Wave Impact on Horizontal Platforms

Olivier de Rooij

Faculty of Civil Engineering
Section of Hydraulic Engineering
WAVE IMPACT ON HORIZONTAL PLATFORMS

Final graduation project
Civil engineering, TU Delft
Final report

O.V. de Rooij
October 2001

under supervision of:

prof. drs. ir. J.K. Vrijling
professor of hydraulic engineering

dr. ir. H.L. Fontijn
head fluid mechanics laboratory

dr. ir. S. van Baars
senior scientific staff member

ir. J.S. Reedijk
senior coastal engineer, Delta Marine Consultants
Preface

This is the final report of the graduation project of O.V. de Rooij. Writing the thesis has been the last assignment of my study Civil Engineering. Over the past nine months I have been working with and thinking about wave impact on horizontal platforms. Now it is almost finished, and I am very happy about it.

During my graduation project I have had the opportunity to work on a scale model experiment performed at HR Wallingford in the UK. Seeing the phenomenon actually happening before my own eyes has been extremely interesting and the numerous discussions of the topic with the "Exposed jetties" project team were very helpful. Professor Allsop, Matteo Tirindelli and Amjad Mohamed-Saleem, thanks for inviting me over.

I also would like to thank everyone who has, on some way or the other, guided me over the past nine months. I would especially like to thank the people of DMC, who made all this possible, and the members of my final theses committee, prof. drs. ir. J.K. Vrijling, dr. ir. H.L. Fontijn, dr. ir. S. van Baars and ir. J.S. Reedijk. Many thanks also to Markus Muttray of DMC for his insights on the matter.

This work could be very useful. I hope it will be.

Gouda, October 2001
# Contents

PREFACE........................................................................................................................................... III  
CONTENTS......................................................................................................................................... IV  
LIST OF SYMBOLS........................................................................................................................... VI  
ABSTRACT AND OUTLINE ................................................................................................................ VII  

## 1. INTRODUCTION .......................................................................................................................... 1  
1.1 GENERAL INTRODUCTION ........................................................................................................ 1  
1.2 THE PROBLEM ............................................................................................................................ 1  
1.3 GOAL AND APPROACH ............................................................................................................. 2  
1.4 WHY HAS THIS PROBLEM NOT BEEN SOLVED YET, AND WHY SHOULD IT BE SOLVED AT ALL? ................................................................. 2  
1.5 BACKGROUND .......................................................................................................................... 3  

## 2. LITERATURE STUDY ON ‘WAVE IMPACT’ .............................................................................. 4  
2.1 THE PHENOMENON .................................................................................................................... 4  
2.2 RESEARCH INTO WAVE IMPACT ............................................................................................ 6  
2.2.1 Introduction .......................................................................................................................... 6  
2.2.2 Wave impact on horizontal platforms ................................................................................. 7  
2.2.3 General remarks .................................................................................................................... 27  
2.2.4 Some words about model tests ............................................................................................ 28  

## 3 CURRENT STATE OF THINKING ............................................................................................ 29  
3.1 INTRODUCTION ........................................................................................................................ 29  
3.2 PEAK IMPACT LOADS, $F_{PEAK}$ ............................................................................................... 29  
3.2 SLOWLY-VARYING POSITIVE LOADS, $F^+$ ............................................................................. 30  
3.3 SLOWLY-VARYING NEGATIVE LOADS, $F^-$ ............................................................................ 30  
3.4 REFLECTION ON MODELS FROM LITERATURE ..................................................................... 31  
3.4.1 Introduction .......................................................................................................................... 31  
3.4.2 Models found in literature .................................................................................................... 31  
3.4.3 How well do the models deal with the issues ...................................................................... 32  
3.4.5 Comparison ........................................................................................................................ 34  

## 4 MODEL TESTS ............................................................................................................................ 37  
4.1 INTRODUCTION ........................................................................................................................ 37  
4.2 MODEL TEST DESCRIPTION ..................................................................................................... 38  
4.3 OBSERVATIONS MADE DURING THE TESTS ......................................................................... 39  
4.4 A LOOK AT THE TEST RESULTS .............................................................................................. 40  
4.4.1 Introduction .......................................................................................................................... 40  
4.4.2 Force-time profiles .............................................................................................................. 40  
4.6 A WORD ON SAMPLING RATES .............................................................................................. 40  
4.7 LONGITUDINAL DISTRIBUTION .............................................................................................. 44  
4.8 THE EFFECT OF A CHANGE IN THE WAVE PERIOD ON THE MAGNITUDE OF THE IMPACT ............................................................................. 47  
4.9 THE EFFECT OF THE DECK CLEARANCE ON THE MAGNITUDE AND SHAPE OF THE IMPACT ............................................................................. 50  
4.10 COMPARING TEST RESULTS TO AVAILABLE MODELS .......................................................... 52  

## 5 ESTIMATING WAVE LOADS ........................................................................................................ 55  
5.1 INTRODUCTION ........................................................................................................................ 55  
5.2 $F_{PEAK}$ .................................................................................................................................. 56  
5.2.1 Validation with model results ............................................................................................. 57  
5.3 $F^+$ .......................................................................................................................................... 58  
5.4 $F^-$ .......................................................................................................................................... 59
## List of symbols

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Dimension</th>
<th>SI-unit</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>L²</td>
<td>m²</td>
<td>area of contact</td>
</tr>
<tr>
<td>B</td>
<td>L</td>
<td>m</td>
<td>wetted deck length (Broughton)</td>
</tr>
<tr>
<td>b</td>
<td>L</td>
<td>m</td>
<td>width of the plate</td>
</tr>
<tr>
<td>c</td>
<td>LT⁻²</td>
<td>m/s²</td>
<td>wave celerity</td>
</tr>
<tr>
<td>c₁</td>
<td>-</td>
<td>-</td>
<td>correction factor Elghamry</td>
</tr>
<tr>
<td>c₂</td>
<td>-</td>
<td>-</td>
<td>correction factor Elghamry</td>
</tr>
<tr>
<td>C_d</td>
<td>-</td>
<td>-</td>
<td>drag coefficient</td>
</tr>
<tr>
<td>C_c</td>
<td>-</td>
<td>-</td>
<td>clearance coefficient</td>
</tr>
<tr>
<td>C_t</td>
<td>-</td>
<td>-</td>
<td>wave period coefficient</td>
</tr>
<tr>
<td>C_o</td>
<td>-</td>
<td>-</td>
<td>coefficient for the bottom line</td>
</tr>
<tr>
<td>C_s</td>
<td>-</td>
<td>-</td>
<td>slamming coefficient</td>
</tr>
<tr>
<td>Cₚ</td>
<td>-</td>
<td>-</td>
<td>slowly-varying positive coefficient</td>
</tr>
<tr>
<td>d</td>
<td>L</td>
<td>m</td>
<td>water depth</td>
</tr>
<tr>
<td>Fₚₚₑᵃᵏ</td>
<td>MT⁻²</td>
<td>N/m</td>
<td>impact peak force</td>
</tr>
<tr>
<td>F⁺</td>
<td>MT⁻²</td>
<td>N/m</td>
<td>slowly-varying positive force</td>
</tr>
<tr>
<td>F⁻</td>
<td>MT⁻²</td>
<td>N/m</td>
<td>slowly-varying negative force</td>
</tr>
<tr>
<td>F_b</td>
<td>MT⁻²</td>
<td>N/m</td>
<td>buoyancy force</td>
</tr>
<tr>
<td>F_d</td>
<td>MT⁻²</td>
<td>N/m</td>
<td>drag force</td>
</tr>
<tr>
<td>F_i₁</td>
<td>MT⁻²</td>
<td>N/m</td>
<td>inertia component of impact force</td>
</tr>
<tr>
<td>F_i₂</td>
<td>MT⁻²</td>
<td>N/m</td>
<td>added mass component of impact force</td>
</tr>
<tr>
<td>F_v</td>
<td>MT⁻²</td>
<td>N/m</td>
<td>total uplift force on the underside of the deck (per unit width)</td>
</tr>
<tr>
<td>H</td>
<td>L</td>
<td>m</td>
<td>wave height (trough to crest), regular waves</td>
</tr>
<tr>
<td>h</td>
<td>L</td>
<td>m</td>
<td>deck clearance</td>
</tr>
<tr>
<td>H_c</td>
<td>L</td>
<td>m</td>
<td>wave crest height (wave top to still water level)</td>
</tr>
<tr>
<td>H_max</td>
<td>L</td>
<td>m</td>
<td>maximum wave height (wave top to wave through)</td>
</tr>
<tr>
<td>Hₛ</td>
<td>L</td>
<td>m</td>
<td>significant wave height (wave top to wave through)</td>
</tr>
<tr>
<td>L</td>
<td>L</td>
<td>m</td>
<td>wave length</td>
</tr>
<tr>
<td>l</td>
<td>L</td>
<td>m</td>
<td>wetted length</td>
</tr>
<tr>
<td>m₂</td>
<td>M</td>
<td>kg</td>
<td>2D added mass</td>
</tr>
<tr>
<td>m₃</td>
<td>M</td>
<td>kg</td>
<td>3D added mass</td>
</tr>
<tr>
<td>P_i</td>
<td>MT⁻²L⁻¹</td>
<td>N/m²</td>
<td>dynamic impact pressure</td>
</tr>
<tr>
<td>P⁺</td>
<td>MT⁻²L⁻¹</td>
<td>N/m²</td>
<td>slowly-varying positive pressure</td>
</tr>
<tr>
<td>P⁻</td>
<td>MT⁻²L⁻¹</td>
<td>N/m²</td>
<td>slowly-varying negative pressure</td>
</tr>
<tr>
<td>r</td>
<td>-</td>
<td>-</td>
<td>( \pi \lambda L )</td>
</tr>
<tr>
<td>t</td>
<td>T</td>
<td>s</td>
<td>time</td>
</tr>
<tr>
<td>tᵯ</td>
<td>T</td>
<td>s</td>
<td>impact duration</td>
</tr>
<tr>
<td>tᵣ</td>
<td>T</td>
<td>s</td>
<td>impact rise time</td>
</tr>
<tr>
<td>v</td>
<td>LT⁻²</td>
<td>m/s²</td>
<td>vertical water particle velocity</td>
</tr>
<tr>
<td>Z</td>
<td>L</td>
<td>m</td>
<td>height of deck (thickness)</td>
</tr>
<tr>
<td>β</td>
<td>-</td>
<td>rad or deg</td>
<td>deadrise angle</td>
</tr>
<tr>
<td>γ_w</td>
<td>ML⁻²T⁻²</td>
<td>N/m³</td>
<td>weight of water</td>
</tr>
<tr>
<td>Λ</td>
<td>L</td>
<td>m</td>
<td>length of plate</td>
</tr>
<tr>
<td>ρ_w</td>
<td>ML⁻³</td>
<td>kg/m³</td>
<td>water density</td>
</tr>
</tbody>
</table>

These notations are used throughout the report, except in figures and tables 'borrowed' from other authors.
Abstract and outline

Introduction

This project is about wave impact on horizontal platforms. It investigates the interaction between ocean waves, mainly during hurricanes, and harbour jetties in coastal areas. Various types of loads can be identified when waves hit these kinds of coastal structures. The focus in this report will be on vertical loads (upward and downward) on platform decks. This subject has only had little coverage in studies on hydraulics or wave mechanics so far. The studies that are available differ substantially in everything from their origin and their goal, to their points of interest and their methods used. All this makes that, before adding a new study to this list, it is very important to thoroughly review the available literature to see which theories have been tested, which references have been looked at, what methods have been used and what results were obtained.

Chapter 1 starts with the introduction to the project, describing the problem under investigation, the goal to be achieved and the approach followed to do this.

Chapter 2 contains a literature review. This review summarises the work done on the subject from around 1970 to today by various researchers and groups of researchers. The first thing that strikes in this review is the non-uniformity and the lack of a clear framework that characterises most projects, at least, as far as the presentation of the respective projects in research papers is concerned. Researchers often follow their own trail, failing to comment on findings by others. Issues that are of considerable importance to the exact understanding of the phenomenon are sometimes not commented on or are solved without proper reasoning. These issues and the way this report deals with them are listed in the end of this chapter.

The literature review does not end in a complete understanding on how wave loads should be calculated. It shows a list of theories, analytical attempts and model tests trying to solve the problem, in lots of different ways for lots of different reasons. In chapter 3 the current state of knowledge is summarised. The wave impact process is separated into three components: $F_{\text{peak}}$, the initial peak force, $F_{+}$, the slowly-varying positive load and $F_{-}$, the negative load. For each of these components, the parameters that influence their shape and magnitude are listed. After this, an overview will be given on how well the various studies found in the literature deal with the issues mentioned above. This will then result in a ranking which reflects the usability of their results. These results will then be quantified for a couple of wave conditions, to see how the models compare.

Chapter 4 will describe model tests performed in the labs of HR Wallingford near Oxford. These tests have not been commissioned or designed especially for this project, but many interesting findings have been obtained from it nevertheless.

The next chapter, chapter 5, shows the way a model predicting the three impact force components could look like. In important notion here is that the bounds of the validity of this model are defined by the characteristics of the scale model it is derived from. The chapter ends with some guidelines for designing a harbour jetty.

The last chapter, chapter 6, contains the conclusions and recommendations this project suggests.
**How this report deals with the important issues of wave impact on horizontal platforms:**

<table>
<thead>
<tr>
<th>No.</th>
<th>Issue</th>
<th>Solved how?</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>The exact parameters determining the size and shape of wave loads.</td>
<td>In paragraphs 5.2 to 5.4 the parameters are listed that determine the three impact force components: $F_{peak}$, $F_{+}$ and $F_{-}$.</td>
</tr>
<tr>
<td>2</td>
<td>The type of structure the project is focussing on.</td>
<td>This project focuses on small to moderate size, concrete harbour platforms, examples of which are the jetties handling cruise ships or general cargo ships in the Caribbean.</td>
</tr>
<tr>
<td>3</td>
<td>How the various impact components actually effect the structure under consideration.</td>
<td>Short duration impact peaks, do or do they not affect the structure as a whole? This study believes they do. More on this issue can be found in the conclusion.</td>
</tr>
<tr>
<td>4</td>
<td>The importance of the sampling rate.</td>
<td>The importance is considerable, see paragraph 4.6. Minimal sampling rates are 200Hz when measuring forces and 1000Hz for measuring pressures. When using lower rates, peak values will not be captured properly.</td>
</tr>
<tr>
<td>5</td>
<td>The importance of a measuring system with the correct specifications.</td>
<td>Test results are often very dependent on the specifications of the measuring system used. Impact pressure peaks are very localised. When measuring these peaks, many small transducers should be used with an adequate range and accuracy. When measuring forces, measures should be taken to make sure only external force effects are displayed.</td>
</tr>
<tr>
<td>6</td>
<td>How to scale model test results to prototype values.</td>
<td>This problem has not been solved yet. Most projects actually dealing with this issue suggest Froude's law should be used. In this work, Froude's law is believed to overestimate forces and reductions are suggested, depending on the model scale used. More on this issue can be found in the appendix.</td>
</tr>
<tr>
<td>7</td>
<td>The probabilistic significance of results.</td>
<td>When testing random waves, the amount of waves should be high enough so that the various definitions of the wave field, like $H_s$, $H_{1/10}$, $H_{0.1%}$, can be related to a sufficient amount of force results. When testing 60 waves, it is no use matching a measured peak value to the $H_{0.1%}$ of the wavefield.</td>
</tr>
</tbody>
</table>

* Issues 1 to 3 are general issues, issues 4 to 7 are issues related to scale model experiments.
1. Introduction

1.1 General introduction

The coastal engineering department at Delta Marine Consultants has on a regular basis been involved in the design of harbours and jetties in coastal zones all over the world. Especially for small jetties in hurricane prone areas like the Caribbean the wave climate is one of the governing design parameters. The direction, length and height of the critical wave determine the required strength of materials and joints and influence the layout, geometry and, ultimately, the final cost of the project. When possible, the height of the structure is chosen in a way that a safe air gap between the highest expected wave and the underside of the deck is provided for. This way, storm waves will never touch the deck, and wave loads on the deck do not have to be considered. Sometimes though, this is not the desirable solution.

First, there can be economical reasons that demand a lower deck level. A higher deck requires a longer and thicker substructure, increasing the cost of material and construction. A higher deck also requires all cargo, both goods and passengers, to be transported downward to the lower harbour or shore level, and from smaller boats to be transported up to the deck level. This increases loading and unloading costs and time. Many existing cruise ships require harbour jetties with deck levels no more than three meters above chart datum, and will sail past towns not offering this kind of facilities.

Secondly, there are practical reasons. High decks require fishermen, sailors and cruise ship passengers to climb up and down on ladders and ramps. It is a lot more practical if the deck would be level with the height of the average boat.

Finally, there is also an esthetical preference for a lower deck. Just imagine an idyllic little Caribbean island with wooded beach cabins and palm trees and a 10 meter high harbour jetty towering out over its shore.

All these reasons make that the coastal structure needs to be able to withstand wave loading.

1.2 The problem

Structures in and around the sea are under the influence of various forces caused by water. These forces can be related to the water level, the current or the waves. This investigation will look at the loads from the waves. These loads can be divided in horizontal and vertical loads. Horizontal loads occur when a broken or breaking wave hits a vertical wall, like a breakwater or a harbour wall. Vertical loads occur when the wave comes into contact with a horizontal object, like the emerged bottom of a ship or the deck of harbour platforms. In the case of the heaving ship, both the ship hull and the water will have a certain velocity. In the case of harbour jetties, only the water will be moving.

At this moment, magnitude and intensity of maximum occurring forces on vertical walls can be predicted with a reasonable degree of accuracy. A lot of questions remain, though, on the precise interaction between a horizontal surface and the wave, especially concerning the effects of air compression and of the layout of the structure on the magnitude of the wave loads.
As for the various kinds of horizontal objects loaded by wave impact, the following list can be considered:

1) deep-water offshore platforms
2) harbour jetties
   a. on flat bottoms
   b. on sloping bottoms
   c. near shore, with or without quay walls
3) ships’ bottoms
4) Catamaran wet-decks

This report will focus mainly on harbour jetties, but theories and experimental results obtained for other objects will be considered too. Within the collection of harbour jetties, most attention will be given here to relatively small platforms under the influence of relatively large wave loads, because for these platforms, wave loads will be critical in the overall design. Larger platforms or platforms in less severe wave climates will be designed on the maximum loads expected on top of the deck, or on the platform dimensions required. Wave loads will be less important for these structures.

1.3 Goal and approach

The primary goal of this study is to develop a better understanding of the process of vertical wave loading on harbour platforms.

To achieve this goal an extensive study of literature will be carried out. This will result in a list of available methods for quantifying vertical wave loads and a list of recommendations on how to minimise the effect of waves on a platform.

The secondary goal of this paper is to offer guidance on how to better predict vertical wave loads when designing a jetty. For this an approach will be proposed based on results found in the literature, modified with the understanding obtained as explained above.

In any jetty design in which wave impact forces appear to cause the critical loads, scale model tests still need to be performed, because an all-solving, accurate model which predicts wave loads on any type of harbour jetty does not exist yet. Scale model tests on one type of deck structure will be looked at in this paper. Only when more tests like these become available, covering a wide range of deck structures and wave conditions, design wave loads can be predicted accurately without performing model tests.

1.4 Why has this problem not been solved yet, and why should it be solved at all?

As explained above, only certain types of structures are affected by vertical wave impact forces. The water impact phenomenon is actually found in other, more relevant areas too, like slamming in shipbuilding and the design of vertical sea and harbour walls loaded by breaking waves. Accurate models do exist in these fields and similarities can be found, but have yet to be proved.

Individual design projects involving the types of structures considered, in general, are not of sufficient size or economical importance to justify thorough fundamental research on wave impact. In some cases, model tests are in fact performed, but they fail to brighten the broad picture, as they often are focussed on very specific jetty styles and dimensions, and use various theoretical backgrounds and experimental approaches.

The problem deserves solving. A more economical design approach, containing fewer uncertainties or risks, will reduce the cost of the design and construction of harbour jetties in general. The market for
small size harbour jetties in the Caribbean might not be too big, but there are also other projects that could benefit from a better understanding. Many big offshore platforms in the North Sea seem to be gradually losing their safe air gap, because of subsiding foundations and ever higher waves. These constructions will need a re-analysis of their structural strength and stability. A good design approach could offer economical benefits for these kinds of projects.

1.5 Background

This thesis has been written as a part of my study Civil Engineering at the Delft University of Technology. It is the final assignment, leading to an 'Ingenieur' degree. It has been carried out for Delta Marine Consultants, the design department of HBG Civiel, Gouda, The Netherlands. Model test results have been obtained from test performed by HR Wallingford, for a project titled "Guidelines for the hydraulic design of exposed jetties". (DETR PII Project 39/5/130 cc2035).
2. Literature study on ‘Wave impact’

2.1 The phenomenon

This project focuses on the vertical loads induced by ocean waves hitting harbour structures. The initial contact between water and structure is called ‘wave impact’ or ‘wave slamming’. When a wave reaches a horizontal platform, the following interaction will occur between wave and platform:

1) the actual wave slam, a rapidly-varying, short duration impact pressure; followed by
2) a slowly-varying positive uplift pressure; followed by
3) a slowly-varying negative suction pressure

![Diagram showing pressure-time history of a typical wave impact](image)

*Figure 2-1. Pressure-time history of a typical wave impact [SHIH AND ANASTASIOU (1992)]*

(The terms ‘rapidly varying’ or ‘slowly-varying’ will always be used with respect to the response frequency of the object under consideration. A force that is rapidly-varying to a structure with a low response frequency, might be slowly-varying to one with a high one.)

The first contact between the water and the platform surface causes the dynamic impact pressure peak. Dynamic, as opposed to quasi-static, because of the very short rise-time and duration of the pressure peak and its often considerable magnitude. The pressure peak can only be called dynamic, when there is a dynamic response of the structure involved. The resonant response of the structure will be dominant at the resonant frequencies of the structure and are related to the inertial and damping characteristics of both hydrodynamic and structural effects. These responses are difficult to scale from model scale to the full-scale situation. Correct scaling of the model would require a hydroelastic model, which are generally very expensive and difficult to calibrate. The other option is to construct the model with sufficient high stiffness, so that its resonant frequency is well above the maximum frequency of interest to the designer and can therefore be ignored.

The impact peak progresses along the platform following the wave crest, so with a forward velocity equal to the wave celerity. The shape of the peak is dependent on the stiffness of the platform deck. For deck with a high stiffness, the peak will be short in duration and high in magnitude. When the force
history from a certain wave condition on a sufficient stiff deck is known, the response parameter should be calculated for any structure of interest.

The spatial distribution of the pressure peak over the area of a platform deck is a subject not yet completely understood. It is not possible to accurately translate pressure peaks into uplift forces. Many different assumptions are used, like the one that applies the peak pressure onto a strip one meter wide, parallel to the wave crest. To omit this problem, the best thing is to measure directly the total uplift forces on the platform under consideration.

After the initial contact between wave and platform, water will start to flow around and over the platform. During this inundation period, the slowly-varying positive pressure will occur, made up of a drag force, due to the flowing water, a buoyancy force, because of the submergence of the platform, and a force acting downward due to the weight of the water landing on the deck.

The drag force component of this positive pressure will slowly diminish and turn negative, as the water surface falls again. This negative drag and the suction caused by the falling water under the platform together with the water flowing over the deck will shape the slowly-varying negative pressure.

The pressure will return to atmospheric level after the wave passes and all the water has dripped off the platform.
2.2 Research into wave impact

2.2.1 Introduction

Research into the effects of wave slamming on maritime structures started on 24 February 1833, when Stevenson measured the first 'wave impact' on a vertical harbour wall in Scerryvore Rocks. His very simple measuring equipment showed a maximum impact pressure of 62.3 kN/m$^2$. In the following period, many researchers tried to measure wave impact, with results varying between 0 and 350 kN/m$^2$. Until 1937, measurements were performed with instruments unsuitable for these short duration phenomena. The first excellent high frequency measurements were executed at prototype scale in Dieppe on the west coast of France, by Rouville et al (1938), who measured a maximum pressure of 690 kN/m$^2$. Bagnold (1939) was the first who tried to explain the wave impact process by performing model tests in a wave flume. He found peak pressure to depend on entrapped air layer thickness, introducing the (later called) Bagnold type impact with air entrapment. Bagnold's project heads of a long list of research into wave impact on vertical walls.

At an international ship convention in Cairo in 1926, wave slamming also started to interest shipbuilders, when wave impact was noticed to occur on ever faster ships and seaplanes. A commission for wave impact forces was installed. Von Kármán (1929) looked at forces on seaplane floats during landing. He idealised the float as a two-dimensional wedge entering the water surface, comparing impulse before and after the impact. After the impact a cylindrical shape of water is added to the equation as extra mass. Von Kármán's 'momentum impact' model starts of the research into wave impact on ships. Later on, when the number of seaplanes decreased, impact pressures on high-speed ships stimulated research, as the model developed by Von Kármán was too conservative for these ships. [RAMKEKA (1978)]

Around 1963 Elghamry [ELGHAMRY (1971)] is the first to look at wave impact forces on horizontal platforms. He describes the magnitude of wave-induced uplift forces on horizontal decks under the action of periodic non-breaking waves and waves breaking on 1:3 and 1:5 beach slopes. Since wave impact on horizontal platforms is the main objective of this paper, literature on this subject will be reviewed in chronological order in this chapter.
2.2.2 Wave impact on horizontal platforms

Elghamry (1971) and Wang (1970), the first steps

The Americans Elghamry and Wang are the first to do experimental tests on horizontal scale models in wave flumes. They notice the short duration impulsive impact to be superimposed on a slowly varying component, explaining it as the abrupt change in the vertical component of the momentum of the incident wave. Their results show an initial peak pressure of considerable magnitude but of short duration, followed by a slowly varying uplift pressure of less magnitude but of considerable duration, and which is typical first positive, then negative. Realising similarities with impact on vertical walls, Wang points out that differences do exist:

"In hydrodynamics a horizontal plate overhanging an otherwise undisturbed gravity wave system may be considered to be a discontinuous boundary of an essentially irrotational flow field, whereas a vertical barrier in a breaking zone is an obstacle in a boundary layer. The mechanisms that cause impact may also differ. For the horizontal plate, impact is caused by the abrupt change in momentum of the fluid flow; for the vertical barrier, impact results from the collapse of a thin air layer. Consequently, for the horizontal plate, impact is expected to occur each time the water is brought into contact with the plate, and for the vertical barrier, impact will not occur unless an air layer of optimum thickness is entrapped."

For periodic waves in constant water depth, Elghamry proposes a deterministic approach with the uplift force dependant on the wave height, the deck clearance and the wave steepness or period. From Stoker's theory he derives the following formula for the total uplift force:

\[
F_r = c_1c_2 \frac{\gamma_w H \lambda}{2} \sqrt{1 + \frac{3r^2}{1 + r^2}}
\]

where:
- \( F_r \) = total uplift force on the underside of the deck (per unit width) (N/m)
- \( c_1 \) = correction factor for the effects of wave steepness, see figure 2-2a for a plot of \( c_1 \) against H/L
- \( c_2 \) = correction factor for the effects of deck clearance, see figure 2-2b for a a plot of \( c_2 \) as a function of the relative deck height (\( \Delta H/H \)) and the relative water depth (\( d/L \))
- \( \Delta H/H \) = (H/2 - h)/(H/2) (-)
- \( h \) = distance between deck and still water level (m)
- \( d \) = water depth (m)
- \( \gamma_w \) = specific weight water (N/m^3)
- \( H \) = wave height (m)
- \( L \) = wave length (m)
- \( \lambda \) = length of the plate (m)
- \( r \) = \( \pi \lambda / L \) (-)

Figures 2-2a+b. The coefficients \( c_1 \) and \( c_2 \) [ELGHAMRY (1971)]
For waves breaking on a beach slope without air entrapment, two peak forces are measured. The first corresponds to the instant of wave breaking and the second corresponds to the effect of the uprush flow. The pressure records show one peak per wave cycle, travelling along the deck at the speed equal to the wave celerity. Peak pressures are found to be one order of magnitude higher than those induced by non-breaking waves. However, the magnitude of the peak force did not increase in the same proportion.

Beams along the edge of the deck might cause an air cushion to become entrapped. In his model tests Elghamry measures peak pressures instantaneous at different locations under the platform. At the shorward end of the plate, pressures are caused by air compression, while at the seaward end, pressures are induced by direct wave action. The magnitude of the peak pressures was approximately double the observed values for the case of no air entrapment. Values for the peak force are an order of magnitude higher than those corresponding to the case of no air entrapment. Scale effects are expected to be an important reason for these high values to occur.

Elghamry performs his tests in a 35 x 0.3 x 1m wave flume with a 1.3 x 0.3m aluminium plate. No details are given about the sampling rate or the capacity of pressure transducers. This information is essential for the test results to have any validity. When choosing a sampling rate or capacity that is too low, exact pressure peaks will be missing and this will severely restrict the accuracy of formulas proposed.

Wang points out the similarities between wave impact and ship slamming problems. Wang describes the uplift pressure induced by a regular wave in a momentum-impact model, as did Von Kármán before him in 1929 for his work on impact loads on seaplane floats during landing.

\[ P_i = \frac{\pi}{2} H \gamma_w \tanh \left( \frac{2 \pi d}{L} \left( 1 - \frac{4 h^2}{H^2} \right)^{1/2} \right) \]

Where:
- \( P_i \) = Impact pressure (N/m²)

When comparing this expression with the Elghamry test results, a reasonable agreement is found. It should be noted again, though, that the exact specifications of the model tests from which these results are obtained are not known.

To better understand the wave impact process, Wang performs model tests in a 27.5m square basin with a platform over a 1:14 sloping beach, using a plunger to produce dispersive wave trains that hit the structure. Again, no details are disclosed about the sampling frequency. Wang’s test results show a lot of scatter but still agree in some way with his momentum-impact model.

**French (1971), Photographic stills**

French finds the rapidly-varying peak pressure to be distributed over a very narrow portion of the platform at any given time, making it of concern only when the strength of individual members of the platform structure is considered. The following slowly varying pressure is of concern to an engineer considering the structural strength of the platform as a unit.

French uses a camera to capture the impact process. For a two-dimensional model in a wave flume he compares pressure diagrams to photo stills. The following description is given of the impact process:

"A solitary wave of height H propagates through still water of depth d, and reaches the horizontal platform mounted at a distance s above the still water level. The wave front, where water first wets the platform, propagates beneath the platform with celerity \( U_d \). The uplift pressure peak, \( p_u \), propagates along the platform with the wave front until the wave front ceases to be defined. The slowly varying pressure, \( p_u \), is first positive, then decreases to zero and becomes negative. Water falls away from the platform in a wave of recession propagating with celerity \( U_w \). The event is over when the last of the water falls from the underside of the platform."
Figure 2-3. Wave impact on horizontal platforms according to French (1971).
French also describes an experimental study with the goal to investigate in detail the hydrodynamics associated with the uplift pressure on platforms, in particular the peak pressure. French uses a wave flume 30m long, 0.4m wide and 0.6m deep with an aluminum test platform with two pressure transducers, which he notices are too big to capture correctly the impact peak. No details are offered, again, on the sampling rate. French tries to correct the transducer problems by extrapolating from a comparison of peak pressures measured under similar conditions but at two different model scales. This method increases peak pressures from a few percent to as much as 50 percent. The accuracy of this method is questionable.

French found normalised peak impulse, or the normalised product of peak pressure and rise time, to be at most 20 percent of values reported for wave breaking against a vertical wall.

The peak pressure and rise time are found to be subject to considerable variance, probably because of the spume and the entrained air in the flow near the wave front. Despite a standard deviation in incident wave height of only about one percent of the mean height, French finds the standard deviation in peak pressure and rise time to be as much as 30 percent or more of the mean value.

**Heathcote and Britton (1980), failure of concrete box unit in surfzone**

In 1980, Heathcote and Britton investigated the failure of two pre-cast concrete box units of a stormwater outfall in the surfzone. Model test results scaled to prototype scale by using Froude’s modelling wave predicted a maximum peak pressure 150 kN/m² for an incident wave height from 1m to 2m, much higher than what the structure should safely withstand. Construction errors were later found to have led to the destruction of the two slabs. 1m to 2m waves reached the structure on many occasions afterwards and it should be noted that it is still in good condition.

**Broughton and Horn (1987), Subsidence of the Ekofisk platform**

Broughton and Horn write a report about a re-analysis of a part of the Ekofisk platform that is influenced by subsidence of the ocean floor. The original design of the platform allowed for the design storm to pass beneath the platform decks with an adequate air gap (deck clearance). With increased subsidence, and hence increased water depth, the platform decks would now be subjected to direct wave action. Broughton and Horn try to define the magnitude and intensity of these wave forces.

They start by predicting theoretically the interaction between the underside of the deck and the wave (Figure 2-4). Following Von Kármán and Wang they calculate inertia loads by considering the momentum of the water when it hits the underside of the deck:

\[ F_v = \frac{1}{4} \rho_w \pi c v B \]

Where:
- \( F_v \) = vertical force upward per unit width \((N/m)\)
- \( \rho_w \) = water density \((kg/m^3)\)
- \( c \) = wave celerity \((m/s)\)
- \( v \) = vertical particle velocity \((m/s)\)
- \( B \) = wetted deck length \((m)\)
- \( t \) = an order of time \((s)\)

![Figure 2-4. Interaction between wave and cellar deck.](image-url)
Noticing the uncertainties attached to this approach, they decide to perform a set of model tests. They test three different geometries for the deck structure:

1) the original platform,
2) a platform modified to reduce horizontal loads by fixing curved beams on the outside frame,
3) a platform modified to reduce vertical loads, by removing the deck plating, after which the deck is reduced to an open framed structure.

These experiments were performed in the large ocean basin at Marintek, Trondheim, Norway. This basin is 80m long, 50m wide and has an adjustable depth of 0 – 10m. The combination of this size and the wave conditions that were to be expected allowed for a 1:50 scale model to be tested at a sampling frequency of 20Hz. Full-scale values of forces were obtained from the test data by applying Froude’s scaling law. In this case, the forces on the full-scale prototype are estimated to be 125000 times (50^3) the values obtained from the model tests. Broughton and Horn model very high waves (H_w = 14.05 m on full scale) hitting a very high deck (12.09 m above still water level).

![Figure 2-5. Vertical force-time history. Horizontal axis displays time (s) [Broughton and Horn (1987)]](image)

Figure 2-5 shows a typical result from the experiments with geometry 1. The vertical force shows a sharp rise upwards, followed by a large force downwards. Broughton and Horn find the negative vertical force acting downwards to be of the same order of magnitude as the vertical force acting upwards, claiming this effect to be in line with previous studies, notably with one by French (1971). This conclusion should be specified further, as it only holds true in this particular case. According to French, the magnitude of the negative slowly varying pressure depends on soffit clearance, platform size and location, and less on the wave height. The positive peak pressure depends somehow on the wave height or the vertical particle velocity. Therefore the negative pressure will remain constant with increasing wave height, but the positive peak will rise sharply, nullifying the result mentioned above.

It should be noted here that the low sampling rate will have influenced this force record. Broughton and Horn test regular wave spectra with a crest elevation of 14.05m on prototype scale, which translates into a wave height of 0.56m on model scale and an impact duration between 10 and 50 milliseconds. A sampling frequency of 20Hz means a time interval of 50ms, clearly to big to properly capture the impact peak. Broughton and Horn, when confronted with these findings, claim that the forces associated with the lower frequency sampling rate are more relevant to the overall global analysis of the platform deck structure. The reason for this, they say, is mainly twofold:

a) The extreme short duration associated with the very high peak loading is far shorter than any possible response time of the platform, deck, or any individual member.
b) As well as being able to note the very high peak force values, it would be necessary to know the likely area over which short duration loading is applied. Measurements of these areas are difficult to obtain.

When comparing the force-time history in figure 2-5 with results obtained by Shih and Anastasiou (1992) or Cornett et al. (1999), as explained below, it becomes clear that this result is very unlikely to
be accurate. The sharp peaked negative force is not found in any other test. An incorrect time scale and a too low sampling rate are suspected to be the cause of this distorted result.

This subject definitely allows for further research. In paragraph 4.6 the effect of the sampling rate on the magnitude of wave loads is discussed.

**Shih and Anastasiou (1992), literature review, semi-empirical approach**

In 1992, Shih and Anastasiou publish a paper describing model tests and their semi-empirical solution to the wave impact problem. Shih et al. are the first to look at the problem in a three-dimensional way, stressing the importance of the direction, in the horizontal plane, at right angles to the wave propagation. They also deal with the effects of scaling and the influence of the sampling rate on the shape of the pressure-time records.

Shih et al. find both the duration and magnitude of the impact to be highly dependent on air entrainment. They note that this effect can only be accurately reproduced in large-scale model tests. Furthermore, the duration of the impact is extremely short and can be recorded accurately only if highly sensitive pressure transducers and fast recording instruments are deployed (Figure 2-8).

Slowly varying pressures, first positive, then negative, are said to be made up of the sum of hydrostatic and hydrodynamic contributions. The hydrostatic head is caused by the difference in elevation between the soffit and the free surface of the undisturbed wave, while the hydrodynamic head is caused by the wave-induced fluid motion underneath the platform.

![Figure 2-8](image)

**Figure 2-8. Importance of sampling frequency in capturing dynamic impact pressures: time history of uplift pressures recorded at (a) high sampling frequency (500 Hz), (b) low sampling frequency (20 Hz). (No information on platform clearance)**
In order to determine the significance of scale effects, experimental work was carried out in two wave flumes. The large flume is 55.0 m long, 2.8 m wide, a maximum operating depth of 1.2 m and a wave generator that can create both regular and irregular wave sequences. The smaller flume is 15.0 m long, 0.3 m wide, 0.3 m deep and can handle regular waves only. For the same prototype wave conditions, the tests carried out in the large flume corresponded to a model scale four times greater than the one of the smaller facility. Measurements were sampled at a rate of 400-500 Hz. The importance of the sampling rate is illustrated in figure 2.8.

Monitoring the wave impact process continuously on their digital oscilloscope while gradually increasing the wave height, Shih et al. noticed the sharp impact peak to grow. This growth continued up to the point were the wave front was no longer stable and the wave crest began disintegrating, thus causing significant amounts of air to be entrained in that region. From this point, although the wave height might still be increasing, the dynamic impact pressure was no longer evident for these breaking or broken waves.

Comments are given on the following topics:

**Effects of air entrainment**
The impact pressure was found to be governed strongly by the effects of air entrainment. Air is entrained in a random manner; a collection of waves with the exact same characteristics still shows a large variation in peak pressure. In most cases, air entrainment reduces the magnitude of the impact peak and enlarges the duration. Extreme impact values seem to depend only on wave kinematics at the point of impact.

**Dependence on clearance**
No clear relation is found, mainly because of the overshadowing effects of air entrainment. Some relation will exist, though, as water particle kinematics like the vertical velocity, depend on the vertical position of the point under consideration.

**Empirical upper limit**
Shih et al. propose the following empirical upper limits for the maximum impact pressure:

\[
P_r = (1.8 - 10 - 7.6) \rho w g H
\]

regular waves

\[
P_r = (4 - 10 - 8) \rho w g H,
\]

irregular waves

whereas previous work by Elghamry suggests a value of 2.5 \( \rho w g H \) as the upper value for regular waves. Shih et al. attribute this difference to the more advanced equipment used in their study, which enables accurate capture of the dynamic pressure transients. These maximum impact pressures are derived from the actual readings of the pressure transducers under the model deck. Shih and Anastasiou do not talk about the expected area of contact of these pressures, so no indication is given of what impact forces could be expected to occur on the full-size deck. This is a shame, since this will be the value an engineer designing a platform deck will be most interested in.

**Scaling of test results**
The impact pressure seems to obey Froude's scaling law. The impact duration does not obey this law, which means that this scaling law cannot be used to scale impulsive pressures.

**Duration of the impact pressure**
The experiment result indicate that the duration of the impact pressure is to a large degree independent of the wave height and the clearance, with the mean duration varying between 8.8 and 16.3 milliseconds, not obeying the relation proposed by Wang (1970).
Slowly-varying positive pressure
This pressure was found to be governed mainly by the wave height and the deck clearance. The following relation is proposed:

\[ P_{sw} = 0.65 \left( H_c - h \right) \rho_c g \]

Where
- \( H_c \) = The crest height of the incident wave above still water level (m)
- \( h \) = deck clearance (m)

French (1971) proposes values for this positive pressure of about 1.0 times this head difference between crest and deck height. The three-dimensional approach adopted by Shih et al. might explain this difference. Higher values of the slowly-varying positive pressure can be expected if two sides of the platform in the direction of wave propagation were to be blocked, so as to prevent water from rising above the soffit level.

Slowly-varying negative pressure
This pressure was found to be governed mainly by the width of the platform and less by the wave height, the deck clearance or the location of the point at which \( P_{sw} \) is measured relative to the centre-line of the platform. An almost linear relation between width of the platform and the magnitude of \( P_{sw} \) could be found.

Kaplan (1992), more momentum-impact with 2-D and 3-D added mass

Kaplan (1992, 1995) also looks at the effect of wave impact on horizontal beams and flat decks on offshore platforms. The vertical force on a flat deck structure is obtained by means of a combined momentum and drag force analysis, assuming the deck is inundated by the wave crest. Kaplan notices similarities between this topic and research done for flat surface slamming of advanced marine vehicles [KAPLAN (1987)]. In the case of the platforms decks, though, there is no motion of the body (i.e. the plate) and the seawater can go over and around the plate.

The vertical force is made up of three components:

1. The actual impact \( (F) \), a vertical momentum component, made up of:
   - an inertia force associated with the fluid acceleration \( (F_{i1}) \)
   - an added mass force associated with the rate of change of added mass \( (F_{i2}) \)
2. A drag component \( (F_d) \)
3. A buoyancy component \( (F_b) \)

So,

\[ F_v = F_{i1} + F_{i2} + F_d + F_b \]

The moment of occurrence of the four force components may be schematised as seen in figure 2-9, with \( F_{a1} \) and \( F_{a2} \) replacing \( F_{i1} \) and \( F_{i2} \). \( F_{a1} \) is negative, as it is associated with the vertical acceleration of the water particles, which is negative when pointing downward during impact.

![Figure 2-9. Four force components (ISAACSON AND BHAT (1994))](image)

*DMC, Gouda 14/100 October 2001*
The impact force is given by:

\[
F_i = \frac{\partial}{\partial t} (m, v) = \rho \frac{\pi}{8} \frac{bl^2}{\left[1 + \left(\frac{l}{b}\right)^2\right]^{1.2}} \cdot v + \rho \frac{\pi}{4} b l c \frac{1 + \left(\frac{1}{b}\right)^2}{\left[1 + \left(\frac{l}{b}\right)^2\right]^{1.2}} \cdot v
\]

Where \( m_3 \) is the three-dimensional added mass, given by:

\[
m_3 = \rho \frac{\pi}{8} \frac{bl^2}{\left[1 + \left(\frac{l}{b}\right)^2\right]^{1.2}}
\]

Where:
- \( b \) = the width of the plate (m)
- \( l \) = wetted length (m)
- \( c \) = wave celerity (m/s)

The parameter \( l \) is determined from the relative degree of wetting of the flat deck underside, which occurs as the incident wave travels along the deck from its initial contact location. The second term on the right continually varies up to the time when the wetted length \( l \) reaches the end of the plate, after which 'c' is zero, remaining so throughout the time that the particular wave elevation is contacting the deck. In the determination of the vertical force for different horizontal deck structures, the value of the vertical particle velocity \( v \) and the vertical acceleration \( \ddot{v} \) is that appropriate to the vertical location of the particular deck. During the impact force time history determination, the value of the velocity and the acceleration is found at each time instant at the location of the geometric centre of the wetted length being considered.

The drag force component is given by:

\[
F_d = \rho \frac{b l C_d \sqrt{v}}{2}
\]

where \( C_d \) is the drag coefficient, here taken as 2.0

And the buoyancy force by:

\[
F_b = \rho g (blz)
\]

where \( z \) = height of deck (m)

This three-dimensional approach is also compared with experimental results. The basic details of these tests are:

Scale: 1:36
Sampling rate: 200 Hz
Filter: low-pass filter; Kaiser filter with a cut-off frequency of 1.0 Hz.

Features:
- Emphasis on procedures for measurement of wave forces and techniques to remove extraneous effects from the model's response to impact loads. A low-pass filter as describes above is found to give the best result.

A good agreement is found between the measured and subsequently filtered data and the theoretical predictions. As an indication, table 2-1 contains some ratios of the measured to the predicted force. In this table, \( F^+ \) is the maximum measured positive force divided by the maximum predicted positive
force, and $F_-$ is the maximum (absolute) measured negative force divided by the maximum predicted negative force.

<table>
<thead>
<tr>
<th>Deck plating</th>
<th>$H_{cr}$ (m)</th>
<th>$T$ (s)</th>
<th>Heading ('')</th>
<th>$F_+$</th>
<th>$F_-$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Solid</td>
<td>19.3</td>
<td>17.3</td>
<td>0</td>
<td>1.05</td>
<td>0.92</td>
</tr>
<tr>
<td>Solid</td>
<td>19.3</td>
<td>17.3</td>
<td>90</td>
<td>1.02</td>
<td>1.46</td>
</tr>
<tr>
<td>Solid</td>
<td>19.3</td>
<td>17.3</td>
<td>0</td>
<td>1.15</td>
<td>1.05</td>
</tr>
<tr>
<td>Grating</td>
<td>19.3</td>
<td>17.3</td>
<td>0</td>
<td>0.86</td>
<td>0.63</td>
</tr>
<tr>
<td>Grating</td>
<td>19.3</td>
<td>17.3</td>
<td>90</td>
<td>1.06</td>
<td>1.14</td>
</tr>
<tr>
<td></td>
<td>19.3</td>
<td>17.3</td>
<td>90</td>
<td>0.71</td>
<td>0.75</td>
</tr>
</tbody>
</table>

*Table 2-1. Ratios of measured to predicted force.*

As can be seen above, for a solid deck, the predictions for the positive force are quite accurate. For the maximum negative force, a larger variation is found. In the case of decks covered with grating instead of plates, theoretically predicted values, both for the positive force and for the negative force, are generally too high.

Kaplan also draws the attention to several important factors that influence the way experimental data is converted to experimental results. One important aspect is the requirement only to measure the external impact forces acting on a structure of interest and to correct for any inertial forces due to the acceleration and subsequent vibration of the structure. Another aspect is related to the data sampling rates and filtering action on the recorded data. As explained before, wave impact is a phenomenon impulsive in nature, and the force occurrence involves rapid rise and fall characteristics. High frequency range measuring rates are required. Similarly, the use of filters should be restricted. The filtering action will possibly delete important aspects of the time history records when the filter frequency cut-off value is too low.

In 1997, Murray and Kaplan describe yet another set of model tests. This time they measure forces on a platform deck with a sampling rate of 500 Hz. Their results show that:

1. There is no significant difference between the impact forces measured in a regular wave as compared to those measured from a wave in an irregular wave group having the same nominal zero-crossing period and crest height. The implies that a maximum expected random sea state could easily be modelled by using an equivalent regular wave, which would be easier to create and reproduce.
2. There is a linear relation between the crest height and the peak impact force.
3. Positive vertical forces are sensitive to both deck clearance and wave period. Increasing the wave period will increase the uplift force, as it increases the wetted length (provided the wetted length has not reached its maximum, being the length of the structure). A smaller deck clearance will also increase forces, as it increases the added mass and the vertical velocity of water particles hitting the deck.

*Klatter, Janssen and Dijkman (1994) Full-scale tests on the Eastern Scheldt Barrier*

Klatter, janssen and Dijkman were involved in the design and construction of the Eastern Scheldt storm surge barrier. A monitoring program was set up to evaluate the hydraulic aspects of the barrier. This program consisted of field measurements of hydraulic loads on the barrier and the response of the structure [Klatter et al. (1990)]. In this program wave impacts on the barrier were also monitored. When considering vertical impact loads, the bottom of the upper beam (figure 2-10) was selected as the critical construction item, as it was a relatively light, prestressed concrete beam that could be lifted from its supports. Wave impact occurs on the bottom of this beam when the gates are open under moderate storm conditions, and with more severe conditions just before the gates start closing. The clearance between the underside of the beam and the still water level is almost zero.
Instrumentation, with specifications as in table 2-2, was mounted on the beam as shown in figure 2-11. Unfortunately, only four of the ten pressure gauges actually functioned during the measuring trail. The maximum pressures recorded with the remaining gauges are presented in table 2-3. It is interesting to see that different pressure gauges, some meters apart, at one time endure completely different pressure magnitudes.

<table>
<thead>
<tr>
<th>Date</th>
<th>Time</th>
<th>Hs (m)</th>
<th>D35</th>
<th>D36</th>
<th>D37</th>
<th>D39</th>
</tr>
</thead>
<tbody>
<tr>
<td>14 Feb '89 05:06:17</td>
<td>1.45</td>
<td>31</td>
<td>59</td>
<td>7</td>
<td></td>
<td></td>
</tr>
<tr>
<td>14 Feb '89 05:09:47</td>
<td>1.45</td>
<td>27</td>
<td>52</td>
<td>4</td>
<td></td>
<td></td>
</tr>
<tr>
<td>14 Feb '89 06:30:37</td>
<td>1.50</td>
<td>14</td>
<td>29</td>
<td>20</td>
<td>22</td>
<td></td>
</tr>
<tr>
<td>14 Feb '89 06:31:20</td>
<td>1.50</td>
<td>3</td>
<td>9</td>
<td>30</td>
<td></td>
<td></td>
</tr>
<tr>
<td>14 Feb '89 13:03:43</td>
<td>1.15</td>
<td>15</td>
<td>30</td>
<td>38</td>
<td>62</td>
<td></td>
</tr>
<tr>
<td>14 Feb '89 13:13:50</td>
<td>1.05</td>
<td>29</td>
<td>17</td>
<td>25</td>
<td>29</td>
<td></td>
</tr>
<tr>
<td>14 Feb '89 13:22:51</td>
<td>1.00</td>
<td>52</td>
<td>28</td>
<td>34</td>
<td>14</td>
<td></td>
</tr>
</tbody>
</table>

Table 2-3. Maximum impact pressures on upper beam

When following the semi-emperical approach introduced by Shih and Anastasiou above:

\[ P_i = (0.2 - 10^5 - 5.4) \rho_w g H \]

Which shows that Klatters maximum fits within the range specified by Shih and Anastasiou.
To illustrate the wave impact on the upper beam and the response of the beam, the time series of two pressure gauges (D35 and D36) recorded at 13:13:50 on 14th Feb. 1989 are presented in figure 2-12. The figure shows a sharply peaked impact, which first hits the North Sea side of the beam bottom and moves with the wave crest to the Eastern Scheldt side. There is hardly any sign of high frequency oscillations that would indicate air entrapment. The observed oscillations are related to the vibration of the structural element itself. The rise time of the pressure peaks was in the order of 10ms and the decline time in the order of 20ms, giving an impact pressure peak duration of about 30ms.

The response of the beam is illustrated in figure 2-13. This figure shows the acceleration of the beam as registered at gauge V33. The response starts as the wave first hits the beam. After the impact the beam vibrates in its natural frequency of approximately 5Hz.
Another interesting result is the response of the support force to the wave impact. Figure 2-14 shows the reading of the force gauge K1. The wave impact force, with a maximum amplitude of about 650 kN, is small in comparison with the dead weight of the beam, which is about 6000 kN.

![Graph showing force gauge reading](image)

*Figure 2-14. Registration of the force gauge in one support of the beam*

During the design phase of the barrier, several scale model tests have been performed. A quantitative comparison of these tests with the field measurements appeared to be quite difficult for different reasons. In the field tests, local pressures were measured, while in most model tests impact forces averaged over a larger area were measured directly. Because the spatial variation is much greater than expected, the average impact force in the field cannot be calculated accurately from the individual pressure registrations. This effect is intensified by the failure of a number of pressure gauges.

Just a little calculation:
The time history of the force measured in the support shows an uplift response force of about 300 kN, corresponding to the pressure peak measured in D35. Assuming both supports experience this response, the total uplift force on the beam is 600 kN. At the moment under consideration, pressure gauge D35 measures 29 kN/m² and D36, although not hit directly, still measures 10 kN/m². D35 and D36 are placed on the edge of the width of the beam, about 5 m apart. When assuming a linear variation of the pressure from D35 to D36, the force on a one metre strip is 100 kN. This would mean that only 6 m of the beam, 45 meters long, is hit at one time.

**Suchithra and Koola (1995), the effect of stiffeners under the platform deck**

Next, there is Suchithra and Koola who perform experiments on a horizontal slab in a wave flume. Their goal is to study the effect of wave frequency, deck geometry and deck clearance on the vertical wave-impact forces. The dimensions of the wave flume are 10.0 x 0.3 x 0.3 m. Acknowledging the slamming phenomenon to be of a high frequency in nature, they use a sampling rate of 1000 Hz.

Suchithra et al. start with the following expression for the vertical impact force:

\[ F_i = \frac{1}{2} C_s \rho_w A v^2 \]

Where:
- \( F_i \) = impact force (N)
- \( C_s \) = slamming coefficient (-)
- \( A \) = area of contact (m²)
- \( v \) = vertical water particle velocity (m/s)
This expression is based on the, so-called, stagnation pressure: $\frac{1}{2} \rho_w v^2$, which is multiplied by a slamming coefficient, $C_s$. The stagnation pressure seems not to be the ideal expression to describe the impact process, as compressibility and air are considered more important than the gravitational acceleration. [RAMKEMA (1978)]

The results of the model tests are presented in the form of a range for the slamming coefficient for four deck geometries. Again, comments are given on the following topics:

**Wave frequency**
At a particular frequency, for different wave heights, the variation of the slamming coefficient is insignificant. This verifies the fact that the impact force is proportional to the square of the velocity.

**Deck clearance**
The test results display a higher impact coefficient for lower deck clearance. The exact relation can not be established, because of the facts that only intermediate values for the deck clearance are studied and because of the considerable degree of scatter found in the graphs.

**Deck geometry**
Four deck geometry’s with and without longitudinal and transversal stiffeners are tested. These stiffeners are observed to have a great effect in the impact force. On platforms with only longitudinal stiffeners, slightly higher wave forces are measured, the difference being too small to indicate a clear trend. When using transversal stiffeners, on the other hand, air pockets can get trapped under the platform, considerably reducing impact forces from incident waves with a certain minimum wavelength. When the wavelength becomes too small, air cannot be trapped between the platform deck, the transversal stiffeners and the wave crest. Waves then hit the deck directly, as they did in the case of the deck without stiffeners.

**Ridderbos (1999), A Wagner formula**

Ridderbos (1999), a former student at Delft university, finishes his final thesis on this subject in 1999. He looks at wave slamming loads on a harbour jetty on the island of St. Maarten in the Caribbean. To get an idea of the impact loads occurring on this jetty, the prototype is modelled at two different scale’s. Ridderbos himself performs a 1:50 scale model test in the laboratories of Delft Hydraulics. Some results of this test are compared to values obtained of a 1:5 scale test performed by Delft Hydraulics in the Deltaflume in Emmeloord.

Ridderbos assumes the magnitude of the wave impact to be dependent on the velocity of the water particles hitting the deck and the direction of impact. The direction of impact he calls $\beta$. Using a formula created by Wagner for impact of wedges in a still body of water, in which $\beta$ is denoted as the ‘deadrise angle’, the magnitude of the wave impact pressure can be found from:

$$p_i = \frac{1}{2} \rho_w C_s(\beta) v^2$$

Where $C_s$ is the slamming coefficient, given by:

$$C_s = 1 + \frac{\left( \pi \cot \beta \right)^2}{2}$$

Again, this expression is derived from the stagnation pressure formula. In Wagner’s case of wedges hitting the water surface, the angle $\beta$ is the angle between one of the faces of the wedge and the horizontal water surface (see figure 2-15). Ridderbos determines $\beta$ by looking at the angle between
the combined vector of the horizontal and vertical velocities of the water particles at the moment of impact and the vertical face (see figure 2-16).

Combining the results of the large scale model tests, the small scale tests and his theoretical model, Ridderbos describes at the effects of the following factors on the magnitude of the wave impact:

Wave height:
With a constant platform level and a constant wave period, wave impact pressures are found to increase somewhat linearly with increasing wave height. This statement seems quite plausible, since water particle velocities increase in waves with an increasing height and constant periods.

Wave period
The wave period is linked to the wave steepness. For waves with constant height and increasing period, water particle velocities will be lower and wave impact pressures will also be lower. Again, this follows from the linear wave theory; vertical water particle velocities decrease with an increase of the wave period T.

Deck clearance
The highest pressure values occur for a large wave hitting the deck just above the still water level. Ridderbos finds the wave impact force to increase “more rapidly than linear” with increasing relative deck clearance.

Cornett et al. (1999), Design of a new cruise ship pier in the Caribbean

In order to quantify the expected wave loads on a new cruise ship pier in Bridgetown, Barbados, a model study is undertaken by Cornett, Tarbottan, Mattila and Gittens. This study is focused on the hydrodynamic forces exerted on the platform deck during extreme storm conditions.

First, some model test details:

Scale 1:50
Sampling rate 200 Hz
Filter 100 Hz anti-aliasing filter
Features:
Pressure transducers both measuring pressures under and on the deck. Low frequency force measuring deck plate. Regular and irregular waves. Differentiating water level/deck clearance, wave height, wave period, deck layout.

Cornett et al clearly describe the phenomenon of water impact on the underside of the pier deck. The pressure fluctuations measured are made up of four components, as sketched in figure 2-17:

1. a rapidly-varying dynamic impact pressure; followed by
2. a slowly-varying positive uplift pressure; followed by
3. a slowly-varying negative suction pressure; followed by
4. an interval with virtual constant atmospheric pressure.
The dynamic impact (shock) pressure component is highly variable and is characterised by a pressure spike with large magnitude and very short duration. These pressure spikes are generated by the initial contact of the free surface with the underside of the pier. The nature of the pressure spikes depends on factors such as air entrainment at the free surface, the vertical velocity of the free surface at the point of contact and the presence or absence of a trapped air pocket.

The slowly-varying uplift pressure prevails for a few seconds (at prototype scale). The duration and magnitude of this pressure is far more constant from wave to wave. The pressure can be attributed to the difference in elevation of the free surface beneath the deck and the wave crest in front of the pier.

The slowly-varying negative pressure prevails while the free surface in the (relatively) undisturbed flow around the pier falls below the level of the deck soffit, leading to a progressive detachment of the free surface from the underside of the deck. Again, this slowly-varying pressure is far more constant from wave to wave than the dynamic pressure.

Cornett et al. placed pressure transducers both on and under the model platform, in order to measure the effect of water flowing over the platform deck. The net hydrodynamic force on a section of deck will be equal to the sum of the spatial integration of the uplift or downward suction pressure acting beneath the deck and the downward pressures due to the weight of water flowing over the deck’s upper surface. Figure 2-18 shows one example of the relation between the pressures recorded on the underside of the pier deck and the forces measured on a 18 x 40 m deck section.

Cornett et al. say that, since wave motion is governed by a balance between gravitational and inertial forces acting on water particles, Froude’s scaling law should be used to relate conditions and results in the model to those at full scale.
Consideration of many extreme loading events such as the one plotted in figure 2-18 led to the following conclusion:

1) The timing of the impact pressure spikes measured on the underside of the platform deck suggests a pressure wave that travels in the direction of wave propagation.

2) The largest uplift forces are generally not caused by the largest waves, as these waves deposit more water on top of the deck, thereby reducing the uplift force.

3) The peak uplift pressures are not necessarily associated with the largest uplift forces or the largest waves.

4) Maximum forces of 25 to 30 MN were recorded at the south end of the pier under wave attack from south and south-west directions. These forces are approximately 30% larger than the dead weight of the deck. The maximum uplift force attenuates slightly along the pier, falling below 25 MN at the north end.

5) In the case of figure 2-19, the maximum uplift force of 25 MN is equivalent to an average uplift pressure of 35 kN/m², or 3.5 m of water. This pressure is slightly larger than the slowly-varying pressures recorded on the underside of the deck, but is much less than the peak shock pressures. This is an important conclusion, as it makes clear that the local shock pressures greatly exceed the spatially averaged pressure.

Influence of wave height
As expected, the results show a steady and consistent trend towards higher loads with increasing wave height. This holds true for:

- pressures and forces,
- regular and irregular waves,
- wave attack from the south and south-west directions, and
- the seaward, landward and central sections of the deck.
The trend was more or less linear in some cases, but significantly non-linear in others. Figure 2-19 shows an example of the non-linear relationship between vertical force and significant wave height for the deck section at the seaward (south) end of the pier under irregular wave attack from the south. The solid line in this figure represents the best-fit power curve through the test data, while the dashed curves represent 95% confidence boundaries on the maximum force to account for the randomness of ocean waves.

![Figure 2-19. Influence of wave height on vertical uplift force.](image)

**Influence of water level**
The underside of the platform deck is at +2 m and the top is at +3 m. The water level is varied between +1 and +3 m. With this variation, maximum vertical forces are found as shown in figure 2-20. As can be seen in this figure, uplift forces go down sharply when the still water level reaches the underside of the deck structure. Cornett et al offer no explanation for this fact. One reason is that for the same wave in higher water, more water will splash on the deck, reducing the uplift forces. This reason, though, does not explain the sudden sharp fall of the maximum uplift force in figure 2-20 around the water level +2 m, as it should also happen below +2 m. Another reason might be that the irregular wave motion gets distorted when the still water level inundates the plate. Water coming down from the plate during the downwards motion of the wave sine will hit the wave as it comes up again.

![Figure 2-20. Influence of water level on maximum vertical force at the seaward (south) and landward (north) ends of the platform.](image)

**Influence of deck perforations**
Cornett et al also look at the effect of deck perforations. They find a perforation of 5% of the deck area to reduce the uplift forces 10 to 25%, depending on the wave height.
Cornett et al. propose the following ideas for the optimisation of the pier design:

- **Deck elevation.** Keeping the deck soffit elevation as high as possible is the most practical means to minimise wave uplift forces.
- **Deck underside roughness.** Pile caps descending below the deck soffit and aligned transverse to the wave direction add roughness to the deck underside. This roughness distorts the wave profile and reduced impact pressures.
- **Deck underside profiles.** Deck underside profiles that create air voids (i.e. precast double tee beams, perimeter edge beams, etc.) can, in the absence of venting, trap air. This trapped air is then compressed by the rising water front, thus reducing the energy of the wave impact actually hitting the structure. But, trapped air will also increase the buoyancy force during the submergence of the deck. It has to be seen if the reduction of peak pressures justifies the increase of the slowly-varying buoyancy force.
- **Structure width.** Investigations by Shih and Anastasiou (1992) suggest that structure width has an influence on the magnitude of uplift forces. Narrower platforms appear to experience lower average uplift pressures. This might be because of the lateral escape of wave energy.
- **Deck perforations.** Deck perforations totalling 5% of the deck areas were responsible for 10-25% reductions in uplift forces. This significant reduction is thought to result from the venting of air and water from beneath the deck, relieving the hydrodynamic pressure below the deck and increasing the volume of water on top of the deck. Figure 2-21 shows a wave attack on a pier with deck perforations. Clearly visible is the effect of venting of air and water from under the deck.
- **Structure mass.** A simple approach to counteract wave uplift is to design a massive structure. Two obvious problems are economy and seismic risk.

![Figure 2-21. Wave attack on the south end of the pier with deck perforations.](image)

**Overbeek and Klabbers (2000), Design of two harbour platforms, subsequently hit by hurricane waves**

Overbeek and Klabbers describe their method of designing jetty platforms. Two of the platforms that have actually been build with this method in the Caribbean were hit by hurricane waves. Both survived the storm with only minor structural damage. Overbeek and Klabbers first predict the design wave height for a harbour jetty in a hurricane storm from wave growth calculations. This procedure will not be dealt with in this project. With the design wave, the design wave loads can be established. From the literature, they obtain the following practical guideline:
For the impact pressure, assumed over strip of 1 m wide, parallel to the wave front:

\[ P_i = C_s \rho_s g H_{\text{max}} \]

For the slowly-varying pressure, assumed over the immersed part:

\[ P_s = 1.0 \rho_s g (H_{cr} - h) \]

Where:
- \( P_i \) = the vertical impact pressure (N/m\(^2\))
- \( P_s \) = slowly-varying positive pressure (N/m\(^2\))
- \( C_s \) = a slamming constant, here assessed as 1.5 (-)
- \( H_{\text{max}} \) = the maximum wave height (m)
- \( H_{cr} \) = the maximum wave crest height (to still water level) (m)
- \( h \) = deck clearance (m)

The constant \( C_s \) is difficult to determine. The values found in the literature range widely. A value of 1.5 was chosen because the measured high impact values are of short duration and their influence on the large concrete mass under consideration was deemed small.

Trapped air is said to increase the peak pressure. For this reason the jetty was designed with beams in only one direction, parallel to the berthing line to avoid the entrapment of the waves in a beam grid. Some reflection is needed at this point, as Suchithra and Koola (1995) claim, as explained above, that a grid of beams which entraps air actually reduces the impact peak pressure. The solution lies here in the definition of the pressure being reduced or increased. Air bubbles attenuate the peak impact pressure, as compressed air evens out some impact energy, but air bubbles also increase the slowly-varying pressure, as the larger platform volume increases the buoyancy force.

Apart from the beams, the deck has gaps in the transverse direction (50 mm wide, every 2 m). For the comfort of the cruise ship passengers these gaps are covered with timber, T-shaped planks that have no fixing and can ‘blow-out’ during a wave attack. This event was actually observed during a storm hitting the harbour. The timber planks were blown-out and fountains of spray appeared through he gaps.
2.2.3 general remarks

Some general remarks concerning this topic are found in the following publications:

Bullock et al. (2000), on the influence of air and scale on wave impact pressures:
- Entrained air reduces the maximum impact pressure and increases rise times.
- Air bubbles that form in freshwater tend to be larger than those in seawater. Consequently air can escape more easily from freshwater than from seawater and larger impact pressure will be measured when using freshwater.
- Application of Froude’s scaling law to transform model results into predictions for full-scale situations leads to a (slight) overestimation of the impact pressure.

Wood et al. (2000), on wave impact on vertical wall:
- 2D model tests overestimate impact pressures, as sideways leakage of air and pressure is neglected.

Takahashi et al. (1994), on dynamic response of sliding breakwater caissons:
- The magnitude of impulsive pressure intensity is quite large, being several times the ordinary wave pressure. However, the effective pressure for caisson sliding is greatly reduced due to the caisson’s dynamic response. The characteristics of the dynamic response are revealed by model experiments and FEM numerical calculations.

Peregrine (1994), on pressures on breakwaters:
- Extremely high pressures are only obtainable on a very small scale. Water surface roughness on full-scale ocean waves causes peak pressures to drop.

Oumeraci et al. (1993), on classification of breaking wave loads:
- Impact forces cannot be described by a single formula.

Schmidt et al. (1992), on impact loads on vertical walls:
- The assumption that impact pressures are not important and thus should not be adopted in the design is wrong.
- Sampling rate’s of 2, 1 and 0.175 kHz, compared to 11kHz, result in a reduction of 2, 7 and 50% for the pressure peaks, 2, 3 and 20% for the force peaks, but no significant change for force impulse.

Oumeraci et al. (1992), on impact loading and dynamic response of caisson breakwaters:
- The occurrence of a sharp force peak, followed by a quasi-static load generally leads to a higher response of the structure than the quasi-static load alone. In other words, Impact pressures should be taken into account when considering the overall structure stability.
### 2.2.4 Some words about model tests

In the available literature, various reports describing model tests are found. Some of these reports do not accurately describe the testing procedure. In the first reports of French, Wang and Elghamry, for instance, no mention is made of the sampling rate used in the experiments. The value of this rate is later found to be very important in accurately capturing the impact process. A reasonable amount of prudence is adopted when referring to the results of reports not accurately describing the test conditions.

**Overview of experiments:**

<table>
<thead>
<tr>
<th>Researcher</th>
<th>Scale</th>
<th>Sample frequency</th>
<th>size wave flume</th>
<th>Filter used</th>
</tr>
</thead>
<tbody>
<tr>
<td>China (2000)</td>
<td>1:20</td>
<td>200 Hz</td>
<td>100 x 1 x 2</td>
<td></td>
</tr>
<tr>
<td>Cornett (1999)</td>
<td>1:50</td>
<td>200 Hz</td>
<td></td>
<td>100 Hz anti-aliasing filter</td>
</tr>
<tr>
<td>Smith WL (1999)</td>
<td>1:5</td>
<td>200 Hz</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Ridderbos (1999)</td>
<td>1:50</td>
<td>400 Hz</td>
<td>30 x 30 x 0.3m</td>
<td></td>
</tr>
<tr>
<td>Kaplan (1995)</td>
<td>1:36</td>
<td>200 Hz</td>
<td></td>
<td>1 Hz Low-pass Kaiser filter</td>
</tr>
<tr>
<td>Klatter RWS (1994)</td>
<td>1:1</td>
<td>1000 Hz</td>
<td>Eastern Scheldt</td>
<td></td>
</tr>
<tr>
<td>Isaacson (1994)</td>
<td></td>
<td>2500 Hz</td>
<td>20 x 0.62 x 0.55m</td>
<td>20 Hz filter</td>
</tr>
<tr>
<td>Suchithra (1994)</td>
<td></td>
<td>1000 Hz</td>
<td>10 x 0.3 x 0.3m</td>
<td></td>
</tr>
<tr>
<td>Anastasiou (1992)</td>
<td>1:x +</td>
<td>400 – 500 Hz</td>
<td>55 x 2.8 x 1.2m + 15 x 0.3 x 0.3m</td>
<td></td>
</tr>
<tr>
<td>Broughton (1987)</td>
<td>1:50</td>
<td>20 Hz</td>
<td>80 x 50 x 10m</td>
<td></td>
</tr>
<tr>
<td>French (1970)</td>
<td></td>
<td>low</td>
<td>30 x 0.4 x 0.6m</td>
<td></td>
</tr>
<tr>
<td>Wang (1970)</td>
<td></td>
<td>low</td>
<td>27.5 x 27.5 x</td>
<td></td>
</tr>
<tr>
<td>Elghamry (1963)</td>
<td></td>
<td>low</td>
<td>35 x 0.3 x 1m</td>
<td></td>
</tr>
</tbody>
</table>
3 Current state of thinking

3.1 Introduction

At this point, an overview is required of the current state of the theory on wave impact loads on horizontal platforms. In this chapter a summary will be given on how the wave forces are believed to be working on a platform deck.

3.1 Peak impact loads
3.2 slowly-varying positive loads
3.3 negative loads

In paragraph 3.4 the various models from literature are commented on, after which they are used to predict wave forces for some wave conditions to see how they compare.

3.2 Peak impact loads, $F_{\text{peak}}$

The impact load is dependent on the following parameters:

- wave height
- vertical water particle velocity
- deck clearance
- air entrapment
- deck/structure layout
- geometry of the bottom line

In general, peak loads increase with increasing wave height, increasing wave steepness and increasing bottom slope.

An essential topic here is the area of contact of the impact pressure. The impact component of known to be of very short duration, and moving along the platform following the wave crest. This would mean that the area of contact is small. Peak pressures should only be expected to occur at the location were water actually comes into contact with the platform.

The impact pressure can be predicted in two ways. First, by considering the vertical water particle velocity at the moment of impact. This velocity will follow from the wave height, the deck clearance (i.e. impact location in wave profile) and the geometry of the bottom line. Secondly, by relating the impact pressure directly to the wave height (significant or crest).

The magnitude of the impact peak pressure is also dependent on the deck/structure geometry and the amount of air entrapment. A grid of stiffeners will reduce impact pressures, by distorting the wave profile and by the compression of trapped air. Supporting piles will attenuate the wave movement.

The impact pressure should then be applied to a strip, with a certain width, parallel to the wave crest.
3.2 Slowly-varying positive loads, $F^+$

The slowly-varying positive load is dependent on the following parameters:

- wave height
- deck clearance
- specific weight of water
- geometry of the platform
- geometry of the structure

Shih and Anastasiou note that, for a particular wave height, the pressure variation as a function of the deck clearance is linear. This behaviour suggests that a global relationship might be established expressing the slowly-varying positive pressure $P_*$ in terms of the wave crest elevation $H_c$ and the deck clearance $d$:

$$P_* = C_* \rho_s g (H_c - d)$$

In which $C_*$ is a coefficient dependant on the platform and structure geometry. Shih and Anastasiou use a value of 0.65 in their model.

Kaplan suggests a combination of drag and buoyancy forces. A comparison between these two methods can be found on page 36.

As soon as the water starts inundating the platform deck, this extra weight will decrease the upward pressure. This implies that the rising water is able to flow over the deck. Large platform decks without gaps or decks with supporting beams blocking the water will experience larger uplift pressures.

3.3 Slowly-varying negative loads, $F^-$

The slowly-varying negative load is dependent on the following parameters:

- platform dimensions
- platform geometry
- wetted platform length

The negative forces are caused by water flowing over and falling of the platform as the wave crest passes the deck. For decks with a small clearance, they can be considerable.

Kaplan (1965) relates the negative force to the negative particle velocity occurring after the wave crest reaches its maximum height and the, always, negative 3-D mass acceleration component. Shih and Anastasiou (1992) find the slowly-varying negative pressure to vary almost linearly with the platform width. Downward force is caused by to the amount of water on the deck. Big, solid decks, where no wave overtopping occurs, will experience little to no downward forces.
3.4 Reflection on models from literature

3.4.1 Introduction
In the literature study various models are presented that calculate the upward impact force. Each of these methods will be commented on briefly.

3.4.2 Models found in literature

Elghamry
Elghamry's model is a deterministic model that gives the maximum uplift force per unit width. This force is dependent on the wave height, the wave period, the length of the plate and the deck clearance. Elghamry's model has been calibrated from the results of model tests. The sampling rate used for these tests is believed to have been too low, which results in an underestimation of the impact peak pressures. The actual sampling rate or the scale used for the tests is not mentioned in the paper, indicating that Elghamry perhaps does not recognise the importance of these parameters. Judging from the dimensions of the wave flume, the scale is believed to be around 1:50. Elghamry's model does not take into account all the parameters identified in paragraph 3.1 as being of influence on the impact peak. The parameters deck/structure layout, geometry of the bottom slope and the possibilities of air entrapment are not considered in the model. This means that Elghamry's result is only valid for harbour jetties with the same deck/structure layout, bottom line and possibilities of air entrapment as the one used in his tests.

Wang
Wang offers an expression for the maximum impact pressure, dependent on the wave height, the water depth, the wavelength and the platform clearance. Wang compares his model with the experimental results of Elghamry, again, underestimating impact peaks. Wang also performs some tests himself. In a 3-D basin he measures the loads from a single wave hitting the structure. For these tests a small scale model is believed to have been used, but again no mention is made of the exact scale or the sampling frequency. Wang gives no insight into the area of contact the impact pressure is expected to work on, so that nothing can be said about the upward forces.

Broughton and Horn
Another simple momentum impact method, dependent on vertical particle velocity at the deck location, deck dimensions and wave celerity. This model compares reasonably well with the results of model test, for which, again, the sampling rate is too low. The effect of the bottom slope is included in the model by the use of the vertical particle velocity, which will increase for an increasing bottom slope. Nothing is known, though, about deck/structure layout or air entrapment. Broughton and Horn test their approach on a scale model of a steel offshore platform. This fact makes it less suitable for use on concrete harbour jetties. Broughton and Horn defend their choice of a low sampling rate (20Hz) by stating that heavy offshore platforms will not notice any higher frequency loads.

Shih and Anastasiou
Shih and Anastasiou consider the local peak impact pressure. They say air entrainment has a big influence on the magnitude of this pressure. Since this air effect is not yet fully understood, Shih and Anastasiou say only an empirical solution, related to a specific deck configuration, can be justified. Impact pressure is related directly to the wave height, multiplied by a slamming constant. They offer no guidance as how this pressure should be translated into uplift forces. Their range of maximum
pressures varies considerably, with the high pressure being a factor 4 higher than the lower value. No clear explanation is given which values should be used for which deck structure.

**Suchithra**  
Derived from the, so called, expression of the stagnation pressure, the model used by Suchithra et al. uses a slamming coefficient $C_s$ to account for the effect of the deck layout on the magnitude of the impact force. As it doesn’t consider any other variables that could be of any importance, accounting for the effect of the deck layout is perhaps the only thing it should be used for.

**Kaplan**  
Kaplan offers a very detailed model, which separates the impact process into four components. The first two describe the actual impact force, which travels along the deck with the wave crest. The third and fourth describe the slowly-varying force, made up of drag and buoyancy components. Kaplan uses a low frequency cut-off filter to clear the force measurements of any inertial response effects. The question still is what effect this procedure has on the high-frequency peak forces. Kaplan validates his model with the help of various tests on scale models of offshore platforms. All these platforms have a very big clearance and the waves tested hit them only with the very top of their crest. This might account for the fact that Kaplan’s model predicts very small impact values.

**Ridderbos**  
Ridderbos also uses the stagnation pressure, multiplied by a slamming coefficient derived from Wagner’s work on ship slamming. The model proposed by Ridderbos is very dependent on deck clearance. A deck with a small clearance will be hit by water particles at an almost vertical direction. The deadrise angle, $\beta$, will be close to zero and the slamming coefficient, $C_s$, reaches the maximum value of 600. Ridderbos compares his model to a small amount of prototype values of scale model test results. No mention is made on how the test results have been scaled to prototype values. This makes it hard to test the accuracy of the test results presented in the report. Parameters like deck/structure layout or air entrapment are not included in the model. This again makes it suitable only for a certain kind of structure. Ridderbos predicts the slowly-varying pressure to be 1/10th of the impact pressure.

**Overbeek and Klabbers**  
Overbeek and Klabbers use a practical expression for the impact pressure similar to the one found by Shih and Anastasiou. Overbeek and Klabbers are interested in total uplift forces. The impact pressure is calculated to work on a strip of 1 meter wide, parallel to the wave front. This expression is derived from a literature review, not from model tests.

### 3.4.3 How well do the models deal with the issues

In the next table, the above models are judged on how well they deal with the issues identified as being of importance to the wave impact phenomenon. This results in a ranking showing the usefulness of the various reports. The possible rankings are:

- Report not useful. To many errors or unclear assumptions
- Report has some use. Although uncertainties on the accuracy exist, results can be used in an order of magnitude comparison
+ Report is useful. The problem is treated in a clear and accurate manner. Most issues are dealt with correctly.
<table>
<thead>
<tr>
<th>No.</th>
<th>Issue</th>
<th>ElGhamry</th>
<th>Wang</th>
<th>Broughton</th>
<th>Shih</th>
<th>Suchithra</th>
<th>Kaplan</th>
<th>Ridderbos</th>
<th>Overbeek</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>The exact parameters determine the size and shape of wave loads.</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>0</td>
<td>-</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>2</td>
<td>The type of structure the project is focusing on.</td>
<td>0</td>
<td>0</td>
<td>+</td>
<td>0</td>
<td>+</td>
<td>+</td>
<td>+</td>
<td>+</td>
</tr>
<tr>
<td>3</td>
<td>How the various impact components actually effect the structure under consideration.</td>
<td>-</td>
<td>+</td>
<td>+</td>
<td>0</td>
<td>-</td>
<td>+/0</td>
<td>-</td>
<td>0</td>
</tr>
<tr>
<td>4</td>
<td>The importance of the sampling rate.</td>
<td>-</td>
<td>-</td>
<td>0</td>
<td>+</td>
<td>+</td>
<td>+/0</td>
<td>+</td>
<td>n/a</td>
</tr>
<tr>
<td>5</td>
<td>The importance of a measuring system with the correct specifications.</td>
<td>+</td>
<td>+</td>
<td>-</td>
<td>+</td>
<td>+</td>
<td>+</td>
<td>+</td>
<td>n/a</td>
</tr>
<tr>
<td>6</td>
<td>How to scale model test results to prototype values.</td>
<td>-</td>
<td>0</td>
<td>0</td>
<td>+</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>n/a</td>
</tr>
<tr>
<td>7</td>
<td>The probabilistic significance of results.</td>
<td>+</td>
<td>-</td>
<td>0</td>
<td>+</td>
<td>-</td>
<td>+</td>
<td>0</td>
<td>n/a</td>
</tr>
<tr>
<td>Ranking</td>
<td></td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>+</td>
<td>0</td>
<td>+/0</td>
<td>0</td>
<td>0</td>
</tr>
</tbody>
</table>

- = No information or wrong assumptions
0 = Some information, not completely conclusive
+ = Proper reasoning, useful information.
(n/a = not applicable)
3.4.5 Comparison

The results of all the models mentioned above are now compared. Which force occurs when an imaginative hurricane wave hits a relatively small harbour deck plate? The formulas used for this comparison can be found in appendix A3.

<table>
<thead>
<tr>
<th>Data:</th>
<th>Wave</th>
</tr>
</thead>
<tbody>
<tr>
<td>height</td>
<td>H</td>
</tr>
<tr>
<td>length</td>
<td>L</td>
</tr>
<tr>
<td>period</td>
<td>T</td>
</tr>
<tr>
<td>angle</td>
<td>α</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Deck</th>
</tr>
</thead>
<tbody>
<tr>
<td>width</td>
</tr>
<tr>
<td>length</td>
</tr>
<tr>
<td>thickness</td>
</tr>
<tr>
<td>clearance</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Water</th>
</tr>
</thead>
<tbody>
<tr>
<td>depth</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Researcher</th>
<th>Impact pressure (kN/m²)</th>
<th>Impact force (kN)</th>
<th>Slowly-varying force (kN)</th>
<th>Max uplift force (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Elghamry (1963)</td>
<td></td>
<td>475</td>
<td></td>
<td>475</td>
</tr>
<tr>
<td>Wang (1970)</td>
<td>83</td>
<td>665*</td>
<td></td>
<td>665*</td>
</tr>
<tr>
<td>Broughton (1982)</td>
<td></td>
<td>800</td>
<td></td>
<td>800</td>
</tr>
<tr>
<td>Shih &amp; Anastasiou (1992)</td>
<td>176-745</td>
<td>1411-5958*</td>
<td>408</td>
<td>1411-5958*</td>
</tr>
<tr>
<td>Suchithra (1994)</td>
<td></td>
<td>499-998</td>
<td></td>
<td>499-998</td>
</tr>
<tr>
<td>Kaplan (1995)</td>
<td></td>
<td>534</td>
<td>291</td>
<td>766**</td>
</tr>
<tr>
<td>Ridderbos (1999)</td>
<td>22</td>
<td>172***</td>
<td>69</td>
<td>172***</td>
</tr>
<tr>
<td>Overbeek &amp; Klabbbers (2000)</td>
<td>147</td>
<td>1176</td>
<td>627</td>
<td>1176</td>
</tr>
</tbody>
</table>

Table 3-a: Impact load from an imaginative hurricane wave according to various researchers.

- Nothing mentioned about the area of occurrence of the impact pressure. The impact force is calculated here by applying the impact pressure onto a strip of one meter wide, over the full width of the structure.
- **Kaplan is the only researcher offering a time history of the forces acting on the deck. That is why his maximum uplift force is not just the biggest of the impact and slowly-varying force (see figure 3-2).
- ***Ridderbos says that the impact pressure should be applied on the total width of the structure, but gives no information on the width of this strip. (a width of one meter is assumed here)

![Church rooftop force history](image1)

![Kaplan force history](image2)

*Figure 3-2. Most researchers assume slowly-varying force to ‘start’ after impact peak fades (left). Kaplan is the only one who predicts a force-time history (right)*
As can be seen in table 3-a and figure 3-3, the values for the impact pressure and forces from the various models available in literature differ substantially. At first glance, it seems that newer models find higher values. This might be because of the fact that early models fail to properly capture the impact process. Shih and Anastasiou are the first that succeed in doing this. They measure impact pressure peaks, but give no information on what forces should be expected to occur. Suchithra and Kaplan, after them, measure impact forces. The magnitude of these forces is far below the values expected when considering the work by Shih and Anastasiou.

Kaplan is the only one offering a pressure-time history of the wave impact process. It shows that the impact peak is more severe in the case of the small harbour deck (a ratio of 4 to 1 to the slowly-varying positive force) than for the big Eastern Scheldt barrier beam (2 to 1). Kaplan's model doesn't take into account any downward forces from water splashing on the deck. This is very correct for a closed off, sufficiently high structure, where the wave crest does not reach the top of the deck, but on a thin platform deck, some downward force will occur.

What is needed to properly judge the accuracy of the models is a good impact force measurement on prototype scale. Some force measurements can be found in the work by Klatter et al. They measure impact pressures and forces on the upper beam of the Eastern Scheldt Barrier. Although not all instrumentation installed on the barrier beam survived the tests, it still offers a good 'order of magnitude' result. Their results and the predictions of the various models compare as follows:

<table>
<thead>
<tr>
<th>Data: Wave</th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>height</td>
<td>H</td>
<td>1.05</td>
<td>m</td>
</tr>
<tr>
<td>length</td>
<td>L</td>
<td>22</td>
<td>m</td>
</tr>
<tr>
<td>period</td>
<td>T</td>
<td>4</td>
<td>s</td>
</tr>
<tr>
<td>angle</td>
<td>a</td>
<td>0</td>
<td>rad</td>
</tr>
<tr>
<td>Deck</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>width</td>
<td>b</td>
<td>45</td>
<td>m</td>
</tr>
<tr>
<td>length</td>
<td>l</td>
<td>5</td>
<td>m</td>
</tr>
<tr>
<td>thickness</td>
<td>a</td>
<td>1</td>
<td>m</td>
</tr>
<tr>
<td>clearance</td>
<td>h</td>
<td>0.05</td>
<td>m</td>
</tr>
<tr>
<td>Water</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>depth</td>
<td>d</td>
<td>5</td>
<td>m</td>
</tr>
<tr>
<td>Researcher</td>
<td>Impact pressure (kN/m²)</td>
<td>Impact force (kN)</td>
<td>Slowly-varying force (kN)</td>
</tr>
<tr>
<td>---------------------</td>
<td>-------------------------</td>
<td>-------------------</td>
<td>---------------------------</td>
</tr>
<tr>
<td>Elghamry</td>
<td></td>
<td>492</td>
<td></td>
</tr>
<tr>
<td>Wang</td>
<td>14</td>
<td>644</td>
<td></td>
</tr>
<tr>
<td>Broughton</td>
<td></td>
<td>813</td>
<td></td>
</tr>
<tr>
<td>Shih &amp; Anastasiou</td>
<td>19-78</td>
<td>833-3519*</td>
<td>681</td>
</tr>
<tr>
<td>Suchithra</td>
<td>391-782</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Kaplan</td>
<td>321</td>
<td>907</td>
<td></td>
</tr>
<tr>
<td>Ridderbos</td>
<td>81</td>
<td>3652**</td>
<td>1826</td>
</tr>
<tr>
<td>Overbeek &amp; Klabbers</td>
<td>15</td>
<td>695</td>
<td>1826</td>
</tr>
<tr>
<td>Klatter</td>
<td>30</td>
<td>1000</td>
<td></td>
</tr>
</tbody>
</table>

Table 4-b. Comparing model results to prototype measurement.

Figure 3-4. Comparing model values to prototype measurements on Eastern Scheldt beam. Bar shows range, square shows mean value

In the first comparison, the prediction by Ridderbos was among the lowest found in literature. In this case, as the clearance between the still water level and the Eastern Scheldt beam is almost zero, his prediction is the highest of all researchers. The Ridderbos model is very much dependent on deck clearance.

Besides Ridderbos, Shih and Overbeek are the only ones safely predicting the wave forces on the Eastern Scheldt beam. It is to early to say that all the other models are wrong, as this is only one comparison or the model results with the forces measured from very specific configuration. Some assumptions had to be made about the exact magnitude of the forces on the beam.

Again, the predictions vary considerably. More model test results are needed to get a more accurate prediction for wave loads on horizontal platforms.
4 Model tests

4.1 Introduction

This chapter will deal with the wave flume model tests performed at HR Wallingford in the UK. It contains the following:

4.2 Short model test description. A more detailed description can be found in the appendix.
4.3 Observations made during testing
4.4 A first look at the test results
4.5 Discussion of experiment

Then some interesting findings obtained from these tests are described:

4.6 Sampling rate: The importance of a sufficiently high sampling rate. When this rate is too low, peak values are not captured properly
4.7 Longitudinal distribution: In most cases the impact peak travels along the platform following the wave crest. In some cases, though, the impact peak only occurs at certain locations.
4.8 Wave period: With constant waterdepth and wave height but decreasing wave period, waves get steeper and impact peaks get higher.
4.9 Deck clearance: The theory that vertical wave forces get higher with lower deck levels is not true. There is a certain range in the relative deck clearance (h/Ho) where the highest loads occur. When the relative clearance is above or below this range, lower forces are measured.
4.10 Comparison of test results to four wave impact models.
4.2 Model test description

In the UK, tests are being performed that are part of a research program titled "Guidelines for the Hydraulic Design of Exposed Jetties". This program is run by HR Wallingford, together with a group of companies from the UK construction, consulting and oil industry. It originates from the idea that all over the world, there is an increasing demand for longer jetties in deeper water, where larger vessels can birth. In these locations the construction of a protective breakwater can be prohibitively expensive and therefore jetties need to be able to withstand wave forces.

Since 1986, there have been repeated request from UK designers and contractors for better guidance and information for the design of exposed jetties, as existing British and European standards do not adequately address design requirements. Whilst the design market for exposed jetties in general is large, individual projects tend to be relatively small, so that new generic methods cannot be developed on any particular project. (http://www.hrwallingford.co.uk/projects/jetty/)

One part of the research program is an experiment featuring a scale model of a jetty. This model is placed in a 2-dimensional wave flume and hit by waves from a random wave field with a JONSWAP spectrum. The vertical loads from these waves on two deck slabs, placed behind each other, are measured. The scale of the model is 1:25. Forces are sampled at 200 and 500Hz.

The notation:

RH22T15Hz500Wav160

describes the results from the test of Random waves with $H_s = 0.22m$, $T_m = 1.6s$, sampling rate 500 Hz and the amount of waves in the test sequence equal to 160.

Eight transducers are installed on the model. Each channels is denoted by a two letter code, the first letter, a to d, for the four elements, the second letter, x or y, for horizontal or vertical load:

- $a_y$: vertical force on first beam element
- $b_y$: vertical force on first deck element
- $c_y$: vertical force on second beam element
- $d_y$: vertical force on second deck element

and

- $a_x, c_x$: horizontal force on first and second beam elements
- $p_x, p_y$: horizontal pressure on two locations on deck front face

This work mainly focuses on the vertical forces on the two deck slabs, $b_y$ and $d_y$. 
4.3 **Observations made during the tests**

During the tests the following observations are made:

1. In general, big waves cause big impacts. Sometimes though, big impacts do not seem to be related directly to big waves. It is the shape of the sequence of waves that determines the size of the impact. A big wave is often followed by a deep through. If after this deep through follows another considerable wave, this second wave will cause a big impact, even though its height might be smaller than the first wave.

2. Wave overtopping causes downward forces on the platform, lasting for about half the wave period. At first, the water flows over the deck from the front edge, then, as the wave crest travels along the platform, water comes in from the two sides as well. On some occasions, these three flows collide with each other in the middle of the deck, almost doubling the negative load on this area of the deck.

![Figure 4-1](image1.png)  
*Figure 4-1. Left, the three overtopping flows, right, colliding in the middle of the deck.*

3. During impact, water, foam and air shoots up through the gaps around the testing elements, with water drops reaching meters high (figure 4-2). It seems that the air shooting out increases the load on the deck, because if the air could have been trapped and compressed, it could even out some impulse. This effect is also recognised in studies on vertical wave impact. The water shooting out, on the other hand, could decrease the load, because, when thinking about the 3-D added mass hitting the deck, some mass is diverted from the deck. It is unclear how to quantify these effects.

4. You can clearly hear the impact occurring.

![Figure 4-2](image2.png)  
*Figure 4-2. Water, foam and air shooting up during wave impact, on d, left and b, right.*
4.4 A look at the test results

4.4.1 Introduction

This paragraph explains some results of the model tests. In general, every test results in a data file containing the readings, in volts, from all four transducers. This data can then be converted into forces by applying the calibration formulas defined for each transducer. By adding the time scale, which follows from the sampling rate, force-time histories are obtained. This paragraph contains the following items:

1. An example of a force-time history, together with the wave probe reading showing what wave causes the load.
2. Examples of slowly-varying positive and negative loads.
3. Example of an overtopping force, that causes a greater than expected downward force in the middle of the deck.
4. Example of single spike impact peak, which leaves the deck oscillating in its own response frequency.

4.4.2 Force-time profiles

![Graph showing force-time history](image)

*Figure 4-3. Force-time history*

![Graph showing wave probe reading](image)

*Figure 4-4. Water level just in front of the deck.*

Figure 4-3 shows a result for the vertical force on the first deck slab, sampled at 500 Hz, from a test sequence in which a wave from a random wave field of 320 waves, with $H_s = 0.22m$, $T_m = 1.5s$, hits the test slab. The slab has a clearance to still water level of 0.06m. Figure 4-4 shows the wave probe record of the actual wave causing this impact. At first, after the initial contact between the wave and the deck, the impact force rises to its peak value. With the uplift force still present, the deck responds in its own resonant frequency. Then, with the deck oscillating, the uplift force fades away and water
starts flowing over the platform, damping out the oscillatory response. After the water flows away, the force returns to the zero level.

One of the response frequencies of the test system is clearly visible in the force signal. The graph shows a sharp peak downward just after the positive impact peak. This, in fact, is just the test element oscillating with a frequency of about 10 Hz, and not an actual downward force occurring on the deck. A requirement of an accurate wave force measurement is to measure the external impact forces acting on a structure of interest only and to correct for any inertial forces due to the acceleration and subsequent vibration of the structure. This has not been done for these test results. The size and duration of the actual forces working on the deck can only be guessed. Because of this fact, these test results are not completely trustworthy from a scientific point of view. From a practical point of view, though, there is still a lot of useful information to be extracted from them. This is illustrated with the help of the next graph:

Why this graph?
Example of the slowly-varying positive and negative forces, visible even though model response influences signal.

Figure 4-5. Force-time history with slowly-varying force.

This graph (figure 4-5) shows the impact from a ‘slower’ and ‘lower’ wave hitting the second deck slab. Here, although troubled by the response oscillation of the deck slab, the peak force and the slowly-varying positive force are clearly visible. The peak force does not rise and fade in one continuous straight line. This means that the force signal is actually measuring external wave forces, and not just the inertial deck response, oscillating after a short duration impulse spike. The actual external wave force occurring most likely is a bit higher than what is measured here.

The water slams into the deck, the deck starts oscillating in its response frequency. The slowly-varying force, from the big mass of water rising up, damps out this oscillation. The positive force fades away, water again runs over the deck, causing a considerable negative force, which lasts for almost 1.5 seconds (half the wave period). The water is observed to flow over the deck from the front and from left and right. These three water flows meet up in the middle of the deck, just over the measuring elements. This collision of water might enlarge the downward force. A comparison of the forces measured on both deck slabs from this wave shows that the downward force on the first slab (seen from the front of the deck), \( b_y \), is a lot smaller than the force on the second deck slab, \( d_y \) (figure 4-6).
Another wave from the same wave field (figure 4-7). This time there is no slowly-varying positive force. The deck oscillates around the zero force level, until the water runs over the deck, causing a downward force. As can be seen in this graph, the response frequency of the deck slab (10Hz) being too close to the wave frequency (0.67 Hz) severely distorts the signal. Right after the initial force peak, the graph suggests a negative peak force to occur, which is in fact is the test set-up responding to the first peak.
4.5 Discussion of experiment

Any experiment on a scaled down model structure has certain inaccuracies. Some of the ones that occur in this test:

1. The pressure and force transducers have certain response characteristics, which differ from those of a full-scale structure. In this case, the response frequency of the set-up seems to be too close to the wave frequency.

2. The four testing elements are fitted in openings through the deck. In order to accurately measure forces predicted by the load cell calibration, they should be able to move freely. As soon as they touch the structure, there will be an error in the force reading. The gaps required around the elements for this will also let air and water escape, something that might not be possible in real life.

   With a model structure and its full-scale prototype both in the open air, the air compression forces on the scale-model are overestimated, because the air pressure should have been scaled down too. The gaps make up for this fact, by letting some of the pressure escape. It is hard to quantify, though, how much of the overestimation is compensated.

3. The end slope normally installed in the absorbing flume is, as the name already shows, able to practically completely damp out incoming waves. For reasons of logistics, another structure was still present in the flume just before this beach. A temporary absorbing slope had to be constructed for these tests. This slope did not completely damp out the wave motion. Some reflection occurs.

   This influences the waves hitting the model. Wave probes on both sides of the structure should help to account for this fact, by measuring the actual incoming wave height.

4. The measuring system used for these tests has a certain maximum output. In some tests, these values are reached, so that the actual peak value can only be guessed by completing the force graph.

5. For this report, all data has to be processed in Excel, limiting the amount of waves in a test run. The more waves are tested in one run, the more accurate the random wave field will be simulated. The value for \( H_{0.1%} \) has a small significance when testing 60 waves.

6. It is not possible to look or to make photos under the deck during the impact. This could be interesting.
4.6 A word on sampling rates

The sampling rate of a certain measuring channel is the amount of readings the computer takes each second. This number defines the accuracy of the channel reading. The wave impact process is known to be of such a short duration, that a high sampling rate is required. In the past, experiments have been performed with a sampling rate for force readings of 25 Hz. In this test, with these specific test conditions, has been found that this rate is too low. Forces have been sampled at 25, 100, 200, 500 and 1000 Hz. The shortest impact force duration measured was 0.04 s. The above mentioned sampling rates will take 1, 4, 8, 20 and 40 reading respectively in this interval. For a proper description of an impact peak, at least 8 readings are required. This means that the force gauge sampling rate should be no lower than 200 Hz. This theory is explained with the help of the figures below. The graphs show 0.4 seconds of the force on deck slab dy during a test with 60 waves, $H_s=0.22m$, $t_{ip}=1.5s$. The force gauge is sampled at 1000Hz (first graph) and subsequently 'sampled down' to 500, 200, 100 and 25 Hz.
Figure 4-8. Time-Force histories of vertical wave impact, sampled at five rates. Every dot on the line indicates a data reading.
In the impact *pressure* measurements, peaks were found to have a duration as low as 0.001 s. Even with a sampling rate of 1000 Hz only 2 to 3 readings describe this peak. The graphs below show the pressure-time record of the same wave, sampled at three different rates. Notice the change in the peak value, as the sampling rate is reduced.

**Figure 4-9.** Time-Pressure histories from one wave, sampled at 1000, 500 and 200 Hz. Dots again indicate data readings. Pressure peaks measured at 1000 Hz are not noticed at lower sampling rates.
4.7 Longitudinal distribution

This graph (figure 4-10) shows the readings from all four vertical gauges at the same time, as one of the biggest waves from the test matrix passes under and over the deck. All gauges are shown to 'feel the wave' in succession. The impact peak seems to be very localised on the second deck, as no other gauge produces high values. This means that not every wave that passes the platform produces impact peaks along the whole deck. Some waves do, as is shown in the next graph:

Figure 4-11 shows the impact peak from another wave actually travelling under the platform deck. All four gauges 'feel the wave' in succession. Deck slabs b_y and d_y have an area about three times larger than the beams a_y and c_y. This clearly shows in the graph, as the forces on the deck slabs are three times bigger.
4.8 The effect of a change in the wave period on the magnitude of the impact

Various sets of tests have been performed with only the wave period changing. Comparing the results from these tests can give an insight into the effect of a change in wave period on the magnitude or shape of the wave impact. The theoretical prediction for this effect has two conflicting components:

1. When the wave period increases with constant wave height, the vertical particle velocity decreases, decreasing the velocity of the water hitting the deck, thus decreasing the magnitude of the impact force on the deck.

2. When the wave period increases with constant wave height, the wavelength increases, increasing the length of the mass of water hitting the deck, increasing the magnitude of the impact force on the deck.

When looking at the test result, it is found that the first component has the strongest effect. As an example, a comparison is made between the results of two test runs, which contain waves with the same wave height (0.22m), but with different wave periods (1.5s and 2.5s). Figure 4-11 shows graphs of 10 seconds from each run. Observation and calculation learns that the wave field with $t_n=1.5s$ produces:

1. a more violent wave field
2. more impact peaks
3. higher impact peaks

The following four graphs illustrating this result.

---

Figure 4-12. 10 seconds from two wave conditions with a different wave period only.
Figure 4-13. 3 seconds from two wave conditions with a different wave period only.

Figure 4-13 shows another comparison of two wave conditions, both again with the same wave height (0.14m), but different wave periods (1.5s and 3s). Apart from the three observations mentioned above, it seems from this comparison that the slower wave produces a stronger slowly-varying positive force.

Recalling the two theories on this matter formulated above, it seems that the first one is valid for peak forces, the second one for the slowly-varying positive force. The amount of waves tested in the two runs compared does not allow for a strong conclusion on this point.
4.9 **The effect of the deck clearance on the magnitude and shape of the impact**

The test program allows to investigate the effect of deck clearance on the wave loads. First, the theoretical prediction: Models depending on the wave particle velocity (Elghamry, Kaplan, Ridderbos) say that the particle velocity at impact and thus the magnitude of impact is higher when the deck is lower. This is probably also the general opinion shared by most people. Shih and Anastasiou say that air effects overshadow the relation. The results from these tests show a clear, but opposing relation. In this experiment, two deck clearances are tested. Of all the wave conditions tested there is no single case where a certain wave shows higher peak forces on the lower deck than on the higher one.

When the deck is low, the wave impact peak is less developed and the magnitude is low. When increasing the deck level, the uplift force increases first, up to the point where the relative clearance ($h/H_o$) is about 0.40. A further increase of the deck level shows a decrease in peak force values.

*Figure 4-14. The effect of deck clearance. The same wave with two different deck heights. In case of a low clearance, wave loads do not really seem to have enough time to shape into an impact peak.*
In figure 4-15, peak impact forces are shown in relation to the relative deck clearance, which is the deck clearance \( h \), divided by the wave crest height (still water level to crest) \( H_c \). This value is 0 when the deck is at still water level and 1 when the top of the wave crest just touches the deck. Wave force models linking the wave force to the water particle velocity (Kaplan, Elghamry, Ridderbos) would assume the peak force to be the highest for a small relative deck clearance, and to decrease with increasing clearance (this expected relation is illustrated by the dotted lines). These tests show another relation, as shown above. Care should be taken though, as only two deck clearances were tested. No values were found in the dataset for a relative deck clearance between 0.15 and 0.29.

For the moment, and with the above considerations in mind, the following relation can be applied to a force dataset:
4.10 Comparing test results to available models

The test results can be compared to some of the available models. Before this is done, some remarks have to be made about the accuracy of the peak force values obtained from the HR test results. As has been shown before, the force signal is distorted by the internal response of the test set-up. After being hit by the impact peak, the force signal shoots of into an oscillatory motion, which is actually influenced by the remaining slowly-varying force effects. This, of course, is very much the wrong way around. The force signal should follow the external force effects, with no or very little influence from the internal response of the test set-up.

A little review of model tests describes in other publications shows that the phenomenon described above is not only seen in these tests. The large-scale flume tests described by Ridderbos and the measurements on the Eastern Scheldt described by Klutter show the same effect.

All this makes that the test results should be used only with a certain accuracy range in mind. This actually is a general rule for all model tests. The hard part is to define the correct range for each test. As a first assumption, without any reference or guidelines, the following accuracy range is used for force values obtained in these tests:

- in model scale: -25 to +25%
- in full-scale: -50 to +50%

Chosen for the comparison of the maximum impact forces are the following models, as each of them uses a different approach to quantify impact:

**Shih and Anastasiou**, both their minimum and maximum estimates. The impact pressure is directly related to the significant wave height. Shih and Anastasiou only consider impact pressures. To make this comparison possible, the impact pressure is supposed to work on a strip parallel to the wave crest, all along the width of the deck, 0.1m wide.

**Kaplan**, 3-D added mass model, impact force depends on particle velocity and acceleration, calculated here with linear wave theory.

**Ridderbos**, impact pressure related to particle velocity squared and the direction of impact. Pressure again working on 0.1m.

**Overbeek and Klabbers**, hydrostatic relationship between the impact pressure and wave height, added to a slowly varying component.

The exact formulas used for this comparison can be found in appendix A3.

And the following waves:

1. Hs = 0.22m  Tm = 2.5s  L = 6.27m
2. Hs = 0.14m  Tm = 2.5s  L = 3.15m
both with waterdepth = 0.75m, deck clearance = 0.06m

and

3. Hs = 0.22m  Tm = 1.5s  L = 3.15m
with waterdepth = 0.75m, clearance = 0.01m
As can be seen in these two figures, the differences are considerable. The models by Kaplan and Ridderbos, both related to the particle velocity, produce very small values. As will be shown below, the Ridderbos formula gets a lot more realistic when the clearance becomes smaller. Kaplan’s model seems to constantly underestimate the impact force.

Figure 4-16 shows a comparison of forces predicted and measured in the case of a low deck clearance of 0.01m. All wave conditions tested show lower forces than in the case of the 0.06m deck clearance. The comparison shows a different result than before. The predictions by Shih and Overbeek do not change, as their models do not consider the deck clearance. Kaplan and Ridderbos predict higher values, with Ridderbos predicting values a hundred times higher.
All these comparisons are made at model scale. When using prototype values, the following values are predicted. The number under the name of the researcher indicates the factor the model value is multiplied with to get the prototype value. The HR tests are scaled in two ways. First by using Froude’s scaling law (scale^3), secondly, by a revised Froude, found in literature on vertical impact (0.5 – 0.7 × Froude). This second approach has never actually been proved to be valid for horizontal platforms too.

![HR14T15 scaled chart](chart.png)

*Figure 4-17. Comparing model results at prototype scale.*

From these four comparisons it seems that only the simple model by Shih and Anastasiou produces comparable values for the magnitude of wave impact. The range between its minimum and maximum values is considerable, nevertheless. Perhaps some specifications related to the deck level and the wave period can the model more accurate.
5 Estimating wave loads

5.1 Introduction

From the current amount of knowledge of wave impact on horizontal platforms, it is not yet possible to construct an accurate impact model that is valid for any kind of horizontal structure. More data is needed first, from more model tests of more different platform configurations, at more scales, including prototype scale. It should also be checked whether the horizontal platform theories are valid for pile caps, deck beams and other smaller elements.

With the results of the tests described in this paper, an impact model will be calibrated here. This model will be valid for jetties with similar configurations in a similar waterdepth loaded by a similar wave climate.

A good wave impact model should predict three things:

1. the maximum short duration peak force, $F_{\text{peak}}$ 
2. the maximum slowly-varying upward force, $F^+$
3. the maximum downward force, $F^-$

paragraph 5.2
paragraph 5.3
paragraph 5.4

These three force components are generated by one wave crest passing the deck. The platform area should be divided into gridlines, spaced one wave length apart, perpendicular to the direction of wave propagation. Each area between lines now contains one wave crest, and one set of forces $F_{\text{peak}}$, $F^+$ and $F^-$ should be calculated on it. From the figure below it also becomes clear what length should be used for the strip on which the peak force works.

![Figure 5-1. waves reaching the harbour platform.](image)

In paragraph 5.5 some general guidelines will be given on designing harbour jetties.
5.2 $F_{\text{peak}}$

The short duration peak force should be allowed to vary with:
- wave height
- wave period
- platform layout (dimensions, presence of beams, presence of gaps)
- air entrapment
- geometry of the bottom line
- deck clearance

A suggestion:

$$F_{\text{peak}} = C_c C_i C_s \rho g H_i x$$

Where:
- $F_{\text{peak}}$ = the maximum peak force on the platform, per unit width (N/m)
- $C_c$ = clearance coefficient (-)
- $C_i$ = wave period coefficient (-)
- $C_s$ = slamming coefficient (4 to 8) (-)
- $C_b$ = coefficient for the bottom line (1.0 for constant depth) (-)
- $\rho$ = specific weight of water (kg/m$^3$)
- $g$ = acceleration of gravity (m/s$^2$)
- $H_s$ = significant wave height ($H_{1/3}$ through to crest) (m)
- $H_c$ = wave crest height (still water to crest) (m)
- $x$ = width of impact strip, depending on size of wave (m)
- $h$ = deck clearance (m)
- $L$ = wave length (m)

![Influence of deck clearance](image)
![Influence of wave period](image)

Figure 5-2a/b. Two coefficients taking into account the deck clearance, $C_c$ and the wave period $C_s$.

This expression is derived from the wave impact theory of Shih and Anastasiou, as it seems to offer safe values for every wave considered so far. In their paper on the subject, Shih and Anastasiou only look at impact pressures. An assumption needs to be made about the expected area this impact pressure is believed to work on. In this report, the area suggested is a strip one meter wide, parallel to the wave crest. For waves hitting a long structure, some maximum crest length should be introduced, as it is very unlikely that a long harbour platform will be hit by a wave crest along its whole length simultaneously.

The coefficient $C_s$ contains the effects of the platform layout and air entrapment. Its range should be determined for each specific platform from model tests or, on a later date, from experience. For the test results used in this report, a range of 4 to 8 seems correct. The coefficient $C_b$ is used to account
for the effect of the geometry of the bottom line. For a non-sloping bottom line $C_p$ equals 1. $C_p$ should increase with increasing slope of the bottom line. This effect has not been quantified in this report.

Two extra coefficients ($C_t$ and $C_c$) are introduced (figure 5-2a/b) to account for the reductions found in this work related to wave period and platform clearance. The value for the crest height $H_c$ that is used in the reduction for deck clearance should be related to a maximum expected wave ($H_{0.1\%}$).

### 5.2.1 Validation with model results

Test run RH22T15 Hz200 Wav320

\[
\begin{align*}
F_{\text{peak}}
C_c & : H_{c, \text{max}} = 0.23m \rightarrow h = 0.06 \rightarrow h/H_{c} = 0.26 \rightarrow C_c = 1.0 \\
C_t & : H/L = 0.065 \rightarrow C_t = 0.85 \\
C_p & : 1.0 \\
C_s & : 4 \text{ to } 8 \\
\rho & : 1025 \text{ kg/m}^3 \\
g & : 9.81 \text{ m/s}^2 \\
H_s & : 0.22m \\
x & : 0.1m \\
b & : 0.2m
\end{align*}
\]

\[
F_{\text{peak}} = 1.0 \times 0.85 \times C_s \times 1025 \times 9.81 \times 0.22 \times 0.1 \times 0.2 = 37.6 \times C_s
\]

\[
F_{\text{peak}} = 150 \text{ to } 300 \text{ N}
\]

Maximum force measured

180 N

Some more test runs (actual values can be found in the appendix):

![Graph comparing predicted results for peak forces to measurements. Bar shows predicted range, square shows measured value.](image)

**Figure 5-3.** Comparing predicted results for peak forces to measurements. Bar shows predicted range, square shows measured value.
5.3  $F^+$

The slowly-varying positive pressure is lower in magnitude and of longer duration than the impact force. This pressure is of interest because it works on the whole area of the deck submerged by the wave. For waves with a relatively low steepness, the slowly-varying component can be larger than the impact peak, which only works on the area around the wave crest.

Following Kaplan's work, the slowly-varying force, per unit width, is made up of two components:

- a buoyancy force, per unit width:
  \[ F_b = \rho g \left( \frac{1}{4} \eta_{left} + \frac{1}{2} \eta_{middle} + \frac{1}{4} \eta_{right} \right) \]

- a drag force, per unit width:
  \[ F_d = \frac{\rho}{2} l C_d v \]

Where:
- \( l \) = wetted length, related to wave celerity (m)
- \( C_d \) = drag coefficient (-)
- \( \eta \) = water level – level underside deck (m)
- \( v \) = vertical water particle velocity (m/s)

Shih and Anastasiou propose a simple hydrostatic relation:

\[ F_\ast = C_\ast \rho g (H_\ast - d) l \]

Where \( F_\ast \) is the slowly-varying positive force, per unit width, and \( C_\ast \) is a slowly-varying positive force coefficient, which Shih and Anastasiou say is around 0.65.

A comparison can be made between the results of these two models and the measured test values:

![Comparing measured to predicted forces](image)

Figure 5-4. Comparing measured forces to predicted values.

Shih's formula, when using a factor 0.65, seems to overestimate the slowly-varying positive forces. Kaplan's solution both over- and underestimates on occasions. Shih's method is a lot simpler to use. The coefficient should be made dependant of the deck layout. High values, around 1, should be used for large and thick platforms, without gaps or openings, where no or little water flows over the deck. Low values, of around 0.20, apply on thin or grated structures. When using this theory on the model test values, a coefficient of 0.50 seems to be correct.
5.4 $F_-$

The negative force should be dependant on the amount of wave overtopping.

$$F_- = C_-(H_{sw} - x)a\rho_wg$$

Where:
- $F_-$ = the maximum negative force on the platform, per unit width (N/m)
- $C_-$ = slowly-varying negative coefficient (-)
- $x$ = level of top of the platform, measured from still water level (m)
- $a$ = length of the platform (m)

Depending on the shape of the platform it could occur that overtopping flows collide, like has been observed in the model tests. This collision can double the downward load on the area deck of the deck where this collision occurs. Long jetties loaded by waves head-on are most likely to experience this effect. When waves hit this jetty normal to the longitudinal axe, the deck will be inundated from one direction only. The coefficient $C_-$ is used to account for this effect.

Again, a comparison is made between the values predicted by the model and the measured values:

![Comparing measured to predicted negative force](image)

*Figure 5-5. Comparing measured values to predicted values of slowly-varying negative forces, using factor $C_-$ =1.5*

Most values are predicted safely. Some small waves are underestimated. Again, the coefficient $C_-$ depends on the deck layout. High values should be used for decks on which overtopping collision can occur. Low values on decks without gaps or openings and with the sides closed off, which prevents overflowing.
5.5 Some guidelines on designing a harbour jetty

5.5.1 Introduction
This paragraph will offer some general guidelines on the design of harbour platforms. The guidelines will focus on the effects of vertical wave loading. Some measures reducing vertical loads might invoke practical or economical problems in other aspects of the design. Choices will have to be made in every project about what the optimal solution would be.

5.5.2 Structure layout
The platform layout has been found to have a considerable effect on the wave loads.

Starting at the bottom. The platform substructure influences the way waves hit the deck. Pile caps, transverse beams and wave screens all distort the wave profile and add roughness to the deck, reducing the vertical wave loads. Bigger pile caps and beams do however increase the horizontal loads of waves breaking on the structure and are more expensive to build. This is one example of the type of design choices explained above.

A grid of deck beams, both in longitudinal and transverse direction, will ‘break up’ the wave shape into smaller patches, again, increasing the roughness and absorbing wave energy. Contact with the sides of the beams distorts the wave profile and mixes air into the waterfront. When a wave crest enters a grid box, air and foam will get trapped in the area between the underside of the deck, the beams and the wave surface. The air will compress and thus even out impact loads. The shape and dimensions of the grid of deck beams will have to be chosen in accordance with the shape of the typical waves.

When deck beams are spaced five meters apart, waves with a length of less than 20m will probably hit the deck without noticing the presence of the beams. The distance between deck beams should be no larger than between 1/8 to 1/10 times the wavelength.

![Wrong](image1)

Wrong

![Right](image2)

Right

Figure 5-6. Dimensions of the grid of beams should follow the shape of the waves expected.

5.5.3 Deck clearance
Deck clearance has been shown to influence the shape and magnitude of peak impact forces. Obviously, a deck clearance greater than the highest expected wave will solve all impact problems. In lower levels, though, there is still some optimisation to be done.

As has been explained in paragraph 4.9, the most severe wave impact occurs in the situations where the relative deck clearance ($h/H_{r,\text{max}}$) is between 0.3 and 0.5. For higher values, the occurrence of smaller forces is easy to explain. In waves touching the deck at higher levels, the vertical particle velocity will be near its minimum and the horizontal particle velocity will be near its maximum, turning the water particle direction away from the deck. This effect can be compared to a case of two cars with the same speed, one hitting a wall head-on, and one hitting the side barrier at a small angle. The last car will be a lot better of.

New insights have been gained for smaller values of the relative deck clearance. The initial thought here is that the higher vertical velocity and the lower horizontal velocity will increase the impact
magnitude even more. It turns out this is not the case. With the deck level being low, the wave profile has not ‘got the time’ to get into shape for a proper wave slam. The wave profile is low and rounded, instead of high and peaked. The impact momentum is not transferred onto the deck only, like in the case of a narrow peaked wave crest, but is also absorbed by the surrounding water mass, moving around and under the deck. A quasi-static force occurs, resembling the slowly-varying uplift force. Those slowly-varying forces, both positive and negative, will still increase with decreasing deck level, as they are almost linearly dependent on the deck clearance.

Care should be taken when determining the deck level in a way that the highest expected wave has a relative deck clearance between 0.0 and 0.2. Although this highest wave will not cause an impact peak, any smaller waves in a different storm field will still do, as their relative clearance is higher. This rule of thumb is better used the other way around: do not put the deck at a level where the highest expected waves will cause the most severe wave impacts. For instance, in an area where the maximum storm is expected to produce waves with $H_s = 6.0 \text{ m}$, $H_{cr, \text{max}} = 5.0 \text{ m}$, do not put the deck between 1.5 and 2.5m above the design water level ($h/H_{cr, \text{max}} = 0.3 - 0.5$).

5.5.4 Deck layout
When designing the actual jetty deck more choices can be made which reduce the wave loads on the deck. The most effective measure is to construct a perforated or grated deck. On a fully grated steel deck almost no vertical loads occur, as waves simply pass through the holes in the deck. Grated decks are used on many offshore platforms. On harbour jetties, a perforation of 5% of the deck area has been shown to reduce vertical loads with 10 to 25%. This significant reduction is results from the venting of air and water from beneath the deck, relieving the hydrodynamic pressure below the deck and increasing the volume of water on top of the deck. On some jetty decks perforations or permanent gaps are not allowed or simply not possible, either because of requirements of cargo handling equipment moving on the deck, or sometimes because cruise ship passengers, especially disabled people in wheelchairs and ladies on high heels, require a solid deck for safe passage. On these occasions a solution proposed by DMC could be used, where gaps between the concrete deck plates are covered up with timber beams, which shoot out as the first waves hitting the deck, thus opening up the gaps.

On a smaller scale, deck underside roughness seems to reduce wave impact loads, again through the principal of compressing air.

If slowly-varying negative forces turn out to be critical in the design of the deck, measures could be taken to reduce wave overtopping. An upstanding ridge could be constructed around the edge of the platform, increasing the height of the platform important to wave overtopping.

5.5.5 Bottom slope
When wind-waves reach a sloping bottom line, their amplitude will increase and their length will decrease. This will continue up to the point where the wave cannot maintain its stable shape and will start breaking. A harbour jetty situated in a shoaling area will experience this same phenomenon. If the jetty is surrounded by harbour walls or steep rocky coasts, there will be the added effect of wave reflection and amplification, which causes waves and breakers under the platform to reach heights of up to twice the incoming wave crest height. Wave slamming loads will increase. The numerical model by Lai and Lee predicts the following increase of the peak force as compared to a non-sloping bottom:

<table>
<thead>
<tr>
<th>Slope</th>
<th>Increase</th>
<th>$C_p$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1/200</td>
<td>20%</td>
<td>1.2</td>
</tr>
<tr>
<td>1/100</td>
<td>40%</td>
<td>1.4</td>
</tr>
<tr>
<td>1/50</td>
<td>60%</td>
<td>1.6</td>
</tr>
</tbody>
</table>

*Table 5-s. Coefficient for bottom slope.*

This relation takes into account the effect of shoaling. It has not been verified in this report.
6 Conclusions and recommendations.

6.1 Conclusions

This report has tried to shed some light onto the subject of wave impact on horizontal platforms. Although the problem has not been solved completely, a framework has at least been created containing the important issues governing wave impact. This framework should support further research on the subject.

The wave impact phenomenon is a pretty complex one. No statical terms seem to be able to accurately describe its occurrence. The shape of the wave hitting the structure has been found to determine the size, shape and duration of impact loads. At the moment of contact, this shape is dependent on three sorts of parameters:

1. Wave parameters, like the wave height and wave period (length, steepness).
2. Parameters describing the sea state surrounding the wave, like the waterdepth, the bottom profile, the amount of reflection from the sides if the basin and the influence of previous or subsequent waves.
3. Parameters describing the structure loaded by the wave, like the size, shape and strength of the platform and substructure, the deck clearance and its response characteristics.

Any wave impact model should take these parameters into account, or at least state for which ranges it is valid for. The actual magnitude of the wave load is dependent on a combination of two things: the available impact energy and the possibilities for this energy to be transferred onto the object. The amount of impact energy is related to the water particle velocity in the direction of the object and the amount of water (mass) in the wave. The possibilities for this energy to be released onto the object are determined by the shape of both the wave profile and the object. With this relation in mind, the influence of each parameter can be determined.

The influence of most parameters is quite straightforward. In higher or steeper waves, the water particle velocity increases, increasing the amount of energy available for impact, increasing the magnitude of the impact peak. The influence of a parameter like deck clearance, though, turns out not to be so straightforward. When the relative deck clearance \((h/H_o)\) is low, the amount of energy available increases, the same way as described above. Measurements showed this effect to be more than compensated by a change in the other halve of the combination determining the magnitude of wave impact, the way the energy is released onto the object. At lower levels, the wave is not yet shaped into a peaked shape delivering peak forces, but more like a body of water overflowing the deck, creating quasi-static loads.

![Figure 6-1. The same wave, hitting the deck at two different levels.](image)

After calculating the wave loads, the second question arises about how these loads actually affect the structure under consideration. In the literature two opposing views exist on what effect peak impact forces have on harbour jetties and platforms.
First there are those who say that the short duration \( t_a \approx 0.04 \sim 0.1 \) s peak forces need not be considered, as their effect on the structure will be negligible. Researchers like Broughton & Horn and Kaplan & Murray claim that heavy platforms with a low response frequency will not notice the high frequency impact spikes and only quasi-static load components should be considered. Broughton and Horn translate this view into their choice of sampling rate. They say that when performing model tests, a sampling rate of 20Hz is sufficiently high, as any higher frequency effects can be disregarded.

The second view is that peak forces should be considered indeed. Depending on the size of the deck slab, peak forces can reach magnitudes three to four times higher than the dead weight of the deck. These pressure spikes will perhaps not completely lift the deck from its supports, but the repetitive pounding blows will surely cause local damage to the deck or to its support structure. This second view is held by researchers like Shih & Anastasiou, Ridderbos, Cornett et al. and also by this report.

Damage found on deck slabs of a jetty designed by DMC in the Caribbean supports the second view. The design of this jetty asked for the deck slabs to be removed in the case of a hurricane approaching. After a hurricane passing the island unexpectedly produced considerable swell waves, the local staff failed to remove the deck slabs in time, and the jetty was hit by waves with a considerable height. After the storm, the deck slabs showed bending cracks on top, indicating heavy loading from below. The structure also suffered local damage due to the movement of deck slabs. The deck has a clearance of about one meter and was hit by waves with a maximum crest height of about two meters. Quasi-static loads could never have caused this kind of damage. Peak loads, however, when using the model proposed in this report for an order of magnitude prediction, could have been between 120 to 240 kN per deck slab, with a slab weighing around 100 kN. This could definitely have caused the damage described above.

A remark should be made here on the impact duration. This duration seems to be related to the speed of the wave and the size of the area under consideration. Smaller pressure transducers measure a smaller impact duration. When the same transducers are used at two different scales, the duration is found to be almost equal. Froude's scaling law predicts the duration to scale as follows:

\[
t_a = \sqrt{\frac{A}{A_b}} t_B
\]

With \( A \) denoting a characteristic measure of length, \( t \) a characteristic measure of time and the subscripts a and b the two different scales.

This relation should only be used to translate impact Force duration from model scale to prototype scale. In the experiments described in chapter 4, the impact force duration on a 0.2 x 0.2m deckslab is found to be between 50 to 150 ms. The prototype, which measures 5 x 5m, will experience a peak force duration between 250 to 750 ms, or 0.25 to 0.75 s. This result equals the time it takes for the wave crest to pass the deck.

### 6.2 Recommendations on designing jetty decks

There are many criteria that influence the design of a harbour jetty. The preliminary study investigating wishes and demands of future users will often result in requirements on size and location of the proposed jetty, including the minimum waterdepth it should be in. There will be an estimate of the deck loads to be expected, from cargo and cargo handling equipment on the deck. The engineer now has to collect wave data for the area, either from an available database or by performing measurements himself. This wave data has to be translated into wave loads with a certain chance of occurrence.

The wave impact loads should first be estimated by using models from literature. If, when applying all the loads on the jetty, these wave impact loads turn out to be critical in the design, further research is necessary to accurately predict the wave loads. An experimental study on a scale model of the future jetty is a way to do this.
For these model tests the following recommendations are made:

On the model scale:
The exact manner in which scale effects influence the test measurements is still not completely understood. A certain error is introduced by the fact that the pressures exerted by entrapped and entrained air are not scaled down properly in regular scale model tests. This error gets bigger at smaller scales. That is why a higher model scale is preferred. When testing at a higher scale, though, the total cost of the experiments is somehow scaled up too, as bigger facilities are needed. An optimisation should be performed, weighing test accuracy to the amount of savings expected. The minimum scale advised here is 1:25. At lower scales, the waves and the structure get too small and air effects take over the process.

On the sampling rate:
This report has illustrated the importance of a correct sampling rate. For force measurements this rate should be at least 500Hz. When measuring impact pressures, the minimum sampling rate advised is 1000Hz.

On the wave conditions:
It is no use testing small waves. The highest forces are going to occur in wave fields with the highest expected significant wave height. The highest peak might not be caused by the highest wave in the spectrum, but it will occur in this test.

On test-structure design
In order to accurately measure the external forces on the model, the response frequencies of the test set-up should be kept well away from the wave frequency. A simple test to measure peak forces only could consist of a horizontal frame with an opening on which a stiff board is placed. This board is then loaded with weights. If a certain wave lifts the board up while passing under the deck, the weight should be increased. If this same wave now does not manage to lift the board, the maximum wave load can be said to be something less than the weight on the board.

On the scaling of model values to prototype scale
When scaling model values to prototype scale, Froude's scaling law should be used if gravitational forces are the predominant mechanism driving the process. At a first glance, this seems to be the case for the wave impact phenomenon. For the slowly-varying force components, Froude's law seems to give adequate results. But, when scaling the peak impact values, prototype values are being overestimated. This overestimation gets bigger as the model scale gets lower. The effects of air compression are believed to be causing this overestimation. In regular model tests, the air pressure is not scaled down. When this relatively high air pressure gets trapped between the wave and the deck, high peak forces are measured.

Most reports found in the literature on wave impact on horizontal platforms use Froude scaling, sometimes even without mentioning it. In research on wave impact on vertical walls, impact pressure scaled up from small scales showed an overestimation, especially of the high peak values, of up to 500%. This result proves that the subject should be looked into further. It greatly affects the accuracy of prototype force values. A simple test should be performed with two scale models of the same structure, scaled at two different rates, if possible one being at least four times bigger than the other.

One way to avoid the problems of scaling would be to perform experiments on prototype scale, like the tests on the Eastern Scheldt Barrier discussed earlier in this report. Air compression pressures can be scaled down by performing tests in a vacuum tank, in which the air pressure can be scaled down according to the model scale used. These last two methods can undoubtedly produce useful fundamental results, but the cost of such tests will be very high.
7. References


Bagnold, R.A. (1939) “Interim report on wave-pressure research” Journal of the institution of Civil Engineers, pp 202-227


Faltinsen, O.M., Sea loads on ships and offshore structures, 1990.


Kaplan, P. (1992) "Wave impact forces on offshore structures: Re-examination and new Interpretations" Offshore Technology conf., OTC 6814, Houston, USA, 4-7 May


Matilla, M., Tarbottton, M., Cornett, A. (1998) "An investigation of wave forces for design of a cruise ship pier Bridgetwon, Barbaros" Proc. Wave '98, Ocean wave kinematics, dynamics and loads on structures. ASCE, Houston, USA


Appendices
A1 Should the Froude law be used to scale model test results to full-scale values?

Description of the Froude scaling law

Measurements obtained from hydraulic models of ocean wave action, in which gravity is the predominant force, are converted to full or prototype scale by application of the Froude law. This law is based on the concept that for dynamic similarity between two geometrically similar systems A and B, the ratio of the inertia force to the gravity force must be the same and hence the two systems should operate at the same Froude number, i.e.:

\[ Fr_A = Fr_B \iff \frac{u_A}{\sqrt{\lambda_A g}} = \frac{u_B}{\sqrt{\lambda_B g}} \]

Where \( u \) is a characteristic velocity and \( \lambda \) is a characteristic length. If \( \rho \) is the density of the fluid, the pressure in system A is then related to the pressure in system B by:

\[ P_A = \frac{\rho_A \lambda_A^2}{\rho_B \lambda_B^2} P_B \]

and the following parameters in the two systems are related by:

- time \( t_A = \frac{\lambda_A}{\lambda_B} t_B \)
- velocity \( v_A = \left( \frac{\lambda_A}{\lambda_B} \right)^{\frac{1}{2}} v_B \)
- mass \( m_A = \left( \frac{\lambda_A}{\lambda_B} \right)^3 m_B \)

Impact forces caused by breaking waves cannot always be modelled using the Froude number. Compression of the trapped air by the breaking wave and subsequent oscillation of air bubbles give model scales which deviate from the above mentioned scale law. Froude’s scaling will over-estimate prototype loads [VINK (1997)]. In the case of horizontal platforms, air entrapment is said to reduce peak pressures. When using Froude’s law to scale model results to full scale, the effect of this reduction will not be taken into account properly. Only in the case of quasi-static wave forces, where the relationships between wave momentum, pressure impulse and horizontal force are relatively simple, the assumption of Froude scaling is realistic.

The general opinion found in the literature on waves on vertical walls is that wave impact in small scale hydraulic model tests will be greater in magnitude and shorter in duration than their equivalent (using Froude) at full scale in (invariably aerated) seawater. This is due to the combined influence of the increase in compressibility of the air/water mixture and the observed change in the wave profile. On the other hand, a comparison between large scale model tests and different small scale tests suggest that Froude scaling should be used to obtain an upper limit for the impact force to be expected. It may be noted that the largest forces occur when the least amount of air is entrained or trapped, and these impacts may be less influenced by scale effects on air compression.

Shinh and Anastasiou (1992) try to explain scaling difficulties by performing tests at two different scales. The tests carried out in the larger facility correspond to a scale four times greater than that of the smaller facility. The following graphs show the ratios between the small and large scale for the
maximum and mean impact pressure, the duration and the impulse for five sets of comparable tests. The ratio expected when applying Froude's law is also displayed. As can be seen, no clear trend can be given on the maximum pressure. Of the five values calculated, three are significantly larger than the expected ratio, two are smaller. Especially the larger values measured make it hard to transfer small scale results to maximum prototype values. The ratios of the mean values are all smaller than the ratio expected. The measured ratios for duration and impulse are both between two and four times smaller than the expected Froude ratio.

![Graphs showing ratios of expected and measured impact pressure, duration, and impulse](image)

*Figure A1-1. Comparison of measured values and values expected using Froude's law, from [SHIH AND ANASTASIOU (1992)]*
In a paper from 1996, Howarth et al. shed some light on the subject. They perform tests on two models: one at prototype scale and an identical one at a scale of 1:32. All dimensions and the wave and tidal conditions measured at the full-scale model are recreated as closely as possible in the laboratory. Any differences in the pressures measured at the model, when scaled using Froude, and prototype scale are therefore likely to be due to the scale effects present in the wave impact process.

Considering wave impact on a vertical harbour wall, both impact magnitude and rise time are found to correlate accurately with a log-normal distribution, one full-scale as well as on model scale. When comparing the distributions of impact magnitude on both scales, though, model results are found to show an increasing overestimation (up to 500%) for increasing magnitude.

The following empirical relation is suggested:

\[
\frac{P_{\text{prototype}}}{\rho g} = \left( \frac{P_{\text{model}}}{\rho g} \right)^{0.684}
\]

Where:
- \(P_{\text{prototype}}\) = Impact pressure on full-scale
- \(P_{\text{model}}\) = Impact pressure scaled from model test using Froude

This relation has been applied to the data from the Shih and Anastasiou flume tests mentioned above. To a certain extent, it seems to correct the ratio between the measured \textit{mean} impact pressure on prototype scale and the overestimated prediction from model scale. The \textit{maximum} impact pressure on prototype scale, on the other hand, was found on occasions to be both over- and underestimated when using Froude on model values. Any underestimation will only get worse after applying the empirical relation mentioned above.

For the impact duration, Howarth et al. claim the opposite holds true. Model results scaled with Froude underestimate the full-scale rise times. The reduced compressibility of the wave front at model scale leads to sharper impact pressures with short rise time and large magnitude. This means, for instance, that when model tests are performed in the design of coastal structures for which a minimum rise time is believed to be of any influence, impact rise times smaller than the value obtained when scaling down with Froude’s law should still be considered.

Howarth et al. say that Impact impulses scale reasonably well when Froudian scale is used. This doesn’t follow from the Shih and Anastasiou data. Perhaps that dataset is too small to draw relevant conclusions from.

Overall, the conclusion should be that the scaling problem of wave forces on horizontal platform has not been solved yet. Applying the straight Froude approach to model results seems to overestimate forces. After reading the work of Howarth et al., a modified Froude rule is adopted in this work, which calculates prototype forces by multiplying the Froude result with a factor. This factor is, without proof, believed to be between 0.5 and 0.7.
A2 Model test description

A.2.1 Test set-up

One part of the program is to measure wave slam and uplift forces on a physical model in a wave flume. The main objective of this test is the measurement of wave forces applied to an idealised model of a jetty structure from various sea conditions. Of major concern are:

- Horizontal slam forces on beams and fenders (not considered in this work),
- Vertical slam forces on beams,
- Vertical forces on deck slabs; slam, +ve and –ve.

The test program discussed here is essentially 2-dimensional with head-on wave attack in a narrow wave flume. The model does not span the whole width of the flume, allowing water rise around the sides of the platform. This introduces a 3-dimensional effect.

A review of several case studies of existing jetties has determined the choice of a prototype structure that is to be simulated by the model and its dimensions. It consists of:
1. a jetty head (wooden frame)
2. a downstanding frame of cross and longitudinal beams (wood)
3. Testing elements: two beams and two deck elements (aluminium)
4. Supporting piles mounted on a base frame (steel)
(See figure A2-1, A2-2 and A2-3)

![Diagram of jetty dimensions](image)

Figure A2-1. Typical jetty dimensions, in meters.
Figure A2-2. Model test device. Plan view from below.

Figure A2-3. Model test device: supporting frame with piles

The facility available at HR Wallingford for this test has an upper limit for the wave height $H_w$ of about 0.20m. A typical prototype wave conditions for an exposed jetty is defined as $H_w \equiv 5m$. This results in a maximum model scale of 1:25, a scale that is believed to be above the value where scale effects influence any major wave force. (For more on this subject, refer to the section on Froude scaling)
With this physical model it is possible to:

- measure the wave loads while varying deck clearance, water level and wave conditions,
- measure wave loads on a flat deck (not dealt with in this report),
- explore the effect of trapped air between downstanding beams under the deck,
- measure the longitudinal distribution of wave loads,
- test the effects of wave obliquity and/or short-crestedness by placing the model in a 3-dimensional wave basin (not dealt with in this report),

A.2.2 Measuring equipment

The testing elements are connected to force gauges that are able to measure the horizontal (in the direction along the flume) and vertical forces acting on the elements. These gauges have been calibrated in a range of about −120 to +120 N. To validate the readings two pressure transducers are placed on the front face of the jetty.
The test set-up is not stiff. The deflection of the deck slabs needed to produce the wave reading is in the order of 5 to 10 millimetres, clearly visible with the naked eye. The response frequency of the test set-up in air is in the order of 10 Hz.

In order to get an accurate correlation between the wave height and the loads on the deck, the wave flume is fitted with three wave probes. One probe is placed near the wave generator and displays the offshore wave height. The second probe measures the incoming wave height just before the model structure, and the last one is placed just behind the model and provides a measure for the amount of reflection from the back-slope in the flume and the effect the model has on the wave propagation.

**A.2.3 Notations**

The notation:

RH22T15Hz500Wav160

describes the results from the test of Random waves with $H_s = 0.22m$, $T_m = 1.5s$, sampling rate 500 Hz and the amount of waves in the test sequence equal to 160.

The channels are denoted by a two letter code, the first letter, a to d, for the four elements, the second letter, x or y, for horizontal or vertical load:

ay vertical force on first beam element
by vertical force on first deck element
cy vertical force on second beam element
dy vertical force on second deck element

and
ax, cx horizontal force on first and second beam elements
pa, pb horizontal pressure on two locations on deck front face

This work mainly focuses on the readings from the vertical force gauges.

A.2.4 The test procedure

A.2.4.1 Introduction

Three computers control the test process. One controls the wave board, located in the beginning of the flume. This wave maker can create regular and random waves, in this case from a JONSWAP spectrum. Then there is a computer reading the wave probes. At a frequency of about 25 Hz it reads the water level in three locations in the flume. At first, after starting the waves, this computer calculates the exact mean water level by looking at about 20 waves. After these 20 waves pass, the third computer is activated, which starts collecting the raw data from the force and pressure transducers. This computer has to be set to collect data for a certain amount of time, at a certain sampling rate.

The Absorbing Wave Flume at HR Wallingford has an array of wave conditions already calibrated. Those chosen for the experiments have three different values of wave steepness (s = H₃ / L), see figure 5-7. These values correspond to two storm situations (s = 0.065 or 0.040) and one swell condition (s = 0.010). From this, the following 16 test conditions follow:

<table>
<thead>
<tr>
<th>Waterdepth. d = 0.60m (model scale), deck clearance: 0.16 or 0.21m</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Prototype</strong></td>
</tr>
<tr>
<td>Wave</td>
</tr>
<tr>
<td>1</td>
</tr>
<tr>
<td>2</td>
</tr>
<tr>
<td>3</td>
</tr>
<tr>
<td>4</td>
</tr>
<tr>
<td>5</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Waterdepth. d = 0.75m (model scale), deck clearance: 0.01 or 0.06m</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Prototype</strong></td>
</tr>
<tr>
<td>Wave</td>
</tr>
<tr>
<td>6</td>
</tr>
<tr>
<td>7</td>
</tr>
<tr>
<td>8</td>
</tr>
<tr>
<td>9</td>
</tr>
<tr>
<td>10</td>
</tr>
<tr>
<td>11</td>
</tr>
<tr>
<td>12</td>
</tr>
<tr>
<td>13</td>
</tr>
<tr>
<td>14</td>
</tr>
<tr>
<td>15</td>
</tr>
<tr>
<td>16</td>
</tr>
</tbody>
</table>
A.2.4.2 **Input**

Choices need to be made concerning the following parameters:

1. Water level (0.60 or 0.75m)
2. Deck clearance (0.16 / 0.21m or 0.01 / 0.06m)
3. Wave conditions (H\(_w\), T\(_m\)) (see above)
4. Sampling rate (25, 100, 200, 500, 1000 Hz)
5. Amount of waves (min 100, max 1000)

In this case, there is a pure practical reason why the combinations of the last two parameters are limited. Due to time related reasons, raw test data had to be processed in Excel. Excel has a limit in the amount of cells it can handle of about 97000. The amount of data points, each of which has to have an Excel cell, is calculated as follows:

\[
X = \text{sampling rate} \times \text{amount of waves} \times \text{mean wave period}
\]

\[
X \leq 97000
\]

In this test procedure, all wave conditions are tested at 500 Hz to make sure that the impact phenomenon is recorded accurately. Some interesting wave conditions are then repeated and sampled at 200 Hz, in order to get a more significant amount of wave data.
A.3 Calculation used for impact comparison

Elghamry

\[ F_v = c_1 c_2 \frac{\gamma_w H \lambda}{2} \sqrt{1 + \frac{3r^2}{1 + r^2}} \]

See page 7.

Wang

\[ P_r = \frac{\pi}{2} H r_w \tanh \left( \frac{2\pi d}{L} \left( 1 - \frac{4h^3}{H^3} \right) \right) \]

See page 8. Peak pressure applied on a strip one meter wide.

Broughton

\[ F_v = \frac{1}{4} \rho_w \pi v B \]

See page 10

Shih and Anastasiou

Impact pressure

\[ P_i = (1.8 - 10 - 7.6) \rho_w g H_s \]

Where:

- \( P_i \) = impact pressure \( (N/m^2) \)
- \( g \) = acceleration of gravity \( (m/s^2) \)
- \( H_s \) = wave height (through to crest) \( (m) \)

This pressure is then assumed to work on a strip one meter wide, parallel to the wave crest, over the whole width of the platform.

Slowly-varying pressure

\[ P_{sv} = 0.65 (H_{cc} - h) \]
Kaplan

**Impact force**

\[
F_i = \frac{\partial}{\partial t} (m_i v) = \rho \frac{\pi}{8} \frac{bl^2}{\left[1 + \left(\frac{l}{b}\right)^2\right]^{1/2}} v + \rho \frac{\pi}{4} \frac{bl}{dt} \frac{1 + \frac{1}{2} \left(\frac{l}{b}\right)^2}{\left[1 + \left(\frac{l}{b}\right)^2\right]^{3/2}} v
\]

**Slowly-varying forces**

\[
F_d = \frac{\rho}{2} bl C_d |v|^2
\]

\[
F_b = \rho gb l \left(\frac{1}{4} d_{left} + \frac{1}{2} d_{middle} + \frac{1}{4} d_{right}\right)
\]

Where:

- \( F_i \) = Impact force (N)
- \( F_d \) = drag force (N)
- \( F_b \) = buoyancy force (N)
- \( m_3 \) = 3-D added mass (kg)
- \( v \) = vertical water particle velocity (m/s)
- \( v' \) = vertical water particle acceleration (m/s²)
- \( \rho \) = specific weight water (kg/m³)
- \( b \) = platform width (m)
- \( l \) = wetted length (m)
- \( d_{left} \) = wave celerity, c (m/s)
- \( d_{xxx} \) = water level – level underside deck (m)

* Vertical water particle velocity calculated from linear wave theory on regular waves.

** Particle velocity in random waves can be 10 to 20% higher.

** Wetted deck length, wave celerity (c) x time from first contact water with deck (t)

Suchithra and Koola

\[
F_i = \frac{1}{2} C_s \rho_{\text{w}} A v^2
\]

See page 19

Ridderbos

**Impact pressure**

\[
P_i = \frac{1}{2} \rho \rho_{\text{w}} C_s (\beta) v^2
\]

Where:

\[
C_s = 1 + \left(\frac{\pi \cot \beta}{2}\right)^2
\]

- \( v \) = water particle velocity a deck location, calculated like Kaplan, with linear wave theory on regular waves (m/s)
Slowly-varying pressure

\[ P_\ast = \frac{1}{10} P \]

Working on the whole deck area.

Wave length in shallow water

\[ L_i = L_0 \left[ 1 - \exp\left( - \left( \frac{2\pi d}{L_0} \right)^{1.25} \right) \right]^{0.4} \]

(WL Delft hydraulics)

Overbeek

Impact pressure

\[ P_i = 1.5 \rho g H \]

assumed over strip one meter wide

slowly-varying pressure

\[ P_\ast = 1.0 \rho g (H_{cr} - h) \]

assumed over whole area of the deck.

Where:

- \( H_{cr} \) = wave crest height (still water level to wave crest) (m)
- \( h \) = deck clearance (still water to underside deck) (m)
A4  **Calibrating the wave load model**

<table>
<thead>
<tr>
<th>Wave no.</th>
<th>$H_s$ (m)</th>
<th>$T_m$ (s)</th>
<th>$H_{cr, max}$ (m)</th>
<th>$h$ (m)</th>
<th>$d$ (m)</th>
<th>$F_{peak, min}$ (N)</th>
<th>$F_{peak, max}$ (N)</th>
<th>$F_{measured}$ (N)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.22</td>
<td>1.5</td>
<td>0.23</td>
<td>0.06</td>
<td>0.75</td>
<td>111</td>
<td>221</td>
<td>180</td>
</tr>
<tr>
<td>2</td>
<td>0.14</td>
<td>1.5</td>
<td>0.16</td>
<td>0.06</td>
<td>0.75</td>
<td>63</td>
<td>127</td>
<td>120</td>
</tr>
<tr>
<td>3</td>
<td>0.10</td>
<td>1.0</td>
<td>0.11</td>
<td>0.06</td>
<td>0.75</td>
<td>46</td>
<td>92</td>
<td>30*</td>
</tr>
<tr>
<td>4</td>
<td>0.10</td>
<td>1.25</td>
<td>0.09</td>
<td>0.06</td>
<td>0.75</td>
<td>34</td>
<td>68</td>
<td>30*</td>
</tr>
<tr>
<td>5</td>
<td>0.22</td>
<td>2.5</td>
<td>0.23</td>
<td>0.06</td>
<td>0.75</td>
<td>98</td>
<td>195</td>
<td>180</td>
</tr>
<tr>
<td>6</td>
<td>0.14</td>
<td>3</td>
<td>0.17</td>
<td>0.06</td>
<td>0.75</td>
<td>55</td>
<td>110</td>
<td>70</td>
</tr>
<tr>
<td>7</td>
<td>0.22</td>
<td>1.5</td>
<td>0.21</td>
<td>0.01</td>
<td>0.75</td>
<td>68</td>
<td>135</td>
<td>120</td>
</tr>
<tr>
<td>8</td>
<td>0.14</td>
<td>1.5</td>
<td>0.13</td>
<td>0.01</td>
<td>0.75</td>
<td>41</td>
<td>82</td>
<td>60</td>
</tr>
<tr>
<td>9</td>
<td>0.10</td>
<td>1.0</td>
<td>0.12</td>
<td>0.01</td>
<td>0.75</td>
<td>33</td>
<td>67</td>
<td>45</td>
</tr>
<tr>
<td>10</td>
<td>0.18</td>
<td>1.5</td>
<td>0.18</td>
<td>0.01</td>
<td>0.75</td>
<td>58</td>
<td>116</td>
<td>80</td>
</tr>
<tr>
<td>11</td>
<td>0.075</td>
<td>0.875</td>
<td>0.09</td>
<td>0.01</td>
<td>0.75</td>
<td>27</td>
<td>54</td>
<td>25*</td>
</tr>
<tr>
<td>12</td>
<td>0.20</td>
<td>1.5</td>
<td>0.19</td>
<td>0.16</td>
<td>0.60</td>
<td>72</td>
<td>144</td>
<td>110</td>
</tr>
<tr>
<td>13</td>
<td>0.20</td>
<td>2.0</td>
<td>0.20</td>
<td>0.16</td>
<td>0.60</td>
<td>70</td>
<td>139</td>
<td>120</td>
</tr>
<tr>
<td>14</td>
<td>0.20</td>
<td>1.75</td>
<td>0.22</td>
<td>0.21</td>
<td>0.60</td>
<td>68</td>
<td>135</td>
<td>30*</td>
</tr>
</tbody>
</table>

Table 5-a. Force peak values compared.  
*measured values under minimum predicted.
Comparing wave impact on vertical walls with impact on horizontal platforms.

A5.1 Introduction

Wave impact on vertical walls has been the topic of research of many projects concerning breakwater or harbour wall stability. Some large-scale experiments have been conducted and various models and theories to quantify the magnitude and intensity of wave impact have been put forward. Horizontal platforms, on the other hand, have not received the same amount of attention. Although differences exist between the two mechanisms of impact, it might still be worthwhile to investigate whether theories and models available for vertical walls can apply to horizontal platforms.
A5.2 Wave impact on vertical walls

A5.2.1 Classification of the wave impact pressures

The wave impact process depends closely on the wall location relative to the wave breaking position [CHAN AND MELVILLE, 1988]. It is, therefore, reasonable to characterise the impact pressures by the colliding conditions of the breaking wave. Similar to Hattori et al. (1984) and Oumeraci at al. (1993) we have:

1. "Flip-through" condition without air entrainment (or upward deflected breaker):
2. Collision of the vertical wave front while entrapping small air bubbles:
3. Collision of plunging breakers with a thin air pocket:
4. Collision of fully developed plunging breakers with a thick air pocket:

1. "Flip-through" condition without air entrainment (or upward deflected breaker):
Transient process from non-breaking wave pressure to single sharp-peaked pressure. Incident waves break as an upward deflected breaker at the wall. Location of maximum pressure moves up with rising free surface at the wall. When increasing the maximum wave height, the water surface becomes steeper and entraps small air bubbles at the contracting region of the water surface. This generates a higher impulsive pressure magnitude than that without air entrainment. A double-peaked pressure is measured where the advancing wave front meets the upward jet, the first peak represents a rapid deflation of the entrapped air and the second peak is owing to the flip-through.

(The four Hattori classifications will be accompanied by wave impact pressure records from model tests in a 20 x 0,3 x 0,55 m wave flume. The graphs P1 – P6 are pressure measurements from the different pressure transducers located as shown in figure A5-1. The sampling frequency of the set-up is 5000Hz.)

Figure A5-1. Arrangement of pressure transducers (units.cm)

Figure A5-2. Dimensionless wave pressure records from flip-through collision without (left) and with (right) air entrainment
2. Collision of the vertical wave front while entrapping small air bubbles.
   The collision of breaking waves with a almost vertical or very slightly curled front surface
   brings about a single impulsive pressure on the wall with a very high magnitude and a short
   duration. The impact pressure front propagates downward from the impact point through the
   water body, distinguishing it from the "flip-through" type, where pressure propagates upward.

Figure A5-3. Dimensionless wave pressure records from collision of vertical wave front
Collision of plunging breakers with a thin air pocket:
In a plunging breaker a jet projects from the crest of the wave, at high velocity, the tip of which is in free fall. However, the forward velocity of the tip is larger than that of the concave wave face. If the wave front curls over before hitting the wall, a small air pocket in the shape of a very thin lens will be trapped under the impinging region of the crest. In the top of this lens shape, the highest pressures are measured, followed by a high frequency oscillation.

Figure A5-4. Dimensionless wave pressure records from collision of plunging breaker with thin air pocket
4. Collision of fully developed plunging breakers with a thick air pocket:
With increasing distance between the wave breaking position and the wall, plunging and
curling of the breaking wave become fully developed. At the instant of impact a large amount
of air will be trapped. After being trapped, this air breaks up into air bubbles, which are
compressed and stretched as a whole with the wave run-up. All along the height of the wall a
regular pressure oscillation with decreasing amplitude will occur, having a significant lower
frequency than the oscillation measured in the third type, and with the highest magnitude at
still water level.

![Wave Pressure Records](image)

Figure A5-5. Dimensionless wave pressure records from collision of fully developed plunging breaker
Some disagreement is found in the literature about the exact description of the ‘flip-through’ motion. In reports by Hattori et al. and Oumeraci et al. the ‘Flip-through’ resembles the Wagner type impact, as discussed below, and the upward deflected breaker, which means relatively low impact pressures and no air entrapment. Cooker and Peregrine (1990) and Vink (1997) compare the ‘flip-through’ effect to the transition type and have it account for their highest measured peak pressures. The general opinion is that highest pressures occur when the converging wave crest hits the wall just over the vertical jet shooting up, so after the stage of ‘flip-through’.

Two kinds of double-peaked pressures are identified. One occurs owing to a sharp depression of the impulsive pressure, when fragments of the entrapped air deflate through to the free surface during the impact. This double peak is characterised by a very short time difference between the two peaks. For another kind, the first peak occurs when the convergent wave crest hits the wall, entrapping a large air pocket, which causes the second peak when being compressed by the advancing wave front. It follows that both kinds of double-peaked pressures require a significant amount of air to be trapped [HATTORI et al. (1994)].

Hattori et al. describe from their experiments a relation between the maximum impact pressure and the rise time:

\[ p_{\text{max}} = 400 t^{-3/4} \]

With \( p_{\text{max}} \) in gsf/cm² and \( t \) in ms.

![Figure A5-6: Relation between \( p_{\text{max}} \) and \( t \). (▲△) Flip-through impacts with and without air entrainment; (+) single sharp peak impact; (●) impacts with damped pressure oscillations.](image)

Vink (1997) and Takahashi (1994) divide the vertical impact process into three types:

**Wagner type**

Named after a German researcher from the 1930’s. In case of a non-breaking wave no air will be entrained in the wave front or entrapped between the wave front and the wall. The pressure at the wall will have a relatively gentle variation in time and will be almost in phase with the wave elevation. With rising relative wave height, though, this quasi-static situation will cease to exist. A certain asymmetry in terms of the steepening of the wave front can cause a fast rise in wave pressure. Lundgren (1969) denotes this as a ventilated shock. Takahashi [TAKAHASHI et al. 1994] refer to it as Wagner type pressure. It is similar to the upward deflected breaker or the “flip-through” breaker mentioned by
Hattori et al. (1994). The pressure time history of the Wagner type impact is characterised by a sudden rise and exponential decay.

**Bagnold type**

When the wave crest converges into a horizontal jet and crashes into the wall, a large air pocket can be trapped. The crashing of the jet into the wall, also called the hammer shock, will cause the first pressure peak in the pressure time history, the compression and the subsequent release of the entrapped air will cause the second peak and the following pressure oscillations and is denoted a compression shock. This type of wave impact is called the Bagnold type impact, because of the fact that the theoretical approach of the characteristics of these wave impacts has first been treated by Bagnold (1939).

**Transition type**

Between the Wagner type of no air entrapment and the Bagnold type with large amounts of air trapped, a third transition zone exists, conveniently named the transition type. The wave crest hits the wall either head on or with small amounts of air entrapped (thin lens shape) or entrained (bubbles) in the wave front. Air is said to have a cushioning effect, so the transition type with its little or now air generally describes wave impacts with the highest impact pressures.

A basic difference between these three types is the angle of attack $\beta$ of the wave front surface against the wall. Together with a measure for the curvature of the wave front, defined as the angle $\delta$ between the tangent and the cord of the wave curve as shown in figure A5-7, the three types can be separated as shown in figure A5-8:

![Diagram showing the definitions of the angles $\beta$ and $\delta$](image)

*Figure A5-7. Diagram showing the definitions of the angles $\beta$ and $\delta$*

![Wave action models for three different types](image)

*Figure A5-8. Wave action models for three different types*
The classifications of wave impact on vertical walls proposed by various researchers compare as indicated below (shaded types result in highest impact pressures):

<table>
<thead>
<tr>
<th>Hattori et al</th>
<th>Oumeraci et al</th>
<th>Vink / Takahashi et al</th>
</tr>
</thead>
<tbody>
<tr>
<td>&quot;Flip-through&quot; condition without air entrapment</td>
<td>Upward deflected breaker</td>
<td>Wagner type</td>
</tr>
<tr>
<td>Collision of the vertical wave front while entrapping small air bubbles</td>
<td>Plunging breaker with a small cushion of air</td>
<td>Transition type</td>
</tr>
<tr>
<td>Collision of plunging breakers with a thin air pocket</td>
<td>Well-developed plunging breaker with a large cushion of air</td>
<td>Bagnold type</td>
</tr>
<tr>
<td>Collision of fully developed plunging breakers with a thick air pocket</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Turbulent bore</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

*Table A-1. Comparison different classifications*

Chan and Melville (1988) present a division between impact types related to the position of the wall in the wave breaking process. When the wall hits the wave just before the start of its breaking process, the wave will develop into a sloshing motion without any impulsive impact on the wall. When shifting the wall downstream, the following impact types will occur:

- Wagner type; upward reflected breakers or "flip-through" breakers without any air entrapment.
- Transition type; little air bubbles or a thin lens shaped air pocket will be trapped between the almost vertical wave front and the wall.
- Bagnold type; a well developed plunging breaker will entrap a large amount of air. A double-peaked pressure time history can be measured.

After a further shift downstream, the broken wave will arrive at the wall containing so much air bubbles and foam that impulsive pressures are damped-out almost completely.

The typical features of a force time history caused by the impact of a breaker plunging on a vertical wall and entrapping a large air pocket are schematically summarised in figure A5-9.

*Figure A5-9. Characteristics of Bagnold type impact forces and their origin [SCHMIDT et al. (1992)]*
A5.2.2 Available wave impact models:

Shock wave model
Wave impact without air on a rigid wall
This model could be used to schematise wave impact with a hammer shock. In this model it is assumed that the water approaches perpendicular to the rigid wall and cannot escape. At the moment of impact, the water is compressed and a shock wave is created in the water, which moves away from the wall. The celerity of the front of the shock wave is equal to the velocity of sound in Water (1480 m/s in pure water [BATTJES (1990)])

This shock wave model gives the absolute upper limit for the wave impact pressure. In reality the pressure will always be lower, because it is very unlikely that the water surface will be completely flat and that it hits the wall parallel and thus not letting the water escape. The maximum prototype pressure found was about ten percent of that calculated by this model [MASSIE et al. (1986)].

Rigid wall and air-water mixture
When taking into account the amount of air entrained in the water, the maximum pressure drops considerably. Depending on the amount and the size of the air bubbles the compression can be either isothermal (no temperature change during the compression, small bubbles) or adiabatic (no heat exchange with the vicinity during compression, large bubbles).

Wave impact on a compressible wall
Compressible water hitting a compressible wall will send off shock waves in both water and wall. Compared with the reduction as a result of air in the water, the reduction due to compressibility of the wall can normally be neglected.

Prediction formulae for wave impact forces on vertical walls
Schmidt et al. derive a formula based on large scale model tests by using waves up to 2m height and with wave periods of 9.4s. The relation between the peak value $F_{\text{peak}}$ per linear meter and the total peak force duration maybe approximated by:

$$\frac{F_{\text{peak}}}{\rho_w g H_{\text{max}}^2} = 1.24 \left( \frac{t_d}{T_p} \right)^{-0.344}$$

Where:
- $t_d$ = impact duration (s)
- $T_p$ = peak wave period (s)
- $H_{\text{max}}$ = maximum wave height (through to crest) (m)

Goda [Vink (1997)] uses the same method to find the upper limit of the dimensionless impact force:

$$\frac{F_{\text{peak}}}{\rho_w g H_{\text{max}}^2} = 15$$

When comparing these two methods to results obtained from the models in the literature on wave impact on horizontal platforms, it seems that for the same wave height, the first two models predict values an order of magnitude greater.

Perhaps some similarity can be found when looking at the pressure-time histories of wave impact on horizontal platforms.
A5.3 Pressure time histories of wave impact on horizontal platforms

Shih and Anastasiou (1992):

Two sampling frequencies compared while capturing dynamic impact pressures. The above graph is the time pressure history of uplift pressures recorded at high sampling frequency (500Hz). A high magnitude peak pressure is clearly visible. The bottom graph shows the same impact recorded at low frequency (20Hz). The pressure peak is not captured.

The pressure-time history resembles the graph of the second condition identified by Chan and Melville (1988), "the collision of the vertical wave front while entrapping small air bubbles". The peak pressure value, though, do not seem to correspond at all. Chan and Melville measure 50 kN/m² from a wave with a height of 0.04m. Shih and Anastasiou measure 10 kN/m² from a wave 0.40m high.
Klatter (1994)

![Graph showing pressure readings from the Eastern Scheldt Barrier.]

Figure A5-11. Two pressure readings from the Eastern Scheldt Barrier.

Pressure time histories from two pressure gauges during the measurement of the highest pressure peak at gauge D35. The response of the beam after the pressure drop shows the vibration of the structure in its own natural frequency of approximately 5 Hz. The pressure graph resembles Chan and Melville's third condition: "Collision of plunging breakers with a thin air pocket". Again, though, the peak pressure values differ considerably.

Smith (1999):

![Graph showing peak pressures measured by Smith.]

Figure A5-12. Peak pressures measured by Smith.
Shows the time pressure records of three pressure cells during the highest upward force. These graphs clearly show that for a high force to occur, pressures need to be high simultaneously on a large area of the platform. These three gauges represent a prototype area of approximately 6 m². Pressure gauges further away show no or very little pressures. This illustrates the localised effect of peak impact pressures.

Figure A5-13. The maximum force measured.

Maximum upward force measured. A model response force is superimposed over the decay of the slowly-varying pressure.
A5.4 Conclusion

This does not seem to be the way to go. The phenomenon of wave impact on vertical walls resembles wave impact on horizontal platforms in some way, but many more differences exist:

1. The magnitude of peak forces on vertical walls is an order of magnitude higher than the magnitude of forces on horizontal platforms.

2. Impact on vertical walls is caused by breaking waves, impact on horizontal platforms is caused by non-breaking waves.

3. Parameters determining the size and shape of the wave loads on vertical walls differ from those determining loads on horizontal platforms.

4. During impact on horizontal platforms, the wave is already touching the platform. The impact is caused by the rising part of the wave ‘flowing’ into the platform. This process is more gentle than the tip of a breaking wave actually slamming into the dry wall.

5. After hitting the horizontal platform with a force peak, the wave crest travels on along the platform, hitting the deck again a bit further on. When a breaking wave hits a vertical wall, the whole mass of water crushes into the platform at that exact same position.

Of course there are similarities too. Both horizontal and vertical wave impact deal with water hitting a structure in a slamming motion. The forces driving the two processes, though, apply in a different manner. The velocity of the water particles hitting a horizontal platform is smaller than the velocity of the water particles hitting the vertical wall.

Figure A5-14. A non-breaking wave ‘flowing’ into a platform and a breaking wave ‘slamming’ into a vertical wall.
Lai and Lee capture some of the preceding experimental results, mainly by French, in a numerical model. Their graph (figure A6-1) shows a sudden rise to a peak pressure, followed by a rapid drop and a slowly-varying pressure, first positive then negative. The slowly varying positive pressure is caused by the head difference under the plate with the rising water around the plate. The negative pressure occurs when the drop of the water surface under the platform creates a vacuum. The suction force that fills this vacuum with air causes the negative pressure. The impulsive peak pressure is said to be important when considering the strength of individual members of the structure. The slowly varying pressures are important for the stability of the structure as a whole.

**Figure A6-1. Non-dimensional pressure on platform at two different locations as a function of time. (The dots are experimental results obtained by French (1971))**

Lai and Lee use a normalised hydrodynamic force per unit width. For $F_u/F_s$, where $F_u$ is the total upward pressure x area platform and $F_s$ is the hydrostatic pressure (weight of shaded part of water in figure A6-2), they find maximum values of 1.0 to 2.0 for flat bottoms and from 3.0 to 8.0 for sloping bottoms and end walls. The other symbols used in the following graphs are:

- $s$ = deck clearance (m)
- $d$ = water depth (m)
- $H$ = wave crest height (m)
- $L$ = platform length (m)
- $S$ = bottom slope (m)
- $C$ = wave celerity (m/s)
- SWL = still water level (m)

**Figure A6-2. Wave impact by Lai and Lee**
The following graph shows the effect of a reflecting boundary, like a quay wall, with a sloping bottom:

![Graph showing the effect of reflecting boundary and sloping bottom](image)

*Figure A6-3. The effect of reflecting boundary and sloping bottom*

As can be seen in figure A6-3, the sloping bottom and the reflecting boundary together significantly increase the positive uplift forces, far beyond values obtained for horizontal bottoms. The maximum negative force does not vary noticeably for different slopes.

Some interesting results are presented in the work by Lai and Lee. One is found in the following graphs. They show the normalised hydrodynamic force, as explained above, against a non-dimensional time, for three values of the relative wave height H/d. The other factors stay constant:

- the relative deck clearance, s/d = 0.2
- the relative platform length, L/d = 4
- the water depth (to which all factors are made relative to), d = 40 cm.

The graphs show both the results of the numerical model and the experimental values found by French (1971). For the relative wave height H/d to increase with a constant d, the wave height H should increase. This increase in H effects the normalised hydrodynamic force F/Fₚ. First, the weight of the water over the deck increases, increasing Fₚ. With this increase, the graph should show lower values for increasing H/d. In fact, this only seems to hold true for the negative part, as the graph actually shows higher values for the positive force component, when comparing graphs a and b, and a constant value when comparing graphs b and c.

So, even though Fₚ increases more than linearly, the total upward pressure increases the same or even more.

One general remark should be made about the validity of this numerical model. The pressure graphs of the numerical approach have been calibrated on experimental results obtained by French (1971) which have been explained above. Quite some questions remain as to the accuracy of his model tests.
Figure A6-4a. Total hydrodynamic force per unit width. $H/d = 0.24$

Figure A6-4b. $H/d = 0.32$

Figure A6-4c. $H/d = 0.4$