component are called wave drift forces.

A moored floating structure can be considered as a nonlinear system. Analysis of the horizontal motions of a moored ship in a seaway show that the response of the structure on the irregular waves include three important components [ref. 21]:

First order:
1. An oscillating displacement of the ship at frequencies corresponding to those of the waves, caused by first order wave forces.

Second order:
2. A mean displacement of the structure, resulting from a constant load component, caused by the mean wave drift force.
3. An oscillating displacement of the structure at frequencies much lower than those of the irregular waves, caused by low-frequency wave drift forces. These are caused by non-linear (second order) wave potential effects, also referred to as wave set-down.
Although the second order forces are much smaller then the first order forces, they are of importance due to the long periods of the slowly varying part.

### 6.3.1 First order wave forces

The contribution of the first order wave forces \( F_{F}^{(1)} \) to the total wave force can be determined using the first order wave parameters and the theory from section 6.2 and Appendix H. The time-averaged value of this wave load and the resulting motion component are zero.

### 6.3.2 Second order wave forces

The ship motions excited by the second order wave forces, the slow-drift motions, are resonance oscillations excited by non-linear interaction effects between waves and the body motion. Slow-drift motions are of equal importance as the linear first-order motions in design of mooring systems for large volume structures. Generally, a moored ship has a low natural frequency in its horizontal modes of motion as well as very little damping at such frequencies. Resonance can lead to very large motions, which could dominate the ship's dynamic displacement. For a moored structure, slow-drift resonance oscillations occur in surge, sway and yaw.

The second order wave forces are generally referred to as wave drift forces. The total drift force consists of a slowly varying drift force (the low-frequency wave drift force) around a mean value (the mean wave drift force).

**Low-frequency wave drift forces**

In a sea state, the wave amplitude provides information about the slowly varying wave envelope of an irregular wave train. The wave envelope is an imaginary curve joining successive wave crests (or troughs); the entire water surface motion takes place within the area enclosed by these two curves. This requires a spectral analysis of the square of this wave envelope. In other words: the spectral density of the square of the wave amplitude gives information about the mean period and the magnitude of the slowly varying wave drift force.

Based on the phenomenon on wave grouping, a general formula of the slow-drift excitation loads, can be derived in terms of the second order transfer functions [ref. 22], (Faltinsen):

\[
F_{k}^{\text{drift}} = \sum_{j=1}^{N} \sum_{k=1}^{N} \zeta_{j}^{*} \zeta_{j} \left( P_{ij} + iQ_{ij} \right) \cdot e^{j\left((\omega_{ij} - \omega_{k})t + (\xi_{ij} - \xi_{k})\right)}
\]

(55)
in which $P_{ij}$ and $Q_{ij}$ can be interpreted as second-order quadratic transfer functions for the difference frequency loads:

$P_{ij}$ = in-phase part of second-order transfer function $P(\omega_i,\omega_j)$

$Q_{ij}$ = out-of-phase part of second-order transfer function $P(\omega_i,\omega_j)$.

Newman [ref. 23] stated that the quadratic transfer function (QTF) is equal to the average of the mean wave drift forces. His approximation implies that an off-diagonal QTF term formed by two frequencies can be approximated by the main diagonal term at the average frequency of the two:

$$P(\omega_i,\omega_j) = \frac{P(\omega_i,\omega_i) + P(\omega_j,\omega_j)}{2}$$

$$Q(\omega_i,\omega_j) = 0$$

Resulting from his findings the formula of the low-frequency wave drift force can be reduced to:

$$F_k^{\text{drift}} = \sum_{j=1}^{N} \sum_{k=1}^{N} \xi_j \xi_j' \left( \frac{P_{ij} + P_{ji}}{2} \right) e^{i(\omega_i - \omega_j) t + (\xi_i - \xi_j)}$$

in which the QTF's $P_{\parallel}$ and $P_{\perp}$ represent the mean drift forces in a regular wave field, i.e. without second order waves.

In shallow water the second order wave effects become more important, because the second order potential to the varying drift forces becomes significant. The contribution of the second order potential to the quadratic transfer functions is zero at the main diagonal and non-zero for the off diagonal terms and thus neglected in Newman’s approximation: The QTF’s are obtained without the contribution of the second order potential.

The contribution of the second order potential to the total wave force will not be introduced by means of the quadratic transfer functions. Its contribution will be considered as a linear second order wave acting on the hull of the vessel.

**Contribution of the second order potential**

Especially when the water depth is finite, the contribution of the second order potential is considerable. The contribution of the second order potential to the wave forces is determined by the phenomenon of wave set down.
In shallow water, irregular incoming waves exhibit the wave set-down phenomenon. This non-linear effect appears as long waves bound to the incoming short waves. Set-down wave elevations are related to second order pressures in the wave field, which in shallow water, is dominated by second order potential effects. The phase of this long wave - relative to the wave group - is such that it has a trough where the wave group attains its maximum wave elevation and a crest where it attains its minimum elevations.

An example of wave set-down can be seen in Figure 6-3.

![Figure 6-3: Wave set down](image)

Based on an idealized sea state consisting of two wave components of frequencies $\omega_i$ and $\omega_j$, and directions $\theta_k$ and $\theta_l$, the second order potential in a bi-chromatic wave group is:

$$
\phi^{(2)} = \sum_{ij} A_{ijkl} \cos(k_i \cos \theta_i - k_j \cos \theta_j) \sin(\omega_i t - \omega_j t - (\omega_i - \omega_j) t)
$$

and has to satisfy the free surface boundary condition, which is:

$$
g \phi_z^{(2)} + \phi_y^{(2)} = -2 \nabla \cdot \phi^{(1)} \cdot \nabla \phi^{(1)} + \phi_t^{(1)} \left( \phi_{zz}^{(1)} + \frac{1}{g} \phi_{zz}^{(1)} \right).
$$

A formulation of $A_{ijkl}$ can be derived by satisfying the free surface condition (Huijsmans [ref. 24])

$$
A_{ijkl} = \frac{1}{2} \left( \omega_i - \omega_j \right)^2 \left( k_i - k_j \right) g \tanh \left( k_i - k_j \right) g^2
$$

with:
Hydrodynamic loads

\[ B_{ij} = \frac{k_i^2}{\omega_i \cosh^2 k_i h} - \frac{k_j^2}{\omega_j \cosh^2 k_j h} \]  

(61)

\[ C_{ijkl} = \frac{2k_i k_j (\omega_i - \omega_j)(\cos(\theta_i - \theta_j) + \tanh k_i h \tanh k_j h)}{\omega_i \omega_j} \]  

(62)

Subsequently the low frequency components on the free surface determine the total wave set-down surface elevation:

\[ \zeta^{(2)} = -\frac{1}{g} \phi_z^{(2)} - \frac{1}{2g} \nabla \phi^{(1)} \cdot \nabla \phi^{(1)} + \frac{1}{g} \zeta^{(1)} \phi_z^{(1)} \]  

(63)

As the contribution of the quadratic terms of the first order quantities already have been taken into account in the calculation of the low-frequency drift forces by the quadratic transfer function \( P_{ij} \), only the contribution of the second order potential is considered. As a consequence the second order free surface elevation is denoted as:

\[ \zeta^{(2)} = \sum_{i=1}^{2} \sum_{j=1}^{2} \sum_{k=1}^{2} \sum_{l=1}^{2} \frac{1}{g} \rho^{(1)} (a_{ijkl} A_g, i) e^{i(k_i \cos \theta_i - k_j \cos \theta_j) x + (k_i \sin \theta_i - k_j \sin \theta_j) y - (\omega_i - \omega_j) t} \]  

(64)

**Dispersion relation**

The incoming wave that results from the low frequency second order potential have a wave number equal to \( k_i^{(2)} = k_i^{(1)} - k_j^{(1)} \) and a wave frequency of \( \omega^{(2)} = \omega_i^{(1)} - \omega_j^{(1)} \). These waves do not satisfy the dispersion equation. If the incoming waves have a frequency of \( \omega^{(2)} = \omega_i^{(1)} - \omega_j^{(1)} \), then the diffracted waves have the same frequency. The wave number follows from the relationship:

\[ \left( \omega_i - \omega_j \right)^2 = kg \cdot \tanh kh \]  

(65)

In order to simplify the situation, the diffracted waves are assumed to have the same wave number \( k^{(2)} = k_i^{(1)} - k_j^{(1)} \) as the incoming waves. Problems refer to a situation where the wave exciting force on the body due to a wave, which has a velocity potential given by equation (58), has to be determined. Diffracted waves have the same wave number as incoming waves. This is solved by considering the ordinary first order wave exciting force \( F^{(1)} \) on the body in a regular wave with wave number equal to \( k^{(2)} = k_i^{(1)} - k_j^{(1)} \) in an ordinary gravity field. For such a case the associated wave frequency \( \omega \) will be in accordance with the dispersion relationship equation. The frequency of the second order wave can be made
equal to the frequency \( \omega^{(2)} = \omega^{(1)} - \omega^{(1)}_j \) of the second order waves by selecting a different value for the acceleration of gravity:

\[
g^*_{ij} = \frac{(\omega_i - \omega_j)^2}{(k_i - k_j) \cdot \tanh(k_i - k_j) h} \quad [\text{ref 21}] (66)
\]

Actually, this is no more than just a trick in order to fit the second order frequency to the second order wave number in order to fulfill the dispersion relationship.

The adaptation of the acceleration gravity implies different corresponding hydrodynamic coefficients for the bound long waves. Regarding the model presented in this thesis it is not useful to determine these new hydrodynamic coefficients, as the waves due to wave set down will be regarded as free long waves while passing the breakwater. However, in order to compare the results of the WSP model with TERMSIM simulations without the presence of a breakwater, the parameter \( \omega^* \) is introduced:

\[
\omega^{*(2)} = \sqrt{k^{(2)}_g \cdot \tanh k^{(2)} h} \quad (67)
\]

\( \omega^* \) is used to determine the hydrodynamic coefficients: in the same matrices but with different frequency.

Figure 6-4: Determination added mass coefficient using \( \omega^* \)

The solutions for \( \omega^{(2)} = \omega^{(1)} - \omega^{(1)}_j \) and \( \omega^{*(2)} \) are both represented in the model. The first has to be used with the presence of a breakwater, because the set-down waves are then regarded as long free waves. \( \omega^{*(2)} \) can only be used incase of bound waves, i.e. without the presence of the breakwater.
For usage in relatively shallow water the wave set-down can be considered as a long wave and subsequently the pressure is hydrostatic and the vertical particle velocities can be neglected.

The contribution of the second order waves to the total wave force is therefore introduced as a linear wave exerting on the ship. The wave forces resulting from the contribution of the second order potential \( F_{k}^{(2)} \) is treated equal to the first order wave forces, i.e. using the strip theory method. These are referred to as "second order waves" in the model.

Since the wavelength of the second order waves due to the phenomenon of wave set down is considerably long, and the breakwater has less influence on reducing the energy of low-frequency waves, it is likely that contribution of the second order waves on the moored ship response will be significant.

### 6.4 Total wave force

The total wave force exerting on the ship's hull is the summation of the different wave force contributions:

\[
F_{k}^{total} = F_{k}^{(1)} + F_{k}^{(2)} + F_{k}^{dref}, \quad \text{with } k = 1, 2...6 \quad (68)
\]

An explanation of the method presented in this section is also described by W. van der Mole, [ref. 25]
7 Wave Simulation Program

7.1 Model set-up of the Wave Simulation Program

Simulation of the new LNG marine terminal concept requires a model set-up that functions as a stepping-stone. The general requirements, project site schematization, co-ordinate system conventions and of the model are presented in this chapter. The model's set-up is given in Figure 7-1:

![Figure 7-1: Model set-up Wave Simulation Program](image)

7.1.1 Requirements of WSP

As a supplement on the currently available ship motion simulation models the Wave Simulation Program must be able to calculate:

- Directional spreading of the waves
  
  *Introducing spectra, defined in both frequency- and directional domain*

- Modified near shore wave parameters, under influence of a breakwater
  
  *Introducing wave parameters which are changed under influence of a breakwater (diffraction, transmission), and/or coast (reflection)*

- Ship-length varying input wave parameters
  
  *Introducing time-series of wave forces which originate from calculations by the Strip Theory Method*

In addition the program must be:

- Well applicable in the preliminary design stage of a LNG mooring facility
- Easy to incorporate in a current ship motion simulation program
7.1.2 Schematization of the model

**Schematization of the project site**
The first step is to schematize the project site. Figure 7.2 presents the project site schematization.

---

**Figure 7-2: Project site schematization**

As can be seen in Figure 7.2, the different areas of the model are defined as follows:

**Offshore area**
The area at the seaside of the breakwater
The offshore wave field is defined by the offshore wave parameters

**Near shore area**
The area between LNG carrier and breakwater and the LNG carrier and the coastline
The modified wave field is defined by the near shore wave parameters

**Ship’s hull**
The hull of the LNG carrier
The wave forces are defined by the near shore wave parameters

Furthermore, point P and Q denote the breakwater-ends. Here they will be referred to as:

**Point P** = right breakwater-end
**Point Q** = left breakwater-end
Schematization of the wave transformation phenomena

Figure 7-3 shows the schematization of the wave transformation phenomena. Diffraction, reflection and transmission are treated separately, without influence on each other.

Diffraction
The incoming waves bend around the two breakwater-ends. Given that the waves, coming from breakwater-end P and Q, have different wave characteristics (amplitude $\zeta$, phase $\phi$ and direction $\mu$), they are treated separately at the hull of the LNGC. The breakwater is assumed to be impermeable.

Reflection
In practice the reflected waves at the coast will be re-reflected at the breakwater. Given that these re-reflected waves would not influence the ship motions substantially, due to dissipation in the breakwater, this will not be included in the model. Since the reflection coefficient is dependent on the Irribarren number, it also depends on the wavelength. Longer waves have a higher coefficient. With the aim of integrating this aspect in the model, the reflection coefficient is chosen to be dependent on the wavelength.

Transmission
As has been shown previously, the transmission formulae do not provide the possibility to include an incident direction spectrum. So the following assumption has been made to include the effect of oblique wave incidence in the transmission:

$$K_{t,\theta} = K_t \cdot \sin(\theta_0)$$  \hspace{1cm} (69)

The transmission formulae are empirically derived for short waves. Therefore only transmission of the first order waves will be taken into account.

7.1.3 Coordinate system conventions
**Earth bound coordinate system** $S(x_0, y_0, z_0)$

A right-handed Cartesian co-ordinate system $S(x_0, y_0, z_0)$ is set in space. The $(x_0, y_0)$-plane lies in the still water surface. The $x_0$-axis is set parallel to the shoreline, with the positive $x_0$-direction pointing to the right. The $y_0$-axis is directed perpendicular to the $x_0$-axis, pointing in landward direction and $z_0$ is directed upwards.

![Figure 7-4: Earth bound coordinate system](image)

**Ship-bound coordinate system** $CoG(x_s, y_s, z_s)$

With respect to the ship another right-handed Cartesian co-ordinate $CoG(x_s, y_s, z_s)$ system is fixed in space. This system is connected to the ship with its origin at the ship’s center of gravity, $CoG$. The directions of the positive axes are: $x_s$ in the longitudinal forward direction, $y_s$ in the lateral direction and $z_s$ upwards. If the ship is floating upright in still water, the $(x_s, y_s)$-plane is parallel to the still water surface.

![Figure 7-5: Ship-bound coordinate system](image)
**Breakwater-bound coordinate systems** $P_p(r, \theta, z)$, $P_q(r, \theta, z)$

In order to transform the off-shore wave field in near-shore wave parameters, two different co-ordinate systems are set with respect to the breakwater. These co-ordinate systems are polar with the origins $P_p$ and $P_q$ lying at the breakwater ends. $r$ is the radial distance from the breakwater tip to the point where the modified wave characteristics are to be determined, and $\theta$ is the angle between the breakwater-end and this radial, $z$ is pointing upwards. The $(r, \theta)$-plane lies in the still water surface.

![Figure 7-6: Breakwater bound coordinate system](image)

Integration of the three coordinate systems is shown in Figure 7-7:

![Figure 7-7: Integration coordinate systems](image)

**7.2 Computer simulation program**
7.2.1 Introduction

A computer simulation program has been developed to perform the calculations on the wave field behind the breakwater and the wave forces acting on the ship’s hull. The program runs under MatLab®, which itself is a computer program that is considerably adapted for making calculations with large matrices.

The set-up of the model has been explained in section 7.1. In this section, the structure of the computer program will be clarified. First the required input data for the program is treated, followed by the different calculation procedures that eventually lead to a modified wave field and wave forces. An output procedure arranges these results in order to produce usable output data. Subsequently the output is integrated in a TERMSIM simulation. Finally the results can be assessed using the assessment tool. This basic procedure of the program is illustrated in Figure 7-8.

Figure 7-8: Flow scheme WSP

7.2.2 Input Files

The input data for the calculations in the simulation module are to be entered in six different input files in MatLab, called M-files. These six M-files are:

- Input_menu
  i.e.: in the menu file, different options of simulation modes must be chosen. The WSP allows different types of simulations. These are specified in Appendix I.
- Input_wave_parameters
- Input_breakwater_parameters
- Input_ship_parameters
- Input_coastal_parameters
- Input_time

The exact content of the input files is described in Appendix I (User manual), and will not be discussed in this section. Note that all input parameters in the five input files are to be entered in SI-units.
7.2.3 Simulation Module

The simulation module of the program uses the input parameters to make all necessary calculations in order to determine the modified wave field parameters behind the breakwater and the wave forces on the ship's hull. This is done using a sequence of programs in MatLab. The main procedure in the simulation module is illustrated in Figure 7-9 where each cell represents a separate calculation module.

Figure 7-9: Flow scheme computer program WSP

The main procedure runs by the M-File 'start'. The procedure within each separate program is described in Appendix I (User manual). In this section, the task of each program shown in Figure 7-9 is only briefly described.
The subprograms in the main simulation procedure are:

Start                  Executes the main procedure of simulation
Hullform               Determines the hull-shape of the LNG carrier and reads the
                        hydro-dynamic file
Coordinates            Sets the local co-ordinates in the earth-fixed co-ordinate
                        system

Offshore:
Pierson_Moskowitz      Calculation of wave components by the evaluating the
                       Pierson-Moskowitz spectrum
Gauss_spectrum         Calculation of wave components by the evaluating the Gauss
                       spectrum
Jonswap_spectrum       Calculation of wave components by the evaluating the
                       JONSWAP spectrum
Set-down               Computes the set-down wave parameters

Nearshore:
Diffraction            Calculation of the diffraction coefficients and pattern using the
                       diffraction theory as described in section 4.2
Reflection             Calculation of the reflection coefficients and pattern using the
                       reflection theory as described in section 4.3
Transmission           Determines the transmission coefficient, using the
                       transmission theory as described in section 4.4

Ship's hull:
Hf-strip               Calculation of first order wave forces, using the strip theory
                       method, described in section 6.2 and Appendix H
Hf-strip2 n° 1         Calculation of second order wave forces, when considered as
                       free waves, see section 6.3.2
Hf-strip2 n° 2         Calculation of second order wave forces, when considered as
                       bound long waves, also see section 6.3.2
Drift                  Calculation of wave drift forces, theory is given in section
                       6.3.2

7.2.4 Output data

The Wave Simulation Program has been designed to provide site specific input data for the
ship motion simulation program. For introduction of the WSP-generated wave forces in
TERMSIM, the time series have to be set in a specific format. As seen in Figure 7-9 this is
done in the output file "shipfor.dat". This file is introduced by saving it in the current directory
of TERMSIM. So it can be considered as the link between the two simulation programs.
Appendix H shows an example of a shipfor.dat file. As WSP generates a large amount of
output parameters, the results are arranged in different types of format for validation and
evaluation purposes. These files, for instance contour plots of the diffracted and reflected wave fields or tables with relevant wave parameters, can be found in the current directory in which WSP is run. Appendix I gives a complete summary and examples of the output files generated by the WSP. Figure on the next presents the flow scheme of the parameters calculated by WSP.

7.2.5 Application of WSP calculations in TERMSIM

Figure 7-10 represents the flow diagram of the modified TERMSIM program. The numerical solution of the equation of motion given by equation (40) in section 5.1 is evaluated in the time domain for all Six Degrees of Freedom. The input of the wave data is replaced by WSP calculations by saving the "shipfor.dat" file in the current directory in which TERMSIM is run.

Figure 7-10: Flow diagram modified TERMSIM

A detailed description of TERMSIM and its hydrodynamic theory, is not discussed here, but can be found in Appendix J.

7.2.6 Ship response assessment tool

Using the assessment tool, provides knowledge of the ship response to the wave field. It provides the analysis of the spectra of motions, mooring line and fender forces. In addition it offers the possibility to investigate the relative increase or decrease of the motions and mooring forces as a function of varying input parameters.
8 Program validation

8.1 Introduction

The best way to validate the WSP program is to compare the output of the program with the results of a laboratory model test. As such a test is not available for the project specified in this research, i.e. a moored LNG carrier sheltered by a detached breakwater, other possibilities of validation have been used. This section outlines the validation process of the Wave Simulation Program and its application in TERMSIM. The main issues in this respect are: the validation of the modified near-shore wave parameters, validation of the wave forces generated by WSP and validation of a correct application of the WSP generated wave force time series in TERMSIM.

Firstly the near-shore wave parameters will be examined. Diffraction and reflection patterns of the modified wave field provide information about the near shore wave parameters. These can be compared with corresponding diagrams given in literature. Secondly the wave forces generated by the WSP are checked. A situation is considered without the presence of a breakwater. The results of the TERMSIM wave force calculations must be in line with the results obtained by WSP. Furthermore, the wave forces must equal the first order transfer functions provided by the hydrodynamic file. Thirdly it has been evaluated whether TERMSIM reads in the wave force time series correctly. Lastly the results with the presence of a breakwater are validated. Comparing the results with DELFRAC simulations, where diffraction around the breakwater is considered, provides this validation. The validation process is schematised in Figure 8-1. All simulation runs, which have been performed for this validation, are tabulated in Appendix K.

---

Figure 8-1: Validation process
8.2 Validation transformation of wave parameters

The validation of diffraction and reflection in fact is a validation and verification of the program. It is set-up as follows: first a comparison of the diffraction coefficients is made with a corresponding diagram given in literature. Then the program is tested to calculate the diffraction and reflection patterns for different angles of wave incidence, both for semi-infinite and detached breakwaters.

8.2.1 Diffraction

**Semi-infinite breakwater**

For the validation of the diffracted wave parameters it is useful to compare the wave field patterns to diffraction diagrams given in literature, for instance by Wiegel [ref 31]. However, the output provided by the WSP program is set in a Cartesian coordinate system, whereas the diagrams provided by Wiegel are drawn in polar coordinates. One diagram is available for lateral coordinates, and it is shown in Figure 8-2. It represents the diffraction coefficients for an incoming wave field with wave incidence of an angle of 90° and a semi-infinite impermeable rigid breakwater. Figure 8-3 shows the results the WSP pattern under the same circumstances.

---

**Figure 8-2: Diffraction 90° after Wiegel**

**Figure 8-3: Diffraction 90° WSP**

**Figure 8-4: Diffraction 45° WSP**

**Figure 8-5: Diffraction 135° WSP**
Figure 8-4 and Figure 8-5 show the diffraction coefficients for wave incidence $\theta_o = 45^\circ$ and $135^\circ$ respectively. These are enclosed in order to show the patterns in case of oblique wave incidence.

An interesting feature demonstrated by Figures 8-2 – 8-5, is that for any approach angle, the value of the diffraction coefficient along a line in the lee of the breakwater that extends from the breakwater tip in the direction of the approaching wave is approximately 0.5. Note also that for a given location in the lee of the breakwater, a one-dimensional spectrum of waves that comes from the same direction will undergo a greater decrease in height (energy density) for successively higher frequency waves in the spectrum. Thus the diffracted spectrum will have a shift in energy density towards the lower frequency portion of the spectrum.

**Detached breakwater**

Figure 8-6 represents the diffraction patterns of an impermeable rigid detached breakwater with a total length of $L = 700$ m (about two times the length of a LNG-carrier). The wavelength of the monochromatic wave $\lambda$ is 30 m. The patterns are shown for two angles of wave incidence $\theta_o = 45^\circ$ and $135^\circ$.

The diffraction coefficients coming from the breakwater tips show values higher than 1, indicating an increase of the surface elevation. The figures also illustrate the strongly varying wave field.

![Figure 8-6: Diffraction around a detached breakwater, wave incidence 45° and 135°](image-url)
8.2.2 Reflection

Semi-infinite breakwater
The reflected wave equals the diffracted wave; mirrored at the coast and multiplied by the reflection coefficient, see Figure 4-3 in section 4.3. The wave field pattern obtained by a reflection calculation for a semi-infinite breakwater, under wave incidence of $\theta_o = 45^\circ$ and $135^\circ$ are presented in Figure 8-7. The contours shown in the reflection plots have a reflection coefficient of $K_r = 1$.

wave incidence $\theta_o = 45^\circ$

Detached breakwater
illustrates the reflection at the coast when in case of a single detached breakwater for wave incidence $\theta_o = 45^\circ$.

wave incidence $\theta_o = 45^\circ$

Figure 8-7: Reflection at the coast, semi-infinite breakwater, wave incidence 45°

Figure 8-8: Reflection at the coast, detached breakwater, wave incidence 45°
In case of oblique wave incidence, the plots show a reflected relative wave height of 1.1 to 1.2 over the whole area. It can be concluded that the influence of reflection could be very important for the response of the moored LNGC in case of a high reflection coefficient defined by coastal characteristics.

**Comparison hydraulic model tests**
For the construction of the breakwater at Limbe in Cameroon, a wave diffraction analysis has been carried out. Details about this study can be found in *“Project construction of breakwater for Limbe shipyard”,* [ref 26]. During a 3D model test, wave measurements have been made at various locations in the basin. These measurements have been used to calibrate the numerical model ANSYS.

**ANSYS**
The calculations have been made using the finite element model ANSYS version 6.1. In this 2-D model the effect of wave reflection against structures, refraction due to local bathymetry and diffraction around objects are included. The model does not include energy dissipation due to wave breaking. Output of the program is given in relative wave heights.

**Simulation runs**
The offshore wave height is always equal to 1 m, whereas the wave period is the actual wave period. Two ANSYS calibration runs were carried out for the calibration of the model with the physical model tests of Delft Hydraulics. Therefore these runs do not include the bathymetry and the reflecting wall, which has been assumed to be fully absorbing.

**Table 1: Simulation runs calibration**

<table>
<thead>
<tr>
<th>Run</th>
<th>Water level</th>
<th>Wave height $H_s$</th>
<th>Wave period $T_p$</th>
<th>Wave direction</th>
</tr>
</thead>
<tbody>
<tr>
<td>01</td>
<td>MSL: +2.4-0 m</td>
<td>1 m</td>
<td>14 s</td>
<td>65°</td>
</tr>
<tr>
<td>02</td>
<td>MSL: +1.50 m</td>
<td>1 m</td>
<td>10 s</td>
<td>90°</td>
</tr>
</tbody>
</table>

Figure 8-9 shows the project site and the output locations (1, 2, 3) of the physical model tests.
Figure 8-9: Project site and output locations breakwater in Limbe

Figure 8-10 demonstrates the diffraction patterns of simulation run 02. The left figure shows the results of WSP, the figure at the right side shows the results of ANSYS.

Comparison with model test
Since the numerical model tests are based on monochromatic waves, the wave pattern is predicted accurately in terms of nodes and anti-nodes. Therefore the numerical results have been obtained for areas of 50 m x 50 m around the output locations. Because the wave heights vary within these areas, the minimum, mean and maximum wave heights are determined. The comparison has been made based on both $H_{m0}$ and $H_{1/3}$. The measurements of Delft Hydraulics of these different wave heights show small deviations.

Table 2 and Table 3 show the comparison of the ANSYS and WSP results with the measured relative wave heights.
Program validation

Table 2: Results run 01, model tests Delft Hydraulics, ANSYS and WSP

<table>
<thead>
<tr>
<th>Location</th>
<th>Measured</th>
<th>ANSYS results</th>
<th>WSP results</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$H_m^0$</td>
<td>$H_{1/3}$</td>
<td>Min</td>
</tr>
<tr>
<td>$T_p = 14$ s $\theta = 65^\circ$</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>0.6</td>
<td>0.5</td>
<td>0.4</td>
</tr>
<tr>
<td>2</td>
<td>1.0</td>
<td>1.0</td>
<td>0.7</td>
</tr>
<tr>
<td>3</td>
<td>0.5</td>
<td>0.5</td>
<td>0.4</td>
</tr>
</tbody>
</table>

Table 3: Results run 02, model tests Delft Hydraulics, ANSYS and WSP

<table>
<thead>
<tr>
<th>Location</th>
<th>Measured</th>
<th>ANSYS results</th>
<th>WSP results</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$H_m^0$</td>
<td>$H_{1/3}$</td>
<td>Min</td>
</tr>
<tr>
<td>$T_p = 10$ s $\theta = 90^\circ$</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>1.0</td>
<td>0.9</td>
<td>0.5</td>
</tr>
<tr>
<td>2</td>
<td>1.5</td>
<td>1.5</td>
<td>1.3</td>
</tr>
<tr>
<td>3</td>
<td>0.6</td>
<td>0.6</td>
<td>0.5</td>
</tr>
</tbody>
</table>

The comparison of ANSYS and WSP to the measured data is given in Table 4.

Table 4: Comparison of ANSYS and WSP to Delft Hydraulic model tests

<table>
<thead>
<tr>
<th>Location</th>
<th>ANSYS to $H_m^0$</th>
<th>WSP to $H_m^0$</th>
<th>ANSYS to $H_{1/3}$</th>
<th>WSP to $H_{1/3}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Run 01</td>
<td>$T_p = 14$ s $\theta = 65^\circ$</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>100 %</td>
<td>111 %</td>
<td>105 %</td>
<td>107 %</td>
</tr>
<tr>
<td>2</td>
<td>87 %</td>
<td>89 %</td>
<td>92 %</td>
<td>95 %</td>
</tr>
<tr>
<td>3</td>
<td>93 %</td>
<td>97 %</td>
<td>100 %</td>
<td>100 %</td>
</tr>
</tbody>
</table>

| Run 02   | $T_p = 10$ s $\theta = 90^\circ$ |                |                    |                  |
| 1        | 80 %             | 85 %           | 70 %               | 78 %             |
| 2        | 103 %            | 103 %          | 102 %              | 102 %            |
| 3        | 111 %            | 111 %          | 109 %              | 109 %            |

The comparison between the model tests and the simulation programs shows that WSP underestimates the wave heights at location 2 of run 01 and location 1 of run 02. On the other hand, the underestimation is smaller than the results of ANSYS.

The difference between the simulation programs and the model test measurements could be explained by the fact that the simulations are based on monochromatic waves, whereas the wave heights of the hydraulic model tests are based on random waves. Overall, it can be concluded that WSP provides good assessment of the wave heights and generally the simulations lead to conservative results.
8.2.3 Transmission

In the input menu of the Wave Simulation Program the user may choose either a constant value of the transmission coefficient $K_t$ or allow the model to calculate values based on wave and structure characteristics. For the latter case, the program provides different transmission formula. Based on the characteristics of the terminal location and the breakwater, a suitable formula can be chosen. The LNG marine terminal needs a submerged breakwater as a consequence of visibility and safety considerations. As a result of the application for rubble mound and solid structures, with a relative low crest freeboard, the d’Angremond formula (equation 32) is very applicable. For that reason the modelling of the d’Angremond formula will be validated in this section.

Validation of the d’Angremond formula

Figure 4-6 in section 4.4.2 shows the dependency of the transmission coefficient to the dimensionless parameter $R_c/H_s$. Considering the d’Angremond formula, the parameters shown in Figure 8-11 influence the transmission coefficient:

![Figure 8-11: Parameters transmission](image)

$H_s$ = Incoming wave height [m]  
$R_c$ = Crest freeboard relative to SWL [m]  
$B$ = Crest width [m]  
$c$ = Permeability parameter [-]  
$\xi$ = Irribaren-parameter $= (\tan(\beta)/H_s/\lambda)^{0.5}$

Dependency on dimensionless parameter $R_c/H_s$

Several runs have been carried out with varying permeability coefficients $c$ [0.64-0.8], in order to determine whether the WSP simulations generate the same distribution as the empirical model tests show. In theory the transmission coefficient cannot exceed the range [0,1]. In practice, limits of about 0.1 and 0.9 are found. The limits in WSP are set on [0.05, 0.95]. This is illustrated in Figure 8.12
According to Figure 8-12 it follows that the transmission coefficient, calculated using the d’Angremond formula, approximates the empirical distribution over $R_c/H_s$.

Dependency on incoming wave direction $\theta_o$

Figure 8-13 illustrates the distribution of the transmission coefficient over the incoming wave direction for different relative crest heights. Decreasing the incoming wave angle $\theta_o$ with respect to the breakwater leads to lower values of the transmission coefficient $K_t$. It is given by the equation (69), see section 7.1.2:

$$K_{t,\theta} = K_t \cdot \sin(\theta_o)$$

Figure 8-13: $K_t$ vs incoming wave direction $\theta_o$
Dependence on peak period $T_p$ and breakwater width $B$

In Appendix L the dependency of the transmission coefficient on the breakwater width and its distribution over the frequency is shown. From this it can be concluded that the transmission coefficient can be considered independent of the wave period and frequency. In practice the frequency distribution of the transmission coefficient over the frequencies probably is significant. But this needs further research and is not included in the program.
8.3 Validation of wave forces

8.3.1 Introduction

This section presents the validation of the wave forces calculated by WSP. The strip theory method, previously described in section 6.2 has been used for the calculation of the wave forces exerting on the ship.

The First order Transfer Functions provided by the hydrodynamic file of the LNG carrier and the first order wave forces have been compared. Furthermore, a spectral analysis of the WSP wave force time series in comparison with the results of TERMSIM has been performed.

8.3.2 Vessel characteristics

A 276.15 m long LNG carrier was considered during the validation process. The main characteristics and stability data are presented in Table 5.

<table>
<thead>
<tr>
<th>Table 5: Carachteristics LNG carrier</th>
</tr>
</thead>
<tbody>
<tr>
<td>Symbol</td>
</tr>
<tr>
<td>Type of vessel</td>
</tr>
<tr>
<td>Type of tanks</td>
</tr>
<tr>
<td>Capacity</td>
</tr>
<tr>
<td>Length between perpendiculars</td>
</tr>
<tr>
<td>Breadth</td>
</tr>
<tr>
<td>Draft</td>
</tr>
<tr>
<td>Displacement volume</td>
</tr>
<tr>
<td>COG above keel</td>
</tr>
<tr>
<td>COG forward of station 10</td>
</tr>
<tr>
<td>Projected side area above waterline</td>
</tr>
<tr>
<td>Projected front area above waterline</td>
</tr>
<tr>
<td>Loading condition in % of max. draft</td>
</tr>
<tr>
<td>Water depth</td>
</tr>
</tbody>
</table>

The general arrangements of the carrier and the mooring configuration can be found in Appendix L.

The strip theory method calculations require a hull-shape file, which provides information about the carrier's hull shape at the cross-section of every strip. Since such a file was not
available for the carrier considered here, a hull-shape file has been made using the cross-sectional figure of a similar LNG-carrier. The hull shape thus obtained has been scaled to fit the LNG carrier used in the validation of the program. The hull-shape file, as well as the figure of the cross-sections, are enclosed in Appendix L. In the Appendix also plots of the added mass and damping coefficients are given.

8.3.3 First order Transfer Functions

The hydrodynamic file provides us information about the wave force First order Transfer Functions (FTF). These are transfer functions between height and phase of the waves and the amplitude and phase of the wave forces.

\[
FTF = \left| \frac{\mathbf{F}}{\zeta} \right| 
\]

\[
F_k = \sum_{\theta=j}^{M} \sum_{\phi=1}^{N} \left| \frac{\mathbf{F}}{\zeta} \right|_{\theta,j,k} e^{i(\theta_{j,k} + \phi_{k})} 
\]

For every combination of frequency and incoming wave direction the FTF’s are specified. The MARIN program DIFFRAC can determine these first order transfer functions. The results are part of the hydrodynamic file. The wave forces obtained in this way have to equal the wave forces derived by the Strip Theory Method.

Generally, the first order wave forces show a good agreement with the First order Transfer Functions. Especially the forces in the most important directions with regard to the mooring line forces, i.e. surge, sway and yaw approximate the values provided by the hydrodynamic file. Apparently, the deviations mainly are located nearby the peaks and troughs. In the higher frequency regions the strip theory method tends to deviate more from the transfer functions.

Figures 8-14 – 8-16 show the results of the comparison between the transfer functions given in the hydrodynamic file and the wave forces calculated using the strip theory method. The most important modes affecting the mooring line forces, in the directions \( \theta = 0^\circ, 15^\circ \), and \( 90^\circ \), are shown in the figures. The plots of the other modes and wave directions are shown in Appendix N.
Program validation

With regard to wave incidence $\theta_o = 0^\circ$, the plots of surge, heave and yaw are shown. The wave forces of the three other modes are negligible (see Appendix M). The surge motion is the most important motion, and shows close correlation to the FTF. The oscillation of the strip theory method is stronger over all frequencies.

Regarding the comparison for wave incidence $\theta_o = 45^\circ$, the sway and yaw motion are almost similar to the FTF's. Again, the stronger oscillation can be noticed in the plot of the surge motion.

With regard to wave incidence $\theta_o = 90^\circ$, the plots of surge, heave and yaw are shown. The wave forces of the three other modes are negligible (see Appendix M). The surge motion is the most important motion, and shows close correlation to the FTF. The oscillation of the strip theory method is stronger over all frequencies.
Figure 8-16 demonstrates that the wave forces, calculated for a wave incidence of 90°, correspond less with the transfer functions. The representation of the difference in surge direction, which is less important in this direction of wave incidence, is a little misleading. The difference between the transfer functions and wave forces is probably the result of the difference between the calculation methods used by WSP (strip theory method) and TERMSIM (panel-method). The strip theory method used by WSP considers the LNG carrier to be a long slender cylinder. The surge motion is determined by defining an equivalent longitudinal cross section, which is swaying. In case of beam waves the method assumes the surge motion to be zero. Regarding the panel-method, the panels are not only orientated in the x-direction, but also in the y-direction. For that reason it is likely that some surge motion occurs.

The sway and yaw are the most important motions in beam waves. As can be seen in Figure 8-16 the sway motion approximates the transfer function closely. However, the graph presenting the yaw motion shows less correspondence.

Theoretically, the yaw motion is likely to approach zero in beam waves. As the carrier’s shape is not symmetrical (i.e. the front and aft side of the vessel differ) some yaw motion is still noticeable. The yaw motion is one order smaller than the yaw motions in other wave directions, so indeed the yaw motion decreases. In section 6.2.1 it has been pointed out that the strip theory method could lead to erroneous results in beam waves, as the total wave force has to be completed by a diffraction part. This is probably the reason that the first order wave force of the yaw motion differs from the First order Transfer function.

Furthermore, the application for beam-waves in the overall model will be of less importance, as the waves are not likely to approach the moored LNG carrier under an angle of 90° when the breakwater is present.

### 8.3.4 Comparison of the wave force spectra

To form a better understanding of the wave forces calculated by WSP, a spectral analysis has been carried out. Using spectral parameters, the wave force spectra of TERMSIM and WSP have been compared for three angles of wave incidence (θ = 0°, 45° and 90°). From section 8.3.3 it follows that the deviation of the wave forces to the First order Transfer Functions is frequency dependent. Hence the wave forces are compared for different wave periods (T_p = 8 s, 13 s, 25 s) as well. The results of wave direction θ = 45° and peak period T_p = 13 s are presented in this section. Plots and tables of the other wave directions and periods can be found in Appendix 0.

**Parameters spectral analysis**
A series of characteristic numbers, called the spectral moments, is related to the spectrum. These numbers, denoted by $m_r$, $r=0,1...$ are defined as,

$$m_r = \int_0^\infty \omega^r S(\omega) d\omega$$

(71)

where $S(\omega)$ is the power density function of the wave forces. The spectral parameters are the combination of spectral moments. The mean frequency and spectral width are important parameters, because the distribution of the spectral density over the frequencies has a high influence on the ship response. They are defined as:

Mean frequency:

$$\bar{\omega} = \frac{m_1}{m_0}$$

(72)

Spectral width:

$$\nu^2 = \frac{m_r m_1}{m_1^2} - 1$$

(73)

The spectral width is the measure of spectrum concentration around the mean frequency. The spectrum is extremely narrow when $\nu^2 \to \infty$ but is wide when the spectral width increases.

Simulations runs

An overview of the simulation runs for the comparison of the wave force spectra is given in Table 6. All runs have been performed both by TERMSIM and WSP. The Pierson-Moskowitz distribution has been used to describe the offshore wave field. The duration $t$ of all simulations is three hours or 10,800 s.

For the calculation of the second order wave forces due to wave set-down the two solutions mentioned in section 6.3.2 are included in the program. Regarding the absence of a breakwater, the second order waves can be assumed as bound long waves, thus the second solution is used.

Table 6: Simulation runs validation wave forces

<table>
<thead>
<tr>
<th>N° run</th>
<th>$T_p$ [s]</th>
<th>$H_s$ [m]</th>
<th>$\theta_o$ [°]</th>
<th>$h$ [m]</th>
<th>Spectrum</th>
<th>$2^{nd}$ order waves</th>
</tr>
</thead>
<tbody>
<tr>
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<tr>
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<td>2.5</td>
<td>45</td>
<td>14</td>
<td>PM</td>
<td>bound</td>
</tr>
</tbody>
</table>

Spectral analysis
Figure 8-17 shows the wave force spectra of wave incidence $\theta_o = 45^\circ$

The spectral parameters of these spectra of wave forces are tabulated in Appendix N in Table N.4. Table 7 shows the difference in terms of percentage between TERMSIM and WSP

**Table 7: Difference spectral parameters TERMSIM and WSP, $\theta_o = 45^\circ$, $T_p = 13$ s**

<table>
<thead>
<tr>
<th>Difference TERMSIM - WSP</th>
<th>$\theta_o = 45^\circ$</th>
<th>Zero moment $m_0$</th>
<th>First moment $m_1$</th>
<th>Mean freq. $(\bar{\omega})$</th>
<th>Spectral width $(\nu^2)$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$F_{\text{SURGE}}$</td>
<td>- 25.0 %</td>
<td>- 23.2 %</td>
<td>2.6 %</td>
<td>- 5.4 %</td>
<td></td>
</tr>
<tr>
<td>$F_{\text{SWAY}}$</td>
<td>3.0 %</td>
<td>- 3.5 %</td>
<td>- 6.4 %</td>
<td>13.8 %</td>
<td></td>
</tr>
<tr>
<td>$F_{\text{HEAVE}}$</td>
<td>100 %</td>
<td>7.1 %</td>
<td>- 50 %</td>
<td>300 %</td>
<td></td>
</tr>
<tr>
<td>$F_{\text{ROLL}}$</td>
<td>9.5 %</td>
<td>12.4 %</td>
<td>2.8 %</td>
<td>- 6.6 %</td>
<td></td>
</tr>
<tr>
<td>$F_{\text{PITCH}}$</td>
<td>- 10.6 %</td>
<td>- 18.3 %</td>
<td>8.1 %</td>
<td>15.7 %</td>
<td></td>
</tr>
<tr>
<td>$F_{\text{YAW}}$</td>
<td>- 2.6 %</td>
<td>- 3.1 %</td>
<td>- 0.4 %</td>
<td>- 8.7 %</td>
<td></td>
</tr>
</tbody>
</table>
Comparing the spectra in Figure 8-17 with the First order Transfer functions, it is remarkable that a small deviation from the FTF leads to higher deviations in the spectra. Furthermore it is surprising to see that the sway motion spectrum shows the highest difference, whereas the comparison with the FTF’s showed very close resemblance. Nonetheless, the mean frequency and spectral width are almost for every motion within a margin of 15%

Heave is the only exception. It strongly differs from the TERMSIM calculations. Once more, this can be explained by the difference between the calculation methods used by TERMSIM and WSP. TERMSIM considers a floating vessel, which is lifted by the waves. As a result the wave forces acting on the hull are small regarding the heave motion. The strip theory method considers the vessel to be a restrained cylinder with an initial position, which is fixed in space (see section 6.2.1). The oscillating motion is calculated by means of the equivalent particle motions. Hence the resulting wave forces on the floating body are much higher. However, the effect of this relatively high wave force is reduced by the hydrostatic restoring component of the equation of motion of heave (see equation 43). Therefore the response will not differ from the TERMSIM calculations to a large extend. Besides that, the heave motion is of minor importance concerning the mooring line forces.

The spectra of motion of the other runs demonstrate that the mean frequency $\bar{f}$ and the spectral width $\nu^2$ are generally within a margin of 20-30% in comparison with TERMSIM. A higher peak period $T_p$ leads to closer correspondence, whereas a small peak shows divergent spectral shapes.

8.3.5 Conclusion validation of wave forces

The first order wave forces calculated by WSP show close correlation to the First order Transfer Functions provided by the hydrodynamic file. Difference has been noticed for the higher frequencies and beam waves. This problem could be solved by using the radiated approach of the strip theory method for a wave incidence $\theta_o$ of 90° (or 270°). In consideration of uniformity, only the relative motion approach is used in the Wave Simulation Program.

The wave force spectra show good agreement. The spectral parameters are generally within a margin of 20-30% in comparison with TERMSIM. The higher the peak period, the lower the spectral resemblance. According to the comparison with the First order Transfer functions, this is not surprising.

Due to the peak in the low frequency region, the heave motion forms the exception. As a consequence of the different calculation methods of WSP and TERMSIM, this difference occurs. However, it will not exert high influence on the mooring line forces, since the heave motion is of minor importance in this respect.
8.4 Application of external wave forces in TERMSIM

8.4.1 Introduction

The link between the Wave Transformation Program and TERMSIM consists of the shipfor.dat file, which is generated by WSP. When this file is stored in the current directory in which TERMSIM is run, TERMSIM will read the input of the wave force time series automatically. Initially, the shipfor.dat file option has been developed by MARIN to provide the possibility to include the influence of a passing ship. In this section the application of the external wave force time series in TERMSIM is investigated.

8.4.2 Starting-up time

The TERMSIM simulation includes a start-up time possibility to damp out the initial ship motions. The actual simulation starts after this starting time, which is often set on 1800 seconds (30 minutes). The data provided by WSP is without the starting-up time. The question is whether the starting-up time influences the results when the wave force time series are given by the shipfor.dat file. Figure O-1 in Appendix O demonstrates the influence of the starting-up time on a simulation of 10.800 seconds, with the wave forces time series generated by WSP. It can be concluded that the motions, although to a small extension, are influenced by the starting time. Similar results have been found for other simulation durations (run 5 – 10; 625 s, 1250 s, 2500 s and 7200 s). Therefore the starting-up time has to be included in all simulation runs, and is set on 1800 seconds.

8.4.3 Duration

Three hours (10800 seconds) is a widely accepted duration for model tests in an irregular sea state. Usually the time step needs to be at least ten times smaller than the smallest natural period of the system that is simulated. Here we can assume that the time step must be ten times smaller than the peak period of the wave spectrum. However, the maximum number of data records in the shipfor.dat file, which can be read by TERMSIM, is 5000. This implies that, for a duration of three hours, the time-step will increase to a value of $\Delta t = 2.16$ s. This value could be too high. It has been investigated for a peak period of $T_p = 13$ s ($\omega_p = 0.48$) whether this time-step leads to erroneous results.

The time step of the wave force time series generated by TERMSIM is constant, namely $\Delta t = 0.5$ s. For a simulation time of 10.800 seconds and peak periods typical for an irregular sea state, this time-step is small enough. For this reason, the wave force time series and spectra obtained from TERMSIM have been compared with the time series and spectra generated...
by WSP. It can be concluded that a time-step, \( \Delta t = 2.16 \) s, seems to be small enough considering a peak period \( T_p = 13 \) s. However, to be sure the time-step will not influence the results, a simulation time of two and a half hour (9000 s) has been used. Considering a peak period of \( T_p = 13 \) s, the time step reduces to \( \Delta t = 1.8 \) s. If higher frequencies are analysed, it is recommended that the time-step and subsequently the duration of the WSP simulation would be decreased within the same ratio.

### 8.4.4 Simulation runs

In order to verify whether TERMSIM reads in the shipfor.dat file correctly, TERMSIM has been run reading its own generated wave forces from such a file. The results have been compared. The procedure is clarified in Figure 8-18.

![Figure 8-18: Validation scheme of external input wave forces in TERMSIM](image)

If the external input of wave forces is read in correctly the results of the wave forces and motions for TERMSIM run 1 and TERMSIM run 2 must be equal. Table 8 lists the simulation runs which have been carried out, all have been carried out for TERMSIM 1 (internal calculation wave forces) and TERMSIM 2 (external input wave forces).

Originally the option for external wave force input has been developed for a passing ship. The wave forces generated by passing ships unequivocally have a smaller wave height, period and duration time. By means of the simulation runs, the influence of these aspects
has to be investigated. For that reason, the runs are carried out with varying incident wave
direction $\theta_o$, peak wave period $T_p$, significant wave height $H_s$ and simulation time $t$.

### Table 8: Simulation runs external input wave forces in TERMSIM

<table>
<thead>
<tr>
<th>No run</th>
<th>Simulation Program</th>
<th>$T_p$ [s]</th>
<th>$H_s$ [m]</th>
<th>$\theta_o$ [$^\circ$]</th>
<th>$h$ [m]</th>
<th>Duration [s]</th>
<th>Spectrum 2$^{nd}$ order waves</th>
</tr>
</thead>
<tbody>
<tr>
<td>11</td>
<td>TERMSIM 1-2</td>
<td>13</td>
<td>2.5</td>
<td>0</td>
<td>14</td>
<td>9000</td>
<td>PM bound</td>
</tr>
<tr>
<td>12</td>
<td>TERMSIM 1-2</td>
<td>13</td>
<td>2.5</td>
<td>90</td>
<td>14</td>
<td>9000</td>
<td>PM bound</td>
</tr>
<tr>
<td>13</td>
<td>TERMSIM 1-2</td>
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<td>2.5</td>
<td>45</td>
<td>14</td>
<td>9000</td>
<td>PM bound</td>
</tr>
<tr>
<td>14</td>
<td>TERMSIM 1-2</td>
<td>8</td>
<td>2.5</td>
<td>45</td>
<td>14</td>
<td>9000</td>
<td>PM bound</td>
</tr>
<tr>
<td>15</td>
<td>TERMSIM 1-2</td>
<td>20</td>
<td>2.5</td>
<td>45</td>
<td>14</td>
<td>9000</td>
<td>PM bound</td>
</tr>
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<td>TERMSIM 1-2</td>
<td>13</td>
<td>0.5</td>
<td>45</td>
<td>14</td>
<td>9000</td>
<td>PM bound</td>
</tr>
<tr>
<td>17</td>
<td>TERMSIM 1-2</td>
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<td>45</td>
<td>14</td>
<td>9000</td>
<td>PM bound</td>
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<td>45</td>
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<td>2500</td>
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<td>1250</td>
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<td>22</td>
<td>TERMSIM 1-2</td>
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<td>45</td>
<td>14</td>
<td>625</td>
<td>PM bound</td>
</tr>
<tr>
<td>23</td>
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<td>8</td>
<td>1</td>
<td>45</td>
<td>14</td>
<td>625</td>
<td>PM bound</td>
</tr>
</tbody>
</table>

Poco, the post-processor of TERMSIM, provides time series and spectra for analysis of the
results. Figure 8-19, illustrates that the external wave forces are correctly introduced in
TERMSIM, for wave incidence $\theta_o = 90^\circ$ and peak period $T_p = 13$ s.

![Spectra motion Surge, 90°](image1) ![Spectra motion Sway, 90°](image2) ![Spectra motion Yaw, 90°](image3)

**Figure 8-19: Re-introduction wave force time series in TERMSIM, for surge, sway and yaw**

### 8.4.5 Conclusions external input of wave forces in TERMSIM

Based on the analysis of the application of the wave force time series in a TERMSIM
simulation, the following conclusions can be drawn:
The duration of the simulation depends on the peak period $T_p$ of the wave spectrum. For a peak period of 13 s a duration time $t$ of 9000 s seems reasonable. If the peak period decreases it is recommended to decrease the simulation time with the same ratio. The starting-up time is set on 1800 s and has to be included when the wave forces are derived from an external file.

Introducing wave force time series in a TERMSIM run leads to similar results, which indicates a correct application of external wave forces in TERMSIM.
9 Influence of breakwater dimensions on ship motions

9.1 Introduction

The application of the Wave Simulation Program in the preliminary design stage of a LNG terminal will be explained in this chapter. With the use of several simulation runs the influence of the breakwater dimensions on the ship motions and wave forces will be investigated.

**LNG carrier**

The same LNG carrier as in the validation process has been considered. The main characteristics and stability data are presented in Table 5 in section 8.3.2. The general arrangements of the carrier can be found in Appendix L.

**Duration**

The duration of the diffraction simulations is 9000 seconds. The starting-up time is set on half an hour or 1800 seconds.

9.2 Simulation runs

Several simulation runs have been carried out for a 130,000 m$^3$ LNG carrier moored at a jetty behind a breakwater. The rubble-mound breakwater, with length $L_b = 800$ m and crest height $R_c = 2$ m, and the jetty are situated at 600 m and 350 m from the coast respectively. The water depth is assumed constant, $h = 14$ m. For the determination of wave reflection a bottom slope of 1:40 is assumed in the surf zone. A typical mooring arrangement is used. The ship is equipped with 16 steel wire ropes with 11 m polypropylene tails.

The results presented in this section are for the terminal dimensions as described above and for different breakwater lengths, crest heights and bed slopes of the coast to investigate the influence of these parameters on the moored ship behaviour. The results are given for the horizontal motions of the carrier, surge, sway and yaw. These degrees of freedom are chosen because they are critical for the safe mooring of the ship. The ship is able to move rather freely in heave, roll and pitch. The mooring system prevents the ship from large horizontal motions. If these motions still become large, line-breaking accidents can occur.
First order Transfer Functions (FTF) are given in Fig. 9-1 to provide more insight in the dependence of the ship behaviour on the wave frequency. The FTF is defined as the ratio between the offshore wave amplitude and the wave force on the ship. Fig. 2 shows the reduction due to the presence of an 800 m long impermeable breakwater for an incident wave direction $\theta_0 = 45^\circ$. Only diffraction is considered here to get a better insight in this contribution. The effects due to reflection and transmission are neglected. For low-frequency waves there is only minor reduction, whereas for higher frequencies the reduction is about 90% for sway and yaw. A remarkable fact is the frequency shift comparing the graphs with and without the breakwater. This shift is due to the change of wave direction in the diffracted wave pattern behind the breakwater.

The results for ship motions presented in this section are for irregular waves. The offshore wave field is defined by a Pierson-Moskowitz wave spectrum, with a significant wave height $H_s = 2.5$ m, peak period $T_p = 13$ s and the calculations have been carried for two different wave directions $\theta_0 = 45^\circ$ and $90^\circ$, where the latter corresponds to waves perpendicular to the breakwater. The vessel motion spectra in Fig. 9-2 and the significant values of the motion amplitudes in Fig. 9-3 are for a wave direction $\theta_0 = 45^\circ$ and different breakwater lengths. The surge motions are dominated by low-frequency behaviour, mainly due to diffracted bound waves. Higher frequencies have no effects on the motions, because the mooring system is very soft in surge. Evidently, a very long breakwater would be necessary to protect the terminal against these long waves. The sway and yaw motions are influenced by both first and second order wave forcing. The breakwater is much more effective for these motions.
Influence of breakwater dimensions on ship motions

Figure 9-3: Significant motions for increasing breakwater length $L_b$, $\theta_0 = 45^\circ$

The investigation of the influence of the crest height and thus the effect of overtopping has been carried out for normally incident waves, see Fig. 9-4. The influence of transmission is in the range of the first order wave frequencies, as the empirical transmission formulae are formulated for first order waves only. Surge motions are low for these transverse waves. Regarding the results for sway and yaw the influence of transmission is large for low crest elevations. In these cases with dominant transmission the model gives an approximation of the ship motions and one should be aware of the necessary rude assumptions on transmitted waves.

Figure 9-4: Significant motions for increasing breakwater crest height $R_c$, $\theta_0 = 90^\circ$

Regarding the reflection at the shore the (second order) low-frequency waves have higher reflection coefficients, and thus dominate the ship motions due to reflection, only in case of a rocky coast there is also considerable short-wave reflection. Results for different bed slopes and for a rocky coast are given in Fig. 9-5. The large difference between a rocky coast and a mild-slope beach shows that reflection can play an important role. In case of oblique waves, the waves reflected from the coast are hardly reduced due to the presence of the breakwater and in case of normally incident waves, where the ship is perfectly sheltered against diffracted waves, the reflected wave energy is dominant for rocky coasts.
Figure 9-5: Significant motions for different shore types; (—) $\theta_0 = 45^\circ$, (−−) $\theta_0 = 90^\circ$
10 Conclusions and Recommendations

The conclusions and recommendations will be given in two categories:

The first category is the development of the Wave Simulation Program (WSP). It is subdivided in the near shore wave parameters, the wave forces and its application in TERMSIM. The second category is the ship motion dependency on the breakwater configuration. For each subject conclusions and recommendations are given.

10.1 Development of the Wave Simulation Program

10.1.1 Near-shore wave parameters

Figure 10-1: Wave Simulation Program

Conclusions

- Comparison of WSP with hydraulic model tests by Delft Hydraulics and the computer program ANSYS, indicates that the Wave Simulation Program correctly calculates the diffracted and reflected waves behind a breakwater.

- Literature on transmission gives empirical derived transmission formulae, which determine the transmission coefficient as a factor of the incoming wave height and breakwater parameters. No practical theory was found on the subject of spectral change behind a detached breakwater due to transmission, and this has not been included in WSP. For the calculation of the transmission coefficient, the d'Angremond formula has been found to be applicable considering a detached rubble-mound breakwater protecting a LNG marine terminal.

- Considering the d'Angremond formula, the wave period has a negligible influence on the transmission coefficient. It is not taken into account in WSP.
Conclusions and recommendations

**Recommendations**

- Transformation of the near-shore wave parameters has only been validated by separate treatment of diffraction, reflection and transmission. Comparison with a hydraulic model test that includes a combination of the three wave transformation phenomena is recommended.

- A more accurate implementation of transmission in the program requires better understanding of the influence of transmission on the spectral shape of the wave field. Integrating theoretical and empirical knowledge on transmission could be a solution. Influence of incoming and transmitted wave direction needs further research as well.

- The computer program provides two options for the calculation of second order waves: one considers the second order wave to be a bound long wave; the other considers it to be a free wave. The difference between these calculation methods has not clearly been investigated, but is recommended.

- In practice combinations of sea and swell can occur. It is recommend that the WSP is adjusted for this possibility.

**10.1.2 Wave forces**

Figure 10-2: Wave force determination

**Conclusions**

- The relative motion approach of the strip theory method proved to be a good method to calculate the wave forces:
  - The wave forces calculated with the strip theory method show close correlation to the First order Transfer function, given in the hydrodynamic file of a moored LNG carrier. Difference has been noticed in case of beam waves. This deviation is generally acknowledged and can be solved
introducing the radiated motion approach. In consideration of uniformity of the computer program, this approach is not included in WSP.

- Differences between the spectra (spectral parameters) of wave forces of WSP and TERMSIM are within reasonable limits (0% - ca. 25%). The higher the peak period, the better the correspondence. In contrast to TERMSIM, heave shows a high peak in the lowest frequency part of the spectrum. This can be explained by the different calculation methods used by TERMSIM (panel-method) and WSP (strip theory method).

10.1.3 Application WSP in TERMSIM

Conclusions

- The external input of wave forces are correctly introduced in TERMSIM.

Recommendations

- 5000 is the maximum number of wave force time series components that can be read in by TERMSIM. For an irregular sea state this could lead to either unacceptably high time-steps or short simulation times. For that reason it is recommended to increase the maximum number of components.
10.1.4 Main conclusions and recommendations development of WSP

**Main conclusions**
The making and testing of the computer simulation program WSP, in which all the calculation procedures were incorporated, lead to the following main conclusion:

- A computer model has been developed to calculate the motions and subsequent mooring forces of a ship moored behind a breakwater. Analytical formulations for diffraction of first- and second order waves by a breakwater are combined with empirical formulations for wave transmission over and through the breakwater and reflection at the shore. Quick computations can be carried out to investigate the influence of varying breakwater dimensions and terminal layouts. This is especially useful in the preliminary design stage of a LNG terminal. The calculations give good approximations of the behaviour of the ship, although model testing remains necessary for particularly the difficult modelling of wave overtopping. Hence, the numerical results can be used as a preparation to the model tests.

**Main recommendations**

- More extensive testing of WSP is advised to validate its use for design purposes. Particularly comparison with a model test is recommended.

10.2 Ship motion dependency on the breakwater configuration and coastal characteristics

![Figure 10-3: Optimization of the breakwater configuration](image-url)

*Figure 10-3: Optimization of the breakwater configuration*
Conclusions

- The results presented in this paper show that a detached breakwater of approximately three times the vessel length efficiently protects the berth against diffracted short waves. However, the protection against long waves is less effective and in case of steep coasts the influence of wave reflection at the shore is dominant. Marginal overtopping over the breakwater can be allowed, although the lack of knowledge about the deformation of the wave spectrum behind the breakwater has to be kept in mind.

Recommendations

- Evidently it is recommended to analyze the influence of the breakwater dimensions on the basis of motions, mooring line and fender forces. A ship response assessment tool has been designed for this purpose.

- Some aspects influencing the ship response need further research
  - Directional wave spreading
  - Choice of transmission formula
  - Combination of wave-, wind- and current forces
References


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Software

DIFFRAC, Maritime Research Institute Netherlands

DELFRA, Pinkster (1993), Delft Technical University

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Websites

[i] www.shipmotions.nl

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[iii] www.ct.tudelft.nl

[iii] www.coastal.ufl.edu