EXPERIMENTAL MODELLING OF LONG ELEVATED AND DEPRESSED WAVES USING A NEW PNEUMATIC WAVE GENERATOR

A DISSERTATION SUBMITTED IN PARTIAL FULFILLMENT OF THE REQUIREMENTS FOR THE DEGREE OF DOCTOR OF PHILOSOPHY Ph.D.

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February 2012
Abstract

Tsunami are long propagating waves caused by the rapid displacement of a body of water. Recent events have shown how devastating tsunami can be to urban infrastructure (e.g. Japan 2011). Tsunami are characterized by their very long period (typically 10 to 40 min), large amplitudes close to the shore, and by a large trough often preceding the positive wave (depending on how the wave is initiated). Their period falls between that of ocean waves and tidal flows. To study these processes, an experimental methodology is applied to analyse how long waves (with or without a depressed component) propagate and move inshore.

This thesis starts by critically reviewing the state-of-the-art in numerical and physical modelling of long propagating waves and identifies the limitations of numerical codes to date as well as previous experimental studies. It is found that while the generation and propagation of a long wave can be reasonably well modelled with numerical codes, the behaviour of the wave nearshore and onshore is much more complex and usually tackled using laboratory experiments. These are themselves often limited by the length and types of waves generated, as well as scale and/or parameters investigated.

A new experimental long wave generation system was developed together with a specialist hydraulic modelling and research facility (HR Wallingford) and this system was used within a wave flume to study long wave dynamics. The novel aspect of the study was that the propagating waves studied had long wavelengths compared to those previously generated in tests and the new generation system enabled leading depressed waves to be analysed for the first time. The propagating waves were investigated by
analysing wave heights and local velocity measurements obtained in the wave flume experiments.

Results of propagating long wave runup, velocities in the shoreline region, and also potential energy are presented and discussed. New runup relationships are proposed.

Pressures on scaled model buildings placed in the onshore region of the flume were also recorded. Some preliminary results about the evolution of local pressures from the onshore flow on a model building are introduced.
Declaration

I hereby declare that the work presented in this thesis is solely my own work and that to the best of my knowledge the work is original except where otherwise indicated by reference to other authors. No part of this work has been submitted for any other degree or diploma.

Ingrid Charvet

February 2012
A mes grand-parents.

Que pour toujours, vous soyez fiers de moi.
“Whether you believe you can, or you believe you can’t--you’re right.”

— Henry Ford
Acknowledgements

I would like to acknowledge the support of my main supervisor, Dr Tiziana Rossetto (UCL), whose enthusiasm, understanding and guidance helped me greatly throughout this difficult journey. I am also extremely grateful to my second supervisor, Dr Ian Eames (UCL), for sharing with me his deep knowledge of the field with patience, and for his guidance and thoroughness. Thank you also to my industrial supervisor Pr William Allsop (HR Wallingford) for providing me with access to the pneumatic wave generator, instrumentation and staff at HR Wallingford, as well as his involvement and help with the first stages of the study.

In the rest of my academic circle, I am very grateful to Dr Tristan Robinson, who helped me through the experimental testing, MATLAB issues and beyond with great patience. Thank you to HR Wallingford’s staff who contributed to the successful development and testing of the new generator, particularly David Robinson and Pierre-Henri Bazin, who I had the pleasure to work with. I also wish to thank Dr Buldakov, Dr Steve Richardson, Dr Simmons, Pr Julian Hunt, Dr Simon Day, Dr Costas Synolakis, Dr Christian Klettner, Dr William Power and Dr Christoforos Koutitas for providing insight, reading materials, and answering my questions at different stages of this work.

I would not have been able to go through this four-year long challenge without my friends and PhD colleagues, who were here every step of the way to share joy, disappointments, knowledge, doubts; and who always listened and cheered me up through the hard times. Thank you to PhD students and post-doc colleagues Melanie
Duncan, Enrica Verrucci, Tristan Lloyd, Randolph Borg, Victoria Sword-Daniels, Amir Torbati, Ionanna Ioannou. Thank you to all my gym buddies at Bloomsbury Fitness for the workouts and the banter! Thank you Rachel Lowe, Stella Vorka, Martina Avellino, Holly Peacock for all the fun, coffees, and reassuring chats. Big thanks to Eric Lacey for all the support provided during the particularly difficult year that was 2008. Finally, special thanks to my dear friends Hayley Whittle and Theodora Hadjimichael who were always here for me.

Last but not least, I am grateful to my loving partner, Igor Kojic, who gave me some very valuable support during the end of my PhD, and with whom I shared unforgettable moments of happiness. I will end this acknowledgements section with a few lines in French, to honour and thank my parents, Eliane and Jean-Claude, for their unconditional love, help and understanding.

Papa, Maman, je vous remercie de tout coeur pour m’avoir epaulee et comprise tout au long de cette thèse. Malgré beausoup de moment difficiles, j’ai toujours su que je pouvais compter sur vous. Sans vous, je n’en serais jamais arrivee la: je tiens a vous dedier mes succes futurs!

This work has been supported by the EPSRC grant EP/F012179/1.
List of publications from this work:


*In preparation:*

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List of Symbols

\( \eta \): surface elevation above mean water level (in m)

\( \gamma \): surface tension coefficient

\( Ir \): Irribarren number, surf similarity or breaker parameter

\( g \): standard gravity (9.81, in m.s\(^{-2}\))

\( c_{p_0} \): theoretical shallow water phase speed (in m.s\(^{-1}\))

\( c_p \): phase speed, if different from \( c_{p_0} \) (in m.s\(^{-1}\))

\( \rho_0 \): density of pure water (1000, in kg. m\(^{-3}\))

\( \rho_s \): density of sea water (1027, in kg. m\(^{-3}\))

\( \rho \): fluid density (in kg. m\(^{-3}\))

\( \mu \): dynamic viscosity (in Pa.s\(^{-1}\))

\( m \): mass (in kg)

\( u \): horizontal velocity (in m.s\(^{-1}\))

\( v \): vertical velocity (in m.s\(^{-1}\))

\( U \equiv (u, v) \): flow velocity (in m.s\(^{-1}\))

\( V \): electrical current (in volts)

\( h \): water depth (in m)
$t$: time (in s)

$H$: wave height (in m)

$a, a^{-}$: positive, negative wave amplitude (in m)

$R$: runup (in m)

$\beta$: angle of slope (in rad)

$X_p$: value of parameter $X$ in the prototype

$X_m$: value of parameter $X$ in the model

$T$: wave period (in s)

$L$: wavelength (in m)

$f$: frequency ($1/T$) (in Hz)

$\omega$: angular frequency ($2\pi/T$) (in rad. s$^{-1}$)
Chapter 1. Introduction

1.1 Introduction to Tsunami

Tsunami are long gravity waves caused by a rapid displacement of a large body of water. A long gravity wave’s main restoring force is gravity (atmospheric effects have a negligible influence on wave propagation), so tsunami can travel large distances without significant loss of energy. The term “tsunami” means harbour \( \text{(tsun)} \) wave \( \text{(ami)} \) in Japanese, because such waves often develop as a resonant phenomena in harbours after offshore earthquakes (Bryant, 2001).

Tsunami are different from wind generated waves. Wind generated waves are initiated due to fluctuations of the atmospheric pressure and grow as wind velocity increases, with gravity forces acting as a mirror to the wave motion. In contrast to tsunami waves, these waves usually dissipate when the wind calms (Massel, 1996). Among other marine gravitational waves, tsunami occupy an intermediate position between tides and ripples (Figure 1.1). Voit (1987) explains they have a period that can vary between 2 and 200min (the most typical tsunami periods being 10 to 40min) and their wavelengths can vary from tens to hundreds of kilometers. Another main characteristic of tsunami is the frequent initial withdrawal of the water as the wave reaches the shore (known as a receding or draw-down wave). Indeed, the wave is not only composed of a large peak but may also be preceded by a trough. During an earthquake generated tsunami the vertical displacement of the seafloor (generally a dip-slip fault) is transmitted to the overlying body of water, creating a deformation of the water surface at equilibrium. In this case, both positive and negative elevation changes to the still
water level are generated. These propagate as waves, which may be led by a peak or trough (depression). When the depression reaches a coastal area, one can see the uncovering of hundreds of meters of the seabed as a warning to the positive wave arrival.

Tsunami are commonly caused by earthquakes generating displacement of the seafloor and subsequent disturbance at the ocean surface. In the deep ocean, an earthquake-generated tsunami travels with quite small vertical displacements and very long wavelengths, until it reaches coastal areas where it shoals up dramatically and may reach 10 meters or more. Tsunami can also be caused by submarine or subaerial explosive volcanic eruptions or landslides travelling into water, underwater explosions and asteroid impacts on a body of water. They can happen not only in the ocean, but also in bays, lakes, or reservoirs (for example, the 1963 Vajont Dam disaster (Panizzo et al., 2005a). Numerous tsunami are generated by seismic sources, more specifically dip-slip faults on subduction zones. When the fault ruptures, the upthrusted and downthrusted crustal blocks displace the overlying water mass accordingly over the whole region; this volume of water now has potential energy to be transferred away from the source, resulting in a tsunami (Figure 1.2). Because the vertical seafloor displacement results in the deformation of the overlying water surface, large earthquakes ($M_W > 7$) have the potential for generating large tsunami. The resulting tsunami size also depends on the depth of the source, and the duration of the seismic event. Tsunami are typically generated by a shallow focus source with low rupture velocity (Synolakis et al., 1997). Earthquakes can also trigger submarine landslides, which can displace very large volumes of water and trigger a tsunami. This was the case for the 1998 Papua New Guinea tsunami (Lynett et al., 2003). Impact tsunami
such as the ones triggered by subaerial landslides or asteroid display a different generation mechanism. The horizontal scale of the water displacement is typically smaller than for earthquake tsunami, but the wave heights are locally much larger. Subaerial landslide tsunami transfer their energy to the water at the impact and during underwater motion of the landslide. According to Panizzo et al. (2005b), the generation mechanism is complex and the features of the generated waves depend on the landslide volume, porosity, density, velocity (at impact and underwater), shape of the front, and on the angle of the slope where it originates. The initial displacement of water in the case of an asteroid generated tsunami is considered to be similar to the displacement resulting from a shock (or multiple shocks), as described by Ward and Asphaug (2003), and Mader (2004). Tsunami triggered by volcanic activity can be a result of volcanic explosions and associated pressure waves, and/or volcano collapse and associated landslides, seafloor collapse. For example, the 1883 eruption of the Krakatoa (see Table 1.1) triggered a paroxysmal explosion and collapse of two volcanic chambers, with subsequent creation of an underwater caldera; these two phenomena together gave birth to destructive tsunami waves (Pararas-Carayannis, 2008). According to Camfield (1980) tsunami with a volcanic origin spread geometrically and do not cause large inundations at locations away from the source, but similarly to impact tsunami, may cause very large waves near the source.

In flat coastal areas without natural or man-made defences, large tsunami can penetrate several kilometers inland. As a result, large tsunami often cause many deaths, and are very costly as they damage or destroy coastal habitations, structures and vegetation (Table 1.1 lists examples of major historic tsunami, their main characteristics and impact for illustration purposes). However, despite the scale of such disasters and the
recent advances of data acquisition techniques, understanding tsunami impact on coasts is still a challenge for the scientific community. Synolakis and Bernard (2006) highlight that tsunami occur less frequently than other types of natural hazards, and historic records of tsunami are unsystematic, especially past century events which were probably under-reported and/or unrecognized. Until the recent developments of instrumental recordings of tsunami (1990s with the NOAA tsunameters), open ocean tsunami data was not available. As regards the tsunami waves close to the shore, the only data available usually comes from tide gauges, which are mostly located in sheltered regions (e.g. harbours) and therefore mainly record the harmonic forcing of the basin, not the actual tsunami.

It is therefore necessary to understand the characteristics of such waves in order to predict their impact on the shores of countries at risk.

1.2 Objectives of this thesis

The objectives of this work are:

- To identify gaps in the understanding of long propagating wave behaviour at the shore,
- To address some of these gaps using a novel wave generator,
- To compare these experimental waves results to previous studies,
- To study the influence of a range of wave parameters on wave runup,
- To test the capabilities and limitations of the pneumatic wave generator and associated experimental set up in meeting the objectives highlighted above.
A new wave generator built by HR Wallingford (described in Chapter 3) allowing for the reproduction of long and depressed waves is tested and used. It should be noted here that actual tsunami are not modelled in this work: no experimental facility in the world simply allows for the resolution of problems on the scale of a tsunami. However, the simpler case of a long propagating waves provides useful information on the inundation and impact of tsunami nearshore and onshore. In this work, long waves are studied experimentally. These waves are generated using the new pneumatic wave generator which allows the displacement of large volumes of water compared to classic wave generation devices.

1.3 Thesis Outline

In Chapter 2, wave theories and classifications are presented, as well as their applications to the modelling of tsunami events. The performance and limitations of wave models, numerical and physical, are critically reviewed. Real tsunami observations complete this review as they provide useful characteristics of long propagating waves. The state-of-the-art in tsunami science is presented and gaps in knowledge are identified.

Chapter 3 presents the main purposes of the experimental testing at HR Wallingford and defines the boundaries set by the scaling used. A schedule of tests is introduced, as well as the pneumatic wave generator, experimental setup, instrumentation, and data acquisition methods for different types of tests. The design of the wave time series used throughout the testing is also presented.
Chapter 4 details the types of measurement made and the analysis techniques applied to the data. The analysis techniques described are used to derive the values of nearshore and onshore flow parameters of interest.

Chapter 5 presents the results obtained for the validation of the pneumatic wave generator, and for investigating its performance and limitations. It also presents the results on the generated wave’s potential for inundation in terms of runup, phase speed and flow velocity, for the different types of waves tested, and discusses the implications of these results. New runup relationships for different wave forms are proposed. Finally, preliminary results on local wave pressures on the front face of model buildings are presented.

Chapter 6 summarizes all findings and presents the conclusions of this research.

The results presented in this thesis are the initial outcomes of a new way of physically modelling long propagating waves. This new data will hopefully lead to a better understanding of tsunami nearshore and onshore behaviour. It is hoped that this new insight will be valuable to academic researchers, practicing engineers and emergency planners.
<table>
<thead>
<tr>
<th>Year</th>
<th>Location</th>
<th>Cause</th>
<th>Max. wave height (m)</th>
<th>Runup (m)</th>
<th>Number of deaths or missing</th>
<th>Cost (USD)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1755</td>
<td>Lisbon, Portugal</td>
<td>Earthquake</td>
<td>6</td>
<td>30 (Algarve)</td>
<td>20000</td>
<td>/</td>
</tr>
<tr>
<td>1883</td>
<td>Java &amp; Sumatra, Indonesia</td>
<td>Volcanic eruption</td>
<td>37</td>
<td>35 (Merak, Java)</td>
<td>36000+</td>
<td>/</td>
</tr>
<tr>
<td>1896</td>
<td>Sanriku, Japan</td>
<td>Earthquake</td>
<td>9.1 (Shirahama)</td>
<td>24.4 (Hawaii)</td>
<td>27000+</td>
<td>/</td>
</tr>
<tr>
<td>1958</td>
<td>Lituya Bay, Alaska</td>
<td>Landslide (aerial)</td>
<td>51</td>
<td>516</td>
<td>2</td>
<td>/</td>
</tr>
<tr>
<td>1992</td>
<td>Nicaragua</td>
<td>Earthquake</td>
<td>1.17 (Puerto Sandino)</td>
<td>9.9</td>
<td>170</td>
<td>25 M</td>
</tr>
<tr>
<td>1998</td>
<td>Papua New Guinea</td>
<td>Submarine landslide</td>
<td>10</td>
<td>4.2</td>
<td>3000+</td>
<td>/</td>
</tr>
<tr>
<td>2004</td>
<td>Thailand</td>
<td>Earthquake</td>
<td>3+ (Sri Lanka)</td>
<td>15+ (Cape Coral)</td>
<td>275000</td>
<td>&gt; 15 B</td>
</tr>
<tr>
<td>2010</td>
<td>Chile</td>
<td>Earthquake</td>
<td>2.5</td>
<td>27 (Constitucion)</td>
<td>400+</td>
<td>&gt; 15 B</td>
</tr>
<tr>
<td>2011</td>
<td>Japan</td>
<td>Earthquake</td>
<td>15</td>
<td>40.5</td>
<td>20000+</td>
<td>122-235B</td>
</tr>
</tbody>
</table>

Table 1.1: Some major historic tsunami features and their impact. The wave height corresponds to the trough-to-peak amplitude of the tsunami recorded close to the shore (i.e. by tide gauge data), and the runup is the elevation above mean sea level of the point of maximum inundation. The information in this table has been taken from the work of Pararas-Carayannis (2008), Camfield (1980), Satake (1995), Lynett et al. (2003), Rabinovich and Thomson (2007), Siripong (2006), Armijo et al. (2010), Wilson et al. (2010), and Chen Chian et al. (2011). (*) Prediction: the modelling from Ward &
Asphaug (2003) indicates the possibility of a global tsunami triggered by a giant asteroid impact. “/” has been used to indicate the information is not available.
<table>
<thead>
<tr>
<th>Motion</th>
<th>Type of wave</th>
<th>Wave period</th>
</tr>
</thead>
<tbody>
<tr>
<td>Propagating</td>
<td>WIND WAVES</td>
<td>0.5s – 30s</td>
</tr>
<tr>
<td>Propagating</td>
<td>TSUNAMI</td>
<td>10min – 40min</td>
</tr>
<tr>
<td>Oscillating / Sloshing</td>
<td>TIDES</td>
<td>12h – 24h</td>
</tr>
</tbody>
</table>

*Figure 1.1: Tsunami are propagating long waves, between short wind waves and tides.*
Figure 1.2: Schematic representation of the tsunami generation process, taken from IOC/UNESCO (2005).
Chapter 2. Literature Review

2.1 Introduction

The focus of this thesis is on long propagating waves interacting with a beach, with the aim of gaining knowledge on tsunami wave runup and behaviour in the nearshore and onshore areas. To understand this, we review some of the key physical processes of water wave dynamics, and introduce the main wave classifications (section 2.2). In section 2.3, numerical tools used for modelling long waves such as tsunami are introduced, and their performance and limitations are critically reviewed. Section 2.4 introduces the physical modelling tools typically used to study long waves and identifies gaps in the ability of these tools to model certain important aspects of long propagating waves. In section 2.5, we present field tsunami characteristics, offshore, nearshore and onshore through observations, records, and case studies of historic tsunami. This final part aims at identifying important field characteristics of destructive long waves such as tsunami, and factors that are influential on the impact of long waves on the coast. In section 2.6, the findings of this review are integrated to identify the gaps to be answered in this thesis.

2.2 Wave Dynamics

2.2.1 Equations of motion and classifications
This section aims at introducing a hierarchy of wave models of decreasing complexity to assess their validity for different wave conditions. A wave is a disturbance of the water surface that propagates, usually with a net transfer of energy from one place to another. Equating the applied forces to the inertia forces of a unit volume of fluid results in the momentum equation. Different forms of this equation exist, depending on the assumptions made regarding the properties of the fluid. One main assumption considered valid for all forms of the momentum equation is that the fluid (water) is incompressible (i.e. \( \rho \) does not vary with pressure inside the fluid, often it is assumed that \( \rho = \text{constant} \)).

This assumption can be challenged in cases such as large earthquakes, landslides / asteroids impacts or underwater explosions, where correct wave generation modelling requires that important density changes are taken into account (Mader, 2004). For wave propagation, the incompressibility assumption can be considered valid. Indeed, despite the density variations (pycnoclines) occurring naturally in the environment, the thin regions of substantial density variation are contained within larger regions of essentially homogenous fluid (Albert et al., 1997), particularly at an oceanic scale. During inundation and flooding, mixing of water with sediment is be expected, inducing significant density changes as the wave travels and at the interface between the ocean and the bathymetry, so compressibility should be taken into account. In this thesis we can assume conservation of mass (continuity) as:

\[
\nabla \cdot \mathbf{U} = 0.
\]

In (2.1), \( \mathbf{U} = \{u, v, w\} \) is the flow velocity. Le Mehaute (1976b) introduces the range of momentum equations:
• The Navier-Stokes equations apply gravity, pressure, and friction to the fluid against inertia. The pressure is assumed to be hydrostatic. These equations provide the basis of most fluid mechanics problems:

\[ \rho \frac{DU}{dt} = -\text{grad}(p + \rho gz) + \nu \nabla^2 U, \]  \hspace{1cm} (2.2)

In (2.2), \( U = (u, v, w) \) represents the three components of velocity and \( \nu \) the kinematic viscosity. The Navier Stokes equations have four unknowns and need to be solved numerically.

• The equations of Euler, which are the Navier-Stokes equations without the viscous forces (i.e. assumption of a perfect fluid):

\[ \rho \frac{DU}{dt} = -\text{grad}(p + \rho gz), \]  \hspace{1cm} (2.3)

At this next level of simplification (3 unknowns), the effects of friction are lost. This is expected to affect predictions for thin flows, where the relative influence of viscosity with respect to depth will be higher, as well as for runup and inundation, regions where viscous forces are more influential on wave dynamics.

• The Boussinesq approximation is derived from the equations of Euler and assumes the pressure is not hydrostatic. There exist a number of Boussinesq-type equations (Madsen and Schäffer, 1999), and they vary according to the
relative influence of non-linearity and dispersion. Boussinesq equations accounting for non-linearity and dispersion to the same order can be found in Mei (1989). For simplicity, only a one-dimensional motion neglecting non-linearity ($a/h<<1$) is introduced here (2.4), along with the associated conservation of mass (2.1). The dependence of flow velocity on the vertical coordinate is removed, and a horizontal velocity is chosen, the depth-averaged velocity:

$$\frac{\partial \bar{u}}{\partial t} + \bar{u} \frac{\partial \bar{u}}{\partial x} + g \frac{\partial \eta}{\partial x} - \frac{h^2}{3} \frac{\partial^3 \bar{u}}{\partial x^3 \partial t} = 0,$$

(2.4)

$$\frac{\partial \eta}{\partial t} + \frac{\partial}{\partial x} (h + \eta) \bar{u} = 0.$$

(2.5)

Boussinesq equations are valid in shallow water (no dependence of velocities on water depth), but only for weakly non-linear and moderately long waves. The same considerations (as for Euler equation) on nearshore and onshore dynamics still stand.

- The shallow water equations do not account for dispersion. They are based on the depth averaged formulation of the Navier Stokes equation, which reduces the number of unknowns. The form of the SW equations presented here has been taken from Moler (2009) and presented in a flux conserving form:
There are non-linear (NLSW) and linear (LSW) forms of the shallow water equations, valid for very long waves in shallow water. In linear shallow water equations, we assume $|\eta| \ll 1$ and the velocity is small enough that the dependence of terms on the vertical coordinate as well as the quadratic terms in $u$ and $\eta$ can be neglected. The neglected terms account for a change of velocity with respect to space. So in the linear form of these equations any rotation or deformation of the fluid is considered to be negligible.

The shallow water approximation (2.6) is useful if both non-linearity and dispersion can be neglected, which implies $a/h \ll 1$ and $kh \ll 1$. A very long
wave will have a small wave number \( k \), so neglecting dispersion for the propagation of long monochromatic waves is reasonable. However, as the wave shoals over the continental shelf, non-linearity will increase which will affect the prediction of such equations nearshore / onshore for linear and non-linear forms of the shallow water equations.

- The Kortvég and de Vries (KdV) equation is based on a weakly non linear form of the shallow water equations. Contrary to all other equations, it involves solving for only one unknown. The form of the KdV equation presented here has been taken from Miles (1980):

\[
\frac{dn}{dt} + c_p \left( \frac{dn}{dx} + \frac{3}{2} \left( \frac{n}{h} \right) \frac{dn}{dx} + \frac{1}{6} h^2 \frac{d^3 n}{dx^3} \right) = 0. \tag{2.7}
\]

In (2.7), \( c_p \) is the shallow water wave speed, conventionally taken as \( c_p = \sqrt{gh} \) (see section 2.2.2). The effects of weak non-linearity and dispersion are accounted for. One solution of the KdV equation for a wave with an infinite length is the solitary wave, which will be presented in section 2.2.3.

Euler and Navier-Stokes equations are respectively the momentum equations for the case of a perfect fluid, and for the case of a viscous fluid. From a mathematical point of view, describing water wave motion consists in solving these equations for which the solution is, generally, the free surface profile. In a water wave problem there can also be other unknown functions such as the pressure, and velocity components. Because a variety of water waves exist, there is no general method of solution, and even in the simple cases, approximations are necessary (Le Mehaute, 1976b; Mader, 2004).
Instead, several methods of solution exist, and for each of them an associated limit of validity.

There are different ways of classifying water waves. Le Mehaute (1976b) only highlights two classifications, physical and mathematical, which are described here. The physical classification gives two families of waves: oscillatory and translator. For oscillatory waves, Le Mehaute (1976b) finds that the orbital particle motion is essentially closed (this is true to first order), and there is no mass transport in the direction of wave travel, with a return flow under the wave crests. For translator waves on the other hand, the net mass transport in the direction of wave travel is important. Examples of translator waves include solitary waves. The mathematical classification allows waves to be categorized according to the choice of simplifying assumptions used to solve the equations of motion. For each set of assumptions there is a different wave theory that can be applied to a particular set of conditions. These simplifying assumptions depend on three characteristic parameters of the wave studied: the wave height \( H \), the wavelength \( L \), and the water depth \( h \), or more precisely on the three characteristic ratios that can be obtained using these parameters: \( H/L \) (wave steepness), \( H/h \) (relative height), and \( h/L \) (relative depth).

Table 2.1 presents the main analytical wave theories and their domain of application, as well as physically, the type of wave motion they represent and mathematically, the type of solution for the problem of free surface motion.

The Airy theory corresponds to the first order of Stokes theory, which is applicable in deep to intermediate water for steeper waves, and described in Le Mehaute (1976b). When it is assumed that \( H/L \), \( H/h \), and \( L/h \) are all small (deep water), linear wave theories can be applied. In shallow water, waves with small \( H/L \) and \( H/h \) can be
described using the linear long wave theory. If the wave steepness is more important, cnoidal and solitary wave theories have to be used. The cnoidal theory is based on the KdV equation (2.7). The solitary wave theory is one exact solution of the KdV equation, it is a wave with an infinite period. Expressions for the profile characteristic length of a solitary wave can be found in section 2.2.3.

Limits of validity for various wave theories with respect to depth and wave height can be found in Le Mehaute (1976b). However as highlighted by the author himself, the graph is merely qualitative, giving an indication of the domains of applicability of various wave theories. Subsequent studies (Fenton, 1990; Hedges, 1995) used experimental, numerical and analytical results to reassess the domains of validity of the wave theories, which are synthesized and plotted in Figure 2.1.

2.2.2 Wave Dispersion

A wave with a constant wavelength propagates with a phase speed $c_p$. Dispersion (or frequency dispersion) occurs when waves of different wavelengths travel at different phase speeds. The dispersion relation relates the phase speed to wavelength $L$ and water depth $h$ for a periodic wave train moving over water of uniform depth (Lighthill, 1966):

$$c_p = \sqrt{\frac{gL}{2\pi \tanh\left(\frac{2\pi h}{L}\right)},}$$

(2.8)

In shallow water, $k$ is small therefore $\tanh(kh) \approx kh$. Knowing $c_p = \frac{L}{T}, k = \frac{2\pi}{L}, \omega = \frac{2\pi}{T}$, and rearranging the terms in (2.20) we obtain, for shallow water conditions:
\[ c_p = c_{p_0} = \sqrt{gh}. \] (2.9)

For the particular case of shallow water, the phase speed depends only on gravity and water depth, not on wavelength. Therefore, it is commonly accepted that shallow water waves are not dispersive in frequency.

Another dispersion phenomenon potentially affecting long propagating waves is amplitude dispersion. There is a dependence of the signal velocity on the wave amplitude if the latter is significant. In shallow water, for the particular case of solitary waves, the speed is not only a function of depth and acceleration of gravity but also wave amplitude (see section 2.2.3). As a result, when several solitary waves interact the ones of greater amplitudes travel faster than the smaller solitary waves. Moreover according to Johnson (1998), solitary waves do not change form when they interact.

### 2.2.3 Different types of propagating waves

**Solitary waves**

When we force a channel of water of uniform depth, in many cases a solitary wave finally emerges from the wave train. A solitary wave is a single positive elevation above the mean water level. This unique type of wave can propagate without change of form, as there is a balance between dispersion and non-linearity. The conditions for which the solitary wave theory may replace the cnoidal theory have been defined by Hedges (1995) to be \( HL^2/h^3 \geq 4000 \).
Miles (1980) presents the Boussinesq profile of solitary waves (Figure 2.2). It can be obtained for the exactly solvable KdV equation (2.7) and is a first order approximation for solitary waves (Halasz, 2009).

\[ \eta(x, t) = a(\text{sech}(x - c_{p_{s1}} t/L))^2, \]  

(2.10)

where \( a \) is the solitary wave amplitude, and \( c_{p_{s1}} \) is the phase speed. The characteristic length \( L_{s1} \) of the solitary wave is given by (Miles, 1980):

\[ \frac{L_{s1}}{h} = 2\sqrt{\frac{1}{3} \left( \frac{h}{a} + 1 \right)}, \]  

(2.11)

which shows that \( L/h \) decreases when \( a/h \) increases, so \( L/h \) is proportional to \( (h/a)^{1/2} \).

The shallow water approximation for the wave speed \( c_{p_0} \) is different from the one used by Russell (1845) who expressed the phase speed not only as a function of depth, but also wave amplitude:

\[ c_{p_{s1}} = \sqrt{g(h + a)}. \]  

(2.12)

To second order (Mader, 2004), solitary waves are described by:

\[ \eta(x) = h + a\text{sech}^2(z) - \frac{4}{3} a \frac{a}{h} \text{sech}^2(z) \left( 1 - \text{sech}^2(z) \right), \]  

(2.13)

with

\[ z = \frac{x}{h} \sqrt{\frac{3a}{4h} \left( 1 - \frac{5a}{8h} \right)}. \]
\[ L_{s2} = \frac{2h}{\sqrt{\frac{4\rho}{gh}(1 - \frac{5a}{8h})}} \]  

(2.14)

\[ c_{ps2} = \sqrt{gh \left( 1 + \frac{1}{2} \frac{a}{h} - \frac{3}{20} \left( \frac{a}{h} \right)^2 \right)} \].  

(2.15)

Integral wave parameters can be defined for solitary waves (Longuet-Higgins, 1974). Excess mass \( A \), wave momentum \( I \), potential and kinetic energy \( E_p \) and \( E_K \) are expressed as:

\[ A = \int_{-\infty}^{\infty} \rho \eta(x) dx, \]  

(2.16)

\[ I = \int_{-\infty}^{\infty} \int_{-h}^{\eta} \rho ud\eta dx, \]  

(2.17)

\[ E_p = \int_{-\infty}^{\infty} \frac{1}{2} \rho \eta(x)^2 dx, \]  

(2.18)

\[ E_K = \int_{-\infty}^{\infty} \int_{-h}^{\eta} \frac{1}{2} \rho (u^2 + v^2) dy dx. \]  

(2.19)

The area under a two-dimensional solitary wave is called its excess mass. The momentum is related to the excess mass as follows (Longuet-Higgins, 1974):

\[ I = c_p A. \]  

(2.20)

Equations (2.16) to (2.20) would give physically meaningful quantities for other type of translator waves where the variables of interest are known.

**N-waves**

Tadepalli and Synolakis (1994) introduce the concept of N-waves (Figure 2.2), these are characterized by a positive and a negative elevation. There are many different forms of N-waves, but for mathematical convenience, Tadepalli and Synolakis (1996) proposed a general form of N-waves to have the profile:
\[ \eta(x) = \varepsilon \eta(x - X_2) a sech^2(\gamma(x - \theta))|_{t=0} \] (2.21)

In (2.19), all variables are dimensionless. \( \varepsilon < 1 \) is a scaling parameter defining the crest amplitude \( a \), \( \theta = X_1 + c_p t \). Graphically, \( X_2 \) corresponds to the horizontal position of the inflection point of the profile, corresponding to \( \eta(X_2) = 0 \) and \( X_1 \) is the position of a positive solitary wave of the same amplitude centered on \( X = X_1 \) at \( t = 0 \), as described by Synolakis (1987) in his description of solitary waves. The wave speed, \( c_p \), has been set by the authors to be equal to 1 when normalized. Therefore, it is assumed the N-wave speed is equal to the shallow water speed \( c_{p_0} = \sqrt{gh} \), \( \gamma = \sqrt{3ap_0/4} \) where \( p_0 \) is a steepness parameter. When the crest and depression heights are equal, \( X_1 = X_2 \) and N-waves can be called isosceles. At this point, the integral measures defined for solitary waves (2.17) to (2.20) are considered to be applicable for N-waves. It is important to note that isosceles N-waves or N-waves with a larger trough will have a null or negative excess mass. However, the total potential energy of the N-wave does not depend on the polarity of the wave, as the energy at play in either the peak or the trough is the energy needed for the water level to return to equilibrium.

2.2.4 Wave Runup

Runup is a local characteristic of a wave flow inland. It is defined as the maximum inundation point above mean sea level (Figure 2.3 - a). Runup is extensively used, compared to other wave characteristics, as an indicator of wave’s potential for inundation and coastal impact.
Early studies have attempted to find relationships linking runup, wave heights and wave lengths for periodic waves (Kaplan, 1955; Shuto, 1967; Togashi, 1981), but their results failed to agree as highlighted by Synolakis (1986). Runup of propagating waves has been investigated analytically, numerically, using the momentum equations presented in section 2.2.1 (Carrier & Greenspan, 1958; Kobayashi et al., 1990; Zelt, 1991), and also in the laboratory. A sample of important runup relationships found in the literature is given in Table 2.2, and the corresponding studies are described in further detail below.

It is difficult to confidently measure the runup of a long propagating wave in the field, and practically impossible to use field data to derive runup relationships. However, a simple runup relationship based on observations by Plafker (1965) following the 1964 Alaskan tsunami was proposed (Synolakis & Bernard, 2006) It states that the maximum local runup does not exceed twice the height of the deformed seafloor $S_d$ offshore:

$$R \leq 2S_d.$$  \hspace{1cm} (2.22)

The rule (2.22) can be expected to work for the specific configuration of water displaced vertically along an idealized straight shoreline or in a closed symmetric basin (such as the area of Prince William Sound, Alaska, where the 1964 tsunami took place). It agrees with some observations, such as: Borroto (2005), the numerical modelling results from Okal and Synolakis (2004) for a linear shoreline, and the numerical modelling of McCloskey et al. (2008) taking into account maximum tsunami wave heights instead of the actual runup. It is interesting to note the study from McCloskey et al. (2008) focused on a potential rupture of the fault located west of Sumatra, and also facing a relatively linear coastline (Padang-Bengkulu), so for such a configuration the Plafker rule is expected to give satisfactory results. However, it is not expected to
perform well in other cases where the runup can be affected by local features such as bays, estuaries or land use. In addition, to predict runup prior to a tsunami event, this rule implies the maximum seafloor displacement is known beforehand. Moreover, this rule seems to be valid for near-field waves and it is likely to be unreliable in areas of extreme coastal steepnesses (Plafker, 1997) or if other amplification factors are present (section 2.5.2).

To simplify the runup problem, a number of studies have considered a single positive wave running-up a beach with a constant slope. Many take into account the influence of bed slope and wave height, starting with the experiments from Hall and Watts (1953), who looked at the interaction of propagating waves with a 45 degree ($\tan \beta = 1$) beach. The experimental waves were unidirectional, generated and propagated over a constant depth region, and climbed the beach. They resembled solitary waves in shape. A runup relationship, dependant on the beach slope, was derived (no distinction was made between breaking and non breaking waves). Moreover, this slope was very steep and the runup processes are expected to be different from the ones on shallower slopes. The theory proposed was:

$$\frac{R}{h} = \alpha(\beta) \left( \frac{H}{h} \right) f(\beta).$$  \hspace{1cm} (2.23)

In equation (2.23) $\alpha(\beta)$ and $f(\beta)$ are empirical coefficients dependant on the beach slope $\beta$. For a 45 degrees slope, the authors obtained $\alpha(\beta) = 3.1$ and $f(\beta) = 1.15$. Synolakis (1987) carried out similar experiments using a 2.88 degrees slope angle ($\tan \beta = 1/19.85$). He calculated the evolution of a solitary wave through the entire runup and rundown process, compared his results with experimental data, and using a weakly non-linear model solved analytically he derived the following runup equation:
\[
\frac{r}{h} = 2.831\sqrt{\cot\beta \left(\frac{d}{h}\right)^{5/4}}.
\] (2.24)

Equation (2.24) is valid for non-breaking waves. The runup here is defined as being maximum water surface elevation above the initial shoreline position. Runup regimes were shown to be different for breaking and non-breaking waves (Figure 2.4). We can see that some agreement was found with (2.24) for non-breaking waves, however the predicted trend moves away from the non-breaking wave data at higher amplitudes. Equation (2.24) is very similar to the result from Hall & Watts (1953) with \(a(\beta) = 2.831\sqrt{\cot\beta}\) and \(f(\beta) = 5/4\), and has been proved right even for steeper slopes (Gedik et al., 2005). Borthwick et al. (2006) validated their numerical results against Synolakis (1987) (for a friction coefficient of 0.008) then investigated the dependence of runup of beach slope numerically. They found there was an upper value to the wave runup for beach slopes comprised between 1/100 and 1/5, which corresponds to equation (2.23) with \(a(\beta) = 3.02\) and \(f(\beta) = 0.91\). These results show that \(a(\beta)\) is one order of magnitude greater for Synolakis (1987), otherwise they would indicate that the value of \(a(\beta)\) may not decrease significantly as the slope steepness increases. The evolution of \(f(\beta)\) versus \(\beta\) for these three examples does not show any strong correlation between bed slope and the coefficient \(f(\beta)\).

Tadepalli & Synolakis (1994) used a non-linear model to obtain expressions for the runup of different types of N-waves numerically. It is important to note that their definition of runup was different from that introduced at the beginning of this section: they defined runup as being the maximum wave amplitude above the initial shoreline position at the maximum penetration of the wave (also in Tadepalli & Synolakis, 1996). Their runup equations are valid for non-breaking waves. They proposed that the runup of an N-wave would be greater than the runup of a solitary wave when they have the
same positive amplitude. They also found that the runup of a LDN (Leading Depression N-wave) is greater than an equivalent LEN (Leading Elevation N-wave), and finally, that the runup of an isoceles LDN wave would be greater than any other wave runup. Tadepalli & Synolakis (1996) used a hybrid KdV-shallow water numerical model to propagate an N-wave with profile (2.21) and develop a general runup equation applicable to different types of N-waves:

\[
\frac{R}{h} = 3.3 \epsilon_{\gamma} p_0^{-\frac{1}{2}} Q(L, \gamma) \frac{R_{sol}}{h}. \tag{2.25}
\]

Equation (2.25) is valid for any type of non-breaking LD N-wave when \(4\gamma \cot \beta \gg 1\) \((\gamma = \sqrt{3ap_0/4})\). In (2.25), \(R_{sol}\) is the runup of a Boussinesq solitary wave of the same amplitude, however the authors do not clearly state whether \(R_{sol}\) is the maximum elevation of inundation above still water level, or the maximum wave amplitude above the initial shoreline position. Moreover, the relationship between \(Q, L\) and \(\gamma\) is not stated. Tadepalli & Synolakis (1996) noted that the maximum runup increases as the trough to crest ratio increases.

Because bores do not make any assumptions on the wave breaking process, they are a very general case of runup to study. The bore runup equation was initially derived by Shen and Meyer (1963), in an analytical study of bore runup. The exact same result was independently derived also by Baldock and Holmes (1999), in their study of swash oscillations, by using laws of motion for a body with constant acceleration and the results of previous studies on bores:
Equation (2.26) describes the unsaturated runup (i.e. runup corresponding to the first swash) as a function of the flow velocity, or the height at the point of bore collapse ($H_b$) and $C$, comprised between 1 and 2. $C$ is a coefficient describing the efficiency of “bore collapse”; which is described as a process involving the rapid conversion of potential to kinetic energy around the shoreline. When a bore front meets a shallow layer of quiescent water, the fluid and bore front velocities approach a common finite value (Yeh et al., 1989). When all the potential energy is converted to kinetic energy, $C$ takes the maximum value of 2. The runup result has been derived for steep beaches, i.e. $\tan \beta \approx 1/10$.

To understand wave runup in three dimensions, Briggs et al. (1995) investigated experimentally the particular problem of variations in runup on a conical island according to wave heights, water depth, and source characteristics. For the range of conditions studied, they derived equations of the form:

$$\frac{R}{h} = \frac{1}{h} (a + bS_D + cS_D^2).$$

(2.27) was derived using regression analysis (i.e. least squares fit), with $a$, $b$, $c$ being quadratic coefficients depending on the water depth and wave height; and $S_D$ representing the tsunami source length offset a distance $D$ from the center of the island ($D$ represents the diameter of the island). This runup relationship has not been tested outside of the range of conditions studied by the authors in three dimensional experiments or with field data.
A small number of studies gives additional information on the factors that influence runup. The results of the numerical modelling from Borthwick et al. (2006) show that for \( a/h > 0.015 \), the runup decreases as the friction coefficient increases, concluding the abrupt change in gradient between the two wave regimes highlighted by Synolakis (1987) is solely due to bed friction. This means that equation (2.24) may be valid for all waves for which \( a/h < 0.015 \). For a frictionless case, Borthwick et al. (2006) found there was no change in runup regime at \( a/h = 0.015 \), in this case equation (2.24) would apply to all waves. Moreover, the results indicate that the runup of solitary waves is greater on steeper beaches for large values of \( a/h \), and had the opposite trend (i.e. smaller values of runup on steeper beaches) for small values of \( a/h \). They also found that for a given value of the friction coefficient, there is an upper limit to the runup irrespective of the beach slope. Gedik et al. (2005) quantified the decrease in runup with increase in slope permeability. Other studies focus on the evolution of the runup process, notably in terms of flow velocities. Yeh (1991) examines the movement of bores in terms of runup and velocities. These are shown to be scattered in the shoreline region, but a clear increase in flow velocities between the upstream and downstream of the shoreline could be observed. Lin et al. (1999) look more closely into the horizontal and vertical components of velocity during wave runup. They use a combined experimental-numerical approach to investigate solitary wave runup and velocity fields on beaches. Variations of horizontal velocity magnitude over the water column during runup were detected for non-breaking waves. For breaking waves, numerical results of velocity under the waves are presented and show the horizontal velocity is nearly constant at the beginning of the shoaling process, and after breaking in the propagating bore (Figure 2.5). During the late runup phase, it is shown that maximum velocities occur at the runup tongue. Synolakis (1986) suggests broken waves (bores) runup higher than non-
breaking waves, and by generating bores of different lengths, shows a dependence between the displacement and duration of plate motion and the maximum runup. However, for bores of a duration greater than 10.8s, all runups seem to tend to a common value. Finally, Li and Raichlen (2003) experimentally look at runup from the point of view of energy balance:

\[
\frac{R}{h} = \left[1 - \frac{E_B}{E_I}\right] \left(\frac{H}{h}\right).
\]

The authors find that runup is directly dependent on the beach slope and wave height, which is consistent with previous results. Over the slope, the potential energy \(E_p\) increases and the kinetic energy \(E_K\) decreases to eventually become 0 at the end of the runup (Li & Raichlen, 2003). Therefore, the focus was on \(E_p\). The potential energy at the time of maximum runup is estimated from a simple energy conservation model based on the total incident energy \(E_I\) and the energy dissipated during breaking \(E_B\), so in (2.28) we have:

\[
\frac{E_p}{E_I} = 1 - \frac{E_B}{E_I}.
\]

The ratio \(E_B/E_I\) (2.29) is determined empirically by the authors:

\[
\frac{E_B}{E_I} = C[A \ln(\cot \beta) + B],
\]

with \(A = -0.47 \left(\frac{H}{h}\right) + 0.534, \ B = 2.165 \left(\frac{H}{h}\right) - 1.154,\) and \(C = 0.19 \left[\ln \left(\frac{H}{h}\right)\right] + 0.969\).

To conclude, existing runup equations are based either on analytical and numerical studies, or on few sources of experiments, which mainly involved solitary waves and
bores. The results show that factors influencing runup comprise wave amplitude, source length, source deformation, beach slope, bed friction, wave energy, breaking.

2.3 Review of results obtained by numerical models for tsunami

Numerical models are based on the equations of motions highlighted in section 2.2.1 and their assumptions. Therefore, some issues may arise when using such tools to address the problems of long wave propagation and interaction with beaches. This section aims at reviewing the results that can be obtained with these tools, particularly in the nearshore environment. The success of numerical models to capture the physics of long waves moving into coastal regions, breaking and interacting with buildings depends on both the important physics and the resolution of the calculations.

The shallow water equations (2.6) have been extensively used to model long wave propagation in the deepest parts of the ocean. For instance, Titov et al. (2005) modelled the 2004 tsunami using the MOST model. The computed offshore amplitudes provided accurate estimates, indeed tsunami wave amplitudes in the deep ocean are much smaller than the wavelength, so shallow water equations are expected to perform well (see section 2.2.1). More examples of computations of waves offshore could be listed, however the focus of this study is on the nearshore behaviour of the wave.

As the wave travels into coastal regions, the decrease in water depth causes the wave amplitude to increase as the phase speed decreases (see (2.9), (2.12) and (2.15)). Shallow water models give satisfying results in terms of wave heights as shown for instance, by Synolakis (1986) and Satake (1995). Mader (2004) confirmed the result
from Satake (1995), by proving that except for shorter waves (periods<600seconds), the Navier Stokes results matched the shallow water long wave results in terms of heights and wave profiles. However, for significant propagation distances, Boussinesq models are more accurate than shallow water models, in terms of predicted surface elevations, as shown by Grilli et al. (2007). Indeed, in the field it is expected that waves of varying lengths would occur, therefore showing a dispersive behaviour for significant propagation distances.

Runup is severely underestimated by shallow water models. These observations are confirmed by testing of the NLSW - OTT model developed by Dodd et al. (1998), which does not produce sensible results if the wave breaks, or if the wave is shorter than the propagation domain. Waves can be run in OTT only if the distance offshore is about one mean wavelength or less. Indeed, when the wave breaks, the detailed physics are not accounted for by shallow water models. More detailed calculations using VoF methods and Navier Stokes solvers can be employed. Mader (2004) also found that the waves modelled with the shallow water equations shoaled higher, steeper and faster than the Navier Stokes waves (therefore they also “break” earlier). The differences between the two models increase as the wave periods become shorter and the slopes get shallower: as shown in section 2.2.1, a shallow water model breaks down when non-linearity becomes significant. In terms of runup, the shallow water model gives satisfactory results only for the first swash, and for wave periods greater than 900s (15min).

Despite recent advances in technology such as PIV (particle image velocimetry), allowing the recording of instantaneous velocity distributions under breaking waves, wave breaking is still not a well understood process. In an attempt to test the influence
of realistic breaking processes on wave inundation results, Lin et al. (1999) used a 2D RANS (Reynolds Averaged Navier Stokes) model (accounting for turbulence) in a study of solitary wave runup and rundown. The numerical tests showed that even with turbulence and viscosity taken into account, the numerical results remained almost the same as for a non-turbulent inviscid model (such as a shallow water model), so the energy dissipation from turbulence and viscous effects are negligible especially for non-breaking waves. However, the vertical distribution of horizontal velocities on steep slopes appeared non uniform, challenging the shallow water models’ assumption of a depth-averaged flow.

**Necessity for accurate bathymetry / topography data**

When assessing tsunami behaviour and impact at the coast, one main issue with shallow water models is the accuracy of the results is highly dependent on the resolution of the bathymetric and topographic data (Titov & Synolakis, 1998; Titov et al., 2005; Mader, 2004), contrary to the intuitive expectation that long waves would not interact with short scale morphological features. However, high resolution bathymetries and topographies are not widely available, or simply do not exist. The cost of acquiring such data is high as hydrographic surveying missions are extremely expensive in terms of equipment, manpower and time. To overcome this problem, Satake (1995) suggested that more comparisons of the observed runup heights with computed tsunami amplitudes on various types of coasts would be needed to find an amplification factor necessary in coastal areas lacking detailed bathymetric data. According to Yean et al. (2006) such an amplification factor defined as $R/H$ can be found for constant slopes, for example the authors used an amplification factor of 3.1
for a very steep slope of 45 degrees (1/1). The runup law (2.26) also indicates the amplification factor is proportional to the bed slope. However, such an amplification factor has not been derived yet for complex coastal bathymetries and tsunami modelers are still dependent on the availability of elevation data.

The numerical grid size also determines the accuracy of the model results, especially for local flooding (Mader, 2004), or runup (Satake, 1995). This is confirmed by the results obtained using OXBOW (designed by Weston, 2004), a Boussinesq code which cannot produce sensible runup results with a low resolution bathymetry. However the finer the grid, the more computationally expensive it is to run the model. Some simple models such as OXBOW encounter problems with very large cell numbers. Satake (1995) highlights that the use of detailed bathymetry data with a small grid size is more effective than the inclusion of non linear terms, so it appears the influence of elevation data and numerical grid resolution is more important than the complexity of the model.

### 2.4 Physical Modelling

Hydraulic models (also called physical models) are an alternative to numerical models when looking at nearshore and onshore flow. In this section, hydraulic models as tools for understanding coastal processes (for the case of tsunami) are reviewed. These models are indeed useful to gain insight into the processes numerical models or theoretical approaches cannot access. However, unless designed specifically to address a particular case scenario, the model is a simplification of reality and the results obtained only give an indication of the real processes at play. Hydraulic models are also
used to validate theories or models that can be used later to extrapolate the experimental results, or applied to a real case scenario.

*Types of waves generated*

In the available literature, physical experiments that aim at gaining knowledge of long propagating waves behaviour often involve the generation of solitary waves. Generating solitary waves using a moving paddle is a widely used method as the generated waves are in good agreement with theoretical waves: according to Hughes (1993), the wavemaker equations (equations relating the displacement of the plate to the wave parameters) have been well-established and extensively applied. Other methods have been used to generate solitary waves. Hammack (1972) tried to obtain an insight of the tsunami amplitudes in the generation region, and designed a 2D experiment where waves were generated by raising or dropping one end section of the bottom of a wave flume, in an attempt to simulate the generation of an earthquake-generated tsunami. The author generated trains of dispersive waves. Bukreev (1999) also generated solitary waves by different means (removal of a baffle, horizontal displacement of a plate, submerged moving object along the tank) to tackle the “free solitary wave” problem (i.e. what would be the behaviour and the limiting amplitude of a naturally occurring solitary wave in the field). The results from his study indicated free solitary waves over the range $0.2 < a/h < 0.6$ cannot be generated by natural processes as even in the laboratory, all the necessary conditions cannot be met, and waves approaching the limiting amplitude are often obtained by forced processes. Long waves such as tsunami have also been physically modelled by a bore, which is assumed
to be the form taken by the wave reaching the coast after breaking offshore (Yeh, 1991).

Stable N-waves are difficult to recreate in an experimental environment. Attempts to generate negative waves or trough led waves have rarely been mentioned in the literature. Miles (1980) describes an experiment consisting in creating an initial depression in a wave tank by withdrawing a weight from a wave flume of constant depth, but the resulting wave train was oscillatory and quickly dispersed. Kobayashi and Lawrence (2004) studied the effect of the polarity of a solitary wave on sediment transport on a beach, using the backwards motion of the classic vertical paddle to create the negative wave. However the negative solitary waves generated changed form rapidly during propagation, significantly more than the positive solitary waves over the sloping beach. This method was also tried by Cox (2008), who explained that researchers tried to reproduce depressed waves in this fashion but again the small trough was seen to dissipate rapidly. The lack of information on this subject possibly indicates the impossibility of experimental facilities to reproduce anything like a N-wave, one reason for this being their relatively short wavelength when reproduced with current laboratory techniques.

Wave periods, amplitudes

To date, experimental solitary waves have relatively short periods: around 4s for the experiments performed in the Caltech wave flume (Goring, 1978), and a maximum of 10s for the waves generated at Oregon State University (Cox, 2007a; Cox, 2007b) and in the Large Hydro-Geo Flume in Japan (Shimosako et al., 2002). This is mainly due to
the limited excursion of the paddle forward (stroke). Observations on limited paddle displacement have specifically been made by Hammack (1972), Goring (1978) (who found there was a time lag between the programmed and the actual motion of the plate), and Synolakis (1986). The latter made a similar observation to Hammack (1972) regarding the total initial displacement of water, as he also noticed the maximum excursion of the piston was limited. As a result, maximum wave amplitudes are also partly determined by the paddle stroke. Briggs et al. (1995) found the maximum normalized wave height that could be generated was 0.20 due this limitation. The Tsunami Wave Basin at Oregon State University is capable of generating a solitary wave 0.8m high in a depth of 1m (29 piston-type wave boards), the stroke of the generator is limited to 2.1m for each paddle. Synolakis (1986) used a constant velocity of the wave paddle and the largest possible displacements to generate motions longer than 10s, in the form of broken waves or bores, however it was shown that these waves changed shape throughout propagation.

\textit{Parameters studied in physical modelling}

Some studies have focused on the evolution of solitary waves in a constant depth region; for example Bukreev (1999) with the free solitary wave problem, Goring (1978) who investigated the various aspects of the propagation of solitary and cnoidal waves over a step and over a shelf, or Hammack (1972) whose study consisted in measuring very precisely the evolution over a constant depth of waveforms resulting from a vertical displacement of the seabed. These studies allow a better understanding of the wave generation and propagation close to the source, and the validation of forms of the momentum equations (Hammack (1972) verified one solution of the KdV
equation (2.7)). However they do not give relevant information on nearshore wave behaviour.

In the nearshore region, most studies focus on wave runup and surface profiles. Synolakis (1986), examines solitary waves running up a 1:19.85 sloping beach. In this study, the experiments were designed to validate an analytically derived expression for wave runup. Synolakis (1987) also showed runup regimes were shown to be different for breaking and non breaking waves. The waves generated were first order solitary waves (as described by equation (2.10)). Other studies have attempted to obtain an insight into the influence of the source length and flow velocities on wave runup (Briggs et al., 1995; Yeh, 1991). Regarding the interaction of waves with structures, only very few experiments have looked into wave impact pressures (Thusyanthan and Madabhushi, 2008; Bullock et al., 2007), and the effect of scour (Tonkin et al., 2003).

**Experimental scale**

Solitary waves studies have been carried out both in wave flumes (Synolakis, 1986; Bukreev, 1999) and in 3D basins (Briggs et al., 1995; Borthwick et al., 2006) of various dimensions, listed in Table 2.3. When studying long waves, the channel / basin needs to be long (since $L/h>>1$), and deep enough so viscous effects are weak over most of the wave evolution. In the laboratory, it is currently impossible to recreate a wave that is several kilometers long and several meters high, so the model wave has to be smaller, however not so small that other forces (such as friction, or capillary effects) start influencing the wave behaviour significantly. For instance, the experiments of Thusyanthan & Madabhushi (2008) and Bukreev (1999) were realized in small wave
flumes (Table 2.3). Although the Bukreev (1999) study was mainly aimed at correlating limiting theoretical and physical amplitudes for solitary waves and was not supposed to address directly the long wave problem, it is interesting to note that in such a small scale experiment surface tension and viscous effects must have been significant, therefore not accounting accurately for the main forces driving long wave generation and propagation in the field (as pointed out by Yim et al., 2004).

However, even for large scale laboratory experiments, the correct reproduction at a reduced scale of a real physical process is challenging due to scale effects. Oregon State University has recently set up a large-scale tsunami research facility in their Wave Research Laboratory (Yim et al., 2004), with the aim of investigating wave breaking and turbulence, wave-structure interactions, runup and velocity but also tsunami triggered by landslides. This facility is under development and few results are yet published.

The experiments presented in Table 2.3 are non exhaustive. The table gives an overview of existing and tested methods for generating solitary waves or bores. Figure 2.6 locates experiments within the wave theories and domains presented in section 2.2.1, and shows that most experiments are located in the intermediate water or shallow water domains. The shallow water experiments correspond mostly to cnoidal wave theory. In comparison, some typical nearshore tsunami wave parameters have also been located on this graph, as well as one actual tsunami record from the 2004 Indian Ocean tsunami (presented later in section 2.5.2). We can see that field tsunami are proportionally longer (and can be higher) than the waves currently possible to generate in a laboratory environment.
2.5 Tsunami Observations

Observations from real tsunami events provide an insight into the behaviour of long waves propagating in the field. Prior to technological advances such as satellite imagery and remote sensing, tsunami were assessed by looking at events after they happened or historical data. Usually this was linked to measurements of earthquake strength, etc. The following section aims at highlighting significant tsunami features as recorded in the field: close to the source and in the propagation region (offshore), then closer to the shore in shallower waters or inland (nearshore / onshore). Because the focus of this experimental study is the reproduction of long propagating waves, the review of tsunami observations presents exclusively earthquake generated tsunami characteristics and does not address the varied and complex characteristics of tsunami waves resulting from other types of generation mechanisms.

2.5.1 Offshore

Wave parameters

Directivity gives an indication of the distribution of maximum tsunami amplitudes. Directivity is considered to be high when the highest waves are spatially concentrated along distinguishable axes, i.e. when this distribution is narrow. For many tsunami, it was observed that an underwater elongated source such as a tectonic fault results in a high directivity of the tsunami, i.e. maximum wave amplitudes and energy are concentrated along an axis perpendicular to the source. Hwang and Divoky (1970)
modelled the 1964 Hilo tsunami and concluded the peak wave heights occur along a path roughly normal to the source. This observation was confirmed later, by Zahibo et al. (2003), who found the 1867 Virgin Island (Caribbean) tsunami propagated mainly southward and eastward. During the 2004 tsunami also, Kulikov and Mendvedev (2005) located an empty wave zone to the north of the wave affected area, accounting for the high directivity of the tsunami. The numerical modelling of this event by Kowalik et al. (2007) showed two main energy lobes: one towards Sri Lanka, one towards South Africa. This again supports the idea of the high directivity of the tsunami also mentioned by {Kulikov, 2006 56 /id}: during propagation, the energy was concentrated along 2 axes roughly perpendicular to the wave source. Maximum amplitudes of the global tsunami were also concentrated on an axis perpendicular to the source during the Japan 2011 event (Chen Chian et al., 2011). These observations imply that the results given by 2-D modelling of tsunami will capture the main tsunami characteristics.

Each tsunami has its unique characteristics for a given source, depth, and propagation distance. A few representative examples of earthquake tsunami characteristics offshore are given here. From their satellite data analysis, Okal et al. (1999) retrieved for the 1992 Nicaragua tsunami an offshore amplitude of 0.08m in one particular location offshore, relevant wave periods of 850s and 350s (14min and 6min), and a wave speed of 210m/s. Close to the source, Fujii and Satake (2007) found that the amplitudes of the initial 2004 tsunami wave ranged from several tens of centimeters to 2 m. The Port Blair tide gauge data, provided by Singh and Gupta (2008) indicates the tsunami at its source was composed already of multiple periods. This tide gauge was the only one that survived the tsunami in the source region. According to the GEBCO bathymetry
data (BODC, 2008), the depth in the region of the tide gauge would have been between 7m and 26m. However, this location is relatively close to the coast and this database is not accurate enough in the nearshore region, so these numbers are just indicative. The analysis of records near the source allowed {Kulikov, 2006 56 /id} to make other interesting observations. The first wave was maximum near the source area, including the Maldives. The tsunami generated had a horizontal extent of about 1400km, and the duration of the earthquake exceeded 200s. Offshore, Kulikov & Mendvedev (2005) analysed the JASON 1 satellite data (cycle 109, track 129), which crossed the path of the 2004 tsunami during its propagation. The authors found that the main tsunami period was 2400s (40min), the main energy being associated with the 1800s to 3000s periods. The maximum amplitude in the open ocean was 0.8m, it was associated with the second peak ({Kulikov, 2006 56 /id}. The wave speed was approximately 200m/s, and the wavelength was approximately 500km. The initial wave of the Japan tsunami was a two-step wave, rising slowly to 2m initially, then to 3-5m as an impulsive wave (Fujii et al., 2011). Multiple waves can be observed offshore, although many of these may not be from the initial wave generated, but reflections (Chen Chian et al., 2011).

Finally, long propagating wave steepnesses vary from low (in the offshore region) to very high (close to the shore and before breaking). Taking the example of the 2004 tsunami, the source wave height did not exceed one meter for a 500km wavelength (Kulikov & Mendvedev, 2005), yielding a steepness \( H/L \) which has an order of magnitude \(-6\). The Mercator data (Siffer, 2005) indicates the 2004 tsunami nearshore steepness has an order of magnitude \(-3\).

\[ Far \text{ field effects: dispersion, Coriolis force, tides } \]
Kulikov, 2006 showed that high frequency components of the 2004 tsunami that were highly dispersive, as some wave components (10km wavelength) were significantly delayed in comparison with the main wave front components (1000km wavelength). By looking at satellite data, Kulikov & Mendvedev (2005) noticed dispersion had significant effects at distances greater than 100km from the source. This observation was confirmed by Grilli et al. (2007) who estimated the dispersive effects of the tsunami using FUNWAVE for propagation modelling and found that East of the source, dispersive effects are less important; the authors highlight the shorter distances of propagation in this direction did not let dispersive effects significantly express themselves.

The effects of Coriolis is to modify the dispersion relationship $\omega^2 = f^2 + ghk^2$, when $f$ is the Coriolis parameter. This is only important when the wave period, $2\pi/\omega$ is comparable to $2\pi/f$. The results from Kowalik et al. (2005) showed the Coriolis force influences wave amplitudes towards the South. In the case of the 2004 tsunami the amplitudes were modelled to be 1cm higher with the Coriolis force than without. The authors highlight this force plays a minor role in trapping the energy of the waves along ridges.

Astronomic tides have also been shown to have an influence on tsunami wave heights. The theoretical study of Mofjeld et al. (2007) on theoretical Pacific tsunami wave trains shows that the maximum wave heights for small tsunami occur on average near mean higher high water (MHHW). For larger tsunamis, the authors found the value of the maximum tsunami height increases less than the sum of the initial tsunami height and MHHW. Only for tsunami amplitudes much greater than the tidal range (i.e. difference between the high tide and succeeding low tide), it was noted that the maximum tsunami
height approaches the sum of the initial tsunami height and mean sea level. These results show an obvious dependency of tsunami heights on tidal levels, which can be explained by the fact that tides have got a much longer period than tsunami (see again Figure 1.1).

An interesting study by Dao and Tkalich (2007) assessed the influence of these new parameters on the global propagation of tsunami and their characteristics. The authors used the improved dispersive TUNAMI N2 NUS code to simulate several real tsunami (Sumatra 2004, Taiwan 2006, Solomon Island 2007). They carried out a sensitivity study for these parameters, including: astronomic tide, Coriolis force, Earth curvature, bottom friction, dispersion. They showed that not only the tsunami height, but also arrival time could be affected by the astronomical tide, pointing out that the misfit between tsunami computations and real data often attributed to bathymetry inaccuracies could in fact be obtained by neglecting astronomical tides. The bottom friction was shown to be important only in shallow water (in the deep ocean the effect was negligible). The effect of dispersion was shown to be stronger in the direction of tsunami propagation and towards deep water, where the wave speed was the largest, so no clear change in wave height was observed at the east side of the source, confirming observation from Grilli et al. (2007). Effects of the Coriolis force and curvature were smaller compared to others, but were still considered relevant for far field tsunami modelling (these effects are expected to be larger at higher latitudes).

*Propagation of wave energy and reflected waves*
Most modelling efforts take into account wave amplitudes for tsunami propagation calculations and tsunami wave energy is commonly not estimated. However, Kowalik et al. (2007) used an energy flux function to model the propagation of the 2004 tsunami. Their aim was to follow the tsunami signal as it travelled from Indonesia to the Pacific Ocean. They noted high energy tsunami travelled slowly over ocean ridges, whereas low energy tsunami waves travelled faster in deeper regions. These ridges acted as waveguides. A similar observation was made by Hebert et al. (2001) who used a Shallow Water code to model four Pacific tsunami (Kurile in 1994, Chile and Mexico in 1995, and Peru in 1996) to determine dangerous azimuths of tsunami propagation for the Marquesas archipelago. They observed submarine reliefs strongly trap tsunami energy to potentially redirect the maximum water heights toward a new direction.

In their numerical study of the tsunami of Flores Island in 1995, Imamura et al. (1995) found that the tsunami reflected off Flores Island to reach Babi Island and cause severe damage to its southern part. Numerical modelling and observations from the 2004 tsunami also indicate that reflected waves may have a more destructive potential than direct waves. Indeed, the numerical model from Kowalik et al. (2007) showed that reflected energy signals seemed to have sent more energy than the direct signal. An example is given in Figure 2.7, where we can see through a given latitudinal cross section the energy flux was greater for the waves reflected from Sri Lanka than for the primary wave. Moreover, the same figure shows reflections from the Maldives were significant compared to the mean energy flux passing through this section after the passage of the initial waves. Late waves were also the highest for some stations undergoing reflection in the Bay of Bengal according to Rabinovich & Thomson (2007).
Interaction with continental slopes and islands

Over most coasts there is a sharp slope in the ocean bottom where the deep ocean meets the continental shelf. Such a feature can lead to deflection and/or reflection of a long wave. Hunt et al. (2007) believe in this case, tsunami waves are deflected and tend to travel parallel to the slope. The waves reaching Sri Lanka during the 2004 tsunami seem to have been deflected through this process, so the amplitude of the waves reaching the eastern shore was reduced whereas waves were generated on the southern and western side of the island. By contrast the amplitude of the waves reaching Sumatra was much greater, showing there were no significant deflections from the slope of the shelf.

In addition, tsunami interact differently with different sites (either close to the source or far field). Rabinovich & Thomson (2007) observed that for sites near the source in the 2004 tsunami, the first wave tended to be the highest, and for distant sites late waves were the highest. Far field, they noticed an increase in distance from the source lead to an increase in oscillations duration and the occurrence of multiple wave trains. The formation of these far-field tsunami features is due to the global and regional topography. Individual trains of incoming waves can enhance local oscillations. Near field, Rabinovich & Thomson (2007) found the energy distribution seemed to be determined by the source configuration. These observations were confirmed the numerical model by Titov et al. (2005): the first few waves are the highest when they follow a direct route from the source, otherwise scattering, reflections, refraction and local resonant effects may strongly amplify the arriving waves. Especially outside the Indian Ocean, they noticed that maximum tsunami wave heights were not associated with the leading waves. This leads to the conclusion that the source determines the
characteristics of tsunami near field, whereas coastal features and bathymetries are much more influential for distant locations.

Islands can be either a good shelter if they are located in the deep ocean, or a bad shelter if they are located in shallower regions where multiple wave amplification processes occur. The Maldives, due to their “pillar” structure did not witness significant wave amplification during the 2004 tsunami (Synolakis & Bernard, 2006). This observation is confirmed by Rabinovich & Thomson (2007), who worked with 45 Indian Ocean tide gauge records, plus 4 in South Africa (Atlantic) and found that isolated open ocean islands are less affected by topographic effects and have a lower background noise than continental/coastal areas (in contrast with continental areas which are affected by resonant characteristics of the shelf and coastline). However one study by Briggs et al. (1995) showed significant local amplification effects around islands. After the 1992 Flores tsunami and the Babi island (located off the coast of Indonesia) disaster, the physical experiments described in section 2.2.4, and solitary waves were shown to split, then move with crests perpendicular to the shoreline as edge waves and finally collide behind it (constructive interference), so the lee side of the island was subjected to very high runups. It is important to notice that Babi island is located over the continental shelf very close to the coastal regions of Indonesia. This setup is very different from the Maldives, and wave amplification and coastal effects would have been definitely more significant than for small amplitude waves in the open ocean.

2.5.2 Nearshore and Onshore
Depressed (or draw down) wave

One key characteristic of tsunami, which has never been reproduced experimentally, is the negative first arrival often reported, prior to inundation by the main wave. In many occasions, eye witness accounts of historic tsunami refer to the sea withdrawal. Such accounts have been used for tsunami reconstitution and analysis, for example the 1956 Greek Archipelago tsunami (Ambraseys, 1960), the 1755 Lisbon tsunami (Baptista et al., 1998), the 1994 Skagway tsunami (Campbell, 1995). More evidence of this phenomenon was given by photos and videos of the 2004 Indian Ocean tsunami (Figure 2.8), as well as tide gauge records (Figure 2.9). According to Fujii & Satake (2007) initial motion of tsunami was downward at stations east of the source (e.g. Tapaonoi or Tarutao) and upward at stations west of the source (e.g., Colombo or Male). However, detailed interpretation of tide gauge data without further processing is difficult. Indeed, tide gauges are usually located in sheltered locations where other wave transformation effects would be predominant (e.g. resonance). Also, the sampling period of these instruments is adapted to tides but not so much to the shorter periods of tsunami waves.

In this context, one particularly interesting record of a tsunami is the “Mercator” trace, displaying a representative trough-led tsunami wave. The echosounder of the Belgian yacht “Mercator” (Figure 2.10) provided a reliable trace of the 2004 tsunami. The ship was located a few kilometres off the coast of South Phuket, and anchored over a depth of approximately 14m. The sampling period of the instrument was 1 minute (an appropriate sampling period for a wave which has a mean period of 40 minutes), and the boat was located in an open area and relatively close to the source. So the wave signal was probably less affected by the coastal configuration that could affect its shape and direction of travel, although waves likeley to be reflections are present on the
record. Moreover, dispersion on this side of the fault was shown earlier to be insignificant.

*Aggravation of inundation*

Tsunami impact is directly linked to the penetration of the waves along the affected coasts and during surveys. This is commonly assessed using inundation, flow depth, and runup. Flow depth is a local measure of the water height above ground, whereas inundation is the distance on the ground between the shoreline and the maximum penetration point of the tsunami. As seen in section 2.2.4, runup is a major indicator of wave impact as it gives an indication of the maximum elevation reached in a region of interest. One problem is runup which is more or less accurately measured in the field and up until 1980s, the definition of runup was interchangeable with wave height (Synolakis & Bernard, 2006), particularly in numerical studies which cannot estimate onshore flow (section 2.3) Therefore, the consistency of a lot of runup data has to be questioned before interpretation. As an example, Titov et al. (2005) noticed that gauge wave heights were not necessarily correlated with runup heights in the vicinity of the wave gauge. The most likely explanation for the authors was the poor reliability of the runup values collected.

Although high runups are usually considered to go together with large incoming wave heights, other explanations have been proposed to justify very high tsunami runups at some locations. After surveying the Andaman seacoast of Thailand, Siripong (2006) suggested the formation of Mach Stem waves could explain very high runups of the 2004 tsunami overtopping cliffs more than 30m high in some places. Mach Stem waves
are the merging of an incoming fast moving wave with the reflected wave in one single wave, and as the wave front moves forward, the height increases steadily (Griffith and Rossenfeld, 2008). Moreover, reflected waves usually move faster than direct waves, as they travel through a medium already travelling at high speed due to the passage of the incident wave. Hence waves undergoing multiple reflections are more likely to form large and fast waves, carrying more total energy than direct waves. Siripong (2006) also highlighted sheltered locations on the lee side of islands were particularly vulnerable to tsunami runup. This observation confirmed the results from Briggs et al. (1995) presented earlier. The observations of section 2.5.1 suggest reflected waves are energetic, so they could have contributed to sending additional energy to coastlines (for examples the multiple reflections in the Bay of Bengal), as shown by the energy flux study of Kowalik et al. (2007). It is important to notice videos from the 2004 tsunami showed a lot of the tsunami waves did not break, and a numerical study of solitary waves runup and rundown on a sloping beach with a 2D RANS model by Lin et al. (1999) showed that non breaking solitary waves undergo continuous reflection during runup. For breaking waves the reflection during runup was insignificant until it reached the maximum runup point. There seemed to be less energy dissipation from turbulence and viscous effects in that case, so it would explain why high reflection from mostly non breaking waves could contribute to important runup and damage in adjacent locations. Finally, local resonance can dramatically enhance overall tsunami inundation by generating additional waves and lengthening the duration of the event. This often happens in harbours (Vela et al., 2010), but also wide continental shelves (Yamazaki and Cheung, 2011).
Natural or man-made barriers to the wave advancement inland can be effective in reducing wave impact. A general set of observations made by Synolakis & Bernard (2006) shows that coastal dunes limit the amount of tsunami penetration. Here are some examples of places spared by the tsunami thanks to shoreline protection: during the 2004 tsunami in Patong beach, a small seawall separating the beach from the road reduced impact velocities (Darlymple & Kriebel, 2005), and in South East India, coastal communities were protected by mangroves (Danielsen, 2005). However, for a large event in the near-field such as the 2011 Japan tsunami, many coastal defences got overtopped or failed, and vegetation was swept away (Chen Chian et al., 2011). In such cases, the resulting debris carried by the flow enhance the risk of damage to structures.

In addition, openings in protecting structures can enhance the destructive potential of the wave: during the Nicaragua Tsunami of 1992, the reef facing El Transito had an opening to facilitate navigation, and the land behind was devastated. The adjacent place was spared. The same observation was made by Fernando et al. (2005) in Sri Lanka, impacted by the 2004 Boxing Day tsunami: coral mining had occurred due to touristic development in the fronting area where a train was derailed by the wave.

In terms of damage to structures, in numerous locations churches and temples were left standing. Whereas the affected populations explained this as a divine intervention, Synolakis & Bernard (2006) highlighted that carefully constructed and engineered buildings were seen to resist better than poorly constructed structures. Also, the presence of multiple circular columns on the ground allowed the tsunami to flow through it with minor impact on the overlying structure. The distance of the structure from the shoreline is also a crucial factor. Although damage as a function of distance has never been assessed from field data, a relevant numerical study from Xiao and
Huang (2007) should be mentioned. They used a RANS model to study wave runup and forces on a house located at different elevations on a plane beach with constant slope. The results showed the wave force and interaction duration were maximum when the house was initially partially submerged; and with every increase of the elevation of the location of the house along the slope, the maximum force was considerably reduced compared to the previous location. Another useful element was brought by Carrier et al. (2003), if we consider the vulnerability of a structure/object according to its location. The authors analysed several wave shapes and found that the maximum momentum flux \( f = hu^2 \), with \( h \) being the inundation depth and \( u \) being the horizontal flow velocity) always occurred in the vicinity of the drawn down location, hence the vulnerability of offshore structures located close to the shore would be more important. Directions of the maximum flow velocity and the maximum momentum flux were dependant on the initial waveform: in the case of a depression, they were directed inshore, in the case of an elevation, they were directed offshore.

**Velocities, Accelerations, pressures on structures**

Flow velocities inland are interesting parameters as they are essential to the determination of the force applied to structures. However, contrary to flow depths, inundation, and runup that can be estimated using, for example, water marks and displaced objects, forces and velocities are difficult to retrieve a posteriori. Information on different tsunami wave velocities at the coast is seldom. Some examples of tsunami velocities onshore are available for recent events thanks to video footage and/or post-tsunami surveys: 6-8m/s in Khao Lak, 3-4m/s in Kamala Beach (2004 tsunami, Pomonis et al., 2006), a maximum wave velocity approaching the shore estimated at
9.72 m/s in Aceh (Synolakis & Bernard, 2006), overland flow speed of 2.5 m/s at Pago Pago (Samoa tsunami 2009, Fritz et al., 2011), and an unbroken wave reaching Kamashi at the speed of 2.8 m/s to 8.3 m/s (Japan tsunami 2011, Chen Chian et al., 2011). Regarding forces, only rough estimates of certain components can be made, if video footage is available (to measure velocities); as shown in the 2004 tsunami field report from Pomonis et al. (2006). As an example, wave pressures on buildings were estimated after the post-2004 tsunami survey, notably in Thailand (one of the most devastated countries): maximums in total lateral pressures were 128 kPa to 144 kPa for Khao Lak and 49.3 kPa to 52.8 kPa for Kamala Beach.

Tsunami have also been reported to accelerate at the moving shoreline, and the observation is consistent with numerical and experimental results. Videos of the 2004 Sumatra tsunami show the wave, as it hits the shoreline, slowed down considerably then accelerated again until it reached the ultimate inundation point, and a video from Aceh showed the wave front approached the shoreline at a speed less than 2.2 m/s then accelerated to 9.7 m/s (Synolakis & Bernard, 2006). Similarly, when comparing numerically the behaviour of different types of theoretical waves, Carrier et al. (2003) found that the maximum flow velocity occurred at the moving shoreline for several waveforms. In more detail, for predominantly positive waveforms the maximum shoreline speed occurred during the drawn down process and for predominantly negative waveforms it occurred during runup. The authors compared results from different types of initial waveforms: positive and negative Gaussian shapes, and leading N-waves for two different types of source (fault dislocation and submarine landslide). The results for the corresponding flow velocities are presented in Figure 2.11. The observations from Tadepalli & Synolakis (1994) could be related to this result, indeed
they found that LDN waves always runup higher than solitary waves and LEN waves: in other words, leading negative waves would climb the beach at higher speed than leading positive waves, hence generating the greatest runup. Finally Synolakis & Bernard (2006) pointed out an experimental result that was ignored for years before the 2004 Tsunami: while generating solitary waves climbing up a sloping beach, Synolakis (1987) noticed the wave front accelerated as it crosses the initial shoreline (Figure 2.12). These observations will be discussed further in section 5.3.1.

In the absence of field data, tsunami velocities inland can usually be determined by numerical models with a very fine grid size (less than 10m) (FEMA, 2008). However, such detailed topography data is often not available. It is a fact that velocity data for the onshore tsunami flow is lacking, so phenomena like the one described above cannot be investigated, and impact forces are almost impossible to predict.

2.6 Conclusions

2.6.1 Critical Analysis

Because oscillatory waves always have equal positive and negative amplitudes and orbital particle velocities, the excess mass carried forward is close to zero. In this study, we are interested in waves that transport a significant amount of mass forward. Therefore, the long propagating waves studied will essentially be long translator waves, such as solitary waves and N-waves. In this work, they will be called long propagating waves as no assumption can be made on their stability. Physically, long propagating waves nearshore have the characteristics of translator waves. They can manifest
themselves as one or several positive waves, waves with a depressed component, or bores: this was the case during the 1946 Aleutian island tsunami (see photograph from IOC/UNESCO, 2005), also in the 2004 Indian Ocean tsunami and the 2009 Samoan tsunami as shown by videos of the waves hitting the coast (Raw Tsunami Videos., 2009; FBI, 2010). Moreover, the runup and destruction involved (the reader can refer to Table 1.1 for examples) do not leave any doubts as to the amount of mass and energy carried forward during the wave propagation. Mathematically, long propagating waves are to be treated as shallow water waves, offshore as well as nearshore. Indeed, their typical wavelengths are so large (e.g. 500km in the 2004 tsunami) that even in the deepest parts of the Earth’s oceans (an average of 4km in the Pacific and Indian Oceans) the relative depth parameter $h/L$ is always smaller than 0.05.

According to the classification of water waves chosen (Figure 2.1) the solitary wave theory is applicable for long period waves in shallow water (like tsunami). A solitary wave or soliton, by definition, is of permanent form, and can interact with other solitons unchanged from the collision (Drazin and Johnson, 1989). However, tsunami should break if they approach a pure solitary form (Sorensen, 1993). Moreover, Madsen et al. (2008) highlight the importance of not systematically assimilating tsunami and solitary waves, as long waves on a geophysical scale generally do not generate solitons in the ocean or on the continental shelf, and bores rarely evolve into solitary waves before reaching the shore. Tadepalli & Synolakis (1996) believe long waves generated close to the coastline (at a distance of one wavelength or less) do not have time to turn into a solitary waveform. Still, solitary waves have been commonly used to model important features of long waves approaching the beach and the shoreline, and have also been used as an input to numerical and physical models of
tsunami (section 2.4). The other form of propagating long wave presented in this review, the N-wave, accounts for the common case where the long wave triggers a receding shoreline prior to the positive wave arrival, as reported in section 2.5.2. However, it has not been as extensively studied as the solitary wave form, especially in the laboratory, where stable N-waves have been virtually impossible to recreate.

The impact of long propagating waves is often assessed using runup. For this reason, researchers have strived to obtain empirical or semi-empirical formulae that help predict the runup of long waves. Validation of runup laws with laboratory data or numerical codes is common. In the field, the Plafker rule can give an order of magnitude of the maximum runup for tsunami in the near-field and simple shoreline configurations. The runup of long propagating waves has been derived empirically mainly as a function of wave amplitude, water depth, and bed slope, whether we consider a propagating bore, a solitary or N-wave shape. There is common agreement that wave amplitude and bed slope have a major influence on runup, with higher slopes triggering a higher runup for a given wave amplitude and higher wave amplitudes triggering higher runups for a given slope. Baldock & Holmes (1999) also took into account the type of energy transfer around the shoreline using a parameter assumed to take a maximum value of 2 if the conversion of kinetic to potential energy is total. For an earthquake-type source where the source parameters are known, runup can also be estimated by taking into account not only wave heights and water depths, but also the length of the tsunami source (Briggs et al., 1995) or the height of seafloor displacement (Plafker rule, according to Synolakis & Bernard, 2006). Wave steepness was shown not to influence runup (Borthwick et al., 2006; Madsen and Fuhrman, 2008). One study by Li & Raichlen (2003) gave the runup as a function of the energy dissipated by the wave
during breaking. However, breaking processes are complex and the dissipated energy will vary with bed slope and wave profiles. The influence of wavelength on runup has only been studied for periodic waves. Therefore, it would be useful to know the contribution of wave parameters implicating the total volume / area of water carried forward initially, such as wavelength and potential energy of the incoming wave, to the wave runup.

In studies aimed at understanding the behaviour of tsunami, shallow water models are seen not to perform well in the nearshore region due to the excessive simplification of the physical processes at play or the low resolution of bathymetric and topographic data (section 2.3). Shallow water models give satisfactory results in terms of offshore wave profiles (for waves longer than 600s) and runup (for waves longer than 900s). These models can neither reproduce shoaling and breaking processes, nor flow velocities, which are important in the nearshore region. Runup can only be reproduced correctly for waves longer than the propagation domain, with a sufficient bathymetry resolution (both field bathymetry and numerical grid size). Flow velocities can only be estimated by more complex models. Unfortunately, numerical codes standards do not exist yet for tsunami modelling, and some tools still perform better than others. Guidelines for benchmarking tsunami numerical models (validation and verification against analytical, laboratory and field data) have been presented by Synolakis et al. (2008), to ensure the model not only solves accurately the equations of motion, but also represents geophysical reality. However, no set of laboratory benchmark data takes into account long wavelengths or depressed wave components yet. Therefore, there is a need for a different type of experimental data: not only to investigate such waves characteristics
and impact nearshore and onshore, but also to provide new data for numerical models validation.

In the laboratory, it has been shown the representative wave periods generated to study a propagating wave running up a beach vary between 1.5s and 10s. These periods, at scale, would be too small to be representative of a long wave. Taking the example of a typical test from Goring (1978): a solitary wave of amplitude \( a = 0.018 \text{m} \) generated over a depth \( h = 0.1 \text{m} \), and using equation (2.12) to find the characteristic length of this solitary wave, we obtain \( h/L = 0.34 \), which places the wave in the intermediate regime (Table 2.1), not in the shallow water (long wave) regime. Similarly, a wave with a 10s period generated over a depth of 4.6m in the Long Wave Flume described in Cox (2007b) would have an amplitude \( a = 0.51 \), therefore a characteristic length \( L = 16.8 \text{m} \) so \( h/L = 0.27 \) (intermediate depth domain). Synolakis (1986) wave profiles match the profile of a first order solitary wave (2.10), so the wavelength of the experimental solitary waves as listed in Synolakis (1987) can be calculated using (2.12): for all waves, \( 0.061 < h/L < 0.54 \), which also places them in the intermediate water domain. In many cases, experimental solitary waves have not been proven to match theoretical solitary wave profiles, therefore it would be necessary to re-examine the above examples of actual wave domains in the light of the experimental wavelengths or phase speed measured. The main issue here is the paddle stroke, which in all cases significantly limited the total displacement of water (therefore not only wavelengths, but also wave amplitudes). This is a significant limitation when it comes to modelling long waves such as tsunami, which displace large volumes of water. Moreover, the main wave parameters investigated in laboratory experiments are amplitudes. Phase celerity, period, potential and kinetic energy, have not been studied experimentally. Maximum
wave energy has often been assumed to be associated with the high amplitude waves and damage, so it is an important parameter to study. Finally, most studies focus on the behaviour of non-breaking waves, when little results are available for breaking waves. Flow velocities and forces on structures have not been extensively studied. In all cases, when reproducing long propagating waves in the laboratory, the model scale has to be large enough for the correct processes to be modelled (i.e. the wave motion is dominated by gravity forces), which requires the use of a large scale experimental setup.

The field characteristics of tsunami give an indication of which factors play an important role on wave characteristics, such as amplitudes, at the coast: source configuration, transoceanic ridges, dispersion, Coriolis forces, astronomic tides, coastal configuration, and interactions with other waves. However, as seen in section 2.2.2, according to the theory shallow water wave speed is independent of wavelength, therefore long propagating waves should not undergo frequency dispersion. The Port Blair tide gauge data and the observations of Okal et al. (2007), who showed through analysis of hydrophone recordings high frequency components of the 2004 tsunami were present offshore, indicating a range of frequencies was present, and not only a single long wavelength (thus explaining the dispersive behaviour of the wave). Dispersive waves were also present during the 1992 Nicaragua tsunami, as shown by the satellite altimetry study of Okal et al. (1999): higher wave numbers were sampled closer to the epicenter, while lower wave numbers were sampled further along the satellite track. Okal et al. (2007) attribute to higher frequency waves a seismic excitation origin, therefore they are a result of the generation process. This indicates that the common representation of a geophysical long wave as a single wave is
inaccurate. When focusing on the main wave component, dispersion only affects the wave far-field (if the propagation distance is greater than one wavelength). Other effects, such as Coriolis forces and tides, only affect wave amplitudes at planetary scale. The source configuration affects the directions taken by the maximum wave amplitudes also at a planetary scale (the largest waves are typically concentrated along a propagation axis roughly perpendicular to the source) but determines the characteristics of the main wave in the near-field. Ocean ridges can redirect the most energetic waves during transoceanic propagation. In a 3D environment, coastal configurations can substantially modify the wave characteristics and subsequent impact with processes occurring such as deflection, reflection, breaking, resonance. Amongst all these processes, reflection seems to noticeably enhance wave impact. Onshore, it has been found the impact of tsunami is also determined by human modifications (eg. openings in natural barriers), poor land use, poorly constructed structures. Pillar structures have been shown to be rather tsunami-proof, either at a small scale (churches and temples) or at a large scale (open ocean islands), as the resistance of the flow against such an object is reduced. Wave forces increase the closer structures and buildings are to the shoreline or the maximum draw-down point, and their vulnerability will be greatly increased if they are poorly engineered. Moreover, the tsunami wave/flow speed increases at the shoreline. This can greatly enhance human vulnerability as when the tsunami enters shallow waters it slows down, creating a false sense of security among the people seeing the wave approaching. The lack of flow velocity data hinders the understanding of this process as well as the estimation of forces on structures. Guidelines have been recently determined for the design of tsunami resilient structures (FEMA, 2008), using mainly empirical models with very conservative assumptions for a general worst case scenario tsunami wave. But the
availability of more experimental data (velocities and pressures) would be essential for a better understanding of tsunami impact onshore.

2.6.2 Scope of the study

This review highlighted major gaps in our current understanding of long propagating waves, particularly for the nearshore wave and onshore flow.

- The various tools available to study these processes are numerical models, full scale observations, and laboratory experiments. Classic long wave models can neither reproduce the complex nearshore processes well (because existing codes break down in the nearshore region), nor the wave loads on buildings. Due to the geophysical scale of the waves studied, full scale experiments are not feasible, and existing field data can only give case specific results. Therefore, novel laboratory experiments need to be designed to study long and leading depressed waves;
- Therefore, physical modelling experiments are the method chosen to investigate these processes;
- Physical modelling of long propagating waves to date is limited to relatively short waves. One aim of this study is to reproduce longer wave periods and wavelengths;
- Experimental studies of wave runup are limited to solitary waves or bores, and a limited number of associated parameters. The next objective of this study is to
understand the effects of wave form (including depressed components) and wave energy on runup;

• Wave velocities and pressures on buildings have not been investigated for long and depressed waves. A final aim of this study is to obtain preliminary results on the flow velocity during runup and the evolution of pressures on model buildings for different types of waves.
<table>
<thead>
<tr>
<th>Water Domain*</th>
<th>Deep</th>
<th>Deep / Intermediate</th>
<th>Shallow</th>
<th>Shallow</th>
<th>Shallow</th>
</tr>
</thead>
<tbody>
<tr>
<td>Type of Wave</td>
<td>Oscillatory</td>
<td>Oscillatory</td>
<td>Oscillatory</td>
<td>Oscillatory</td>
<td>Translator</td>
</tr>
<tr>
<td>Steepness**</td>
<td>Low</td>
<td>High</td>
<td>Low</td>
<td>High</td>
<td>High</td>
</tr>
<tr>
<td>Type of solution</td>
<td>Exact</td>
<td>Numerical</td>
<td>Exact</td>
<td>Numerical</td>
<td>Exact</td>
</tr>
</tbody>
</table>

Table 2.1: Main wave theories, domain of applicability, physical nature of the waves and type of mathematical solution, from Le Mehaute (1976b). *Deep: $h/L<1/2$, Intermediate: $1/20<h/L<1/2$, Shallow: $h/L<1/20$. **There are no quantitative limits between “low” and “high” steepnesses, however it is commonly accepted that linear waves ($H<<L$) have a low steepness and other waves ($H>L$, $H>>L$) have high steepnesses.
<table>
<thead>
<tr>
<th>Type of wave</th>
<th>Theory Name (Year)</th>
<th>Runup R</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Solitary</td>
<td>Hall &amp; Watts (1953)</td>
<td>( \frac{R}{h} = \alpha(\beta)\left(\frac{H}{h}\right)^{\gamma} ) (2.23)</td>
<td>Synolakis &amp; Bernard (2006)</td>
</tr>
<tr>
<td>Field tsunami</td>
<td>Plafker Rule (1964)</td>
<td>( R \leq 2S_d ) (2.22)</td>
<td>Synolakis &amp; Bernard (2006)</td>
</tr>
<tr>
<td>Solitary wave</td>
<td>Runup Law (1987)</td>
<td>( \frac{R}{h} = 2.831 \sqrt{\cot \beta \left(\frac{H}{h}\right)^{5/4}} ) (2.24)</td>
<td>Synolakis (1987)</td>
</tr>
<tr>
<td>Solitary wave</td>
<td>No name (1995)</td>
<td>( \frac{R}{h} = \frac{1}{h} \left(\alpha + bS_D + cS_D^2\right) ) (2.27)</td>
<td>Briggs et al. (1995)</td>
</tr>
<tr>
<td>N-wave</td>
<td>N-wave runup law (1996)</td>
<td>( \frac{R}{h} = 3.3 \epsilon_\theta p_o \frac{1}{Q(L_\gamma)} \frac{R_{sat}}{h} ) (2.25)</td>
<td>Tadepalli &amp; Synolakis (1996)</td>
</tr>
<tr>
<td>Bore</td>
<td>Ballistic Theory (1963, 1999)</td>
<td>( \frac{R}{h} = \frac{U^2}{2gh} = \frac{c^2H_b}{2h} ) (2.26)</td>
<td>Shen &amp; Meyer (1963), Baldock &amp; Holmes (1999)</td>
</tr>
<tr>
<td>Bore</td>
<td>Energy balance model (2003)</td>
<td>( \frac{R}{h} = \left[\frac{1 - \frac{ke}{kl}}{1.5e} \right] \left(\frac{H}{h}\right) ) (2.28)</td>
<td>Li &amp; Raichlen (2003)</td>
</tr>
</tbody>
</table>

Table 2.2: Empirical wave runup relationships. \( R \) is the runup, \( h \) is the water depth, \( H \) is the total wave height, \( \beta \) is the slope angle, \( S_D \) is the length of the wave generating source, \( \epsilon_\theta \) and \( p_o \) are respectively a scaling and a steepness parameter as defined for (2.21), \( R_{sat} \) is the runup of a Boussinesq solitary wave of the same height as the N-wave considered, \( Q \) is a function of \( L \) and \( \gamma \) as defined for (2.21), \( H_b \) is the height of bore collapse, \( C \) is a coefficient describing the efficiency of the conversion of potential energy to kinetic energy.
energy to kinetic energy, and $U_s$ is the speed of the bore at the shoreline (shoreline velocity). $E_i$ is the incident wave energy, $E_B$ is the energy dissipated by breaking, and $\delta$ is a parameter characterizing the shape of the runup tongue ($\delta \approx 0.18$).
<table>
<thead>
<tr>
<th>Reference</th>
<th>Laboratory</th>
<th>Flume / Basin dimensions</th>
<th>Wave Generator</th>
<th>Stroke (or displacement of device)</th>
<th>Slope</th>
<th>a/h, T</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hammack (1972)</td>
<td>Caltech</td>
<td>31.73m x 0.39m x 0.61m</td>
<td>Raise / Drop of end section of flume</td>
<td>0.18m</td>
<td>Flat bed</td>
<td></td>
</tr>
<tr>
<td>Goring (1978)</td>
<td>Caltech</td>
<td>31.73m x 0.39m x 0.61m</td>
<td>Vertical paddle</td>
<td>2.44m</td>
<td>5.95°, 2.97°, 1.98°</td>
<td>a/h=0.1</td>
</tr>
<tr>
<td>Synolakis (1986)</td>
<td>Caltech</td>
<td>31.73m x 0.39m x 0.61m</td>
<td>Vertical paddle</td>
<td>2.44m</td>
<td>1:19.85 (2.88°)</td>
<td>0.03&lt;a/h&lt;0.6</td>
</tr>
<tr>
<td>Briggs et al. (1995)</td>
<td>USAE</td>
<td>27m x 30m x 0.6m</td>
<td>60 vertical paddles</td>
<td>0.305m</td>
<td>1:4</td>
<td>0.05&lt;a/h&lt;0.2</td>
</tr>
<tr>
<td>Bukreev (1999)</td>
<td>Lavrentev Institute of Hydrodynamics</td>
<td>7.3m x 0.2m</td>
<td>Baffle removal, Paddle, submerged</td>
<td>N/A</td>
<td>Flat bed</td>
<td>0.05&lt;a/h&lt;0.2</td>
</tr>
</tbody>
</table>
### Table 2.3: Examples of physical experiments generator, facilities and basic wave results.

<table>
<thead>
<tr>
<th>Reference</th>
<th>Facility Details</th>
<th>Wave Generation Method</th>
<th>Wave Characteristics</th>
</tr>
</thead>
<tbody>
<tr>
<td>Yim et al. (2004)</td>
<td>O.H. Hinsdale Wave Laboratory, Basin: 48.8m x 26.5m x 2m, Flume: 106m x 3.7m x 4.7m</td>
<td>Body moving</td>
<td>Varied</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Basin: 2m Flume: 0.8m semi-stroke</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>a/h(max)=0.8 0.5s&lt;T&lt;10s</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>0.5s&lt;T&lt;10s</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Flume: a/h(max)=0.4 6</td>
</tr>
<tr>
<td>Hunt (2003)</td>
<td>UKCRF Plymouth University, 72m x 27m Paddles (72)</td>
<td></td>
<td>1:20</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>a/h=0.2</td>
</tr>
<tr>
<td>Thusyantha &amp; Madabhushi (2008)</td>
<td>Schofield Centre, 4.5m x 1.5m x 1m 100kg rect. block dropped</td>
<td></td>
<td>N/A</td>
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<td>a/h=0.2 T=1.5s</td>
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Chapter 3. Experimental Design and Methodology

The aim of this chapter is to present the experimental design and methods used for data collection. The purpose of the experiments is stated in section 3.1, and scaling considerations are discussed. In section 3.2, all experimental equipment is presented: this includes the functioning of the pneumatic wave generator, the wave flume, the wave probes, the velocity probes, the pressure transducers, and the design of simplified model buildings. In section 3.3, the design of various wave time series is described, and followed by the presentation of the testing schedule.

3.1 Introduction

3.1.1 Purpose of Experiments

The literature review in Chapter 2 has identified a gap in the understanding of long waves, particularly leading depressed waves, interacting with coastal bathymetry. The loading of such waves on buildings also needs to be better understood.

The aims of the testing are:

- To assess the performance and limitations of a new long wave generator and the associated experimental setup in reproducing long, and leading depressed, waves;

- To measure all wave parameters characterizing the wave form, for two wave types (elevated and N-waves) and a range of wavelengths;
• To obtain measurements of a wave’s potential for inundation (i.e. runup, flow velocities, phase speed);

• To obtain data describing the impact of the wave on a simplified built environment (i.e. wave pressures on buildings, flow velocities around buildings).

Three main types of waves are considered: periodic waves, elevated waves, and leading depression N-waves. Periodic waves are initially generated to test the ability of the new generator to reproduce classic, well-known waves in a controlled manner, for a range of amplitudes and wavelengths. Elevated waves are recreated to assess their similarities and differences with traditional solitary waves, and serve as benchmark data for comparison with the new types of waves (the N-waves). All waves studied in these experiments are shallow water waves (i.e. \( \frac{h}{L} < \frac{1}{20} \)), see Table 3.1 and Table 3.2. However, given the wide range of wave periods to be reproduced, from this point in the thesis the term “short” wave will refer to a wave with a period of less than 10s, and the term “long” wave will refer to a wave with a period longer than 10s (unless otherwise stated in the text). This limit has been chosen to distinguish between wave periods that are comparable to previous studies or current possibilities (see section 2.4) and longer periods.

3.1.2 Scaling of laboratory experiments

This purpose of this section is to ensure the main forces driving the prototype wave are well reproduced in the laboratory environment. Potential scale effects that may affect
experimental results are also presented. Since long waves (the depth to wavelength ratio is smaller than 1/20) are modelled, it is most appropriate to apply Froude scaling. The following section aims at assessing a typical prototype wave is well reproduced in the model using Froude scaling, and gives example model values for wave height, period, flow velocity and pressure to be expected during the experiments. Scale effects which may affects the experimental results are discussed in at the end of this section.

**Scaling for long and depressed waves**

The scale ratio \((N_X)\) is the ratio of a parameter in the model \((X_m)\) to the value of the same parameter in the prototype \((X_p)\):

\[ N_X = \frac{X_m}{X_p} \]  

(3.1)

The Froude number is a measure of the relative influence of inertial and gravity forces in a hydraulic flow. In this study the Froude number is defined as:

\[ Fr = \frac{U}{\sqrt{gh}}. \]  

(3.2)

In equation (3.2), \(U\) represents a characteristic velocity. Undistorted Froude scaling is used, as in most coastal engineering models involving long waves (i.e. the vertical length scale is the same as the horizontal length scale). The Froude criterion requires that the Froude number in the model must be the same as that in the prototype \((Fr_p = Fr_m)\), and is given by:

\[ N_{Fr} = \frac{U}{\sqrt{N_g \cdot N_L}} = 1 \]  

(3.3)
\( L \) is a characteristic length (usually taken as the depth \( h \)), and \( g \) is the acceleration of gravity. This condition must be met by the scale ratios between prototype and model. In an undistorted Froude model, geometric similarity requires that all lengths scales (depths, wave heights, distances, etc.) considered in the model are represented by \( N_L \), as defined by equation (3.1). Taking \( U = c_p s_1 = \sqrt{g(h + a)} \) in equation (3.3), we obtain:

\[
\frac{F_{rm}}{F_{rp}} = \sqrt{h_p(h_m + a_m)}/\sqrt{h_m(h_p + a_p)}
\]  

(3.4)

According to the preliminary study of the long wave generator from Bazin (2008), the maximum positive water level elevation \( a = 0.12 \text{m} \) can be reached using the smallest possible flume water depth \( h = 0.46 \text{m} \). Using relation (3.4) with a 4m high prototype wave (Figure 2.10) approaching the shore (i.e. travelling over a depth of for example, 14m (Figure 2.3- b)) we obtain a scale ratio of 0.99 for the Froude number.

Therefore, Froude scaling is likely to be maintained for this study. The scaling requirements for the parameters of interest have been summarised below, with reference to the similarity principles from Hughes (1993).

Geometric similarity imposes that a typical 5m high prototype tsunami wave approaching the shore would, at model scale \((1/100 < N_L < 1/50; L_m = N_L \cdot L_p)\), have a height of between 0.05m and 0.1m.

Kinematic similarity poses specific requirements for all time and lengths dependant quantities. Time and velocity scales are respectively expressed as:

\[
N_p = N_L^{1/2} \cdot N_\rho^{1/2} \cdot N_\gamma^{-1/2}
\]  

(3.5)
\[ N_F = N_L^{1/2} \cdot N_{p}^{-1/2} \cdot N_{g}^{1/2} \quad (3.6) \]

\[ N_{g}^{1/2} = \sqrt{N_{p} \cdot N_{g}} \] represents the square root of the product of the scales for density \((\rho)\) and gravity \((g)\).

Considering that \(N_p \approx 1\) \((N_p = 0.97)\) and with a gravity scale of unity, gives:

\[ N_F = N_L = N_{L}^{1/2} \quad (3.7) \]

Given the flow velocity values given in section 2.5.2 at model scale, according to Froude scaling we should expect velocities from 0.3m/s to nearly 10m/s.

Dynamic similarity poses specific requirements for all mass, time and length dependant quantities, including pressures. As the aim is to reproduce long wave loads on buildings at scales of 1/100 to 1/50, the values presented in section 2.5.2 can be scaled according to the Froude scaling criteria:

\[ N_p = \frac{N_L}{N_{p}^{1.5} \cdot N_{g}} \quad (3.8) \]

Again, if we consider that \(N_p \approx 1\) and the gravity scale is unity, the pressure scale becomes:

\[ N_p = N_L \quad (3.9) \]

Thus at model scale pressures up to at least 3kPa \((\approx 306 \text{ mmH2O})\) can be expected.

The example values given above give an indication of the order of magnitude of the parameters of interest in this study. Table 3.3 summarises example model values corresponding to a long prototype wave nearshore and subsequent flow onshore for each variable of interest.
If all experimental procedures and assumptions are correct, non reproduction of the right order of magnitude of the model values calculated with Froude scaling assumptions would indicate that additional physics are at play in a small two dimensional flume.

Reflections

One important scale effect of long wave modelling is the reflection process. Long-period waves (compared to short waves) reflect more of their energy when they meet solid bodies or boundaries; and hence, reflections are more apt to be an important aspect in long-wave physical models. The level of reflected wave energy can be predicted using the Iribarren number (or surf similarity parameter) $Ir$ and the reflection coefficient $Cr$. Different forms of empirical equations for $Cr$ have been presented in Allsop and Hettiarachchi (1988).

$$Ir = \frac{\tan \beta}{\sqrt{\frac{H}{\tau}}}.$$  \hspace{1cm} (3.10)

Allsop & Hettiarachchi (1988) used different sets of experimental data to show that the reflection coefficient $Cr$ tends to unity as $Ir$ (3.10) becomes larger. Therefore, for a wave travelling over a constant slope, it is expected that large wavelengths will trigger reflections with energy that may match that of the incoming wave. For geometrically undistorted models, the reflection from steep beaches or structures in a model will sometimes be different from the wave reflection that occurs in the prototype, depending on the specific circumstances (Hughes, 1993). Le Mehaute (1976a) showed that waves reflected from smooth walls tend to be smaller in the model than in the prototype.
because of increased friction in the small scale model (the surface is relatively rougher than in the prototype). The same phenomenon may happen over a smooth slope, such as the one used in these experiments. Reflected waves and their correspondence with geophysical reality will not be specifically investigated in this study.

Scale effects

Scale effects in long-wave hydrodynamic models can arise. These result primarily from the scaling assumption that gravity is the dominant physical force balancing inertial forces. According to Hughes (1993), scaling based on this supposition incorrectly scales the other physical properties of viscosity, elasticity, surface tension, etc., under the assumption that these forces contribute little to the physical processes. Scale effects that potentially exist in the model have to be recognised.

Surface Tension: damping, wave attenuation

Another potential scale effect in long wave models is surface tension, which influences wave speed and can induce wave damping (Le Mehaute, 1976b), principally for periodic waves. This study aims to reproduce principally non-periodic waves, thus wave damping due to surface tension should not be an issue. However, a presentation of the surface tension effect is relevant as it may affect waves in very shallow water (i.e. close to the shoreline).

Hughes (1993) highlights that surface tension effects occur when water depths become very shallow ($h < 0.02m$), especially in undistorted models. At these small depths,
surface tension is significant and the model experiences wave motion damping that does not occur in the prototype. According to Rossetto et al. (2011), when looking at simple flows (as in single wave run-up) flow depths of 2cm or less may be sufficient. Moreover, the authors confirm that the effects of surface tension are generally negligible for model wave heights greater than 2cm or wave periods greater than 0.5s. Focussing on wave breaking, Stive (1985) also pointed out that scale effects are minimal, even nonexistent, if the undistorted model wave height is greater than 10 cm. Table 3.3 shows that wave heights smaller than 10cm can be expected, however the breaking process is not the focus of this study.

Regardless of the considerations above, care should be taken during operation of long-wave models to try and avoid dust contamination of the water surface as this would increase the coefficient of surface tension, thus increasing the surface tension scale effect.

In case wave heights or the water depth do become small enough for surface tension to potentially influence wave speed, Hughes (1993) provides a mean of estimating the relative influence of surface tension and gravity in the linear theory expression for wave celerity:

\[ c_p^2 = (gL/2\pi) \cdot \tanh(2\pi h/L) + (2\pi \sigma/\rho L) \cdot \tanh(2\pi h/L). \] (3.11)

The surface tension coefficient \( \sigma \) for fresh water at a temperature of 8°C (present experimental conditions) has a value of 0.075 N/m (result derived using the experimental values from Markov et al. (2008), Figure 3.1). Therefore, for any given wavelength, it is possible to calculate the ratio of the 2\(^{nd}\) term (surface tension term) to
the first term (gravity term) which would give the percentage contribution of surface tension to the speed term (see section 5.1.1).

**Viscous forces and Reynolds number**

According to Hughes (1993), long-wave hydrodynamic models scaled according to the Froude criterion do not correctly simulate viscous forces because the Reynolds number differs between that of the prototype and the model. Long waves are attenuated by internal friction and by bottom boundary layer friction arising from the water viscosity, particularly when the wave travels over considerable distances. The Reynolds number is a measure of the relative influence of inertial and viscous forces in a hydraulic flow and is crucial in assessing the scale effect due to viscous forces. In this study the Reynolds number is defined as:

\[ Re = \frac{\rho h U}{\mu} , \]  

(3.12)

where \( \rho \) is the density of the liquid, \( h \) is the depth, \( U \) is a characteristic velocity, and \( \mu \) is the dynamic viscosity of the liquid. Similarly to the Froude criterion, the Reynolds criterion requires that the Reynolds number must be the same for both the model and the prototype (\( Re_p = Re_m \)). It is usually intended for use in modelling flows where the viscous forces predominate, which should not be the case here. However, it is necessary to check prior to the experiments that the viscous forces in the model would not be significant compared to the ones in the prototype, otherwise they would constitute a scale effect to be taken into account. The Reynolds criterion is given by:
\[ N_{Re} = \frac{N_{vN_{L}N_{p}}}{N_{\mu}} = 1 \]  

(3.13)

Using \( U = c_{p_{s1}} = \sqrt{g(h + a)} \) as an approximation of the phase speed in (3.12), we obtain:

\[ N_{Re} = \frac{\mu_{p} \rho_{0} h_{m} \sqrt{h_{m} + a_{m}}}{(\mu_{m} \rho_{s} h_{p} \sqrt{h_{p} + a_{p}})} \]  

(3.14)

For seawater (salinity 35g/kg) at 20 °C the dynamic viscosity \( \mu \) is 0.00108 and for freshwater at 8 °C the dynamic viscosity is 1.41 (Lide, 2004). Taking the example values (prototype and model waves) given for the calculation of the Froude criterion above, we obtain \( Re \approx 800 \) for equation (3.12); and the ratio of model Reynolds number to prototype Reynolds number (3.14) is 3.10^{-6}. Therefore, the viscous forces in the experiments are significant, but not representative of prototype viscous forces. This major difference between the laboratory waves and the geophysical waves is expected, at the depth and phase speed of a prototype wave cannot be matched by any practical laboratory setup.

The distance travelled by the present experimental waves is very small in comparison to their wavelengths, so it is expected that wave attenuation / damping will be negligible (this is confirmed by the actual wave measurements, see section 4.1.1). In physical models with noticeable wave attenuation, it is possible to theoretically correct or calibrate the initial wave height to compensate for wave attenuation so that the nearshore wave is correctly scaled with respect to the prototype condition.

**Density effect**
When estimating any parameter dependent on water density it is necessary to account for the density scale effect. Such parameters include wave momentum, potential energy and the different force components. Scale model experiments are usually conducted using fresh water, if the prototype condition is salt water, there is about a 3% difference in density (for Froude modelling it is assumed that this difference is negligible: \( N_p \sim 1 \)) and this affects the value of the prototype wave parameter of interest (Le Mehaute, 1976b).

3.2 Experiment Equipment and Procedures

3.2.1 The Long Wave Generator

A collaboration between University College London (UCL) and Hydraulics Research Wallingford (HRW) was established in 2007 to build a new type of wave generator, capable of generating long waves and draw down waves. The functioning of this device was inspired by the previous HRW tide generator, designed to add tide effects to the usual model waves, which was able to reproduce a prototype tide of 12.5h in 7.5min (Bazin, 2008). The functioning principle of this device was to withdraw a certain volume of water from a basin into a tank, then release it in a controlled manner. This was done using a fan linked to the “tide box” (tank) and a valve controlled by a motor unit to regulate the air pressure inside (Wilkie and Young, 1952). However, the period of a propagating long wave is smaller than that of a tide’s. In addition, the new device aims to generate waves in a flume rather than in a 3D environment: as a consequence, the
new generator has to exchange water more quickly between the tank and the propagation channel, and its overall geometry has to be different.

The general description of the long wave generator and flume setup is schematically presented in Figure 3.2. In this thesis, the tank refers to the wave generator, the flume refers to the propagating channel including the sloping bathymetry and flat land area, and the basin refers to the total wet volume of the system tank – flume. The tank is placed at one end of a 45m long flume, in front of existing paddles, at the other end a bathymetry is built with a sump next to the end wall. This sump prevents reflections from the biggest waves reaching the end of the flume. A pump at the bottom of the sump allows water to be brought back to the flume, keeping volumes constant.

*General Principle of Operation*

The tsunami generator’s mode of operation is represented in Figure 3.3. Two valves (security and control valve) are located at the top of the tank and the pump is pumping air through the tank roof. In order to optimise system speed and limit flow perturbations within the tank (Bazin, 2008), the pump is located in the middle and the two valves at opposite ends of the tank roof (Figure 3.4 – right). The safety valve can only be operated manually and its opening should (ideally) not change between related tests so as to avoid uncontrolled pressure changes. It is there to prevent the water from rising too high and being extracted in the pump, which is not water resistant. However, too conservative an approach to the degree of opening of the safety valve affects the performance of the generator, therefore some changes in the safety valve opening can be necessary in exceptional circumstances. The control valve is operated by a motor
controlled by a computer based system of control and acquisition. The valve position is recorded in motor units, which range from 0 (valve shut) to 300000 (valve opened). When the valve is open, this corresponds to a value of 42 degrees (for an absolute maximum opening of 45 degrees). However, the three missing degrees don’t have much influence on the air flow (Bazin, 2008). The user can operate the control valve in different ways through a user interface previously designed by HRW’s staff in the LabView software. It is possible to enter valve values one by one for a real time control of the valve (“Control From Dial”), so as to generate a periodic rise and fall of water in the tank of a given amplitude with a periodic opening and shutting of the valve (“Control form Sine”), or to enter valve time series corresponding to a specific wave (“Control From Time Series”).

Design Characteristics

Desk calculations (Bazin 2008; Robinson, 2008) were performed by HRW prior to the experiments to find the volumes, speeds and heights of water entering and leaving the tank necessary for an example wave (Figure 2.10) to be generated, considering the largest undistorted scale of operation of 1:50. These calculations are summarised in Rossetto et al. (2011), who demonstrated that significant oscillations of the water surface would not occur within the tank at any point during, or at the end of wave generation; they also calculated the necessary tank size as well as the pump and valve requirements. During construction, the panels were braced to avoid breakage or distortion of the tank during operation due to the pressure differences that build up between the inside and outside of the tank. Baffles were also placed within the long
wave generator to dampen oscillations within the tank without significantly reducing the peak flow rates (Figure 3.4 - left).

The tank is 1.15m wide, 1.8m high, and 4.8m long: a picture and schematic representation of the tank and its components at one end of the flume is illustrated in Figure 3.5.

### 3.2.2 The Wave Flume and set up

The pneumatic wave generator and flume are located in the hydraulic modelling building of HRW. The concrete flume is 45m long and 1.2m wide, and the concrete immovable bathymetry is composed of an even bottom followed by a constant 1/20 slope and flat land area. Due to the tank dimensions and the presence of wave paddles in the flume, the effective length along which the wave propagates is not greater than 28m. The effective propagation length also varies according to the bathymetry. A removable slope extension is used for certain tests (runup) resulting in the absence of flat land area for these specific tests. Within the flume walls, there are two glass windows (both 3.3m wide and 1.7m high): the edge of the first is located 25cm away from the mouth of the tank whilst the second window is situated at the end of the flume (close to the shoreline / dry area with the present bathymetry), 27.7m away from the tank.

The water used in these experiments is fresh water. Its temperature is relatively cold (as most of the testing described in this thesis took place during autumn-winter 2008). During the winter months the lab was slightly heated to ensure the water would not
freeze, which brought the average temperature of the water to 8°C (the temperature of the water not only affects the surface tension coefficient and dynamic viscosity of the water, but also the instruments). It is important to note that the water contains organic and non-organic debris, possibly coming from its source, pipes or the laboratory environment. For the most part, these debris are not directly visible, but may have been in high enough concentration to affect some instruments, including the velocity probes (see section 4.1.2).

3.2.3 Measurement of water elevation

The water elevation in the flume is measured using resistance probes and a probe monitor. A resistance probe is 0.6m long. Resistance probes measure the electrical current flowing in an immersed probe, consisting of a pair of parallel stainless steel wires. Each wire has a diameter of 0.006m and the distance between the centre of each wire is 0.05m. The current flowing between the probe wires is proportional to the depth of immersion and this current is converted into an output voltage proportional to the instantaneous depth of immersion. The output voltage can be calibrated in terms of wave height by varying the depth of immersion of the probe in still water by a known amount and noting the change in output signal level (HR Wallingford, 2008). The sampling frequency used in these experiments is 50Hz, and the accuracy of measurements is +/- 0.5mm. The wave probes have to be recalibrated every time water is added to the model, as this will trigger changes in the conductivity of the water.
The total number of probes available varies during the testing; the maximum number of probes used during a test is 12. Due to the limited number of wave probes for the total flume length, we concentrate successively on the “offshore” area (i.e. constant depth region of the flume) and on the “nearshore” area (i.e. sloping beach), for different sets of experiments. Table 3.4 shows the probe patterns used for the tests and associated results presented in this study. For each set of experiments described later, the associated probe pattern will be referred to using its number so the reader can find the necessary information regarding probe name and location in this table.

3.2.4 Measurement of runup

The measurement of runup and rundown is important for comparing the characteristics of the present waves with existing studies. The ratios of $a/h$ for the runup study range between 0.02 and 0.18, for both elevated and N-waves.

Runup is measured using tape placed horizontally along the flume wall and graduated every centimeter: therefore, an accuracy of about $10^{-2}$m can be expected for runup measurements (without accounting for naked eye accuracies in estimating intervals between objects). The end of the tape marked zero is placed at the toe of the bathymetry, the position of the shoreline $x_s$ for a still water level is measured for each test. The position of the maximum inundation point $x$ along the slope of angle $\beta=1/20$ is recorded for each wave, thus the runup $R$ is determined as follows:

$$R = (x - x_s)/\cot \beta$$

(3.15)
Each wave runup as defined by (3.15) is recorded 4 to 6 times without significant spread of the values: thus each wave runup is taken to be the average of all measurements. Runup and rundown of N-waves are measured in the same way.

### 3.2.5 Measurement of Flow Velocity

To measure velocities in the nearshore/onshore area we need an instrument capable of recording in small depths. PIVs (Particle Image Velocimeters) have been considered, as they give velocity fields through a vertical or horizontal layer of water (according to their placement) with a laser. However, they require clean and transparent side walls all along the area to be studied, which is not the case at HRW. Using Acoustic Doppler Velocimeters (ADV) is another technique for measuring flow velocity, however, the instrument requires a minimum 20cm of water for immersion. Since the main interest is to measure the flow velocities close to the shore, ADVs do not constitute an appropriate option. Instead, current flow meters (velocity probes) are chosen for these experiments as they can record in depths of a few centimeters only.

The STREAMFLO miniature velocity probe system available at HRW is designed to measure velocities from $5.10^{-2}$ m/s to $150.10^{-2}$ m/s for fluids in open channels. In optimum measuring conditions they have an accuracy of +/- 2%.

The measuring head ($2r = 15\text{mm}$ in diameter) consists of a five blade rotor mounted on a hard stainless steel spindle (Figure 3.6). The spindle terminates in fine conical pivots which run in jewel bearings mounted in an open frame. The head is attached to the end of a stainless steel tube containing an insulated gold wire which terminates
1mm away from the rotor and is connected to an electronic measuring unit. When the rotor is fully immersed in a fluid, the passage of the rotor blades past the gold wire tip slightly varies the measurable impedance between the tip and the tube. This variation is used to modulate the carrier signal, and after amplification and filtering a square wave signal is obtained. In a digital indicator (tachometer) the pulses are counted over a known time period to obtain a digital reading (HR Wallingford, 2006). With each velocity probe and with each tachometer, a calibration chart is provided. The probe calibration chart gives the number of counts per second (or frequency) as a function of velocity, and the tachometer calibration chart gives the output voltages as a function of frequency, so the velocities can be obtained from the output voltages. The velocity effectively measured by the probe represents a local average of the flow velocity close to the bottom of the flume:

\[
\mu_u = \frac{1}{\pi r^2} \int u \hat{n} dt. \tag{3.16}
\]

In equation (3.16), \( \hat{n} \) is the normal to the cross sectional area of the head. This approximation is correct as long as the flow is uniform. This is true if we consider that because of the high Reynolds number of the flow (see section 3.1.2), the boundary layer is thin (Schlichting, 1979). If the previous assumption is not true, the instrument may be affected by shear. It may also measure the unsteadiness of the flow.

The velocity probes are used to obtain a velocity profile around the moving shoreline and inland for different types of waves. To do so, four velocity probes are placed at regular intervals along the flume bathymetry both the nearshore and onshore (positions on Figure 3.7). The aim of this set of experiments is to compare the velocity profile for different waveforms, heights, and periods.
3.2.6 Buildings design and instrumentation

The tests described in this section were designed and carried out as part of this work, but only preliminary results are presented in section 5.3.2. Full analysis and results will be given in subsequent work (Lloyd, 2011).

To measure the wave/flow forces on structures, several types of model buildings are designed. Some of these buildings are equipped with force recording devices such as pressure transducers. Building constructions, materials and dimensions are very different across the world: however, here the aim is not to recreate a specific case scenario at model scale, but to obtain general information on long waves interacting with structures of different shapes, sizes and locations. For this purpose, it remains necessary to recreate realistic buildings dimensions, so a sample of different kind of buildings and their specifications from different tsunami-prone countries around the world was taken from the World Housing Encyclopedia (2008). General building dimensions are deduced and scaled to 1:100 to create the two types of model buildings used in the experiment.

Foam buildings

A hollow, cube-shaped foam building 0.15m to-side empty is used to estimate an initial approximate wave force from an elevated wave. To adjust the mass of the cube, the empty space on its middle was filled with a varying number of lead shots. Impermeable foam was chosen so the mass of the cube would be negligible compared to the mass of lead shots. A first experiment consisted of obtaining a rough estimate of the force
profile as the wave travels inland. This was carried out using the largest elevated wave which can possibly be generated (test ID 135). The single weighted foam building is successively placed at different distances from the shoreline. The mass \( m \) of the foam building is constantly adjusted so that when the wave hits it, it just starts to move (i.e. the force of the wave is a little bit larger than the resistance of the building). Having measured the friction coefficient \( \mu \) of the wooden platform beforehand using a force gauge, the force applied to the building of mass \( m \) at each distance can be deduced using the relation:

\[
F_{\text{wave}} = \mu \cdot m \cdot g.
\]

(3.17)

The equation above does not take into account buoyancy, which was estimated for different weights of the foam building beforehand by varying the depth beneath a weighted foam building and observing the onset of floatation. The results of these buoyancy tests are presented in Figure 3.8, and represent the necessary water depth for an object of a given mass to start floating. Data from this graph was collected experimentally by using foam buildings of varying masses and recording the water depth at which they start being lifted through buoyancy alone. If the flow depth on the wooden platforms for a given building mass is in the orange area, buoyancy effects are non-negligible, thus (3.17) can only be used to calculate the wave force outside of this area. This method for assessing the force of the wave is not very accurate (i.e. the estimation of the building motion by eye is relatively subjective), but it allows for an overview of the possible force profile of a tsunami flow inland which can be compared with further tests (see section 5.3.2).

The next set of experiments is designed to collect flow velocity data around regularly shaped and spaced structures in order to examine how this pattern affects flow velocity
values. Several foam buildings are placed in a cluster to demonstrate how the flow velocity changes when travelling around buildings. Six foam buildings are filled with lead shots to make them immovable, these would correspond for example to a set of single family houses in the USA (Arnold, 2002). They are placed inland, on a wooden platform, forming two rows of equally spaced buildings. The four current flow meters measured the flow velocities in front of buildings and in between. Because only a limited number of instruments is available, each required several positions (the different velocity probes setups are illustrated in Figure 3.9, Figure 3.10 and Figure 3.11). The aim of this set of experiments is to compare the flow velocity with buildings to the flow velocity without buildings (previous section), and to determine by how much the flow is slowed down due to the presence of structures. This data was collected as part of the testing but is not analysed here.

Wooden buildings

A set of wooden buildings are equipped with pressure transducers: three on the front face, two on the side and one on the back face. These buildings are empty in the middle to allocate space for the multiple wires. There is a 30x15 building of dimensions 0.15m×0.250m×0.30m (Figure 3.12) and a cube-shaped 30x30 building 0.30m to-side. Despite the fact that a wooden structure has its own resonating frequency to be accounted for when analysing wave force, building these structures with wood was a cost efficient way of ensuring their timely readiness for the experiments. It is believed the structure’s resonance component can be removed during data processing if necessary. The 30x15 building dimensions correspond at a scale of 1/100 to a 8 storey reinforced concrete building (e.g. hotel), according to a US standard (Faison et al.,
2004). The 30x30 building is a variation of the 30x15 structure allowing the capture of wave forces on a greater width and allowing the creation of a different kind of obstacle to the flow. Such dimensions also consider the space needed to contain the instruments and their wires.

All buildings are able to rotate so that the effects of the wave can be compared for a 90 degree angle and a 45 degree angle.

Three wooden platforms (0.6m x 0.7m), with a gap underneath allowing the wires of the instrumented buildings to run underneath and out of the flume to the recording equipment (Figure 3.13), are placed on the flat land area. This equipment allows forces and pressures on buildings to be measured at different distances. The thin building is successively placed at distances d1, d2, and d3 from the toe (same distances as the runup measurements), to record pressures as the structure gets further away from the shoreline (d1= 14.25m; d2=14.95m; d3=15.65m). d1, d2 and d3 are taken at the centre of each wooden plate (see Figure 3.13), so the location of impact on the building is offset by plus or minus half the dimension of the structure tested (plus for pressure on the back of the building, minus on the front). At distance d1, the building can be rotated at 45 degrees to assess how the orientation of the structure affects the impact (Figure 3.14). The building can also be rotated 45 degrees in the other direction but, as its base is not square, it cannot be assumed the patterns of pressures will be similar. The 30x30 building is tested only for d1 at 90 degrees (facing the shoreline), as the number of tests is constrained by time. The role of this building is only to assess how the shape of the building affects the flow and the pressure distributions along the front face, the results will not be presented here but in subsequent work.
**Pressure Transducers**

The TRAFAG pressure transducers available at HRW are able to record pressures in the range of 0 to 160mbar (16kPa), in accordance with the order of magnitude of the wave properties shown in Table 3.3. They can cope with wet-dry conditions, have an accuracy of 0.25% and are flush mounted on the wooden buildings described above.

### 3.3 Design of time series

#### 3.3.1 Periodic Waves

Finite wave trains of sinusoids of different periods and amplitudes are generated. The open loop control allows generation of periodic waves of periods between 50s and 200s, using the “Control from Sine” functionality of the LabView software. The parameters of the periodic waves tested are listed in Table 3.1. It is apparent that the maximum amplitude of 0.045m also corresponds to the maximum wave period of 200s. In some instances, the wavelengths generated in our experiments exceed the flume length, so major reflections would be expected. However, contrary to translator waves, sine waves are essentially orbital and relatively little wave momentum is carried forward.

A wide range of waves are generated with good sinusoidal shapes (Figure 3.15). Tests involving different amplitudes of valve opening have been carried out and we it is apparent that the valve opening does not affect the wave shape. The valve opening only affects the wave amplitude in the flume: if for a given period the valve opening range is
greater, the waves will be higher (Bazin, 2008). On several occasions an interruption in the regularity of the signal is observed, and can be interpreted initially either as a parasitic reflection due to the geometry of the flume, or due to resonance of the basin configuration (Figure 3.16). Waves that are not purely sinusoidal have been flagged ** in Table 3.1. The energy spectra of waves of different periods displaying such irregularities give a similar dominating peak in energy, which corresponds to a period of 26s (0.24Hz) (Figure 3.17). This indicates the signal disruption is likely due to the natural resonance of the system rather than reflections from the bathymetry. In the latter case the spectrum would display a main frequency that would vary with the individual input signal period. In reality the orbital particle trajectories are open and some forward motion may occur. So the presence of reflected waves from the bathymetry may also affect the signal.

Period output is always consistent with the period input from the control valve, and apart from the particular case of the 50s waves which seem to be more sensitive to signal disruptions, amplitudes are also consistent. These initial tests confirm that the generator is capable of generating waves in a controlled manner.

### 3.3.2 Elevated waves

With elevated waves, the reflections problem is expected to be more significant due to the forward motion of positive excess mass. Therefore, it is necessary to work with short elevated waves (4-8s) for the design of the wave form. If an appropriate type of valve time series is found, it is possible to design longer waves, useful for velocity and impact tests, but their shape is not as “clean” as the shorter ones.
According to Bazin (2008), the most relevant way of reproducing the desired profile is to create a valve time series that triggers a constant rise and drop of the pressure in the tank (Figure 3.18 - a). The system which has been designed for long wave generation cannot release water quickly enough to create the steeper theoretical wave profile of a Boussinesq solitary wave (equation (2.10)), but the wave generated (example in Figure 3.19) can potentially be compared to theoretical wave shapes in terms of amplitude or volume.

Elevated waves were reproduced in the flume with amplitude \(a\) to water depth \(h\) ratios ranging from \(a/h = 0.014\) to \(0.26\) (as measured in the constant depth propagation region, close to the tank). This range was mainly limited by the height of the tank, as wave heights for a given period increase with the head difference between the flume water level and the water level in the tank. It is however noted that the tank design can simply be altered in the future if a greater range of \(a/h\) ratios are required. The period of the “short” waves ranged from 4.2s to 9.3s (see Table 3.1), the period of the longest propagating elevated wave generated throughout the testing was 92.2s. The creation of longer elevated waves (up to 107s in period) was attempted, however they would not trigger significant inundation and were therefore impossible to study in terms of impact. This is believed to be due to the amount of reflection triggered by such long waves. The longest wave periods imply much longer wavelengths and therefore lower wave steepness. These waves have much greater surf similarity or breaker parameters \(Ir\) than the tested periodic waves, and it has been shown by Allsop & Hettiarachchi (1988) that for smooth slopes, the largest values of \(Ir\) are associated with reflection coefficients approaching 1.
3.3.3 N-waves

N-waves, especially leading depression N-waves, have not been properly studied in the laboratory due to the difficulties associated with propagating stable trough-led waves. N-waves have however a particular interest here as they can produce steeper positive wave fronts after the initial depression has passed than elevated waves. Due to its novel method of wave generation, the tsunami generator has the ability to produce waves with depressed components. N-waves of different shapes and amplitudes were therefore reproduced in the laboratory (see Table 3.1).

Short N-waves

The design of N-waves starts by generating relatively short waves, for the same reason as for elevated waves. In terms of profile, N-waves can be seen as mirror solitary waves on either side of the mean water level, a negative one followed by a positive one. This is the assumption used to derive the N-waves valve time series, as no relationship has been established between the N-wave equation (2.21) and the valve opening. The rise between the trough and the following peak is steep for short waves: as such it is best to generate this part of the wave (or more) using ramp or square time series for the opening/shutting of the valve (Figure 3.18- b). A combination of the two techniques can also be considered, for example by using a constant drop in pressure in the tank to generate the trough and a quick opening to start the peak, shutting the valve slightly at the end to absorb part of the returning flow thus avoiding excessive flattening of the end of the peak (Figure 3.18- c). The theoretical N-wave profile (2.21) is compared to a
typically generated N-wave (Figure 3.20). This level of agreement is representative of the match obtained for other N-wave profiles.

N-waves with relatively short wave periods (7s to 9.8s, see Table 3.1 and Table 3.2, for typical N-waves displaying $a/h$ ratios between 0.011 and 0.16) have been reproduced first. These short waves are a necessary point of comparison with theoretical N-waves (Tadepalli & Synolakis, 1994; Tadepalli & Synolakis, 1996) and solitary wave (Miles, 1980) theories, as well as acting as a base reference for comparison with longer waves. A range of N-waves with progressively longer periods are also generated with $a/h$ ratios up to 0.21. The longest N-wave reproduced throughout the testing had a period of 161.5s.

Long N-waves

Despite the reflection problem that hinders the proper generation of long elevated waves, they can theoretically be reproduced by stretching the shorter waves’ time series. However, this method is not appropriate to generate long N-waves. Indeed, stretching a valve time series where the trough is defined as a ramp leads to major instabilities in the flume, as the water level can only be dropped rapidly for very short periods of time. If the ramp time is extended, the water level cannot stay at trough amplitude for long as the whole flume level naturally goes back to equilibrium. In other words, as long as the wave is short, only a small volume of water (directly in front the mouth of the tank) enters the tank before more is released; but if a bigger volume of water has to be drawn into the tank, the whole flume water level needs to stabilise. The
height of the outlet also limits the size of the trough: when the water level drops fast, it is more likely to reach the top of the outlet quickly and be unable to drop any further. Therefore, for long N-waves, only the constant drop and rise of pressure in the tank technique is applicable. This technique was used to recreate a long N-wave close to the shore (see Figure 3.18- d); resembling the wave in Figure 2.10. Indeed, the field data effectively matches the profile of a very long N-wave (Figure 3.21).

The long N-wave is initially generated using two methods. The first method consists of a closed loop control system (PID) based on the water level reading within the tank. It creates a very convincing wave profile, but the behaviour of the wave at the shoreline is not realistic, as the flow velocity at the shore is so low that the shoreline is hardly moving (consequently there is no inundation). This phenomenon is likely due to the fact that the PID calculations are based on the water height within the tank, which is constantly adjusting by taking and releasing water, over short time intervals. This means that velocities inside the flow are constantly changing direction and would thus end up cancelling each other. So the water surface matches the input profile but the particles and total flow velocities are not representative of a propagating wave. Therefore, in these experiments this method of long N-wave generation has been rejected in favour of the open loop system.

In open loop, it is necessary to allow the water level time to stabilise as water is being withdrawn within the tank. Therefore a reverse solitary wave valve profile (constant drop of pressure) is created, followed by the valve profile corresponding to an elevated wave for the positive amplitude, stretched over time. The valve stays at 0 and 300000 for longer compared with the previous solitary profiles in order to allow more water to be sucked in/released at the same time as matching the long period of this N-wave. We
notice that when using the whole tank volume to generate it, the wave profile at a scale of 1/50 can be reproduced. The positive amplitude generated at this scale is 0.073m, and the negative amplitude -0.062m, for a period of 114.6s. According to Froude scaling, it is expected that for the same wave at a scale of 1:100 the possible period to generate for the Mercator would be about 81s, with a positive amplitude of 0.036m and a negative amplitude of -0.031m.

Using this method the trough is very well generated, however the peak recorded next to the tank is “interrupted” by a reflected wave (the time of occurrence of the reflected peak at this location is consistent with the travel time of the direct signal, using the expression for the phase speed of a shallow water wave (2.9)). Unfortunately, the present setup negates the ability to generate a better peak close to the tank. The shoaling process and reflections happening over the sloping bathymetry, give a much bigger peak closer to the shore (Figure 3.21). Since this wave, even at model scale 1:50, is very long (almost 300m) compared to the propagation length of the flume (about 30m), it is acceptable to compare model and prototype profiles anywhere along the flume.

It can be seen that the principal wave characteristics of a long and depression led N-wave, especially the long trough preceding the peak, were reproduced well up until the point at which the leading edge wave returned to the generator. A longer flume would allow waves with greater periods to propagate without being affected by reflections in the offshore region. An improved control system might also remove the reflected component and produce waves at a larger scale. These improvements are discussed in section 5.1.4.
3.3.4 Testing schedule

Testing of the wave generator

The first stage of the testing is to assess the ability of the wave generator to reproduce different types of waves and find the limitations in terms of wave periods, amplitudes, repeatability of wave measures.

The first type of wave considered is a periodic wave, as they are simple wave forms so theoretical profiles and laboratory measurements are straightforwardly comparable. The ability of the wave generator to reproduce periodic waves of various characteristics correctly and consistently is a necessary step before any further testing. Testing of the wave generator continues once the other wave shapes of interest (i.e. elevated waves and N-waves) have been designed.

Wave parameters and applications for testing of the generator, runup, velocities and pressures are presented in Table 3.1 and Table 3.2. A summary of the valve time series IDs used for the whole testing is presented in Table 3.5, whilst valve time series for the main waves are presented in Appendix A.

Reproduction of elevated waves and N-waves

The wave generator functions in open loop thus there is no feedback system and the profile of the waves generated depends only on the valve opening and water depth. Apart from periodic waves, during all tests the “Control from Times Series” functionality of LabView is used so that for each depth of operation (CWL – Control
Water Level), every time series of valve opening recreates one particular wave in the flume. Details of the times series design is given in section 3.3. A number of test waves are designed, and a representative subset of each wave category is chosen for different sets of experiments.

Each designed valve time series is given an identification (ID) number corresponding to the original test number. This time series can be reused for running wave tests at different water levels than the one initially used to design the time series: this results in a new different wave. Therefore, the valve / wave ID shall be distinguished from the test number.

Runup and velocity tests

Runup and rundown tests are carried out for elevated waves (runup only) and N-waves. Waves IDs135, 136, 137, 138, 139, 146, 147, 148, 149 are used for the study of runup of elevated waves. Waves IDs 113, 130, 131, 132, 133, 280 and very long N-waves 307 and 314 are used for the study of runup and rundown of N-waves (Table 3.5).

Velocities in the shoreline region are measured, first without obstacles interfering with the flow, then between fixed regular shaped foam buildings inland. A small subset of representative waves of different amplitudes, types and periods is created (core tests): elevated waves: 135, 138; long elevated waves: 415; N-waves: 113; long N-waves: 307.

Pressure tests
For pressures tests, a subset of waves of different heights and shapes is taken. The original subset comprises 14 waves as it includes wave ID 440. 440 is a series of solitary waves (multiple elevated waves) but due to reflection problems and instabilities in the flume after the passage of the first wave, it is not possible to carry on with this test. Consequently, a subset of 13 waves is taken. Three elevated waves are kept (135, 136, 138), showing a range of $a/h$ ratio and being large enough to reach the model buildings when running up. Three longer elevated waves (441, 415, 442) are also used for pressure tests (in spite of their profile being affected by reflections from the slope) because it is still possible to compare their behaviour with shorter elevated waves. The three shorter N-waves most likely to reach far inland (113, 130, 133) are kept. Finally, four long N waves (307, 434, 435, 439) are used for these tests. Only the core tests are taken in cases where, due to time constraints, it is not possible to test the 13 waves. In this thesis, only the results corresponding to wave ID 135 (short elevated) are presented later in section 5.3.2.
<table>
<thead>
<tr>
<th>Test Time (G)</th>
<th>Valve Time (G)</th>
<th>Water Type (G)</th>
<th>Purpose of test</th>
<th>Depth (m)</th>
<th>Period (s)</th>
<th>Wave height (m)</th>
<th>Amplitude (m)</th>
<th>Wave length (m)</th>
<th>Rise (s)</th>
<th>H/L (G2)</th>
<th>Weight loss (g)</th>
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<td>200</td>
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<td>0.045</td>
<td>0.050</td>
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Table 3.1: Table of the initial set of waves tested. (*) The valve time series ID corresponds to the name of the input file to be used at a given depth of operation to
reproduce the same wave. For sine waves, there is an inbult function where only wave period and range of valve motion (in motor units) are necessary for wave generation. (***) These waves resulted in a non-sinusoidal signal. This was likely to be an effect of parasitic reflections, or resonance for a particular period. Calculation of period/wavelength: The wave peaks - for all wave exceeding the effective flume length – are likely to be affected by reflections, especially the second half of the positive amplitude, and represent a composite wave (reflections and direct signal). Therefore, the period has been calculated: for solitary waves, by assuming symmetry and doubling the first half period, for N-waves, by calculating the full trough period and assuming symmetry for the positive part of the wave (resulting in the same calculation as for a solitary wave). (***) The recording of wave 307 is a model wave at an undistorted scale of 1/50 compared to the prototype Mercator (as measured Nearshore).
Table 3.2: Table of other waves tested, for which experimental design, data processing and results are reported in this thesis. The test IDs for part of the testing refer to the date and test number, with the following format: yymmdd_number. When average parameters were estimated for several consecutive tests, the first and last test numbers are displayed as: yymmdd_(number, number). (*) The valve time series ID corresponds to the name of the input file to be used at a given depth of operation to reproduce the same wave. (**) The full tests IDs are, in order: for wave 135, 081028_2, 081028_(10,12), 081028_(19,21), 081028_(28,30), 081028_(37,39); for wave 415: 081028_(25,27), 081028_(34,36), 081028_(43,45); for wave 113, 081031_(25,27), 081101_(13,15), 081101_(19,21), 081103_03; for wave 307, 081031_(22,24), 081031_(28,30), 081101_(04,06), 081101_(07,09), 081101_17, 081101(22,24), 081103_(05,07).

<p>| Test ID (ID) | Valve time series ID (***) | Wave type (W) | Purpose of test | Depth (m) | Period (s) | Wavelength (m) | Amplitude (m) | Wave height (m) | 1st (ID) | 2nd (ID) | 3rd (ID) | 4th (ID) | 5th (ID) | 6th (ID) | 7th (ID) | 8th (ID) | 9th (ID) | 10th (ID) | 11th (ID) | 12th (ID) | 13th (ID) | 14th (ID) | 15th (ID) | 16th (ID) | 17th (ID) | 18th (ID) |
|-------------|-----------------------------|---------------|----------------|-----------|-------------|----------------|---------------|----------------|----------|----------|----------|----------|----------|----------|----------|----------|----------|----------|----------|----------|----------|----------|----------|----------|----------|----------|----------|</p>
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<td>170s</td>
<td>0.85m/s</td>
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*Table 3.3: Example model values for a prototype nearshore tsunami and associated onshore flow.*
Table 3.4: Wave probes positions used in this study. Each pattern corresponds to a set of experiments. The distances have been measured from the mouth of the tank.

<table>
<thead>
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<th>Probes (pattern 1)</th>
<th>Distance from tank (m)</th>
<th>Probes (pattern 9)</th>
<th>Distance from tank (m)</th>
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<td>Offshore 1</td>
<td>1.3</td>
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<td>Nearshore 6</td>
<td>21.58</td>
<td>Offshore 8</td>
<td>11.6</td>
</tr>
<tr>
<td>Nearshore 7</td>
<td>22.58</td>
<td>Offshore 9</td>
<td>12.8</td>
</tr>
<tr>
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<td>23.73</td>
<td>Offshore 10</td>
<td>14</td>
</tr>
<tr>
<td>Probes (pattern 2)</td>
<td>Distance from tank (m)</td>
<td>Toe</td>
<td>15.2</td>
</tr>
<tr>
<td>Offshore 1</td>
<td>1.3</td>
<td>Nearshore</td>
<td>20.7</td>
</tr>
<tr>
<td>Toe</td>
<td>15.2</td>
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</table>
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Chapter 4. Diagnostics/Data Processing

The aim of this chapter is to describe how the data was processed and the diagnostic applied to interpret it. In section 4.1, the calibrations, repeatability and nature of different types of measurements are described. In section 4.2, sources of experimental errors are identified, and signal processing operations are reported. In section 4.3, the parameters derived from wave elevation time series are introduced, including integral measures of the wave. In sections 4.4, 4.5, and 4.6 additional considerations regarding the processing and interpretation of runup, velocity, and pressure data respectively, are presented.

4.1 Measurement Types

4.1.1 Water Elevation and Runup

*Calibration of resistance probes*

The water elevation is proportional to the voltage circulating in the probes. The relationship between the voltages and water elevation is given by (HR Wallingford, 2008b):

\[ \eta(V) = A \cdot (V - B) \]  

(4.1)

\( V \) represents the voltage measured, \( A \) and \( B \) are constant coefficients for one particular wave probe and one calibration file. A new calibration file is created every time water is added to the testing channel (due to the subsequent changes in the conductivity of the
water), and the probes are re-calibrated. The relationship between the recorded voltages and the water surface elevation is linear, therefore any processing can be performed on the converted values.

**Repeatability**

Repeatability for the whole population of experimental waves was assessed by using $N(x)$ samples of elevated and N-waves, of varying size $n$ for each variable $x$ (i.e. maximum amplitude, minimum amplitude, period, runup). The same time series was launched five times in a row or more and the offshore wave profiles were recorded (i.e. measurements made at probe Offshore 1). Elevated waves appear to be repeatable when launched one after the other at similar control depths (Figure 4.1 - a). The repeatability of N-waves has been tested in the same way as for solitary waves, and they appear to be extremely repeatable (Figure 4.1 - b).

The variation between individual measurements in a sample for $x$ is generally measured using the statistical variance $s_x^2$:

$$s_x^2 = \frac{\sum(x_i - \bar{x})^2}{N-1},$$  \hspace{1cm} (4.2)

After verifying the largest standard deviation was not larger than twice the smallest standard deviation for each sample in the pool (for variable $x$), we can calculate the pooled variance $s_{pX}$ and standard deviation for the combined samples (Moore and McCabe, 2003):
Taking $x$ as $a$ for positive amplitude, $a^-$ for negative amplitude, $T$ for period and $R$ for runup, the standard deviations of these different variables are listed in Table 4.1. The standard deviation for each individual wave was calculated over 3 or 4 measurements for elevated waves, over 3 measurements for N-waves, and over 3 to 6 runup measurements for all waves. There were 9 individual elevated waves and 4 N-waves to calculate the pooled standard deviations for amplitudes and period. There were 26 runup standard deviations calculated for elevated waves and 18 for N-waves. These results make no assumption on the variations of wave amplitudes further down the flume (i.e. it is expected that $\sigma_a$ will increase with $x$).

The waves that can be generated with the generator have orders of magnitude ranging from $10^{-2}$m to $10^{-1}$m. The standard deviation for positive and negative amplitudes being one to two orders of magnitude smaller, the repeatability of wave amplitudes for the generated waves is confirmed. Similarly, wave periods from 4s to 200s can be repeatably reproduced in these experiments as the order of magnitude for the standard deviation of wave periods is $10^{-2}$s to $10^{-1}$s. Finally, wave runups have orders of magnitude ranging from $10^{-2}$m to $10^{-1}$m. The associated standard deviation being at least two orders of magnitude smaller, runup measurements are also repeatable. The standard deviations of the samples $s$ are only estimates of total population standard deviation $\sigma$ (i.e. standard deviation for all waves tested).
4.1.2 Velocities

*Particle velocity*

In the case of propagating waves such as elevated waves and N-waves (see section 2.6.1), orbital velocities will be called particle velocities, since the particle motion is always in the direction of wave propagation (Longuet-Higgins, 1974). For long waves, the fluid velocity is approximately horizontal (Grimshaw, 2004). The horizontal component of a fluid flow beneath a periodic wave train is well known (Robinson et al., 2011):

\[
u(x, t) = \sum_{j=1}^{J} \omega_j \frac{\cosh(y+h)k_j}{\sinh(hk_j)} (-a_j \sin(k_j x + \omega_j t) - b_j \cos(k_j x + \omega_j t)), \quad (4.5)
\]

For each wave component \(j\) in the series, \(a_j, b_j\) are the Fourier coefficients, \(k_j\) is the wave number and \(\omega_j\) is the angular frequency. Moreover, for long waves, \(k_j\) is small, so the maximum speed for (4.5) is approximately:

\[
u_0 \approx \frac{\omega_j}{hk_j} \sqrt{a_j^2 + b_j^2}. \quad (4.6)
\]

The total wave amplitude \(a\) is the magnitude of coefficients \(a_n\) and \(b_n\). Setting \(c_{p_0} = \omega_j/k_j\), so (4.6) becomes:

\[
u_0 \approx \frac{a}{h} c_{p_0}. \quad (4.7)
\]

The validity of this approximation will be tested in section 5.3.1, after processing and analysis of velocity measurements.

*Calibration of velocity probes*
The velocity probes effectively measure the horizontal velocity of the flow, \( u = u_0 \). For the tachometer, the relationship between the frequency of counts (in Hz) and the output voltage \( U \) (in volts) is linear, whereas for the velocity probe the relationship between the number of counts per second and the flow velocity \( v \) (in cm.s\(^{-1}\)) is only linear if \( u > 6\text{cm.s}^{-1} \). Therefore, a quadratic relationship between the voltages and the flow velocity exists to take into account the full range of voltages (HR Wallingford & Lloyd, 2008e; HR Wallingford & Lloyd, 2008d; HR Wallingford & Lloyd, 2008c; HR Wallingford & Lloyd, 2008b; HR Wallingford & Lloyd, 2008a), and is given by:

\[
u(V) = (AV + B)^c + DV \tag{4.8}\]

Each velocity probe is associated with one particular tachometer for the whole testing process. Despite the non-linearity observed for small velocity values, the processing can be performed on the actual velocities as values smaller than 0.06m/s typically correspond to the magnitude of instrumental noise.

**Repeatability of velocity measurements**

The repeatability of the phase speed (speed at which the wave travels) was assessed by looking at three or four values of experimental phase speed \( c_{p_{\text{exp}}} \) measured for the same wave. \( c_{p_{\text{exp}}} \) was assessed by recording the time between two recorded wave peaks at known probes locations. The repeatability of flow velocities with reliable current flow meter readings is assessed in the same way, this time with three to four consecutive tests (Figure 4.2). The repeatability of the patterns seems satisfactory, but
the peak velocities \( u_{max} \) appear slightly less reliable, this was due to the clogging triggered by water debris (section 3.2.2).

When standard deviation is assessed for a single sample of size \( n \), the classic definition of standard deviation is used:

\[
 s_x = \sqrt{\frac{\sum(x_i - \bar{x})^2}{n-1}}. \tag{4.9}
\]

Here, the variable \( x \) is successively taken as being the peak particle velocity \( u_{max} \) and the experimental phase speed \( c_{p\text{exp}} \). Table 4.2 lists the typical standard deviations obtained for different types of nearshore flows (short waves, long elevated waves, long N-waves). It appears that current flow meters make more accurate measurements of the peak flow velocities triggered by the longer waves (with the exception of P1).

The minimum velocity that can be recorded is 0.05m/s, and maximum velocities up to 1.1m/s have been recorded. Orders of magnitude of standard deviations for all velocity probes range from \( 10^{-1} \)cm/s to \( 10^{0} \)cm/s. Therefore, for all current flow meters, recorded velocities below \( 10^{-1} \)cm/s cannot be considered repeatable. Higher velocities (1 cm/s to 100 cm/s) are repeatable only with instruments P2 and P3, and only for long waves such as generated by times series ID 415 and 307. Probe P4 was changed after test 081113_10 because of malfunction, this did not affect the repeatability tests. Peak velocities approaching 10 cm/s can be considered repeatable with all instruments and for all waves.

Limitations of velocity probes
The velocity probes’ rotor can spin in both directions, therefore recording forward and reverse flow. However, the tachometer does not return negative values of velocities. The measurements return the velocity magnitude, effectively $|u|$. When the flow changes direction, the velocity returns to zero (example in Figure 4.3: this type of recording is characteristic of the wet region upstream the shoreline, where backswash always follow the forward flow).

Further inland we only record the velocity time histories of one velocity peak (forward flow). Indeed, the sump located at the end of the flume absorbs the initial wave.

Generally, the repeatability of the measurements made using the current flow meters is found to be unsatisfactory, as it is not consistent and highly dependent on the values of the velocity measured and the type of wave tested. For the conditions that allow repeatability to be satisfactory as highlighted above, velocities remain reliable as long as the current flow meters do not get clogged by floating debris. Unfortunately, only advanced stages of fouling are noticeable so some velocity results may be difficult to interpret.

### 4.1.3 Wave Pressure

*Calibration of pressure transducers*

All pressure transducers display a perfectly linear relationship between voltages and pressures (in mm of water), with zero volts corresponding to a dry transducer (HR Wallingford, 2008a). The relationship between the voltages $U$ and the pressures $P$ is given by:
\[ P(V) = A.V. \]  \hspace{1cm} (4.10)

**Repeatability**

The repeatability of pressures with transducers has been assessed with data from 4 consecutive tests with the same wave for one short wave, and 3 different instruments for the long wave. The respective definitions for standard deviation (4.9) and (4.4) were used to calculate the results in Table 4.3. Only two typical examples of elevated waves are treated here, and due to time constraints N-wave pressure data is not treated in this thesis. Pressure time histories are very repeatable, for long and short waves (Figure 4.4). Peak pressures are repeatable for long waves but not for short waves. This is expected, as impact pressures occur over a time scale smaller than the sampling frequency of the instruments.

**4.2 Filtering Operations and signal corrections**

**4.2.1 Sources of experimental error**

*Experimental signal errors*

Random errors are variations in the individual values for a repeatedly measured quantity. These can arise from the measuring apparatus itself, as fluctuations due to changes in temperature, battery charge, or electrical interactions with other sources. Observable repeatability of all measured quantities has been assessed (see section 4.1), however it is necessary to quantify the magnitude of such errors to assess measurement
accuracy. An estimation of standard errors for the main quantities of interest is made in section 4.3.4. Random errors such as electronic noise can be removed through filtering. The time difference between the time stamp of the two boxes differed by a short period for the second part of the experiments. The error in synchronization of the boards for probes pattern 1 is identified and corrected. Errors in nearshore wave heights have also been identified, these are likely calibration errors, which were inconsistent. As it was not possible to repeat the experiments or to correct the calibration file directly, the wave profiles have been corrected by using the linear relationship between the nearshore shoaled amplitudes. All shoaling waves have been shifted by the difference between expected and actual wave heights defined by this linear relationship and the non-attenuated wave amplitude at the toe of the beach. Values of the wave profile between the start of the shifted profile and the zero value (still water surface) have been calculated using a cubic spline interpolation. Due to beach reflections potentially affecting the second half of the profile, the latter was reconstructed by symmetry with the first half (for elevated waves).

Temperature changes can also affect the resistance probes and result in a drift in voltages; however, this issue can be ignored here as the frequent calibration of the probes due to the varying control water levels used in this study automatically avoids this error.

Finally, when the stainless steel probes are in contact with water, capillary action may occur. A combination of surface tension and adhesion forces (i.e. molecular attraction between different bodies) draw the interface down or up by a certain distance as the water level respectively rises or drops. Capillary action can distort the recorded voltage at small water depths, where surface tension effects are likely to be more significant.
(i.e. close to the shoreline, see section 3.1.2). The capillary length $l_c$ is the distance of rise or drop of water along the wire due to capillary action, and gives the subsequent error in the probe’s measurement.

$$l_c = \sqrt{\frac{y}{\rho \alpha g}} \quad (4.11)$$

For a surface tension coefficient of 0.075 N/m, we obtain $l_c = 0.002m$. In these experiments, all probes were 0.6m long, so such a phenomenon could have affected some of the nearshore probes, especially the ones located close to the shoreline.

**Probe movement**

Resistance probes can vibrate because they are flexible. High frequency oscillations can be directly measured on the wave signal (example on Figure 4.5). This parasitic probe movement can be due to the probe’s bending frequency of oscillation, loosening, or vortex shedding. The probe’s first bending mode $f_r$ can be estimated as follows (Harris & Piersol, 2002):

$$f_r = \frac{A}{2\pi} \sqrt{\frac{E_c2l_0}{\mu cl_c^5}} \quad (4.12)$$

$$m_c = \rho_c \pi \left(\frac{D}{2}\right)^2 l_c. \quad (4.13)$$

We consider the instrument is a system of two fixed cylinders of stainless steel. In equation (4.12), $A$ is an empirical coefficient, determined for the first bending mode of a fixed-free cantilever to be 3.52; $E_c$ is the Young modulus of stainless steel, $l_0$ is the moment of inertia (second moment of area) of one cylinder ($l_0 = \pi D^4/64$), $l_c$ is the
length of the probe, and $\mu_c$ is the mass per unit length of a cantilever (in this case, we take $\mu_c = 2m_c/l_c$). Each cylinder’s Young’s modulus $E_c$ has a value of $2.10^{11}$ Pa (Cunat, 2000), and the mass of one cylinder $m_c$ is determined by its volume and density using (4.13). In (4.12) and (4.13) $D$ is the diameter of the cylinder, and $\rho_c$ the density of stainless steel which has a value of approximately 7800 kg/m$^3$ according to Cunat (2000). Substituting (4.13) into (4.12) and rearranging, a new expression for $f_r$ can be obtained:

$$f_r = \frac{A}{2\pi} \sqrt[3]{\frac{E_c D^4}{16 \rho_c l_c^2}}. \tag{4.14}$$

(4.14) gives a value of 11.82Hz for the first bending mode of the probe, which is a lot higher than the frequency of oscillation that can be observed on noisy records.

Then the hypothesis of vortex shedding at the back of the probe’s wires is tested: when a fluid flows past a fixed object, vortices are created at the back of this object. For a small cylinder such as a resistance probe, vortex shedding creates parasitic oscillations of the probe. The frequency at which the vortex shedding takes place $f_s$ in this case is related to the Strouhal number $St$ by the following equation:

$$f_s = St u_0 / D. \tag{4.15}$$

According to Techet (2005) for circular cylinders $St$ has a value of about 0.2 for smooth surfaces and for Reynolds numbers $Re$ ranging from approximately 200 to 80000 (see section 3.1.2).

Using the parameters of the waves in Figure 4.5 as a typical example, Table 4.4 shows the shedding frequency expected for each wave of the time series and compares it to the actual wave frequency ($f_w = 1/T$). The first wave displaying visible oscillations is wave
3, on Figure 4.5 these oscillations can be estimated to have a frequency of 1.4Hz, which corresponds to the shedding frequency calculated using equation (4.15).

As the particle velocity depends on the individual wave amplitudes (see equation (4.7)), the shedding frequency becomes lower with damping. Therefore, oscillations appear later in the time series, when the shedding frequency becomes low enough to affect the wave frequency.

**Sloshing**

As shown in section 3.3.1, natural resonance creates low frequency oscillations that can perturb the propagating wave signal. Such oscillations are also visible on the full-length signals recorded (Figure 4.6, the sloshing has a period of 26s). The velocity recording of the wave in the shoreline region (Figure 4.3) clearly shows periodic forward and backward flow of the same period. They are triggered by the initial perturbation of the system (generated wave), and can significantly affect the signal after the launch of the first wave (see Figure 4.7, discussed further in section 4.3.2.3). The largest velocities are triggered by sloshing on Figure 4.3) so only the first wave of the time series represents the signal of interest.

**Signal processing operations**

From this review of measurement problems, necessary corrections to the recorded signal can be identified:

- high frequency noise has to be removed,
• low frequency sloshing has to be removed,
• repeatability tests have been performed to assess the significance of random errors,
• the synchronization error of recording boards has been corrected.

Sloshing is an issue that can be solved by truncating the signal. Indeed, it starts to affect the signal as soon as the initial disturbance occurs, therefore all waveforms other than the initial wave can be dismissed without hindering the quality of the analysis. There is only minor drift and it only concerns the end portion of a small amount of records, therefore it would be removed at the same time as the sloshing disturbances. Capillary action cannot be corrected post-processing so any dubious records (such as the signals from Nearshore 8) should be dismissed. Due to the linear relationship between voltages and the water elevation recorded by resistance probes, signal processing operations are performed on the calibrated wave heights for the removal of high frequency noise. These operations are described in the following section.

4.2.2 Filtering methods

Local averaging

Smoothing is a useful technique for removing random errors due to electronic noise. It acts as a low pass filter, which consists in creating a function that aims at capturing important patterns in the data, using their statistical properties to reduce scatter. High frequency noise and other small scale features are dismissed. One of the most common smoothing methods is the moving average. A moving average centered on point $i$
calculates each new value $y(i)$ in a series of $x(i)$ values for a window/span of $n$ data points, following the expression:

$$y(i) = \frac{\sum_{m=-n/2}^{n/2} x(i)}{n}. \quad (4.16)$$

In equation (4.16), $m = n - 1/2$ if $n$ is odd and $m = n/2$ if $n$ is even. The beginning and end points in the series are handled with smaller windows. A simple moving average (or running average) calculates each new value $y(i)$ in a series of $x(i)$ values by taking the mean of the previous $n$ successive points:

$$y(i) = \frac{x(i) + x(i-1) + \cdots + x(i-n+1)}{n}. \quad (4.17)$$

The results in Figure 4.8 are given by a centered moving average (4.12). Figure 4.8 – b shows that the moving average method using small spans, for instance $n=5$ does not totally get rid of all the high frequency noise. Increasing the value of $n$ ($n=15$ in Figure 4.8 – c) gives better results in high frequency reduction (some roughness still visible), with no loss of signal magnitude. For $n=30$, the noise is completely removed but the magnitude of the maximum wave becomes slightly reduced compared to the original signal (Figure 4.8 – d). The performance of the moving average is satisfactory for removal of this type of noise, however it gives mixed results with vortex shedding noise, of much lower frequency. The results given by filtering wave data with equations (4.16) and (4.17) are similar, particularly for small spans. For very large spans ($n>15$), the centered moving average performs better than the running average. For this reason, only the moving average described by (4.16) will be discussed in the rest of this section.
Contrary to elevation data, the moving average method gives satisfactory results with velocity data, which is mostly affected by high frequency electronic noise. Figure 4.9 shows the effects of a moving average filter on a velocity time series, again the data can only be made noise-free for very large spans, which results in a slight loss of signal. However, the velocity data is characterized by very large signal-to-noise ratio compared to the elevation data, so a moving average with a small enough span allows satisfactory smoothing of the noisy features without losing peak velocities below the given threshold of accuracy (+/-2%, see Chapter 3).

More advanced smoothing techniques also exist. For example, a widely used category of smoothers in signal processing are Kernel smoothers. Particularly relevant to time series data, the Kernel regression finds a non-linear relation between a pair of random variables, classically using a Gaussian kernel:

$$K(x) = \frac{1}{2\pi} e^{-\frac{1}{2}x^2}$$

(4.18)

$f$ is an unknown function, and is usually estimated using the whole $n$ data points in the time series as:

$$f(t) = \frac{\sum_{i=1}^{n} K(b(t - t_i))y_i}{\sum_{j=1}^{n} K(\frac{t - t_j}{b})}$$

(4.19)

Here the index $b$ is defined as the bandwidth of the kernel (i.e. width of the kernel used for estimation). Equation (4.19) has been applied to the data. First, bandwidths are varied and a fixed number of data points arbitrarily chosen as half of the length of the time series ($n=1100$) is chosen (Figure 4.10). It can be observed that for small bandwidths the wave amplitude is conserved (Figure 4.10- b), however some noise is still present. Slightly increasing the bandwidth helps to remove more noise (Figure 4.10
- c), but not completely. A greater increase in bandwidth removes all noise but the wave amplitudes are significantly reduced (Figure 4.10–d). Second, the bandwidth giving the best results in terms of conservation of signal and noise reduction for the previous test \((b=4)\) is fixed, and the length for the time series (number of data points \(n\)) is varied (Figure 4.11). When the original length of the time series is reduced by half (Figure 4.11 - b), some noise is still present. When the length of the time series is reduced further, noise is significantly reduced (Figure 4.11 - c) or virtually absent (Figure 4.11 - d), but at the cost of visibly underestimating wave heights. Optimizing the Kernel smoother (i.e. finding the best combination of \(b\) and \(n\)) for signals with relatively low levels of noise might be possible. However it is expected that this technique will need to be individually tailored to different signals and will not give satisfactory results for higher noise levels. Therefore Kernel smoothing will not be used for the analysis of wave profiles.

**Wavelet**

Wavelets are mathematical functions of varying shapes that cut up data into different frequency components, and each one is treated with a resolution matched to its scale. A wavelet prototype function \(\psi\) is chosen to analyze (decompose) a function \(f(t)\) so the signal can be represented in terms of a wavelet expansion. The wavelet transform is represented by \(W_f\):

\[
W_f(a, b) = \int f(t)\psi(at + b)dt. \tag{4.20}
\]

\(\psi(at + b)\) is the shifted and scaled version of the wavelet at time \(b\) and at scale \(a\) (Weeks, 2007). The wavelet decomposition results in approximated and detailed
coefficients, respectively called $h(k)$ and $g(k)$. $h(k)$ represents the lowpass (scaling) filter, and $g(k)$ represents the highpass (wavelet) filter, with $k$ being the index of the wavelet coefficient (starting at $k=0$).

The wavelet denoising procedure consists in applying the wavelet transform to the noisy signal to retrieve the noisy wavelet coefficients. A new level of decomposition is created every time $h(k)$ is re-decomposed / analyzed. Then, through a method called thresholding, the noises are removed: if the detail of the signal is smaller than a given threshold (that can be chosen by the analyst), the corresponding coefficients are set to zero (hard thresholding) so these details can be omitted without affecting the important features of the data (Graps, 1995). Finally, the inverse wavelet transform is applied to the thresholded wavelet coefficients to obtain a reconstructed, denoised signal.

This method is suitable for data containing discontinuities and sharp spikes that are relevant to the time history: contrary to Fourier analysis which would delete all high frequency components in a signal regardless of their magnitude, wavelet analysis is able to distinguish between the high frequencies that can be dismissed and the ones that are relevant at different locations in the time series. Figure 4.12 represents the results of a wavelet filter with maximum thresholds for all coefficients (using the basic wavelet “square-shaped” function Haar (4.21)) at different levels of decomposition.

$$\Psi(t) = \begin{cases} 
1 & 0 \leq t < \frac{1}{2} \\
-1 & \frac{1}{2} \leq t < 1 \\
0 & \text{otherwise} 
\end{cases} \quad (4.21)$$

Figure 4.12 shows that high levels of decomposition are necessary to remove high frequency noise, however some loss of signal can be observed (Figure 4.12 – c, d). Lower levels of decomposition preserve the signal better, some noise can still be
present (Figure 4.12 – b). It would be possible to improve the quality of the filtered signal at a slightly higher level of decomposition (e.g. level 5-6) by progressively lowering the thresholds. The levels of decomposition are not expected to vary much between each data series, however this type of processing requires careful thresholding of all coefficients one by one. In addition, the length of the signal has to be divisible by $2^{\text{level}}$ for wavelet analysis to be feasible, which requires signal extension of all the data series, most of which already have varying lengths. Given that the data to be analyzed does not contain interesting sharp features / high frequency components to be preserved for interpretation, the use of wavelet analysis is not relevant here, and the more time efficient Fourier analysis approach giving similar results is adopted.

**Fourier**

The Fourier analysis is based on the decomposition of a signal into a series of cosine and sine waves. It retrieves the frequency contents of the signal using cosine and sine waves, and can remove unwanted high frequencies. Fourier is therefore more appropriate than the other described techniques for filtering elevation data, as it can remove both electronic noise and probe vibration noise. For a signal defined over time $t$, the theory known as the Fourier Transform allows one to obtain the frequency spectrum $F(\omega)$ of a time history $\eta(t)$ over a time period $T$ by applying:

$$F(\omega) = \int_0^T \eta(t)e^{-i\omega t} dt.$$  \hspace{1cm} (4.22)

From (4.22) the energy spectrum is defined as $|F(\omega)|^2$ and represents the energy magnitude of the different frequencies represented. It gives an indication of which high frequencies have a noticeable energy compared to the main wave signal, these can be
regarded as noise and must be removed from the data (example given in Figure 4.13).
The energy spectrum is evaluated using a trapezoidal method. The Inverse Fourier Transform retrieves from the Fourier Transform the wave signal into the time domain and within the desired frequency limits (i.e. filtered signal), in this case $0 < f < \omega/2\pi$:

$$\eta(t) = \frac{1}{2\pi} \int_{-\omega_c}^{\omega_c} F(\omega) e^{i\omega t} d\omega \quad (4.23)$$

Figure 4.14 shows the results given by the Fourier analysis for different frequency limits (or cut-off frequencies). It is shown that the main signal is relatively well preserved even at low cut-off frequencies, and that all instrument noise is removed. The optimum results for the particular wave of this example are given for a cut-off frequency $|\omega_c|=4\text{rad/s}$ (Figure 4.14 – c). Equations (4.22) and (4.23) are therefore chosen to filter the data.

### 4.2.3 Sensitivity analysis / choice of filtering parameters

**Velocities – Choice of span for moving average**

Figure 4.15 shows that the size of the span directly affects the resolution of the filter. For large values of $n$, the loss of signal is significant only for the smallest velocity peaks (Figure 4.9 - d). For the particular signal analyzed in this figure, a span of $n=10$ (Figure 4.9– c) gives satisfactory results both in terms of noise and conservation of peak values. However, smaller peaks can be recorded. A span of $n=6$ (Figure 4.9– b) smoothes the noise well enough for the signal to be clear so this value is considered
appropriate to filter the velocity data. Different spans may be applied for particular cases \((n<6\) for very small peaks, \(n>6\) for higher levels of noise and/or wide peaks).

The filtered signal can be compared to the original signal by using the Inverse Fourier Transform (4.23). Ideally the filtered signal should not be distorted compared to the original one, and all the very high frequencies masking the lower wave frequencies of interest should have disappeared.

*Surface elevation - Number of modes for Fourier analysis*

The number of modes is the number of frequencies used to represent the original signal in the frequency domain (energy spectrum). It should be high enough to avoid a loss of signal (Figure 4.16) and not to distort the original signal but not too high, so that the analysis remains computationally efficient. Longer waves require a higher number of modes for filtering compared to shorter waves, as the frequencies to be considered stretch from much lower frequencies to the same higher ones (noise). It is found that short waves (e.g. test ID 135) require representation over a domain of 190 modes, slightly longer waves (test ID 415) 250 modes, and very long waves (test ID 307) 350 modes.

*Surface elevation - Cut-off frequency*

An unfiltered signal is used to graphically estimate the period of the main wave and the period(s) of parasitic oscillations (See again Figure 4.5). From this we can calculate the approximate frequencies that carry the most energy (i.e. basin sloshing and main wave
frequency), and the frequencies that need to be removed (cut-off frequency and above). Once the cut-off frequency has been found we can limit the area of integration of the Inverse Fourier Transform to the frequencies of interest. However, a compromise should be found so as not to lose signal by filtering excessively: Figure 4.17 shows the lower the cut-off frequency, the more signal is lost.

One particular number of modes and one particular cut-off frequency do not give satisfactory results for every experimental wave. Waves with similar wavelengths require a similar number of modes. Noise frequency components tend to vary slightly, and cut-off frequencies have to be adjusted accordingly. Several runs of the Fourier Transform pair, varying these parameters slightly, may be required to obtain the most appropriate filter and adjust the values for each wave if necessary. In choosing the filter frequencies and the number of modes the following requirements are sought:

- When comparing filtered and unfiltered signal, the filtered signal should be free from unwanted high frequency components such as instrument-related noise, and not excessively smoothed out.
- When looking at the energy spectrum, the high frequency components should not carry a significant amount of energy (examples of energy spectra for different cut-off frequencies are shown in Figure 4.19).
- When comparing maximum and minimum amplitudes before and after filtering, there should be no loss or minimum loss of height.

In most cases, it is not possible to obtain perfect filters matching all three requirements presented above. The most important consideration is the potential amplitude loss of the main original wave, which may happen if not enough modes are taken into account for filtering or if too many frequencies are removed. Indeed, most of the calculations
for the analysis following the filtering process are based on wave profiles, as well as maximum and minimum amplitudes. In case an insufficient amount of frequencies is the cause of the loss of signal, we can also observe a distortion of the filtered signal, particularly obvious with long waves. If it is the low cut-off frequency causing the loss of signal, changing the number of modes will not improve the quality of the filtered wave. In this case it is necessary to increase the value of the cut-off frequency. As a consequence, some filtered waves still display a certain amount of “roughness” (visible high frequencies) as lowering the filter limit would have excessively diminished the amplitude of main signal. For such waves a compromise needs to be found between minimal loss of signal and optimum filtering. However, in many cases, reducing the amount of high frequencies to be removed does not significantly alter the quality of the filtered signal as the energy carried by these modes is negligible compared to the energy of the main signal.

4.2.4 Standard errors

Standard errors need to be estimated in order to know the confidence intervals associated with the population mean $\mu_x$ for a variable $x$ (i.e. for any measurement, we know the interval in which the true value lies). Given the observable repeatability of successive measurements for all quantities of interest (surface elevation, runup, velocities, pressures – see section 4.1), it is reasonable to assume for a series of $N$ measurements of each quantity $x$, that the best value in the sample is given by its mean $\bar{x}$. While the standard deviation $s_x$ gives a measure of the spread of the values of the
sample, the standard error of the sample mean \( SE_{\bar{x}} \) expresses the error in \( \bar{x} \) (Moore & McCabe, 2003):

\[
SE_{\bar{x}} = \frac{s_x}{\sqrt{n}} 
\]  

(4.24)

For multiple samples pooled standard error \( SE_{p,x} \) can be expressed as:

\[
SE_{p,x} = s_{p,x} \sqrt{\frac{1}{\sum_{i=1}^{n} \frac{1}{n_i}}}
\]  

(4.25)

The standard error associated with \( \bar{x} \) is \( SE_{\bar{x}} \) so that \( \bar{x} - SE_{\bar{x}} < \bar{x} < \bar{x} + SE_{\bar{x}} \). The confidence intervals for the mean of the population \( \mu_x \) (all waves, for the variables in Table 4.3 and Table 4.4) are given by:

\[
\bar{x} - SE_{\bar{x}} t^* < \mu_x < \bar{x} + SE_{\bar{x}} t^*
\]  

(4.26)

In (4.26), we will replace \( SE_{\bar{x}} \) by \( SE_p \) to obtain the confidence intervals for the variables in Table 4.1. \( t^* \) is the value retrieved for \( t(n-k) \) from a \( t \) distribution as described in Moore & McCabe (2003). \( t \) values can be found in NIST/Sematech (2006), \( k \) being the number of samples considered (i.e. \( k=1 \) for one sample). The authors show that \( t \) distributions can be used when data are close to normal (\( n<15 \)) or for large samples (\( n>15 \)). Table 4.5 gives the value of \( t^* \) for different degrees of freedom \( n-k \) and common confidence interval levels.

We take \( x \) as \( a \) for positive amplitude, \( a^- \) for negative amplitude, \( T \) for period, \( U \) for flow velocity, \( R \) for runup and \( p \) for pressures.

Applying (4.26) to the data and considering a relatively conservative confidence interval (90%) we obtain confidence limits for each average measurement (Table 4.6).
Table 4.6 indicates that the accuracy of measured wave amplitudes is greater than the accuracy of measured periods for both types of waves and very satisfactory in comparison with the instrument accuracy (see section 3.2.3). The sampling period of the wave signal was 0.05s, so the accuracy of N-wave periods was satisfactory. Given the variations in repeatability of the velocities recorded by the current flow meters (Table 4.2), the standard error is calculated separately for each instrument, and shall be considered to vary according to the type of wave flow measured. It is noted at this point that all velocity measurements that do not display a degree of repeatability (i.e. noise, inconsistent patterns) will be simply dismissed.

4.3 Wave Elevation Time Series

4.3.1 Wave amplitude, period, wavelength

Maximum and minimum wave amplitude is calculated for each probe from zeroed time series. It is the parameter usually used for the assessment of wave inundation potential compared to other wave parameters. However, many parameters are needed to fully characterize a wave.

Wave period and wavelength are also retrieved from wave elevation time series. However, due to the reflection issues highlighted in section 3.3, in many cases the second half of the positive part of the wave may not strictly correspond to the direct signal. Therefore, the period $T$ and wavelength $L$ are calculated using the first half of the positive wave, assuming symmetry:
\[ T = 2(t_{\eta_{\text{max}}} - t_0), \]  \hspace{1cm} (4.27)

\[ L = c_{p_{\text{exp}}} T. \]  \hspace{1cm} (4.28)

In (4.27), \( t_{\eta_{\text{max}}} \) is the time of occurrence of the wave peak, and \( t_0 \) corresponds to the time when the value of the wave elevation is 1\% of the maximum wave height \( (t_{\eta_{\text{max}}} > t_0) \). In (4.28), \( c_{p_{\text{exp}}} \) is the experimental phase speed. Note that, for N-waves, the trough does not trigger any reflections from the slope, so the parameters corresponding to the negative part of the wave are calculated on the full negative profile.

These considerations also apply to the estimation of the integral measures of the wave presented below.

### 4.3.2 Integral Measures of Wave Properties

An exact representation of the wave profile in the frequency domain can be determined, however the objective here is to find a measure representative of the wave shape and length. Therefore, integral measures of wave properties are defined for all waves following the definitions presented in section 2.2.3.

#### 4.3.2.1 Excess Mass
The excess mass $A$ of the wave for a unit width between the two positions corresponding to the beginning and end of the wave (respectively $x_1$ and $x_2$) is expressed as:

$$A = \int_{x_1}^{x_2} \rho \eta(x) \, dx. \quad (4.29)$$

For small turbulent circulations, the Taylor hypothesis allows the conversion of time into space by using the large scale mean velocity (Taylor, 1938). The Taylor hypothesis approximation can be used when the contributions of small scale velocities ($u$) to the motion of the flow are small in comparison with the large scale velocity. In the case of a long propagating wave, the phase velocity $c_\rho$ drives the forward motion of the flow, so this hypothesis can be used and $dx = c_\rho \, dt$. Using the phase speed experimentally measured $c_{\rho \exp}$, and noting $t_1$ and $t_2$ the times corresponding to the beginning and end of the wave profile, the excess mass can be expressed as:

$$A = \int_{t_1}^{t_2} \rho \eta(t) c_{\rho \exp} \, dt. \quad (4.30)$$

### 4.3.2.2 Momentum

The wave momentum $I$ of a wave of mass $m$ can be expressed the same way as the linear momentum of an object of mass $m$ using:

$$I = mc_{\rho \exp}. \quad (4.31)$$

Considering a unit flume width for the two dimensional laboratory experiments, the momentum can be expressed using:
The integral measures described above were calculated for all waves. However because excess mass and momentum can be negative or equal to zero for N-waves, wave energy will be the preferred integral parameter to be investigated.

4.3.2.3 Potential Energy

The potential energy \( E_p \) of the experimental wave defined between positions \( x_1 \) and \( x_2 \) is expressed as:

\[
E_p = \int_{x_1}^{x_2} \frac{1}{2} g \rho_0 \eta(x)^2 \, dx. \tag{4.33}
\]

Using the Taylor hypothesis, the potential energy of the wave in the constant depth region of the flume can be expressed as:

\[
E_p = \int_{t_1}^{t_2} \frac{1}{2} g \rho_0 \eta(t)^2 c p_0 \, dt. \tag{4.34}
\]

As an example, the cumulative potential energy of the wave from in Figure 4.6 is shown in Figure 4.7. The first energy plateau reached by the wave corresponds to the initial wave of the time series, the launched wave. The other plateaus correspond to the subsequent waves. We have shown that such waves are mainly due to the sloshing of the basin. Figure 4.7 shows that the sloshing’s contribution to the overall potential energy of the signal is significant, and for the purpose of this study the energy of the sloshing waves is not of interest. Therefore, the potential energy is calculated using the initial wave of the time series only, using \( t1 \) and \( t2 \) as in equation (4.30).
4.3.2.4 Kinetic Energy

The kinetic energy $E_K$ of the wave is expressed as:

$$E_K = \int_{t_1}^{t_2} \int_{x_1}^{x_2} \frac{1}{2}(u^2 + v^2) \, dy \, dx. \quad (4.35)$$

Using the Taylor hypothesis and considering, in the constant depth region, a long wave so $\delta v / \delta y$ is small, the vertical velocities can be neglected and the kinetic energy of the wave becomes:

$$E_K = \int_{t_1}^{t_2} \frac{1}{2} hu(t)^2 c p_0 \, dt. \quad (4.36)$$

Using equation (4.7) as an approximation of $u$ for any value of surface elevation $\eta$, we obtain:

$$E_K = \int_{t_1}^{t_2} \frac{1}{2} h \rho_0 \left( \frac{\eta(t)}{h} \sqrt{gh} \right)^2 c p_0 \, dt. \quad (4.37)$$

Simplifying and rearranging the terms in (4.37), we obtain expression for the kinetic energy, which is equal to the wave potential energy (4.34):

$$E_K = \int_{t_1}^{t_2} \frac{1}{2} g \rho_0 \eta(t)^2 c p_0 \, dt. \quad (4.38)$$

So we expect $E_p \approx E_K$ between the tank and the toe of the sloping beach. Between the toe of the beach and the breaking location, we expect $E_p$ to increase and $E_K$ to decrease, as a result of the shoaling process where the wave grows and slows down. After breaking, $E_p$ is expected to decrease due to the energy dissipation occurring during breaking. No velocity data is available on the slope before wave breaking, moreover the total kinetic energy is expected to become null at the end of the runup.
process (Li & Raichlen, 2003), so the evolution of kinetic energy during shoaling is not considered here. Therefore, the results presented in Chapter 5 consider exclusively potential energy.

4.4 Wave Runup and Rundown

The wave runups were calculated using equation (3.14). Rundowns were calculated for a sample of short and long N-waves, using \( x_s - x \) instead of \( x - x_s \) in (3.14). In the case of rundown, \( x \) is the distance uncovered on the beach before the positive wave’s arrival. Each wave runup and rundown was averaged over the number of repeats, no further processing of the raw data was performed. The analysis carried out to derive the runup results is presented in section 5.2.

4.5 Wave Velocity Time Series

*Velocity time history*

The experimental set up did not allow for the measurement of surface elevations at the same location as flow velocities. Therefore, it is not possible to verify directly approximation (4.7), however the height profiles are reproducible. Moreover, the waves are long enough that a height measurement made at a close enough location may give a reasonable estimate of the measured velocity.
For all waves, the velocity patterns on the slope and in the vicinity of the shoreline are complex with multiple flow directions. This is particularly easy to observe for long wave records, where the flow velocities change directions a number of times. Figure 4.3 gives an example of a complex velocity pattern upstream of the shoreline, triggered by the generation and propagation of a long elevated wave (of period 83.3s). The initial small amplitude velocity corresponds to the initial forward motion of the water mass triggered by the generation of the long wave: it happens 10.4s after the observed rise in water level (see dashed blue wave in Figure 4.3), which is the approximate time taken by a long wave to travel the distance separating Offshore 1 and the velocity probe. The long wave envelopes the rest of the velocity recording, showing that the wave contains flows constantly moving in opposite directions. The period of a reverse flow corresponds to the period of basin sloshing (~26s), therefore is not characteristic of the input wave. The same type of velocity pattern can be observed on the slope downstream close to the shoreline, due to forward flow and back swash. However once the flow reaches the flat land area, the velocity time histories only display a single peak, characteristic of the flow velocity inland only travelling in one direction (the sump absorbs potential reflections). This evolution is similar for all waves tested.

Peak velocity

Figure 4.18, shows the peak velocities of four different waves with distance along the sloping beach. A number of outliers can be observed. Although most graphs show an upward trend, many points fall significantly below. One possible method to remove outliers would be to calculate the local average over four or five points, and remove all readings deviating more than two standard deviations from the local average. This
method is expected to give satisfactory results for a small number of outliers (Figure 4.18–d). However, in most cases the relatively large number of outliers would significantly influence the value of the local average, so that at best no outliers would be detected, and at worst good data would also be deleted. An alternative option would be to select reliable data according to the energy spectrum of the velocity time history. By taking as a reference the energy spectra of a sample of trusted readings (i.e. repeatable, clean readings with consistent magnitudes and patterns), a constant decay in the energy can be observed for angular frequencies higher than $\omega_m=1\text{rad.s}^{-1}$ (Figure 4.19). According to Hunt and Vassilicos (1991), signals with discontinuities (such as the velocity time series presented here) have energy spectra of the form $E(\omega) \propto \omega^{-2p}$ as $\omega \to \infty$; for sharp discontinuities $p=1$, for other discontinuities $p=2$.

\[
Y(\omega) = Y(\omega_m).\omega^{-N}.
\]  

(4.39)

Therefore, the relationship (4.39) is compared to the velocity energy spectra for all $\omega > \omega_m$ and all data for which $2 < N < 4$ is kept, and is otherwise dismissed. Examples are shown in Figure 4.19. This method efficiently removes outliers.

For waves in the vicinity of the shoreline, the largest velocities may correspond to a sloshing motion. In such cases, the peak velocity considered is either the first peak of the series (which can only correspond to a forward motion of the wave) or the second peak if identified as being a forward motion (such as the example in Figure 4.3). This can be reinforced by taking into account the predicted wave arrival time.

4.6 Wave Pressures
The pressure transducers on the front face of the building (T4, T5 and T6, see Figure 3.12) record a signal for the highest elevated wave (test ID 135). However for smaller wave amplitudes, only T4 and T5 (test ID 415) or T4 alone record a signal (test ID 138). The back transducer T3 never records any pressure at the back of the building, due to the flow pattern that creates an empty triangular area here. T2 never records any signal either, as the flow heights inland were too small to reach it. T1 records a weak signal in the case of the longest waves (test ID 415), which should correspond to the component of the hydrostatic force on the side wall.

Wave pressures give an indication of the wave impact at a particular point on a surface. To retrieve wave forces from wave pressure data would require integration of pressure over a given area of the structure and knowledge of the evolution of flow depth in contact with this area with time $h_f(t)$. This is beyond the scope of this thesis. However, in cases where the full area surrounding a given pressure transducer is in contact with fluid (i.e. the data indicates that the flow reaches the next transducer up), a crude approximation of the wave force on this area can be obtained by multiplying pressure and area. Preliminary results of wave pressure data will be introduced in Chapter 5 and briefly discussed along with the foam building force experiment described in section 5.3.2.
### Wave Type Properties

<table>
<thead>
<tr>
<th>Wave type</th>
<th>$s_{p_a}$ (m)</th>
<th>$s_{p_a^-}$ (m)</th>
<th>$s_{p_r}$ (s)</th>
<th>$s_{p_R}$ (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Elevated waves</td>
<td>$2 \times 10^{-3}$ (4,9)</td>
<td>/</td>
<td>0.48 (4,9)</td>
<td>$3.4 \times 10^{-3}$ (6,25)</td>
</tr>
<tr>
<td>N-waves</td>
<td>$1.1 \times 10^{-4}$ (3,4)</td>
<td>$1.3 \times 10^{-4}$ (3,4)</td>
<td>$2.9 \times 10^{-2}$ (3,4)</td>
<td>$1.9 \times 10^{-3}$ (6,18)</td>
</tr>
</tbody>
</table>

**Table 4.1:** Pooled standard deviations (equation (4.4)) of positive and negative amplitudes, period, and runup, for elevated and N-waves. The first number in the brackets is the sample size ($n$) used to calculate the standard deviation for an individual sample $s_x$, the second number is the total number of waves $N(x)$.

### Peak Flow Velocity Values

<table>
<thead>
<tr>
<th>Wave type</th>
<th>$s(u_{max})_{p_1}$ (cm/s)</th>
<th>$s(u_{max})_{p_2}$ (cm/s)</th>
<th>$s(u_{max})_{p_3}$ (cm/s)</th>
<th>$s(u_{max})_{p_4}$ (cm/s)</th>
<th>$s(c_p \ exp)$ (m/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Short Elevated</td>
<td>1.18 (3,1)</td>
<td>2.10 (3,1)</td>
<td>4.2 (3,1)</td>
<td>4.27 (3,1)</td>
<td>0.011 (3,1)</td>
</tr>
<tr>
<td>Long N</td>
<td>2.02 (3,1)</td>
<td>0.32 (3,1)</td>
<td>0.62 (3,1)</td>
<td>1.14 (3,1)</td>
<td>0.082 (3,1)</td>
</tr>
<tr>
<td>Long Elevated</td>
<td>1.69 (3,1)</td>
<td>0.17 (3,1)</td>
<td>0.61 (3,1)</td>
<td>1.78 (3,1)</td>
<td>0.04 (3,1)</td>
</tr>
</tbody>
</table>

**Table 4.2:** Standard deviations of the peak flow velocity values recorded by each instrument ($P_1$, $P_2$, $P_3$, $P_4$) and for different lengths of wave time series. The sample size and total number of waves respectively are indicated in brackets.
### Table 4.3: Standard deviations of the peak pressure values for two main elevated wave time series. The sample size and number of samples \((n, N(x))\) are indicated in brackets.

<table>
<thead>
<tr>
<th>Wave type</th>
<th>(s(P_{\text{max}T4})) (mmH20)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Short elevated</td>
<td>4.63 (4,1)</td>
</tr>
<tr>
<td>Long elevated</td>
<td>0.54 (4,3)</td>
</tr>
</tbody>
</table>

### Table 4.4: Wave amplitudes, orbital velocities, frequencies and shedding frequencies for the record in Figure 4.5. The number in the first column indicates the position of the wave in the time series. The wave frequency corresponds to the frequency for waves 2, 3 and 4.

<table>
<thead>
<tr>
<th></th>
<th>(a_+ \text{ (m)})</th>
<th>(c_{p0} \text{ (m/s)})</th>
<th>(U_0 \text{ (m/s)})</th>
<th>(f_s \text{ (Hz)})</th>
<th>(f_w \text{ (Hz)})</th>
<th>(f_w/f_s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wave 1</td>
<td>0.033</td>
<td>2.4</td>
<td>0.135</td>
<td>4.5</td>
<td>0.088</td>
<td>0.019</td>
</tr>
<tr>
<td>Wave 2</td>
<td>0.023</td>
<td>2.4</td>
<td>0.094</td>
<td>3.14</td>
<td>0.038</td>
<td>0.012</td>
</tr>
<tr>
<td>Wave 3</td>
<td>0.01</td>
<td>2.4</td>
<td>0.041</td>
<td>1.36</td>
<td>0.038</td>
<td>0.028</td>
</tr>
<tr>
<td>Wave 4</td>
<td>0.068</td>
<td>2.4</td>
<td>0.028</td>
<td>0.93</td>
<td>0.038</td>
<td>0.041</td>
</tr>
<tr>
<td>Degrees of freedom (dof)</td>
<td>2</td>
<td>3</td>
<td>8</td>
<td>9</td>
<td>25</td>
<td>70</td>
</tr>
<tr>
<td>-------------------------</td>
<td>----</td>
<td>----</td>
<td>----</td>
<td>----</td>
<td>----</td>
<td>----</td>
</tr>
<tr>
<td>90% confidence</td>
<td>2.92</td>
<td>2.353</td>
<td>1.86</td>
<td>1.833</td>
<td>1.708</td>
<td>1.667</td>
</tr>
<tr>
<td>95% confidence</td>
<td>4.303</td>
<td>3.182</td>
<td>2.306</td>
<td>2.262</td>
<td>2.06</td>
<td>1.994</td>
</tr>
<tr>
<td>99% confidence</td>
<td>9.925</td>
<td>5.841</td>
<td>3.355</td>
<td>3.250</td>
<td>2.787</td>
<td>2.648</td>
</tr>
</tbody>
</table>

Table 4.5: $t^*$ values to be used in equation (4.26) for commonly used confidence intervals, and degrees of freedom for the range of sampled variables (table values from Moore & McCabe (2003)). For $u$ and $c_{p \exp}$ dof=2, for pressure data (short wave) dof=3, for pressure data (long wave) dof=9, for N-waves $a$, $a^-$ and $T$ dof=8, for N-waves runup dof=70, for elevated waves $a$ and $T$ dof=25, and for elevated waves runup dof=90.
<table>
<thead>
<tr>
<th></th>
<th>Standard error (mean or pooled)</th>
<th>$SE \times t^* ; SE_p t^*$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$a$ (elevated wave)</td>
<td>0.0031m</td>
<td>0.0053m</td>
</tr>
<tr>
<td>$a$ (N-wave)</td>
<td>0.00011m</td>
<td>0.0002m</td>
</tr>
<tr>
<td>$a^-$ (N-wave)</td>
<td>0.00013m</td>
<td>0.00024m</td>
</tr>
<tr>
<td>$T$ (elevated wave)</td>
<td>0.75s</td>
<td>1.27s</td>
</tr>
<tr>
<td>$T$ (N-wave)</td>
<td>0.029s</td>
<td>0.054s</td>
</tr>
<tr>
<td>$R$ (elevated wave)</td>
<td>0.0084m</td>
<td>0.014m</td>
</tr>
<tr>
<td>$R$ (N-wave)</td>
<td>0.0038m</td>
<td>0.0064m</td>
</tr>
<tr>
<td>$c_{p \exp}$ (elevated wave, short)</td>
<td>0.0063m/s</td>
<td>0.018m/s</td>
</tr>
<tr>
<td>$c_{p \exp}$ (elevated wave, long)</td>
<td>0.023m/s</td>
<td>0.067m/s</td>
</tr>
<tr>
<td>$c_{p \exp}$ (N-wave, long)</td>
<td>0.047m/s</td>
<td>0.14m/s</td>
</tr>
<tr>
<td>$u_{\max p4}$ (elevated wave, short)</td>
<td>2.46cm/s</td>
<td>7.2cm/s</td>
</tr>
<tr>
<td>$u_{\max p4}$ (elevated wave, long)</td>
<td>1.03cm/s</td>
<td>3cm/s</td>
</tr>
<tr>
<td>$u_{\max p1}$ (N-wave, long)</td>
<td>1.17cm/s</td>
<td>3.4cm/s</td>
</tr>
<tr>
<td>$p$ (elevated wave, short)</td>
<td>2.3mm$H_2O$</td>
<td>5.45mm$H_2O$</td>
</tr>
<tr>
<td>$p$ (elevated wave, long)</td>
<td>0.47mm$H_2O$</td>
<td>0.86mm$H_2O$</td>
</tr>
</tbody>
</table>
Table 4.6: Standard errors and confidence intervals calculated for all variables and wave types. For wave amplitudes, periods and runup, no distinction was made between long and short waves as the observable repeatability was the same. For particle velocity measurements, due to the differences between probes (see Table 4.2), only the least repeatable measurements were listed for a conservative estimate of the error.
Figure 4.1: Repeatability of: (a) elevated waves, (b) N-waves.
Figure 4.2: Repeatability of velocities for one velocity probe p1; (a) for a short wave (test ID 135), (b) for a longer wave (test ID 415), (c) for a very long wave (test ID 307). The complex flow velocity profile (c) is due to forward and backward flow in the wet region of the slope close to the shoreline.
Figure 4.3: Example of a velocity times series (test 08_081028 probe 2) recording forward ('F') and reverse flow ('R'). The flow reverses due to waves moving backward and forward. The blue dashes correspond to the wave recorded by Offshore 1 at the same time as the velocity time series. The black line is the same wave record offset as to envelop the velocity record. Note the flow velocity always increases and decreases before the flow direction changes. The second velocity peak is likely to be a forward motion corresponding to the main wave, as the rise of the first part of the waves matches closely the rise in velocity, this pattern is characteristic of all long positive waves over the slope.
Figure 4.4: Repeatability of pressures, (a) for a short elevated wave (tests14-17_081107), (b) for a long elevated wave (tests 30-33_081107).
Figure 4.5: Typical effect of vortex shedding on a wave probe (here Nearshore 1 for test 06_081028). The period of oscillation observed on wave 3 is 0.7s, giving a corresponding frequency of 1.4Hz.
Figure 4.6: Example wave signal (test 20_081028) displaying natural sloshing of the basin. The associated spectrum (upper right corner) shows the lowest frequency corresponds to the period of sloshing (0.24Hz - 26s); the generated wave having a slightly higher frequency (0.4Hz - 15s).
Figure 4.7: Cumulative potential energy for the whole signal in test 20_081028. The first plateau reached corresponds to the input wave, the other plateaus to each other wave of the time series (reflections and sloshing).
Figure 4.8: Results given by a moving average filter applied to Offshore 1 probe on test 20_081028: (a) unfiltered signal, (b) span n=5 points, (c) span n=15 points, (d) span n=30 points. The blue line represents the original maximum wave height.
Figure 4.9: Effects of a moving average filter on velocity time series, here recorded by PI for test 20_081028. The blue lines correspond to the minimum values of the peak velocities expected given the instrument accuracy, for (a) unfiltered data, (b) moving average filter with a span $n=6$ points, (c) moving average filter with a span $n=10$ points, (d) moving average filter with a span $n=14$ points.
Figure 4.10: Results given by a Kernel smoother applied to Offshore 1 probe on test 20_081028 for n=1100; (a) unfiltered signal, (b) with a bandwidth of 2, (c) with a bandwidth of 4, (d) with a bandwidth of 8. The blue line represents the original maximum wave height.
Figure 4.11: Results given by a Kernel smoother applied to Offshore 1 probe on test 20_081028 for $b=4$; (a) unfiltered signal, (b) $n=1100$, (c) $n=500$, (d) $n=250$. The blue line represents the original maximum wave height.
Figure 4.12: Results given by a wavelet transform filter applied to Offshore 1 probe on test 20_081028; (a) unfiltered signal, (b)- 3 levels of decomposition,(c) 4 levels of decomposition, (d) 5 levels of decomposition. One level of decomposition corresponds to one wavelet coefficient, the thresholds have been kept to a maximum for all graphs presented here. The blue line represents the original maximum wave height.
Figure 4.13: Energy spectra for an example wave (389), cut-off frequency in rad/s: (a) 4; (b) 5; (c) 6. The first energy peak at low frequencies corresponds to the natural frequency of the flume, and the highest frequencies carrying noticeable energy correspond to instrumental noise (circled in red).
Figure 4.14: Results given by Fourier filtering applied to Offshore 1 probe on test 20_081028 for a fixed number of modes; cut-off frequency in rad/s: (a) unfiltered signal, (b) 3; (c) 4; (d) 5. The blue line represents the original maximum wave height.
Figure 4.15: Difference between the unfiltered and filtered peak velocity values according to the span of the moving average.
Figure 4.16: Effect of the number of modes on the loss of signal, here for test 30_081031.
Figure 4.17: Effect of the cut-off frequency on the loss of signal, here for test 20_081028.
Figure 4.18: Peak velocities vs. distance recorded by the velocity meters with readings, (a) for wave ID 135 (elevated wave), (b) for wave ID 415 (long elevated wave), (c) for wave ID 113 (N-wave), (d) for wave ID 307 (very long N-wave). The red dashes correspond to the position of the shoreline.
Figure 4.19: Energy spectra of the four velocity probes for test 28_081028 (Elevated wave 135). The red line corresponds to (4.39) for $N=3$ ($p_1$, $p_3$, $p_4$), and for $N=0$ ($p_2$). The data recorded by $p_2$ is therefore removed. The red dashes correspond to the observable onset of energy decay at $1$ rad/s.
Chapter 5. Experimental Results

This chapter aims at presenting the results obtained using the experimental methodology described in Chapter 3, and the data processing described in Chapter 4. In section 5.1, the capabilities and limitations of the long wave generator are discussed in relation to the objectives of this work. In section 5.2, runup results for the different types of waves tested are compared to previous studies, new runup relationships are derived using a semi-empirical approach, and differences and similarities between the results are explained. In section 5.3, other results obtained are presented: these include phase velocities for different types of waves, flow velocities around the shoreline and inland, and finally some preliminary results on wave pressures on buildings.

5.1 Validation of the new HRW Long Wave Generator

The validation of the new wave generator consists in assessing:

- whether the scale of experiments was appropriate to represent the driving forces of long propagating waves in the field,
- whether the generator successfully reproduced short and long propagating elevated and N-waves,
- whether the current setup addressed the weaknesses of previous physical experiments in reproducing long and depressed wave components as well as shallow water conditions,
- the limits of the current generator and setup in terms of wave size.
These points are treated in turn in the next sections.

5.1.1 Scale of experiments

The preliminary calculations in section 3.1.2 have predicted that Froude scaling is appropriate for modelling long waves approaching the shore, given the maximum $a/h = 0.26$ for a positive elevation at model scale. It is now proposed to estimate, using an example prototype wave, whether this holds for the range of waves tested. For this purpose, it is first necessary to determine the phase speed of the experimental waves.

*Phase speed*

The phase speed of the wave can be measured directly using the surface elevation from a series of resistance probes placed in the constant depth region of the flume, and taking into account the distance travelled and time of occurrence of the peak elevation. The slope of the linear fit in Figure 5.1 gives the phase speed of the wave of interest. Given the $h/L$ ratios at play in the experiments, it is expected that the experimental waves would travel with the shallow water wave speed: $\sqrt{gh}$. However, the experimental phase speed were observed to be 3% to 24% larger than the shallow water wave celerity for both N-waves and elevated waves (Table 5.1).
Like solitary waves for which the phase speed can be described by (2.12) and (2.15), it is likely the amplitude of elevated waves influences the phase speed. Using a third order solution for a solitary wave, Klettner and Eames (2011) describe the phase speed of a solitary wave as:

\[
c_p = \sqrt{gh} \left(1 + \frac{1}{2h} + \frac{3}{20} \left(\frac{a}{h}\right)^2 + \frac{3}{56} \left(\frac{a}{h}\right)^3 \right). \tag{5.1}
\]

We have seen (section 3.3.2) that experimental elevated waves could be generated with 0.014 < \(\frac{a}{h}\) < 0.26. Therefore, if we apply (5.1) to the case of elevated waves the second and third order terms can be neglected, and taking \(\lambda = 1/2\), we obtain:

\[
c_p = \sqrt{gh} \left(1 + \lambda \frac{a}{h}\right). \tag{5.2}
\]

Figure 5.2 shows the phase speed of the wave increases with \(a/h\). For N-waves, we take into consideration the total wave height so \(a/h\) becomes \(H/h\). (5.2) fits the elevated wave data with minimal errors for \(\lambda = 0.88\); for N-waves \(\lambda = 0.86\) with the sum of errors squared being one order of magnitude greater compared to elevated waves. It can be observed in Figure 5.2 that the smallest waves are close to the linear fit for \(\lambda = 1/2\), and deviate from it as the waves get larger. The waves tested here are not in the linear regime as shown by the comparison between experiments and wave theories in Figure 5.3, which may explain the differences in predicted phase speeds.

The phase speed of the wave changes with depth and wave amplitude in the nearshore region. An example of the evolution of \(c_p\) compared to \(c_{p_0}\) is presented in Figure 5.4. Until the approximate point of wave breaking, the nearshore phase celerity drop up to 15% of the phase celerity in the offshore region \(c_{p_0}\).
**Froude scaling**

The phase velocity \( c_p \) of the experimental waves is described by (5.2). So it is necessary to know the variations of \( Fr_m/Fr_p \) (\( Fr_m \) being the Froude number in the model, and \( Fr_p \) being the Froude number in the prototype) for the range of waves studied, using the actual phase velocities. With the shallow water velocity \( c_{p0} = \sqrt{gh} \) and taking \( U \) in equation (3.2) as being the phase velocity described by (5.2), the Froude criterion (equation (3.3)) becomes:

\[
\frac{Fr_m}{Fr_p} = \frac{1 + \lambda \frac{H_m}{h_m}}{1 + \lambda \frac{H_p}{h_p}} \quad (5.3)
\]

Figure 5.5 shows how well the processes driving an example long prototype wave in the nearshore region are reproduced with the present laboratory setup, using equation (5.3). 46 experimental waves have been used. For the example of a prototype 4m wave travelling over a depth of 14m, the experimental Froude number ranges between 81% and 98% of the prototype Froude number for all of the investigated \( a/h \) ratios for short and long elevated waves, and N-waves (see sections 3.3.2, 3.3.3). This means that the scale of these experiments is large enough for gravity and inertia forces to be dominant and be representative of typical long propagating waves approaching the shore, which is sufficient for the purpose of these experiments. However, depending on the purpose of other/future applications, such as the reproduction of a specific tsunami case scenario, larger experimental waves may be needed to represent very large prototype waves.

**Considerations of scale effects**
In terms of scale effects, surface tension was expected not to affect results despite some small wave heights being generated (section 3.1.2). However, in the analysis of velocity probe results we found that contaminations due to water debris could affect wave measurements. Using equation (3.11), and increasing the surface tension coefficient, $\sigma$, by an arbitrary 10% to represent organic contamination, the relative influence of surface tension and gravity is calculated for the shortest and largest wave as $3.7 \times 10^{-6}$ and $2.1 \times 10^{-9}$, respectively. Surface tension is therefore confirmed to be negligible for all wave processes considered in the experiments. The significant influence of viscous forces has been shown not to induce significant wave damping (section 4.1.1).

In this study, we only investigate model pressures and potential energy. However, it is important to note that the estimation of prototype pressures from the model results, would require to take into account the density effect (i.e. difference between the model density (freshwater) and the prototype density (seawater): see section 3.1.2). From equation (3.8), we obtain:

$$N_p = \frac{N_L}{0.97}. \quad (5.4)$$

Potential energy is partly determined by water density, and the density effect can be taken into account using the following scale ratio:

$$N_{E_p} = N_p N_L = 0.97 N_L. \quad (5.5)$$

In the presentation of the results, no comparison will be made between the model potential energies measured and field potential energies (data not available), therefore equation (5.5) will not be used here but is relevant to the estimation of field potential energy in future work.
5.1.2 Generation of long and depressed wave components

It has been shown in previous chapters that waves are reproduced in a controlled manner (periodic waves, see section 3.3.1), and all waves are repeatable (section 4.1.1). This section demonstrates the other properties of the setup used and discusses whether the other requirements of the pneumatic generator are fulfilled.

\[ \text{Attenuation} \]

Attenuation \( (\Delta a = (|a_{01}| - |a_{010}|)/X_{010} - X_{01}) \) along the constant depth region of the flume has been calculated for a major sample of elevated waves and N-waves (Table 5.2). Attenuation has been proven to have orders of magnitude ranging from \(-6\) to \(-4\) for all waves (calculated between Offshore 1 and Offshore 10 for probe pattern 9, see Table 3.4).

As most waves are long compared to the propagation domain, small attenuation was expected. The dissipation taking place can be due to friction on the bottom boundary layer. Alternatively, because the standard deviations of positive and negative amplitudes recorded are of the same order of the attenuation or more (Table 5.2), the apparent attenuation may only be the manifestation of measurement variability.

\[ \text{Shallow water conditions} \]

It has been shown in section 2.6.1 that most current laboratory techniques do not allow for the generation of shallow water waves (defined as being waves for which \( h/L < 1/20 \)). Let us define the shallow water wave limit as \((h/L)^* = 0.05\). Taking data
from Table 3.1, which includes short waves tested for runup, and calculating \( h/L \) for each wave, the difference \((h/L)^* - h/L\) is positive in all cases except for tests 366 and 395. These tests involved a slightly shorter elevated wave (wave ID 148), however the \( h/L \) values for tests 366 and 395 are respectively 0.055 and 0.051. These values are very close to the shallow water limit so they can still be considered as shallow water waves. All subsequent waves tested had a period equal or larger than the waves of Table 3.1. Therefore the long wave generator and current setup allowed shallow water conditions to be present for all experiments. Figure 5.3 places present elevated waves and N-waves into the wave theories and domains of applicability presented in section 2.2.1. It can be observed that all waves generated are shallow water waves, and a number of experiments (for long waves) match nearshore field tsunami conditions.

Wave profiles (including trough)

The experimental elevated wave profiles at generation are not as steep as first or second order theoretical solitary waves (Figure 3.19). As a consequence, the results obtained in the present experiments cannot be assimilated or directly compared to pure solitary wave results. However, it is expected that in the field, propagating long waves are not generally of a solitary wave form (see section 2.6.1), therefore the results obtained for elevated waves are relevant for understanding long propagating wave behaviour in the nearshore region. The theoretical profile of an N-wave (2.21) matches well the experimental N-waves (Figure 3.20), particularly for short waves. For long N-waves (such as wave ID 307, Figure 3.21), the peak length is always reduced due to interactions with the beach, therefore compromising the match for the positive part of the wave. Despite the satisfactory match between a theoretical N-wave and one
nearshore signal recorded during the 2004 tsunami (Figure 3.21), N-waves are only a theoretical profile accounting for depressed wave components. They have not been extensively studied in the laboratory or in the field (see section 2.6.1). Therefore, the present experimental waves with positive and negative component can provide useful information on the wave behaviour at the shore. It is expected that the long wave generator placed in a wave flume of more than one wavelength would allow for the full theoretical and experimental profiles to be similar.

*Long wave periods*

As seen in section 3.3.1, the successful generation of very long periodic waves is possible with the long wave generator and current set up. However, the longer the waves, the more reflections affect the main wave signal (see section 3.1.2).

Allsop & Hettiarachchi (1988) provide experimental data indicative of the reflection coefficient $Cr$ that can be expected given the Iribarren number $Ir$ of a given wave. Table 5.3 gives examples of the reflection coefficients that can be expected with the present experimental setup. We can see that the longest waves can produce reflections with half the energy of the direct signal.

For the waves of interest in this study, i.e. long propagating elevated waves and N-waves, the maximum wave period for a given depth is limited by the length of the constant depth region (before the reflection process occurs). In section 3.3.2, it was highlighted that elevated waves with a period of approximately 100s would significantly reflect off the sloping beach giving little or no motion of the water at the shoreline. Instead, an elevated wave of period 92.2s or less would propagate and run up
the beach. The maximum elevated wave period allowing significant motion of the shoreline was not assessed. The period of long N-waves can reach slightly higher values, as only the positive part of the wave reflects off the slope. The positive part of the wave can be affected by reflections (as explained in section 3.3.3), in the same way as an elevated wave. However, one can observe the positive part of the N-wave always remained relatively narrow compared to the wave trough despite the length of valve time series applied. Because the narrow wave peaks often had a period of 20s to 28s, it is believed sloshing (~26s) could have also affected the signal. Experimental N-waves all triggered significant motion at the shoreline. The maximum period tested for a long N-wave with the present setup was 160.5s (test 28_081031).

These observations show that very long waves can be successfully generated with the present wave generator. However, it has been shown the propagating signal can be significantly affected by the configuration of the experimental setup. A longer flume (ideally twice the wavelength of the longest wave to be tested) would allow for longer waves to be generated and propagated. Moreover, as seen in section 3.1.2, larger values of the Iribarren number as defined by equation (3.10) would result in larger reflections; so a shallower slope would weaken reflections. Sloshing would always occur but at a different frequency / period than with the present setup.

Wave stability

Short elevated waves and N-waves appear stable (i.e. do not change form) during their propagation along the constant depth region of the flume (Figure 5.6 and Figure 5.7). The shortest elevated waves (L<15.2m) also conserve their shape in the constant depth
region of the flume, with the initial form of the wave seen to preserve its shape during propagation along the constant depth region of the flume. However, for all N-waves, the propagation distance (15.2m) is equal or larger than one wavelength. For example, the period of the experimental N-wave in Figure 5.7 is 7s (shortest N-wave), with $L = 15.5$ m. Therefore, it is not possible to confidently know with the present set-up whether the N-waves produced by the new long wave generator would be stable over longer distances. It is expected that long N-waves will be more stable than the short N-waves that have previously been reproduced with paddles in other studies, such as the negative solitary waves from Kobayashi & Lawrence (2004) (see section 2.4). A flat bottom propagation region of two or more wavelengths would be necessary to assess the change or conservation of the wave shape, which is not available for these waves in our current setup.

5.1.3 Wave heights

Elevated waves with $0.014 < a/h < 0.26$ and N-waves with $0.011 < a/h < 0.21$ have been generated (note that $a$ refers only to the positive amplitude of the wave for the latter). Specifically for N-waves, the range of trough amplitudes was $0.024 < a^-/h < 0.1$, in terms of total wave height N-waves could be generated for $0.07 < H/h < 0.21$.

Let us compare these values with field conditions. Using again the data in Figure 2.10 as an example of a long propagating wave nearshore, the initial wave positive amplitude gives $a/h = 0.26$. Moreover, a range of typical nearshore conditions when long waves hit the shore can be fixed, with depths over the continental shelf ranging
from 10m to 100m and wave heights (as reported in the literature) ranging from 3m to 16m, so representative $a/h$ ratios would range from 0.05 to 0.64 (Table 5.4). It is seen that the present setup allows the reproduction of the lower third of this range of $a/h$ ratios. In terms of wave troughs in the nearshore region, Figure 2.10 gives $a^-/h =0.2$. The data in Figure 2.9 should be interpreted with caution because of the sheltered location of the tide gauges and the unavailability of still water depths at these particular locations, however it indicates wave troughs around 1m can also be expected. Taking a range of trough sizes from 1m to 4m and continental shelf depths of 100m, 50m and 10m, we obtain a range of field values $0.01 < a^-/h < 0.4$ (Table 5.5). The current generator and setup allows for the reproduction of most of these $a^-/h$ ratios. However, very large depressed components would require larger laboratory water depths and an increased tank capacity.

The maximum $a/h$ that can be reproduced with the present generator and setup is limited by the head difference between the maximum water height reached in the tank and control water level (at the beginning of the test), as well as the length of the constant depth region of the flume (i.e. the total length for which the wave does not reflect off the slope). For N-waves, the trough amplitude is limited by the height of the outlet and the wave period (long N-waves can reach higher trough amplitudes compared to short N-waves). Reducing the height of the outlet above the bottom of the flume would increase the head difference, however it would compromise trough generation. Friction losses at the mouth of the tank would also increase, reducing the energy imparted to the propagating waves. Increasing the tank height and pump power would be an alternative and more suitable solution for the generation of larger waves. Increasing the total flume propagation length to be greater than the longest wavelength
to be tested would not only help full profiles to be reproduced, but also allow for the generation of slightly longer elevated waves, and for the stability of N-waves to be correctly assessed. Reproduction of a greater range of troughs would be possible with overall higher water depths (to be allowed for in the design of the bathymetry).

Finally, the wave attenuation was shown to be negligible (see section 4.1.1). However, one can notice for long elevated and N-waves, the wave height at the Toe is significantly greater than the height of the generated wave (Table 5.6). The snapshot graph (Figure 5.8, for two examples of long waves) show the wave actually recorded at the Toe is likely to be a composite wave, made of both reflections and direct signal. Initially, for the positive parts of both waves, a slow even rise in water level between Offshore 1 and Toe can be observed. Then, the water level tips towards the Toe and rapidly as the bulk of the wave comes in, the water level rises at the Toe (reflections are believed to enhance this process). The flume level slowly decreases to return to its equilibrium position. For the N-wave (Figure 5.8 - b), it is interesting to note that the water level decreases faster at the toe of the slope than close to the tank, at Offshore 1. The positive wave reaching the slope is likely to split, and part of it will travel towards the offshore region of the flume: firstly, by inertia to restore the water level equilibrium; secondly, through the reflection process.

For all long waves, the processes taking place at the toe of the bathymetry are complex interactions between the propagating wave, sloshing and reflections triggered by the presence of the slope, as described above. Unfortunately, all waves of large wavelengths will significantly reflect off the slope ($1 < Ir < 3.5$ so $0.1 < Cr < 0.5$ as shown in Table 5.3). A milder model slope would reduce this effect but not eradicate it.
with an improved wave generator and setup, the researcher should expect some reflections to take place in the flume.

### 5.1.4 Summary

To conclude, the range of Froude numbers in the present study confirms that gravity and inertia forces were dominant, so the model waves were representative of long propagating wave dynamics. Moreover, surface tension effects have been proven to be negligible, even for small waves. Shallow water conditions \((h/L<1/20)\) were respected throughout the testing, so the model waves were shallow water waves. Elevated waves and N-waves were successfully generated with the present equipment. However the signal during wave propagation, particularly for the longest waves, was affected by reflections and sloshing. For the longest waves such as wave ID 307 the reflection coefficient \(C_r\) could reach 0.5 (i.e. according to Allsop & Hettiarachchi (1988), reflecting half of the energy of the incoming wave). Elevated waves were stable. However N-wave stability could not be assessed as the propagation distance was not larger than one wavelength for all tests. The wave heights \(H/h\) for elevated waves and N-waves can be representative of a range of long propagating wave over the continental shelf. Wave attenuation was negligible, but the height of long waves close to the toe of the slope was affected by complex reflection processes taking place.

The correct reproduction and propagation of long wave profiles could be improved by minimizing the reflections from the slope. To do so, a longer wave flume would be needed. This requirement would also allow for correct assessment of N-wave stability.
The length of constant depth bathymetry necessary would depend on the range of wavelengths to be tested but would need to be at least twice the length of the longest wave. Taking as an example the range of waves from Table 3.1 some of the longest waves were 22.8m-44.6m long, so a constant depth region of 45.6m-89.2m would be desirable. Reflections would still be expected with the very long waves ($L>300$m, such as presented in Table 5.3), these can be minimize by using a milder slope. For example, using the data in Table 5.3 and replacing $\tan \beta$ by 1/50, the largest value of $Ir$ would be 1.3 which would correspond to a reflection coefficient of only 0.15 for test 081031_29. A larger tank capacity, improving its vertical dimensions, would allow the generation of larger $H/h$ ratios for both N-waves and elevated waves. Larger water depths would allow for the generation of larger troughs, however they will reduce the total head difference between the top of the tank and the control water level; which in return would prevent larger $H/h$ ratios to be generated. This has to be considered if improvements to the design of the generator are made in the future.

5.2 Runup

5.2.1 Comparison with previous runup relationships

With this section, the measurements taken of runup are compared with predictions from existing equations. The analysis of experimental data then leads to new runup relationships, which are later discussed. All runup results presented in this section correspond to the wave conditions of Table 3.1 and Table 3.2.
The runup of elevated waves ($T<10s$)

A regression analysis was applied to the data, and it is found that the general runup equation from Hall & Watts (1953) (2.23) is consistent with the runup of our experimental elevated waves for $\beta=1/20$, when $\alpha(\beta)\approx2.14$ and $f(\beta)\approx0.77$. Table 5.7 presents the range of values for $\alpha(\beta)$ and $f(\beta)$ from previous studies (Hall & Watts, 1953; Synolakis, 1987; Borthwick et al., 2006). Figure 5.9 shows that despite the range of slopes and experiments, all results look consistent. The values of $\alpha(\beta)$ and $f(\beta)$ are approximately constant throughout the range of slopes considered (Figure 5.10), potentially indicating that these empirical coefficients are not strongly dependent on the beach slope.

Table 5.7 shows that despite the range of slopes and experiments, values of $f(\beta)$ are relatively clustered ($0.582 < f(\beta) < 1.25$), and close to 1. This suggests that a linear relationship may be used to approximate runup. The value of $\alpha(\beta)$ for the non-breaking waves from (Synolakis, 1987) is much larger than the other experimental and numerical values, including experiments that were performed using a similar slope. This can be due to the fact that the corresponding expression for runup was derived analytically with the corresponding trend moving away from the data for higher values of $a/h$ (see Figure 2.4), with the runup being defined as the maximum surface elevation at the initial position of the shoreline in Synolakis (1987), which is different from the definition used in experiments.

The present experimental waves follows the same trend as Synolakis’ results for a range of $a/h$ ratios, but the new waves have a slightly higher runup (Figure 5.11). As reviewed in Chapter 2, the runup of large waves such as the ones considered here can be influenced by the friction coefficient of the slope. Synolakis (1986) used a smooth
aluminium beach whereas the present runup experiments were performed on a concrete slope with relatively greater roughness. Therefore, if affected, the runup of our waves should be slightly lower than the runup of Synolakis (1987), in a similar fashion to the results from Borthwick et al. (2006). As the contrary is seen, the friction coefficient of the slope is unlikely to be the cause of the observed discrepancies in runups. This suggests the amplitude is not the only parameter affecting the runup of a long propagating wave. The wavelength may be one explanation for this phenomenon, as pointed out by the initial observations of Synolakis (1986) on bores (section 2.2.4).

The runup relationship (2.29) from Baldock & Holmes (1999), which gives the runup as a function of flow height, underestimates the runup of the present experimental waves for $H_b = \alpha_{\text{nearshore}}$ and $C=2$ (total conversion of potential to kinetic energy), by a factor of 1.75, as shown in Figure 5.12. This difference is probably due to the fact that the height of the bore $H_b$ was taken here as being the wave height close to breaking, which is probably larger than the height of the resulting bore.

Figure 5.13 presents the runup results of the experiments and those of Synolakis (1987) and compares them to those predicted by the Li & Raichlen (2003) relationship (2.30) which relates runup to the energy balance after breaking and normalized wave height. The longer waves (this study) still runup slightly higher than the shorter waves. This is likely due to the fact that (2.30) has been derived empirically as a function of beach slope and $a/h$ (Li & Raichlen (2003)), instead of being derived from the full wave profile. Moreover, when $a/h > 0.45$, the trend disappears, so this relationship does not seem to be applicable to large wave amplitudes.
The runup of N-waves ($T<10s$)

Figure 5.11 shows runup from N-waves with relatively small $a/h$ ratios ($a/h<0.04$) match those of Synolakis (1987) solitary waves. This result is not in agreement with the theoretical findings from Tadepalli & Synolakis (1994), which suggest that N-waves runup higher than a solitary wave of the same amplitude. Even though only a small sample of N-waves was tested under small $a/h$ conditions. More data would be necessary to confirm this result. However for larger $a/h$ ratios, N-waves and elevated waves of similar wavelengths are seen to follow the same runup.

The runup of long waves ($T>10s$)

The runup of elevated waves and N-waves with periods greater than 10s is also shown in Figure 5.11. The periods of three of the longest elevated waves were 60s, 76s, and 94s. They are not seen to runup significantly higher than the other elevated waves from the present experiments. The long N-waves had periods of 105.4s, 106.5s, and 111.5s, however large $a/h$ ratios could not be reached at generation. The three waves are around the limit $a/h>0.045$. One displays a runup that follows the same trend as shorter N-waves, the others runup significantly higher. More data would be necessary to draw conclusions on the the runup of long N-waves.

5.2.2 Correlation between runup and measures of the wave form

It is now proposed to look at the correlation between runup and measures characterizing the wave form for short and long elevated, as well as N-waves. The
variables potentially having an influence on runup (section 2.6.1) are: wave amplitude, beach slope but also wave length and potential energy. We aim to find a relationship relating the parameters characterising the wave form to the runup $R$. The present data, but also the data from Synolakis (1987) is used for this purpose to test a large range of wavelengths. Moreover, because the slope is an invariant between these two datasets, only the parameters characterizing the wave form ($a, H, L, E_p$) are determining $R$. It is necessary to estimate the unknown parameters ($L$ and $E_p$) of other experimental waves for comparison with the present experiments. Knowing Synolakis (1987) wave amplitudes and the profiles correspond to a first order Boussinesq profile (Figure 3.19), it is possible to calculate wave lengths and potential energy for the dataset using (2.11) and (4.31), respectively.

The functional relationship between two (or more) parameters is usually assessed in coastal engineering using dimensional analysis and/or knowledge of the physical relationship between several parameters. In other disciplines, pure empirical methods are often used, as they always provide the best fit to the data. However, this approach generates results which are very specific to the data set, and the parameter combination may not take into account the physics of the phenomenon. Semi-empirical regression analysis is a good alternative combining the best of both approaches, as an analytical solution may not be possible due to the complex nature of long wave runup. Therefore, a two-phase semi-empirical approach is proposed. First, dimensional analysis is used to reduce the number of variables to be considered and allow to investigate the form of the relationship for runup. Secondly, regression analysis will be used to find the empirical parameters of the equation that give the best fit to the experimental data.
Referring back to sections 2.2.4 and 2.6.1, we saw the parameters that possibly influence wave runup $R$ are $H, a, a^-, L, h, \beta, E_p, \rho$ and $g$. $a$ corresponds to the positive amplitude of any wave, and $a^-$ corresponds to the negative amplitude of an N-wave; and $|a| + |a^-| = H$ (for an elevated wave, $a = H$). $E_p$ is the total potential energy of a given wave. For N-waves, this can be split into the potential energy of the trough, $E_p^-$, and the potential energy of the peak, $E_p^+$ (for elevated waves, $E_p^+ = E_p$). $\beta$ is the angle of the slope, $\rho$ is the water density, and $g$ is the acceleration due to gravity. Figure 5.14 and Figure 5.15 confirm that for the data at hand, some correlation between runup and all these parameters exists. One exception appears in Figure 5.15, where there is no clear trend between wavelength and runup. However for consistency with the analysis of elevated waves, this parameter will be included in the runup analysis of N-waves.

Dimensional analysis is a useful tool to identify, from a set of originally uncorrelated parameters, which dimensionless products can be formed and the possible functional relationships between them. The first step in a dimensional analysis is the careful choice of which parameters are independent (i.e. they cannot be expressed in terms of the other parameters). In this case, $H$ is a parameter dependent on $a$ and $a^-$ (see previous section). Equations (2.11) and (2.14) also indicate that $L, h$ can be related, at least in the case of first or second order solitary waves, to the previously mentioned parameters. A potential correlation between $L, h$ and $a$ has been checked for the elevated waves generated in these experiments, without success. Therefore, we will consider $L, h, a$ and $a^-$ to be independent. Without knowing which one of these parameters is the most influential on wave runup, a characteristic length parameter $L^*$ can be introduced for the dimensional analysis. As three dependent potential energies
can be considered (i.e. \(E_p, E_p^+, E_p^-\)), a characteristic energy \(E^*\) is also introduced. Considering the following independent variables: \(L^*, E^*, \beta, \rho, g\), the functional relationship between these variables can be expressed as:

\[
R = f(L^*, E^*, \beta, \rho, g).
\] (5.6)

The Buckingham Pi theorem (Hughes, 1993) states that the number of dimensionless products that can be formed from a set of \(n\) variables is equal to \(n-r\), with \(r\) being the number of dimensions at play in the system. \(\beta\) is removed from the set of variables to perform dimensional analysis as it has no dimension. \(f(\beta) = \Pi_3\) is a separate dimensionless product that would be taken into account in the functional relationship between variables, however in our case we are only interested in the wave form parameters in relation to runup, so \(\beta\) is a constant and this dimensionless product is dismissed. There are three dimensions at play: length [L], mass [M], and time [T]; and five variables left, including runup. So we can expect to find a set of two dimensionless products (\(\Pi_1\) and \(\Pi_2\)), which, following the same theorem, are related as follows:

\[
\Pi_1 = \Psi(\Pi_2). \tag{5.7}
\]

Each of the products \(\Pi\) in (5.7) will have the form given by the expression:

\[
\Pi = (L^*)^a (E^*)^b \rho^c g^d R^e. \tag{5.8}
\]

\(a, b, c, d,\) and \(e\) are integers. Then, each parameter in equation (5.8) is expressed as a function of its dimensions:

\[
L^*[=][L], \tag{5.9}
\]

\[
E[=][ML^2T^{-2}], \tag{5.10}
\]
Substituting equations (5.9) to (5.13) into the dimensional form of equation (5.8), we obtain:

$\Pi = [L]^a [ML^2T^{-2}]^b [ML^{-3}]^c [LT^{-2}]^d [L]^e.$

(5.14)

Rearranging the terms, equation (5.12) becomes:

$\Pi = [L]^{(a+2b-3c+d+e)} [M]^{(b+c)} [T]^{(-2b-2d)}.$

(5.15)

For $\Pi$ to be dimensionless, it is necessary for each of the powers in (5.12) to be zero thus creating a system of three equations and five unknowns to be solved:

\[
\begin{aligned}
(a + 2b - 3c + d + e &= 0 \\
b + c &= 0 \\
-2b - 2d &= 0
\end{aligned}
\]

(5.16)

Using system (5.16) $a$, $b$ and $c$ can be expressed as functions of $d$ and $e$ and replaced in (5.8):

$\Pi = (L^*)^{(4d-e)} E^{(-d)} \rho^d g^d R^e.$

(5.17)

In (5.17), the first dimensionless product $\Pi_1$ can be identified easily because $e$’s and $d$’s can be grouped, leaving only one dimensionless product $\Pi_2$ to be formed:

$\Pi_1 = R/L^*,$

(5.18)

$\Pi_2 = \frac{(L^*)^4 \rho g}{E}.$

(5.19)
Replacing (5.18) and (5.19) into (5.7) we obtain:

\[
\frac{R}{L^*} = \Psi \left( \frac{(L^*)^4 \rho g}{E} \right). 
\]  

(5.20)

By plotting \(\Pi_1\) against \(\Pi_2\) for a sample of simple combinations of \(L^*\), we can see that the data can be described by a power law (Figure 5.16). Therefore, we infer the functional relationship (5.20) to be of the form:

\[
\frac{R}{L^*} = K \left( \frac{L^*^4 \rho g}{E} \right)^k. 
\]  

(5.21)

In (5.21) \(K\) is a constant, \(K\) and \(k\) cannot be defined with the present dimensional analysis, however, runup has been shown to be a function of beach slope so one can predict one or both of these values would correspond to a function of \(\beta\). \(L^*\) can be the flume width \((w)\), wave amplitude \((a\) or \(a^-)\), height \((H)\), wavelength \((L)\), or water depth \((h)\). Because the present experiments were carried out in 2 dimensions, \(w\) can be taken as a unit width so equation (5.21) can possibly apply to a number of combinations of three possible variables \(L^*\). Regression analysis is necessary to identify the forms of equation (5.21) that can give a satisfactory fit to the data available.

5.2.2.2 Regression analysis

Dimensional analysis has shown that a power law (equation (5.21)) may well describe physically the relationship between wave runup and a number of characteristic wave parameters. However, it did not allow the determination of the empirical multiplier \(K\) and the power \(k\). Moreover, the nature of the characteristic length \(L^*\) needs to be determined, as well as the nature of \(E^*\) for N-waves. Finally, dimensional analysis does
not quantify the uncertainty associated with the equation obtained. Regression analysis allows for the determination of the optimum values for $K$ and $k$ and the most significant length parameters $L^*$ to be taken into account, by reducing the error associated with the prediction. Moreover, it gives an indication of the uncertainty associated with the given equation.

Simple linear regression can be performed using the variables in equation (5.21); if the latter is linearised as follows:

$$
\log \left( \frac{R}{L^*} \right) = \log K + k \log \left( \frac{L^* \rho g}{E} \right) + \epsilon. 
$$

(5.22)

In (5.22), $\log K$ and $k$ represent the regression coefficients of the model. $y = \log \left( \frac{R}{L^*} \right)$ is the response variable, $x = \log \left( \frac{L^* \rho g}{E} \right)$ is the explanatory variable and $\epsilon$ represents the error associated with the prediction. For every fixed value of $x$, the errors are assumed to be independent random quantities normally distributed with mean zero and a common variance $\sigma^2$. These assumptions indicate that the estimated values of $\log K$ and $k$ are unbiased (Chatterjee et al., 2000), and they need to be checked as part of the regression analysis.

Regression analysis consists in obtaining the best estimation of values for $\log K$ and $k$. These are calculated by reducing the total error between the response data and the predicted response. For this purpose, the residuals $e_i$ have to be calculated and the sum of squared residuals $SSE$ has to be minimized:

$$
e_i = y_i - \hat{y}_i, 
$$

(5.23)

$$
SSE = \sum_{i=1}^{n} e_i^2. 
$$

(5.24)
In (5.23) and (5.24), for \( n \) data points, \( y_i \) is one observed response to an input variable \( x_i \), and \( \hat{y}_i \) is the response predicted by (5.22) (for \( \varepsilon \approx 0 \)).

We define the uncertainty associated with (5.22) as being minimal when the coefficient of determination, \( R^2 \), approaches 1 and the mean of the deviations (squares) from the fit \( \bar{e} \) is zero. \( R^2 \) indicates the proportion of the total variability in the response variable which is accounted for by the predictor variable (Chatterjee et al., 2000). The coefficient of determination is calculated using the total sum of squares \( SST \):

\[
SST = \sum_{i=1}^{n} (y_i - \bar{y})^2, \tag{5.25}
\]

\[
R^2 = 1 - (SSE/SST), \tag{5.26}
\]

\[
\bar{e} = \frac{\sum_{i=1}^{n} e_i}{n}. \tag{5.27}
\]

One way to deal with uncertainty is to explore the source of error, by assessing the dependence of the residuals on one or more parameters and refine the original fit. Another way is to define the confidence intervals associated with a particular fit. The latter will be applied later in this section.

Potentially, one runup relationship taking into account all the relevant parameters may describe runup well. However, runup laws may vary between different types of waves, so the wave data is divided into different populations. Each wave group is identified with a letter, as listed in Table 5.8. Each wave group is used to find candidate runup equations, which are then compared.

Waves smaller than 10s are chosen as the “short” waves, in comparison with the waves periods typically generated in other studies (see section 2.4). Because we are not only interested in the influence of the wavelength, but also the wave form, we choose to
group waves primarily by type (i.e. elevated waves, N-waves), not by length. If all short or long waves do have a similar behaviour, the grouping above will still work (i.e. similar results will be associated to group A and D).

*Short elevated waves (group A)*

The first subset of data to be used in the regression is short elevated waves. Relationships for runup found using this subset should be comparable to existing equations that are mostly based on data such as this. Synolakis (1987) data is added to the current experimental data in order to enrich the database. The combinations of $k$, $K$, $L$, $h$, and $a$ resulting in a strong $R^2$, a zero mean error, and satisfying all the linearity assumptions will be kept. Table 5.9 presents the regression coefficients, characteristic lengths variables and uncertainties associated with the combinations displaying a significant degree of linearity between $x$ and $y$ ($R^2 \geq 0.90$).

When outliers were present, they were removed from the data set and the regression analysis was repeated. In the present analysis, outliers are defined as data which associated residuals are located more than 2.5 standard deviations away from their mean $\bar{e}$.

The normal distribution of residuals in Table 5.9 has been checked in two ways. First, a normal probability plot was used to graphically assess the fit of the data to a normal distribution. If the data only deviated slightly from the normal distribution, an additional conservative statistical test of normality (Lilliefors, 1967) was performed to test the null hypothesis ($H_0$: the residuals are normally distributed):
\[ D = \max_{e} |F^*(e) - S_n(e)|. \quad (5.28) \]

In (5.28), \( F^*(e) \) is the cumulative normal distribution function with mean \( \bar{e} \) and standard deviation \( s^2_e \), and \( S_n(e) \) is the sample’s cumulative distribution function. To validate \( H_0 \), \( D \) shall not exceed a critical value, the latter is determined according to the sample size \( n \) and the level of significance desired \( \alpha \). \( \alpha \) represents the probability that the test will reject \( H_0 \) when it is in fact true (Moore & McCabe, 2003). In the examination of residuals, we choose to detect not only pure normal distributions but close to normal distributions so we set \( \alpha = 0.001 \). If the residuals follow a normal distribution, the equation is rated as “Yes” in the last column of Table 5.9.

In addition, standardized residuals have been plotted against the predictor variable to detect any violation of the linearity assumption: if the assumption holds, no correlation should be detected. The standardized residuals \( z_i \) are easier to interpret than the ordinary residuals \( e_i \), as they are centered and scaled as follows:

\[ z_i = \frac{e_i - \bar{e}}{\sigma}. \quad (5.29) \]

In (5.29), the standard deviation of the residuals \( \sigma \) can be calculated from the estimated variance of the residuals, and given by:

\[ \sigma^2 = \frac{SSE}{(n - 2)}. \quad (5.30) \]

For short elevated waves, grouping of the residuals between two distinct populations (i.e. data from Synolakis (1987) and the present data) can be observed (Figure 5.17), and arguably one population does not appear random. However, considering all the residuals together, there is no overall trend. Therefore, the residuals are considered to be uncorrelated. As seen in Table 5.9, in some cases the residuals display a trend,
identified as being quadratic. This suggests that for these combinations of parameters, a
different type of relationship between $x$ and $y$ has to be considered, by adding a
quadratic term to the right side of equation (5.22). Moreover, the plot of the residuals
against $x$ allows to visually check that there is no heterogeneity of $\sigma^2$.

The results of Table 5.9 indicate that for short elevated waves, there is a unique
combination of the parameters $a$, $h$, $L$ and $E_p$ that gives a strong linear relationship
($R^2 = 0.96$) with unbiased estimates of $\log K$ and $k$:

$$ log \left( \frac{R}{L} \right) = -1.26 + 0.81 \log \left( \frac{a^3 \rho g}{E_p} \right). $$

(5.31)

The regression coefficients in (5.31) are close to -1 and 1 and are tested against the
hypotheses: $H_{01}: \log K = -1$ and $H_{02}: k = 1$. If either or both of the null hypotheses
are not rejected, a simplified form of the equation can be used.

The testing of these hypotheses is done using a $t$-test as described in Chatterjee et al.
(2000). A $t$-test can be used in any case if $n \gg 15$; if $n \approx 15$ and outliers have been
removed, or if $n < 15$ and the response variable is normally distributed (Moore & Mc
Cabe, 2003). In other cases, this test should not be used. For wave group A, (one outlier
for equation (5.31)), $n = 97$. The test statistic requires the calculation of the standard
errors associated with $k$ and $\log K$:

$$ SE_{\log K} = \sqrt{\frac{\sigma^2}{n} + \frac{\sum x^2}{\sum_i=(x_i-\bar{x})^2}}, $$

(5.32)

$$ SE_k = \sqrt{\frac{\sigma^2}{\sum_i=(x_i-\bar{x})^2}}, $$

(5.33)
\[ t_{\log K} = \frac{\log K - C_{\log K}}{SE_{\log K}}, \quad (5.34) \]
\[ t_k = \frac{k - C_k}{SE_k}. \quad (5.35) \]

In (5.32) and (5.33), an unbiased estimate of the variance of the residuals is given by (5.30), and the \( t \) statistics (5.34) and (5.35) are distributed as a Student’s \( t \) with \( n-2 \) degrees of freedom (Chatterjee et al., 2000). \( C_{\log K} \) and \( C_k \) are the constants chosen, to be compared with \( \log K \) and \( k \), respectively. If the magnitude of the \( t \) statistic is greater than the critical value from the \( t \) distribution \( t_{(n-2,\alpha/2)} \), then the corresponding null hypothesis (in our case, \( H_{01} \) or \( H_{02} \)) is rejected so \( \log K \) and \( k \) are significantly different from -1 and 1, respectively. The \( t \) values distribution table (used for all wave groups) can be found in NIST/SEMATECH (2006). We choose a level of significance \( \alpha = 0.01 \), so \( t_{(95,0.005)} = 2.629 \). We obtain \( |t_{\log K}| = 8.08 \) and \( |t_k| = 12.11 \). Hence, both hypotheses are rejected and equation (5.31) cannot be simplified.

When the errors are normally distributed, it is possible to construct the confidence intervals for the regression parameters (\( \log K \) and \( k \)) of equation (5.31). Confidence intervals are expected to contain the true value of the parameter of interest at the level of confidence chosen. The \( (1 - \alpha') \).100% confidence interval for \( \log K \) and \( k \) respectively are given by (Chatterjee et al., 2000):

\[ \log K \pm t_{(n-2,\alpha'/2)}SE_{\log K}, \quad (5.36) \]
\[ k \pm t_{(n-2,\alpha'/2)}SE_k. \quad (5.37) \]

We choose here to obtain 95% confidence intervals for \( \log K \) and \( k \) using (5.36) and (5.37); these are calculated in Table 5.16.
The $(1 - \alpha'').100\%$ confidence interval for the prediction of response variables $\hat{y}_{0t}$ for a range of chosen variables $x_{0t}$, is:

\[ \hat{y}_{0t} \pm t_{(n-2,\alpha''/2)}SE_{\hat{y}_{0t}}, \]  

(5.38)

with

\[ SE_{\hat{y}_{0t}} = \sqrt{\sigma^2 \left( 1 + \frac{1}{n} + \frac{(x_{0t}-\bar{x})^2}{\sum_{i=1}^{n}(x_i-\bar{x})^2} \right)}. \]  

(5.39)

Because the regression results are only valid between the minimum and maximum values of $x$ tested, confidence intervals for the whole range of values $x_{\min} < x_{0t} < x_{\max}$ are constructed and plotted in Figure 5.18 along with the regression fit (5.31) and the confidence intervals for the regression parameters. Here we choose $\alpha' = \alpha''$ so 95\% confidence intervals for the regression parameters and the prediction variable are obtained.

The same procedure is applied to all the other groups of waves.

**Long elevated waves (group B)**

Table 5.10 presents the results obtained for long elevated waves. The results indicate that for long elevated waves, there are two combinations of the parameters $a, h, L$ and $E_p$ that satisfy all the linearity assumptions (residual plots are presented in Figure 5.19). Combination 3 would give results consistent with wave group A. However, the combination giving the strongest relationship ($R^2 = 0.96$) with unbiased estimates of $logK$ and $k$ is:
\[
\log \left( \frac{R}{h} \right) = 1.71 - 0.56 \log \left( \frac{ahpg}{E_p} \right). \tag{5.40}
\]

In wave group B, \( n = 8 \) and the response variable is normally distributed, therefore a \( t \)-test can be used. The regression coefficients in (5.40) can be tested against the hypotheses: \( H_{01}: \log K = 1 \) and \( H_{02}: k = 0 \). The \( t \) statistics (5.34) and (5.35) are calculated for the regression parameters in equation (5.40) and compared to the critical values of the \( t \) distribution: \( t_{(6, 0.005)} = 3.707 \), \( |t_k| = 11.83 \), and \( |t_{\log K}| = 3.08 \). Therefore, \( H_{01} \) is validated and \( H_{02} \) is rejected, so (5.40) becomes:

\[
\log \left( \frac{R}{h} \right) = 1 - 0.56 \log \left( \frac{ahpg}{E_p} \right). \tag{5.41}
\]

The 95% confidence intervals for \( \log K \) and \( k \) using (5.36) and (5.37) are given in Table 5.16. Confidence intervals as defined by equation (5.38) for the whole range of values \( x_{min} < x_{0i} < x_{max} \) are constructed and plotted in Figure 5.20 along with the regression fit (5.41) and the confidence intervals for the regression parameters.

**All elevated waves (group C)**

Using the same methodology, we now intend to find a significant linear relationship \( (R^2 \geq 0.90) \) describing the runup of all elevated waves (group C) with an acceptable level of uncertainty. Table 5.11 summarizes the results obtained.

The results of Table 5.11 indicate that for all elevated waves, there are two combinations of the parameters \( a, h, L \) and \( E_p \) satisfying all the linearity assumptions, giving a strong linear relationship \( (R^2 = 0.96) \). However, the errors for combination 4 are one order of magnitude smaller than for combination 5. Therefore, combination 4 is
chosen as being the best fit to the data in group C. The residual plots are presented in Figure 5.21.

\[ \log\left(\frac{R}{L}\right) = -1.27 + 0.87\log\left(\frac{a^2\rho g}{E_p}\right). \]  
\hspace{2cm} (5.42)

No outliers were removed so \( n = 106 \). The regression coefficients in (5.42) can be tested against the hypotheses: \( H_{01}: \log K = -1 \), \( H_{02}: k = 1 \). The \( t \) statistics (5.34) and (5.35) are calculated for the parameters in equation (5.42) and compared to the critical value of the \( t \) distribution: \( t_{(104,0.005)} = 2.576 \), \( |t_k| = 7.7 \) and \( |t_{\log K}| = 7.0 \). These results show that both hypotheses are rejected. Therefore equation (5.42) cannot be simplified. The 95% confidence intervals for \( \log K \) and \( k \) using (5.36) and (5.37) are given in Table 5.16. The confidence intervals for \( \log K \) and \( k \) for equation (5.42) include the values of \( \log K \) and \( k \) for equation (5.31), therefore the two equations can be considered similar. Confidence intervals as defined by equation (5.38) for the whole range of values \( x_{\min} < x_{0i} < x_{\max} \) are constructed and plotted in Figure 5.22 along with the regression fit (5.42) and the confidence intervals for the regression parameters.

*Short N-waves (group D)*

Runup relationships for N-waves are derived in a similar way; this time adding the negative amplitude \( a^- \), the potential energy of the peak \( E_p^+ \) and the potential energy of the trough \( E_p^- \) to the set of parameters tested for elevated waves. Results for short N-waves are presented in Table 5.12.

The results of Table 5.12 indicate that for short N-waves, the strongest relationship \( (R^2 = 0.98) \) verifying all the linearity assumptions can be expressed as:
\begin{equation}
\log \left( \frac{R}{h} \right) = 1.75 - 0.4 \log \left( \frac{Lh^2 \rho g}{\varepsilon^2} \right). \tag{5.43}
\end{equation}

For wave group D, after removal of outliers for equation (5.43), \( n = 16 \). The corresponding residual plot is presented in Figure 5.23. Arguably a slight quadratic trend might be detected on the residual plot, however it is very weak so a linear relationship is still considered to be suitable to describe the data. The regression coefficients in (5.43) can be tested against the hypotheses: \( H_{01} : \log K = 1 \) and \( H_{02} : k = 0 \). The \( t \) statistics (5.34) and (5.35) are calculated for the regression parameters in equation (5.43) and compared to the critical values of the \( t \) distribution for the corresponding degrees of freedom: \( t_{14,0.005} = 2.977 \), \( |t_k| = 25.9 \) and \( |t_{\log K}| = 6.33 \). These results show that both hypotheses are rejected, therefore equation (5.43) cannot be simplified.

The 95% confidence intervals for \( \log K \) and \( k \) using (5.36) and (5.37) are given in Table 5.16. Confidence intervals as defined by equation (5.38) for the whole range of values \( x_{min} < x_{0i} < x_{max} \) are constructed and plotted in Figure 5.24, along with the regression fit (5.43) and the confidence intervals for the regression parameters.

\textit{Long N-waves (group E)}

For long N-waves, only three different waves gave reliable runup data. Because the regression analysis was carried out using this very small number of points, a number of very strong linear correlations could be observed (Table 5.13). However, a larger number of data points would be needed to truly assess the strength of the correlation, as
well as the characteristics of the residuals. The results presented in Table 5.13 need to be interpreted with caution.

The results of Table 5.13 indicate that for long N-waves, there are two strong relationships \( R^2 = 1 \) verifying all the linearity assumptions and with the same mean error. However, if we consider the coefficient of determination for combination 3 with higher precision we obtain \( R^2 = 0.997 \), against \( R^2 = 0.996 \) for combination 5. Therefore, we choose combination 3, for which the residual plots are shown in Figure 5.25.

\[
\log \left( \frac{a}{a^*} \right) = -1.29 - 1.17 \log \left( \frac{\bar{a} \rho_p}{\bar{E}_p} \right). 
\] (5.44)

The regression coefficients in (5.44) cannot be tested using the \( t \) statistics (5.34) and (5.35) because \( n = 3 \) and the response variable does not appear normally distributed. However, because the errors appear normally distributed, it is possible to calculate confidence intervals for equation (5.44), given in Table 5.16.

Confidence intervals as defined by equation (5.38) for the whole range of values \( x_{min} < x_0 < x_{max} \) are constructed and plotted in Figure 5.26 along with the regression fit (5.44) and the confidence intervals for the regression parameters.

*All N-waves (group F)*

There are two results in Table 5.14 giving a very strong relationship \( R^2 = 0.99 \) between runup and the predictor variable (combinations 6 and 9). Given the moderate size of the sample, we decide to focus on the combination giving the smallest number of outliers, which corresponds to combination 6 (residual plots are shown in Figure 247.
5.27). Moreover, this combination also corresponds to the smallest mean error by one order of magnitude, and gives the following relationship:

\[
\log \left( \frac{R}{a^{-}} \right) = 2.37 - 0.45 \log \left( \frac{(Lu)^2 \rho g}{E_p^+} \right).
\]

(5.45)

For wave group F, after removal of outliers for equation (5.45), \( n = 20 \). The regression can be tested against the hypotheses: \( H_0_1: \log K = 1 \), \( H_0_2: k = 0 \). The \( t \) statistics (5.34) and (5.35) are calculated for the parameters in equation (5.45) and compared to the critical values of the \( t \) distribution for the corresponding degrees of freedom:

\[
t_{(38,0.005)} = 2.878, \ |t_k| = 35.59 \text{ and } |t_{\log K}| = 33.12.
\]

Both hypotheses are rejected therefore equation (5.45) cannot be simplified.

The 95% confidence intervals for the regression parameters in equation (5.45) are calculated and presented in Table 5.16.

Confidence intervals as defined by equation (5.38) for the whole range of values \( x_{min} < x_{0i} < x_{max} \) are constructed and plotted in Figure 5.28 along with the regression fit (5.45) and the confidence intervals for the regression parameters.

All waves (group G)

This analysis is applied one final time to all waves (group G, Table 5.8). We now intend to find a strong linear relationship \( (R^2 \geq 0.90) \) describing the runup of all waves, with an acceptable level of uncertainty. The parameters \( a^{-} \) and \( E_p^+ \) are now removed for the parameters to be tested, as they would not be useful to describe the runup of elevated waves. However, \( E_p \) and \( E_p^+ \) will still be tested separately because in
the absence of $E_p^-$, for N-waves, these two parameters are independent. The same applies to $a$ and $H$. Table 5.15 summarizes the results obtained.

In Table 5.15, there are five combinations (1, 2, 3, 6 and 7) of parameters satisfying the linearity assumptions, giving a strong relationship ($R^2 = 0.97$). The smallest number of outliers is given by combinations 6 and 7 and the smallest mean error by combination 7. The residual plots for this combination of parameters are shown in Figure 5.29.

$$\log \left( \frac{R}{L} \right) = -1.28 + 0.87 \log \left( \frac{a^3 \rho g}{E_p} \right). \quad (5.46)$$

For wave group G, after removal of outliers for equation (5.46), $n = 126$. The regression coefficients can be tested against the hypotheses: $H_{01}: \log K = -1$, $H_{02}: k = 1$. The $t$ statistics (5.34) and (5.35) are calculated for the parameters in equation (5.46) and compared to the critical values of the $t$ distribution for the corresponding degrees of freedom: $t_{(124,0.005)} = 2.576$, $|t_k| = 9.78$ and $|t_{\log K}| = 7.24$. All hypotheses are rejected, therefore (5.46) cannot be simplified.

The 95% confidence intervals for the regression parameters in equation (5.46) are calculated and presented in Table 5.16.

Confidence intervals as defined by equation (5.38) for the whole range of values $x_{\min} < x_{0_i} < x_{\max}$ are constructed and plotted in Figure 5.30 along with the regression fit (5.46) and the confidence intervals for the regression parameters.

The equation for wave group G is greatly influenced by the high number of data points for elevated waves ($n = 106$ for wave group C equation (5.42)), in comparison with N-waves (without outliers, $n = 20$ for wave group F equation (5.45)). This results in equations (5.42) and (5.46) to be equivalent.
Three methods have been explored to solve this issue, however none of them were proven adequate to treat the data without introducing error. The first method would consist in randomly sub-sampling the waves in group A, to obtain a smaller sample (i.e. \( n = 10 \)), comparable in size to group B. Group C would then be formed using group A sub-sample and group B. However, this method would dismiss a large number of data points in group A, as well as modifying the regression results obtained for the total number of points and increasing the uncertainty (example shown in Figure 5.31). The second method would consist in inversely weighing the points in the smaller groups (i.e. by \( 1/\sqrt{n} \)). However this would be equivalent to giving significantly more confidence to the points of the smaller samples only because they belong to such samples, which would not physically make sense. A final method would create (for group A) a chosen number of bins of equal size in \( x \), and average in \( y \) all points falling into each bin to reduce the total number of points. However, this method cannot be applied to data points that are independent (Motulsky and Christopoulos, 2003), which is the case for the present data. This method would give wrong regression results, particularly if the number of points in each bin varies.

Therefore, the aim of investigating a possible common relationship for all wave forms would require more long elevated and N-wave data. However, such a relationship may not exist. Indeed, the previous results indicate that the runup of elevated waves and the runup of N-waves should be treated as two separate processes, as the negative components of N-waves \((a^-, E^-)\) often appear in the best fit.

Finally, the grouping of data in two distinct regions for wave groups A, C and D (corresponding to Synolakis (1987) data and the present data) indicate that the range of
parameters for these two groups did not overlap. An additional wave group (i.e. “very short” waves) may have been considered.

Regression analysis: summary and discussion

Laws of the form of equation (5.21) are summarized in Table 5.17, with confidence intervals for \( k \) and \( K \), for each group of waves. The results from this table are discussed below.

A number of previous studies on runup have determined that \( R \sim a \) (see section 2.2.4 and Table 2.2). Posing \( 0.81 \leq k' \leq 0.87 \), equations (5.65), (5.67) and (5.71) imply that:

\[
\frac{R}{l} \sim \left( \frac{\rho g a^3}{E_p} \right)^{k'}.
\]  

(5.47)

Moreover, if we consider that \( E_p \sim \rho g a^2 L \), equation (5.47) becomes:

\[
R \sim a \left( \frac{a}{L} \right)^{k' - 1}.
\]  

(5.48)

Because \(-0.19 \leq k' - 1 \leq -0.13\), the term \((a/L)^{k' - 1}\) is small. Therefore, this result is consistent with previous studies for short elevated waves, i.e. the runup approximately scales as the amplitude of the wave. A similar reasoning can be applied to the relationship obtained for short N-waves, with equation (5.68) implying that:

\[
\frac{R}{h} \sim \left( \frac{E_p}{\rho g L h^2} \right)^{\frac{1}{2}}.
\]  

(5.49)

So
Replacing \( h \) by \( a^- \), the same result can be obtained for equation (5.70), showing that for N-waves also, \( R \sim a \). However, it is believed the constant term \( K \) is dependent on the potential energy (therefore on the wave form) so it is more accurate to take into account this parameter in the estimation of runup. These results do not indicate that N-waves runup higher than elevated waves. The runup of long N-waves is not explained here due to the difficulty of interpretation of the regression results.

For long elevated waves, the same combination of parameters as for (5.65) and (5.71) would have also given a regression result satisfying all the linearity assumptions. However, the best fit was given by equation (5.66), which indicates a contribution of the wavelength to the same order as the amplitude. This result can be explained if we consider that the potential energy \( E_{ps} \) of a mass of water \( m \) as it climbs up a slope \( \theta \) is:

\[
E_{ps} \approx \beta Rm\cos \theta.
\]

In two dimensions, \( m \) can be approximated by \( m \approx \rho aL \). Moreover, with \( \beta \) being constant and assuming \( E_{ps} \sim E_p \), in terms of runup equation (5.51) is equivalent to:

\[
\frac{R}{h} \sim \frac{E_p}{\alpha \rho g^2}.
\]

Simplifying equation (5.52) in the same way as for equations (5.65), (5.67) and (5.71), we obtain \( R \sim \sqrt{\alpha} \). The present results suggest that there is a stronger dependence on wavelength for long waves than for short waves, indicating the presence of two different regimes. Interestingly, these two regimes (\( R \sim a \) and \( R \sim \sqrt{\alpha} \)) are similar to the ones presented by Synolakis (1987), i.e. breaking and non-breaking. Indeed, if a power
law is fitted to these two regimes (using a simple least squares regression), we find that

\[ R \approx 4a \] for non breaking waves, and \[ R \approx \sqrt{h}a \] for breaking waves (Figure 5.32).

This can be explained considering that, for Synolakis (1987) data, \( \frac{L}{h} \) increases with \( \frac{a}{h} \) (Figure 5.33). When the \( \frac{h}{L} \) ratio becomes smaller, the relative bottom friction increases, which eventually leads to the wave breaking. The study from Borthwick et al. (2006) also shows that runup regimes are dependent on friction (see section 2.2.4).

Therefore, both studies’ results are consistent with the present results and the idea that runup regimes are dependent on wavelength. The weaker dependence on amplitude for long waves may be due to the large amount of wave energy reflected back during the runup process: Table 5.3 shows that the energy reflected back could be up to half that of the direct wave.

The rundown results (Figure 5.34) do not display a strong correlation between runup and rundown, for short N-waves. For long N-waves, not enough data was collected to give conclusive results. However, drawing lines of best fit through the short and long N-wave data, respectively, would indicate a decrease in runup with an increase in rundown. This would be consistent with the idea that the runup decreases as the negative amplitude and potential energy increase, as shown by the corresponding trends in Figure 5.15. It has to be noted that the range of troughs that could be generated, especially for short waves, was small (see section 5.1.2) so such results should be interpreted with caution.

5.3 Other results

5.3.1 Flow velocities
The purpose of this section is:

- to give a more detailed insight into how the flow is coupled to the wave movement,

- to explain the motion in the channel prior to the arrival of the first wave.

Measurements carried out with the velocity probes effectively measure the velocities at the bottom of the flume. They therefore correspond to a local average of the flow velocity over the cross sectional area of the probe, and over time, although it is believed the measurements are not affected by the bottom shear layer (see section 3.2.5). Moreover, all velocity probes are located downstream of the breaking location of the waves so the horizontal flow velocity can be considered uniform as shown in section 2.2.4 (Lin et al., 1999). Therefore, the velocities recorded are representative of the actual flow velocity.

Velocity time history

Figure 5.35 represents, for a short elevated wave, the wave height recorded next to the wave generator (Offshore 1) and the velocity times series recorded close to the mid-point of the slope (velocity probe p2). The first velocity peak occurs at the expected wave arrival time at this position, using the empirically determined phase speed (5.2) with $\lambda=0.88$ for the wave velocity.

Equation (4.7) provides an estimate of the maximum particle velocity that can be expected. Lin et al. (1999) showed the horizontal velocity is only constant at the early
shoaling stages. When the wave reaches a large steepness further up the slope the horizontal velocity is not constant anymore. Because the velocity probes used in these experiments recorded bottom velocities and were located downstream of the breaking location of the waves, it is unlikely this approximation would predict the peak velocities observed in the data.

An alternative approach is to consider that as the waves are broken when they reach the velocity probes, the resulting flow has the characteristics of a bore. Lin et al. (1999) have shown that the horizontal velocity is nearly constant over a vertical column in a broken wave (see section 2.2.4), with the magnitude of the horizontal velocities being larger at the tip of the bore. In this case it is reasonable to consider bottom velocities are representative of the horizontal velocities of the flow. Baldock & Holmes (1999) give the equation of particle motion at the tip of a bore, at the location of the shoreline $X_s$ as:

$$X_s(t) = U_s t - \frac{1}{2} g t^2 \sin \beta. \quad (5.53)$$

In (5.53), $U_s = C \sqrt{g H_b}$ is the initial shoreline velocity, and $\beta$ is the angle of the slope (in our case, $\beta = 1/20$). $t$ is the time elapsed since bore collapse. The velocity of a particle can therefore be expressed as:

$$\frac{\partial X_s}{\partial t} = U_s - gt(\sin \beta). \quad (5.54)$$

In the experiments, the surface elevation data show the waves break between Nearshore 4 and Nearshore 5, therefore we take $H_b = a_{Nearshore4}$, and $C=2$. Nearshore 7 is the wave probe closest to the shoreline, therefore the velocities estimated at the time the wave reaches the probe are likely to be representative of the velocities recorded close to
the shoreline. Using (5.54) to estimate the particle velocity at the time it reaches Nearshore 7, we obtain a value of 58.95 cm.s\(^{-1}\) for the wave in Figure 5.35, which corresponds to the value of the first peak recorded by the velocity probe.

However in the case of long waves (i.e. Figure 4.3), an initial small velocity peak which does not correspond to the main wave can be observed. Knowing that the whole flume is set in motion prior to the arrival of the main positive peak (particularly in the case of long waves), this early rise in water level may be due to the effect of the wave being felt from the first few second of wave generation at the beach. It is possible to estimate the velocity \(V_s\) due to the initial rise of water level over the slope, as the sum of \(u\) and \(v\). The variation of water level \(da\) over the slope corresponds to the rise in water level at Offshore 1 over the time \(dt\), total time of the increase in velocity as observed on the record. Because \(\beta\) is small (\(\tan \beta = 1/20\)), \(u \sim V_s\) so \(V_s\) can be expressed as:

\[
V_s = \cot \beta v = \cot \beta \frac{da}{dt}. \tag{5.55}
\]

Taking the example of Figure 4.3, with \(da = 0.8\) cm and \(dt = 3.9\) s, the initial velocity predicted using (5.55) is \(V_s = 4.1\) cm.s\(^{-1}\), which is close to the value of the observed velocity (6.7 cm.s\(^{-1}\)) for the first peak, considering that (5.55) is a simplified model of the velocity over the slope. Therefore, the initial peak observed on the record is likely due to the initial motion of the water at the other end of the flume, as described above.

\[Evolution\ of\ peak\ velocities\ at\ the\ moving\ shoreline\]

The method described in section 4.5 effectively removed peak velocity outliers, as
shown in Figure 5.36. This figure shows that for all waves an increase in peak particle velocities has been recorded in the shoreline region. These observations remind of the ones from Synolakis (1987), Carrier et al. (2003), and Synolakis & Bernard (2006) regarding the increase of velocities at the shoreline location (see section 2.5.2). An apparent linear relationship of velocities versus distance can be fitted for all waves, with a coefficient of determination $0.78 < R^2 < 0.90$. The rate of increase $\partial u_{\text{max}}/\partial x$ is similar for both types of elevated waves (short and long), and is more than twice as large for long and short N-waves.

This result ($\partial u_{\text{max}}/\partial x > 0$) is initially surprising if we know, from (5.53) and (5.54), that the acceleration of the particle is:

$$\frac{\partial^2 x_f}{\partial t^2} = -gsin\beta. \quad (5.56)$$

Equation (5.56) shows the acceleration of the fluid particle at the tip of a bore over a slope is always negative (in the case of our experiments $\left|\frac{\partial^2 x_f}{\partial t^2}\right| = 48.9\, \text{cm.s}^{-2}$). A likely explanation for this phenomenon is that the total acceleration (5.56) (also given by the Navier Stokes equation (2.2)) has a local component $\partial u/\partial t$ and a convective component $u\partial u/\partial x$, so the apparent acceleration with distance measured from the probes may not correspond to the total acceleration but to either of those components. If we consider motion in the $x$ direction and integrate over depth, from (2.2) we obtain:

$$h \frac{\partial u}{\partial t} + hu \frac{\partial u}{\partial x} = -gsin\beta. \quad (5.57)$$
Moreover,

\[ h \frac{\partial u}{\partial t} = \frac{\partial (uh)}{\partial t} - u \frac{\partial h}{\partial t}, \quad (5.58) \]

\[ hu \frac{\partial u}{\partial x} = \frac{\partial (hu^2)}{\partial x} - u \frac{\partial (uh)}{\partial x}. \quad (5.59) \]

Because mass is conserved (equation (2.1)), \( \frac{\partial h}{\partial t} + \frac{\partial (uh)}{\partial x} = 0 \), substituting (5.58) and (5.59) in (5.57) gives:

\[ \frac{\partial (uh)}{\partial t} + u^2 \frac{\partial h}{\partial x} + h \frac{\partial u^2}{\partial x} = -gsin\beta. \quad (5.60) \]

Considering only the space variations of \( u \), let’s compare the second and third terms of (5.60). The second term is always negative, because the flow gets thinner as it travels up the slope \( \left( \frac{\partial h}{\partial x} < 0 \right) \). The third term on the other hand can be positive. Even though this term is not equal to \( \frac{\partial u_{max}}{\partial x} \), equation (5.60) shows that even if the total acceleration of a particle moving up the slope is negative, positive components of acceleration can be present.

An attempt to understand the respective contributions of convective and local accelerations is made. Table 5.18 summarizes the estimated convective accelerations \( \bar{u}_{max} \frac{\partial u_{max}}{\partial x} \) at the shoreline retrieved from the data in Figure 5.36, for the total distance \( dX \) between the first and the last probe recording. When compared to the acceleration of gravity, it can be noted that over a distance, all particles are only weakly accelerating.

Figure 5.37 represents the evolution of estimated convective accelerations with distance for changes of velocities with smaller distances \( dx \). Convective accelerations can take positive and negative values, although the present estimations indicate that the convective accelerations are often positive. Moreover, it can be seen that, in contrast
with the relatively low values from Table 5.18, convective accelerations can be locally quite high.

The local acceleration \( \partial |u| / \partial t \) is plotted in Figure 5.38 with the velocity \( u \), for the same data as in Figure 5.35. The filtered acceleration \( \partial |u| / \partial t \) corresponding to the action of the initial wave on the beach is plotted on Figure 5.39, along with the value of the minimum local acceleration and the value of the total acceleration. While the initial acceleration peak is due to the fast moving front of the broken wave (Shen & Meyer, 1963; Yeh, 1991), the minimum local acceleration corresponds to the bulk of the flow climbing up the beach. We can see that the local acceleration at this location contributes to approximately a third of the total acceleration over the slope.

These results indicate that while a mass of water climbing up a slope essentially descelerates, the convective and local accelerations can be alternatively positive and negative.

These observations cannot be directly compared to the observations from Synolakis (1987) and Synolakis & Bernard (2006) (see section 2.5.2), because in these studies, the horizontal velocities \( u \) were not directly measured. However, it has been shown by Shen & Meyer (1963), and Yeh (1991) that bore collapse involves the rapid conversion of potential to kinetic energy, triggering an acceleration of the fluid velocity at the bore front. Bores are often assumed to collapse at the shoreline (Ballock & Holmes, 1999), which would coincide with the apparent acceleration of the wave at this location, as reported in field and laboratory studies. The study from Carrier et al. (2003) estimated flow velocities variations with space and time (see Figure 2.11), and we can see that there are local changes in the flow velocity magnitude for all types of waves, which is consistent with the present results.
5.3.2 Wave pressures on structures

In their structural design guidelines for tsunami, FEMA (2008) showed that wave loading on structures is made of several components for the total force. Buoyant forces are only a concern for structures that have little resistance to upward forces. Here, the wooden buildings are bolted to the flat land area so buoyant forces are not relevant. Debris impact forces and damming of waterborne debris are not at play in the present laboratory setup. Therefore, for the case of the experiments described here, only hydrostatic, hydrodynamic, and impulsive forces are of interest. Their respective expressions are given below:

\[
F_h = \frac{1}{2} \rho gbh^2. \tag{5.61}
\]

\[
F_d = \frac{1}{2} \rho C_d b (hU^2)_{\text{max}}. \tag{5.62}
\]

\[
F_s = 1.5F_d. \tag{5.63}
\]

\(b\) is the breadth of the structure or component considered, and \(C_d\) is drag coefficient, recommended to be taken as \(C_d = 2\) (FEMA, 2008).

Only initial results for elevated wave forces on model buildings are presented in the following section.

The peak pressure value’s repeatability is disputable whereas the time dependent pressure is repeatable (see section 4.1.3). Moreover, in the field building damage is likely to depend significantly on the time of interaction of the flow with the structure, as shown, for example, by videos from the 2011 Japan tsunami (Figure 5.40). Therefore we are considering here an average pressure, depending on time. With \(\Delta t\) being the
total time of interaction between the flow and the structure as indicated by the pressure
time series, the average pressure recorded by one transducer is:

\[ \bar{p} = \frac{\int_{i_1}^{i_2} \rho \, dt}{\Delta t}. \]  \quad (5.64)

Figure 5.41 shows the evolution of average pressure (5.64) with time of interaction of
the short elevated wave with the structure \( \Delta t \), and with the height of the transducers
(see Figure 3.12), for one building facing the flow at distance \( dI \). As highlighted in
section 4.6, only the largest elevated wave for this series of experiments records data
for all 3 transducers on the front face, therefore only the results for this wave are
presented.

Unfortunately without the local flow depth around the building it is not possible to
calculate precisely the corresponding force. However, if the flow reaches transducer
T5, we can consider that water is in contact with the whole area surrounding T4. Then
we can assume that the product of the local average pressure and the area surrounding
T4 is representative of the force acting on this area. The average pressure seems to vary
linearly with the height of the transducers facing the flow, potentially suggesting that
the flow velocity during the interaction of the wave with the structure is constant.
However, due to the small number of data points these results should be interpreted
with caution.

Results for the experiment of the force applied on a single foam building (section 3.2.6)
are presented in Figure 5.42. The total force of the elevated wave, which is
approximately \( F_h + 2.5F_d \) (see (5.61), (5.62), (5.63)), decreases as a 2\textsuperscript{nd}
order polynomial as it travels inland. Considering a simplified triangular-shaped runup
tongue, the flow depth \( h \) decreases linearly as the flow progresses inland. Therefore,
this trend can be expected if the velocity $U$ is either approximately constant, or decreasing, with distance.

The evolution of average pressure with distance retrieved from the wooden buildings experiments described in section 3.2.6 is shown in Figure 5.43. For the bottom transducers facing the flow, the force appears to decrease linearly with distance. This is also the case for wave 138, where only transducer T4 recorded a signal for all distances. However, the wooden building could only be placed at three different locations from the shoreline, and extra data would be required to verify the evolution of average pressure and trend further away from the shoreline. Indeed, the foam building experiment indicated a quadratic decrease in force instead of linear; and a limited amount of data usually does not allow for the distinction between those two trends. The top transducer facing the flow displays no significant variation of force recorded. It appears that above a set height off the ground, the average pressure is approximately constant regardless of distance, but here again more data would be needed to verify this observation.
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Table 5.1: Comparisons between experimental phase speed and the theoretical shallow water phase speed ($\sqrt{gh}$). The first series of tests corresponds to elevated waves, and is separated from the second series of waves, the N-waves. Waves with a period greater than 10s are in bold. The characteristic parameters of these waves can be found in Table 3.1 and Table 3.2.
Table 5.2: Representative sample of wave attenuations

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*Table 5.3: Typical experimental waves of varying lengths and amplitudes and their associated Iribarren number and reflection coefficients. The coefficient $Cr$ has been deduced from data presented in Allsop & Hettiarachchi (1988) and is only indicative of the amount of reflection.*
### Table 5.4: Range of continental shelf depth and long propagating wave conditions in the field (wave heights). In column H/h (H=5m), the wave height H is fixed, the water depths h range from 10m to 100m. In column H/h (h=25m), the water depth is fixed, the wave height ranges from 3m to 16m.

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<th>h (m)</th>
<th>H(m)</th>
<th>H/h (H=5m)</th>
<th>H/h (h=25m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>300</td>
<td>10</td>
<td>3</td>
<td>0.500</td>
<td>0.120</td>
</tr>
<tr>
<td>1200</td>
<td>25</td>
<td>4</td>
<td>0.200</td>
<td>0.160</td>
</tr>
<tr>
<td>600</td>
<td>40</td>
<td>5</td>
<td>0.125</td>
<td>0.200</td>
</tr>
<tr>
<td>1800</td>
<td>50</td>
<td>7</td>
<td>0.100</td>
<td>0.280</td>
</tr>
<tr>
<td>3600</td>
<td>60</td>
<td>10</td>
<td>0.083</td>
<td>0.400</td>
</tr>
<tr>
<td>5400</td>
<td>70</td>
<td>11</td>
<td>0.071</td>
<td>0.440</td>
</tr>
<tr>
<td>7200</td>
<td>80</td>
<td>12</td>
<td>0.063</td>
<td>0.480</td>
</tr>
<tr>
<td>9000</td>
<td>90</td>
<td>15</td>
<td>0.056</td>
<td>0.600</td>
</tr>
<tr>
<td>10800</td>
<td>100</td>
<td>16</td>
<td>0.050</td>
<td>0.640</td>
</tr>
</tbody>
</table>

### Table 5.5: Range of continental shelf depth and long propagating wave conditions in the field (depressed wave).

<table>
<thead>
<tr>
<th>Trough amplitude (m)</th>
<th>$\alpha^-/h$ (h=10m)</th>
<th>$\alpha^-/h$ (h=50m)</th>
<th>$\alpha^-/h$ (h=100m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.1</td>
<td>0.02</td>
<td>0.01</td>
</tr>
<tr>
<td>2</td>
<td>0.2</td>
<td>0.04</td>
<td>0.02</td>
</tr>
<tr>
<td>3</td>
<td>0.3</td>
<td>0.06</td>
<td>0.03</td>
</tr>
<tr>
<td>4</td>
<td>0.4</td>
<td>0.08</td>
<td>0.04</td>
</tr>
<tr>
<td>Test</td>
<td>Wave</td>
<td>Offshore1 (m)</td>
<td>Toe (m)</td>
</tr>
<tr>
<td>-------------</td>
<td>------</td>
<td>---------------</td>
<td>---------</td>
</tr>
<tr>
<td>081028_07</td>
<td>415</td>
<td>0.0907</td>
<td>0.1034</td>
</tr>
<tr>
<td>081028_08</td>
<td>415</td>
<td>0.0927</td>
<td>0.1027</td>
</tr>
<tr>
<td>081028_016</td>
<td>415</td>
<td>0.0914</td>
<td>0.1042</td>
</tr>
<tr>
<td>081028_018</td>
<td>415</td>
<td>0.0926</td>
<td>0.1064</td>
</tr>
<tr>
<td>081031_30</td>
<td>307</td>
<td>0.0267</td>
<td>0.0372</td>
</tr>
<tr>
<td>081101_21</td>
<td>307</td>
<td>0.0344</td>
<td>0.0639</td>
</tr>
<tr>
<td>081101_22</td>
<td>307</td>
<td>0.0317</td>
<td>0.0593</td>
</tr>
<tr>
<td>081103_05</td>
<td>307</td>
<td>0.0231</td>
<td>0.0309</td>
</tr>
</tbody>
</table>

Table 5.6: Variations of wave amplitude at the toe for long waves. The wave amplitudes at Offshore 1 and at the Toe are shown, as well as the expected wave amplitude at the toe after attenuation (0.0001). (*) The difference between the amplitude expected at Offshore 1 and at the Toe is shown. (**) The proportion of Toe amplitude compared to the amplitude at Offshore 1.
<table>
<thead>
<tr>
<th></th>
<th>$\alpha(\beta)$</th>
<th>$f(\beta)$</th>
<th>$\cot(\beta)$</th>
<th>$\frac{\alpha(\beta)}{\sqrt{\cot(\beta)}}$</th>
<th>Range $a/h$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hall &amp; Watts</td>
<td>3.1</td>
<td>1.15</td>
<td>1</td>
<td>3.1</td>
<td>0.05-0.43</td>
</tr>
<tr>
<td>(1953)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Synolakis</td>
<td>12.61</td>
<td>1.25</td>
<td>19.85</td>
<td>2.831</td>
<td>0.005-0.048</td>
</tr>
<tr>
<td>(1986)*</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Synolakis</td>
<td>1.109</td>
<td>0.582</td>
<td>19.85</td>
<td>0.056</td>
<td>0.056-0.633</td>
</tr>
<tr>
<td>(1986)**</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Borthwick et al. (2006)**</td>
<td>3.02</td>
<td>0.91</td>
<td>5</td>
<td>1.35</td>
<td>0.15-0.3</td>
</tr>
<tr>
<td>Borthwick et al. (2006)**</td>
<td>3.02</td>
<td>0.91</td>
<td>10</td>
<td>0.3</td>
<td>0.075-0.15</td>
</tr>
<tr>
<td>Borthwick et al. (2006)**</td>
<td>3.02</td>
<td>0.91</td>
<td>30</td>
<td>0.1</td>
<td>0.012-0.03</td>
</tr>
<tr>
<td>Borthwick et al. (2006)**</td>
<td>3.02</td>
<td>0.91</td>
<td>50</td>
<td>0.06</td>
<td>0.005-0.012</td>
</tr>
<tr>
<td>Borthwick et al. (2006)**</td>
<td>3.02</td>
<td>0.91</td>
<td>100</td>
<td>0.03</td>
<td>0.001-0.005</td>
</tr>
<tr>
<td>Present work</td>
<td>2.14</td>
<td>0.77</td>
<td>20</td>
<td>0.5</td>
<td>0.046-0.18</td>
</tr>
</tbody>
</table>

Table 5.7: Values of $\alpha(\beta)$ and $f(\beta)$ obtained by different studies on wave runup. (*) for non-breaking waves, (**) for breaking waves, when runup is measured at the maximum position of the advancing shoreline. (***) The authors provided $\alpha(\beta)$ and $f(\beta)$ obtained for the maximum value of $R/h$ for different slopes, contrary to the other results presented in this table, these were obtained numerically.
<table>
<thead>
<tr>
<th></th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>Short elevated waves ($T&lt;10s$) and data from Synolakis (1987)</td>
</tr>
<tr>
<td>B</td>
<td>Long elevated waves ($T&gt;10s$)</td>
</tr>
<tr>
<td>C</td>
<td>All elevated and solitary waves</td>
</tr>
<tr>
<td>D</td>
<td>N-waves ($T&lt;10s$)</td>
</tr>
<tr>
<td>E</td>
<td>N-waves ($T&gt;10s$)</td>
</tr>
<tr>
<td>F</td>
<td>All N-waves</td>
</tr>
<tr>
<td>G</td>
<td>All waves</td>
</tr>
</tbody>
</table>

*Table 5.8: Subpopulations of wave data for regression analysis.*
Table 5.9: Wave group A, short elevated waves: response variable, combinations of \( k \) and \( \log K \) giving a minimal value for equation (5.24), coefficient of determination, mean of residuals and distribution of residuals for plots displaying a degree of linearity. The value in brackets next to the coefficient of determination indicates the number of outliers removed, if no value is shown, there were no outliers.
<table>
<thead>
<tr>
<th></th>
<th>$\frac{R}{L'}$</th>
<th>$L^{-3}$</th>
<th>k</th>
<th>log K</th>
<th>$R^2$</th>
<th>$\bar{e}$</th>
<th>${e_1, e_2, ..., e_n}$</th>
<th>${e_1, e_2, ..., e_n}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>$R/h$</td>
<td>$aLh$</td>
<td>-0.56</td>
<td>1.71</td>
<td>0.96</td>
<td>$6 \times 10^{-16}$</td>
<td>Yes; Yes</td>
<td>Yes</td>
</tr>
<tr>
<td>2</td>
<td>$R/h$</td>
<td>$Lh^2$</td>
<td>-0.33</td>
<td>1.33</td>
<td>0.96</td>
<td>$-4 \times 10^{-16}$</td>
<td>Yes; Yes</td>
<td>No</td>
</tr>
<tr>
<td>3</td>
<td>$R/L$</td>
<td>$a^3$</td>
<td>1.1</td>
<td>-0.88</td>
<td>0.91</td>
<td>$-7 \times 10^{-16}$</td>
<td>Yes; Yes</td>
<td>Yes</td>
</tr>
</tbody>
</table>

Table 5.10: Wave group B, long elevated waves: response variable, combinations of $k$ and logK giving a minimal value for equation (5.24), coefficient of determination, mean of residuals and distribution of residuals for plots displaying a degree of linearity. There were no outliers.
Table 5.11: Wave group C, all elevated waves: response variable, combinations of $k$ and $\log K$ giving a minimal value for equation (5.24), coefficient of determination, mean of residuals and distribution of residuals for plots displaying a degree of linearity. The value in brackets next to the coefficient of determination indicates the number of outliers removed, if no value is shown, there were no outliers.
<table>
<thead>
<tr>
<th>$\frac{R}{L}$</th>
<th>$E$</th>
<th>$L^3$</th>
<th>$k$</th>
<th>$\log K$</th>
<th>$R^2$</th>
<th>$\bar{e}$</th>
<th>${e_1, e_2, \ldots, e_n}$</th>
<th>${e_1, e_2, \ldots, e_n}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$R/h$</td>
<td>$E_p^+$</td>
<td>$L^3$</td>
<td>-0.43</td>
<td>4.94</td>
<td>0.94</td>
<td>$3.10^{-15}$</td>
<td>Yes; Yes</td>
<td>Yes</td>
</tr>
<tr>
<td>$R/h$</td>
<td>$E_p^+$</td>
<td>$hL^2$</td>
<td>-0.43</td>
<td>3.40</td>
<td>0.97 (1)</td>
<td>$-1.10^{-16}$</td>
<td>Yes; Yes</td>
<td>Yes</td>
</tr>
<tr>
<td>$R/h$</td>
<td>$E_p^+$</td>
<td>$\alpha L^2$</td>
<td>-0.39</td>
<td>1.75</td>
<td>0.92</td>
<td>$4.10^{-16}$</td>
<td>Yes; Yes</td>
<td>Yes</td>
</tr>
<tr>
<td>$R/h$</td>
<td>$E_p^+$</td>
<td>$Lh^2$</td>
<td>-0.40</td>
<td>1.75</td>
<td>0.98 (2)</td>
<td>$6.10^{-16}$</td>
<td>Yes; Yes</td>
<td>Yes</td>
</tr>
<tr>
<td>$R/h$</td>
<td>$E_p^+$</td>
<td>$Lh^2$</td>
<td>-0.37</td>
<td>0.32</td>
<td>0.91</td>
<td>$-2.10^{-16}$</td>
<td>Yes; Yes</td>
<td>Yes</td>
</tr>
<tr>
<td>$R/a^-$</td>
<td>$E_p^+$</td>
<td>$L^2 a^-$</td>
<td>-0.46</td>
<td>5.47</td>
<td>0.97 (1)</td>
<td>$4.10^{-16}$</td>
<td>Yes; No*</td>
<td>Yes</td>
</tr>
<tr>
<td>$R/a^-$</td>
<td>$E_p^+$</td>
<td>$Lh a^-$</td>
<td>-0.45</td>
<td>3.77</td>
<td>0.96 (1)</td>
<td>$5.10^{-16}$</td>
<td>Yes; No*</td>
<td>Yes</td>
</tr>
<tr>
<td>$R/a^-$</td>
<td>$E_p^+$</td>
<td>$L(a^-)^2$</td>
<td>-0.42</td>
<td>2.34</td>
<td>0.97 (1)</td>
<td>$-4.10^{-16}$</td>
<td>Yes; Yes</td>
<td>Yes</td>
</tr>
<tr>
<td>$R/a^-$</td>
<td>$E_p^+$</td>
<td>$h(a^-)^2$</td>
<td>-0.41</td>
<td>0.90</td>
<td>0.96 (1)</td>
<td>$-3.10^{-17}$</td>
<td>Yes; Yes</td>
<td>Yes</td>
</tr>
<tr>
<td>$R/a^-$</td>
<td>$E_p^+$</td>
<td>$(a^-)^3$</td>
<td>-0.37</td>
<td>-0.17</td>
<td>0.98 (2)</td>
<td>$5.10^{-16}$</td>
<td>Yes; No*</td>
<td>Yes</td>
</tr>
<tr>
<td>$R/a^-$</td>
<td>$E_p^+$</td>
<td>$a^3$</td>
<td>0.31</td>
<td>2.78</td>
<td>0.92</td>
<td>$3.10^{-16}$</td>
<td>Yes; Yes</td>
<td>Yes</td>
</tr>
<tr>
<td>$R/L$</td>
<td>$E_p^+$</td>
<td>$L^3$</td>
<td>-0.40</td>
<td>1.04</td>
<td>0.97 (2)</td>
<td>$3.10^{-15}$</td>
<td>Yes; Yes</td>
<td>Yes</td>
</tr>
</tbody>
</table>

Table 5.12: Wave group D, short N- waves: response variable, combinations of $k$ and $\log K$ giving a minimal value for equation (5.24), coefficient of determination, mean of residuals and distribution of residuals for plots displaying a degree of linearity. The
value in brackets next to the coefficient of determination indicates the number of outliers removed, if no value is shown, there were no outliers. (*) In some cases, the variance does not appear constant when only a limited number of points is available at the extremities of x. More points in these regions would be needed to confirm heteroscedasticity.
<table>
<thead>
<tr>
<th></th>
<th>$\frac{R}{L^*}$</th>
<th>$E$</th>
<th>$E^3$</th>
<th>$k$</th>
<th>$\log K$</th>
<th>$R^2$</th>
<th>$\bar{e}$</th>
<th>Un-correlated; $\sigma^2=$constant</th>
<th>normally distributed</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>$R/h$</td>
<td>$E_p$</td>
<td>$La^2$</td>
<td>-0.91</td>
<td>-1.85</td>
<td>1</td>
<td>$-7 \times 10^{-17}$</td>
<td>Yes; Yes*</td>
<td>No**</td>
</tr>
<tr>
<td>2</td>
<td>$R/h$</td>
<td>$E_p$</td>
<td>$La^2$</td>
<td>-0.85</td>
<td>-1.91</td>
<td>0.99</td>
<td>$-5 \times 10^{-16}$</td>
<td>Yes; Yes*</td>
<td>No**</td>
</tr>
<tr>
<td>3</td>
<td>$R/a^-$</td>
<td>$E_p$</td>
<td>$La^2$</td>
<td>-1.17</td>
<td>-1.29</td>
<td>1</td>
<td>$7 \times 10^{-17}$</td>
<td>Yes; Yes*</td>
<td>Yes**</td>
</tr>
<tr>
<td>4</td>
<td>$R/a^-$</td>
<td>$E_p$</td>
<td>$La^2$</td>
<td>-1.09</td>
<td>-1.36</td>
<td>0.99</td>
<td>$-2 \times 10^{-16}$</td>
<td>Yes; Yes*</td>
<td>No**</td>
</tr>
<tr>
<td>5</td>
<td>$R/H$</td>
<td>$E_p$</td>
<td>$La^2$</td>
<td>-1.15</td>
<td>-1.37</td>
<td>1</td>
<td>$-7 \times 10^{-17}$</td>
<td>Yes; Yes*</td>
<td>Yes**</td>
</tr>
<tr>
<td>6</td>
<td>$R/H$</td>
<td>$E_p$</td>
<td>$La^2$</td>
<td>-1.08</td>
<td>-1.44</td>
<td>0.98</td>
<td>$-3 \times 10^{-16}$</td>
<td>Yes; Yes*</td>
<td>No**</td>
</tr>
</tbody>
</table>

Table 5.13: Wave group E, long N-waves: response variable, combinations of $k$ and $\log K$ giving a minimal value for equation (5.24), coefficient of determination, mean of residuals and distribution of residuals for plots displaying a degree of linearity. (*) It is not possible to assess homoscedasticity using the residuals for only 3 data points, the results shown here indicate if the points appeared evenly spaced along the fit. Moreover, residuals never appeared correlated (i.e. aligned), but more points would be necessary to correctly assess this. (**) with 3 data points, normality could only be examined using the normal probability plot. ‘Yes**’ indicates all points were very close to the normal distribution.
<table>
<thead>
<tr>
<th></th>
<th>$\frac{R}{L}$</th>
<th>$E$</th>
<th>$L^{-3}$</th>
<th>k</th>
<th>log $K$</th>
<th>$R^2$</th>
<th>$\bar{e}$</th>
<th>${e_1, e_2, \ldots, e_n}$</th>
<th>${e_1, e_2, \ldots, e_n}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>$R/L$</td>
<td>$E_P$</td>
<td>$L^3$</td>
<td>-0.46</td>
<td>1.88</td>
<td>0.98 (1)</td>
<td>$-3.10^{-15}$</td>
<td>Yes; Yes</td>
<td>Yes</td>
</tr>
<tr>
<td>2</td>
<td>$R/L$</td>
<td>$E_P$</td>
<td>$a^{-1}L^2$</td>
<td>-0.72</td>
<td>0.37</td>
<td>0.96</td>
<td>$-1.10^{-15}$</td>
<td>Yes; No</td>
<td>Yes</td>
</tr>
<tr>
<td>3</td>
<td>$R/H$</td>
<td>$E_P$</td>
<td>$L(a^-)^2$</td>
<td>-0.41</td>
<td>1.04</td>
<td>0.91</td>
<td>$9.10^{-17}$</td>
<td>Yes; No</td>
<td>Yes</td>
</tr>
<tr>
<td>4</td>
<td>$R/H$</td>
<td>$E_P$</td>
<td>$LH\alpha^-$</td>
<td>-0.58</td>
<td>1.72</td>
<td>0.90</td>
<td>$3.10^{-16}$</td>
<td>Yes; No</td>
<td>Yes</td>
</tr>
<tr>
<td>5</td>
<td>$R/a^-$</td>
<td>$E_P$</td>
<td>$L(a^-)^2$</td>
<td>-0.71</td>
<td>2.24</td>
<td>0.95</td>
<td>$-2.10^{-16}$</td>
<td>Yes; No</td>
<td>Yes</td>
</tr>
<tr>
<td>6</td>
<td>$R/a^-$</td>
<td>$E_P$</td>
<td>$(a^-)^2L$</td>
<td>-0.45</td>
<td>2.37</td>
<td>0.99 (1)</td>
<td>$-2.10^{-16}$</td>
<td>Yes; Yes</td>
<td>Yes</td>
</tr>
<tr>
<td>7</td>
<td>$R/a^-$</td>
<td>$E_P$</td>
<td>$L^2\alpha^-$</td>
<td>-0.45</td>
<td>5.36</td>
<td>0.98 (1)</td>
<td>$1.10^{-15}$</td>
<td>Yes; Yes</td>
<td>Yes</td>
</tr>
<tr>
<td>8</td>
<td>$R/a^-$</td>
<td>$E_P$</td>
<td>$HL^2$</td>
<td>-0.55</td>
<td>6.72</td>
<td>0.97 (1)</td>
<td>$2.10^{-15}$</td>
<td>Yes; Yes</td>
<td>Yes</td>
</tr>
<tr>
<td>9</td>
<td>$R/a^-$</td>
<td>$E_P$</td>
<td>$LH\alpha^-$</td>
<td>-0.56</td>
<td>3.11</td>
<td>0.99 (2)</td>
<td>$-6.10^{-14}$</td>
<td>Yes; Yes</td>
<td>Yes</td>
</tr>
<tr>
<td>10</td>
<td>$R/a^-$</td>
<td>$E_P$</td>
<td>$(\alpha^-)^3$</td>
<td>-0.44</td>
<td>-0.63</td>
<td>0.95</td>
<td>$-3.10^{-16}$</td>
<td>Yes; No</td>
<td>Yes</td>
</tr>
<tr>
<td>11</td>
<td>$R/h$</td>
<td>$E_P$</td>
<td>$Lh^2$</td>
<td>-0.4</td>
<td>1.73</td>
<td>0.97 (2)</td>
<td>$-1.10^{-14}$</td>
<td>Yes; Yes</td>
<td>Yes</td>
</tr>
</tbody>
</table>

Table 5.14: Wave group F, all N-waves: response variable, combinations of k and log K giving a minimal value for equation (5.24), coefficient of determination, mean of residuals and distribution of residuals for plots displaying a degree of linearity. The value in brackets next to the coefficient of determination indicates the number of outliers removed, if no value is shown, there were no outliers.
<table>
<thead>
<tr>
<th>R/L</th>
<th>E</th>
<th>( L^3 )</th>
<th>k ( K )</th>
<th>( R^2 )</th>
<th>( \bar{e} )</th>
<th>Un-correlated; ( \sigma^2 = \text{constant} )</th>
<th>normally distributed</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>( R/L )</td>
<td>( E_p )</td>
<td>( H L^2 )</td>
<td>-0.89</td>
<td>2.71</td>
<td>0.97 (3)</td>
<td>1.10(^{-15} )</td>
</tr>
<tr>
<td>2</td>
<td>( R/L )</td>
<td>( E_p^+ )</td>
<td>( H L^2 )</td>
<td>-0.87</td>
<td>2.62</td>
<td>0.97 (6)</td>
<td>2.10(^{-15} )</td>
</tr>
<tr>
<td>3</td>
<td>( R/L )</td>
<td>( E_p )</td>
<td>( L^3 )</td>
<td>-0.45</td>
<td>1.74</td>
<td>0.97 (5)</td>
<td>2.10(^{-15} )</td>
</tr>
<tr>
<td>4</td>
<td>( R/L )</td>
<td>( E_p^+ )</td>
<td>( L^3 )</td>
<td>-0.45</td>
<td>1.73</td>
<td>0.98</td>
<td>-1.10(^{-17} )</td>
</tr>
<tr>
<td>5</td>
<td>( R/L )</td>
<td>( E_p )</td>
<td>( H a^2 )</td>
<td>0.86</td>
<td>-1.28</td>
<td>0.97 (2)</td>
<td>-2.10(^{-16} )</td>
</tr>
<tr>
<td>6</td>
<td>( R/L )</td>
<td>( E_p^+ )</td>
<td>( a L^2 )</td>
<td>-0.90</td>
<td>2.72</td>
<td>0.97 (1)</td>
<td>-9.10(^{-16} )</td>
</tr>
<tr>
<td>7</td>
<td>( R/L )</td>
<td>( E_p^+ )</td>
<td>( a^3 )</td>
<td>0.87</td>
<td>-1.28</td>
<td>0.97 (1)</td>
<td>2.10(^{-16} )</td>
</tr>
<tr>
<td>8</td>
<td>( R/h )</td>
<td>( E_p )</td>
<td>( L h^2 )</td>
<td>-0.36</td>
<td>1.18</td>
<td>0.93 (4)</td>
<td>-3.10(^{-16} )</td>
</tr>
<tr>
<td>9</td>
<td>( R/h )</td>
<td>( E_p^+ )</td>
<td>( L h^2 )</td>
<td>-0.35</td>
<td>1.17</td>
<td>0.93 (2)</td>
<td>3.10(^{-16} )</td>
</tr>
<tr>
<td>10</td>
<td>( R/h )</td>
<td>( E_p^+ )</td>
<td>( a L h )</td>
<td>-0.70</td>
<td>1.90</td>
<td>0.93 (5)</td>
<td>-5.10(^{-16} )</td>
</tr>
</tbody>
</table>

Table 5.15: Wave group G, all waves: response variable, combinations of \( k \) and \( \log K \) giving a minimal value for equation (5.24), coefficient of determination, mean of residuals and distribution of residuals for plots displaying a degree of linearity. The value in brackets next to the coefficient of determination indicates the number of outliers removed, if no value is shown, there were no outliers.
<table>
<thead>
<tr>
<th>Wave group</th>
<th>Equation</th>
<th>n</th>
<th>SE_k</th>
<th>SE_{log K}</th>
<th>$t_{(n-2, \alpha/2)}$</th>
<th>C.I k</th>
<th>C.I logK</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>(5.31)</td>
<td>97</td>
<td>0.016</td>
<td>0.0317</td>
<td>1.985</td>
<td>(0.77; 0.40)</td>
<td>(-1.32; -1.19)</td>
</tr>
<tr>
<td>B</td>
<td>(5.41)</td>
<td>8</td>
<td>0.0477</td>
<td>0.232</td>
<td>2.447</td>
<td>(-0.68; -0.45)</td>
<td>(1.15; 2.28)</td>
</tr>
<tr>
<td>C</td>
<td>(5.42)</td>
<td>106</td>
<td>0.017</td>
<td>0.0387</td>
<td>1.960</td>
<td>(0.83; 0.90)</td>
<td>(-1.35; -1.20)</td>
</tr>
<tr>
<td>D</td>
<td>(5.43)</td>
<td>16</td>
<td>0.0186</td>
<td>0.269</td>
<td>2.145</td>
<td>(-0.44; -0.37)</td>
<td>(1.49; 2.0)</td>
</tr>
<tr>
<td>E</td>
<td>(5.44)</td>
<td>3</td>
<td>0.0571</td>
<td>0.0287</td>
<td>12.706</td>
<td>(-1.89; -0.44)</td>
<td>(-1.65; -0.92)</td>
</tr>
<tr>
<td>F</td>
<td>(5.45)</td>
<td>20</td>
<td>0.0128</td>
<td>0.0413</td>
<td>2.101</td>
<td>(-0.48; -0.43)</td>
<td>(2.28; 2.45)</td>
</tr>
<tr>
<td>G</td>
<td>(5.46)</td>
<td>126</td>
<td>0.0135</td>
<td>0.0382</td>
<td>1.960</td>
<td>(0.84; 0.89)</td>
<td>(-1.35; -1.20)</td>
</tr>
</tbody>
</table>

*Table 5.16: For each wave group: equation number, number of individuals (after removal of outliers), standard errors for the regression coefficients and associated confidence intervals (C.I.).*
### Table 5.17: Power laws of the form of (5.21), with associated confidence intervals (C.I.) for the regression parameters k and K.

<table>
<thead>
<tr>
<th>Wave group</th>
<th>Equation</th>
<th>C.I. k</th>
<th>C.I. K</th>
</tr>
</thead>
<tbody>
<tr>
<td>A (S.E)</td>
<td>$\frac{R}{L} = 0.28 \left( \frac{\rho g a^3}{E_p} \right)^{0.81}$</td>
<td>(5.65)</td>
<td>(0.77; 0.40)</td>
</tr>
<tr>
<td>B (L.E)</td>
<td>$\frac{R}{h} = 2.72 \left( \frac{L}{\rho g L h} \right)^{0.2}$</td>
<td>(5.66)</td>
<td>(-0.68; -0.45)</td>
</tr>
<tr>
<td>C (all E)</td>
<td>$\frac{R}{L} = 0.28 \left( \frac{\rho g a^3}{E_p} \right)^{0.87}$</td>
<td>(5.67)</td>
<td>(0.83; 0.90)</td>
</tr>
<tr>
<td>D (S.N)</td>
<td>$\frac{R}{h} = 5.75 \left( \frac{E_p^{+}}{\rho g L a^{2}} \right)^{0.4}$</td>
<td>(5.68)</td>
<td>(-0.44; -0.37)</td>
</tr>
<tr>
<td>E (L.N)</td>
<td>$\frac{R}{a} = 0.27 \left( \frac{E_p^{+}}{\rho g L a^{2}} \right)$</td>
<td>(5.69)</td>
<td>(-1.89; -0.44)</td>
</tr>
<tr>
<td>F (all N)</td>
<td>$\frac{R}{a} = 10.7 \left( \frac{E_p^{+}}{\rho g L (a^{-2})} \right)^{0.45}$</td>
<td>(5.70)</td>
<td>(-0.48; -0.43)</td>
</tr>
<tr>
<td>G (all)</td>
<td>$\frac{R}{L} = 0.28 \left( \frac{\rho g a^3}{E_p^{+}} \right)^{0.87}$</td>
<td>(5.71)</td>
<td>(0.84; 0.89)</td>
</tr>
</tbody>
</table>
Table 5.18: Accelerations of four different types of waves over the total distance where measurements were taken; comparison with acceleration of gravity. \( \bar{u}_{\text{max}} \) is the average of all peak velocities, \( \partial u_{\text{max}} \) is the difference between the initial and the final peak velocity recorded along a given distance \( \partial X \), and \( \bar{u}_{\text{max}} \frac{\partial u_{\text{max}}}{\partial X} \) is the estimated convective acceleration in the shoreline region.
Figure 5.1: Position of an elevated wave (test 321) in the constant depth region (offshore probe positions) vs. peak time occurrence. The slope of the linear fit is the experimental phase velocity of the wave.
Figure 5.2: Phase celerity versus $a/h$ for (a) elevated waves, and (b) $N$-waves. The red linear fit (red) corresponds to (5.2) for $\lambda=0.88$ for (a); and $\lambda=0.86$ for (b), in both
cases the green trend corresponds to (5.2) with \( \lambda = 1/2 \). The error bars are based on the standard error (0.04) for the phase speed.
Figure 5.3: Present experiments for elevated waves and N-waves located in the wave theories graph and domain of applicability, as presented in section 2.2.1. Previous experiments, as well as typical nearshore field tsunami are also placed in the graph.
Figure 5.4: Evolution of phase celerity in the nearshore region with respect to the offshore depth wave celerity, for test 20_081028.
Figure 5.5: Froude number ratio between the range of experimental waves and a typical long propagating wave in the nearshore region (Figure 2.10).
Figure 5.6: Short elevated wave propagating along the flume (test ID 366, $T=4.2\text{s}$, $L=9.46\text{m}$). (*) Each probe recording is represented by a vertical offset of 0.05m with respect to the previous probe.
Figure 5.7: Short N-wave propagating along the flume (test ID 340, T=7s, L=15.5m).

(*) Each probe recording is represented by a vertical offset of 0.1m with respect to the previous probe.
Figure 5.8: State of the water level in the flume at different times during the generation of a long elevated wave (a – test 081028_018), and a long N-wave (b – 081101_22).
Figure 5.9: Runup data for short elevated waves (present experiments), and relationships of the form of (2.23) obtained in previous work for different values of $\alpha(\beta)$ and $f(\beta)$ as listed in Table 5.7.
Figure 5.10: $(\alpha, f)$ vs. cot$(\beta)$ for the values presented in Table 5.7. No obvious correlation seems to exist between the value of the slope angle and the two undefined functions $(\alpha, f)$. 
Figure 5.11: Runup results for the long wave generator: elevated waves, N-waves, with short and long periods, compared to Synolakis (1987) data.
Figure 5.12: Comparison between the present runup data for elevated waves and Baldock & Holmes (1999) ballistic theory; for $H_b = a_{\text{Nearshore}}$ and $C=2$ (red line). The runup seems underestimated by the theory by a factor of 1.75 (green line of best fit).
Figure 5.13: Normalized runup for the data from Synolakis (1987) (blue lozanges), and the present data, (red squares) vs. normalized potential energy, as described by (2.28) (Li & Raichlen, 2003).
Figure 5.14: Scatter plot of variables potentially influential on runup, here for short elevated waves. The waves generated with the pneumatic generator are represented by blue triangles, the solitary waves from Synolakis (1987) are represented by green circles. A trend can be observed in all graphs, suggesting they should all be included in the runup analysis.
Figure 5.15: Scatter plot of variables potentially influential on runup, here for short N-waves. A trend can be observed in all graphs except for wavelength. However for consistency with the analysis of elevated waves, all parameters are included in the N-wave runup analysis. The plot for positive and negative potential energies indicate that there is a balance between both potential energies during the wave runup process.
Figure 5.16: Example correlations between the dimensionless products of equation (5.24) for different variables \( L' \). These indicate a power law relates the two dimensionless products.
Figure 5.17: (a) Residual plot corresponding to the best fit for wave group A (short elevated waves). The data on the right-hand side corresponds to the data from Synolakis (1987), the pneumatic generator data is grouped to the left-hand side. This plot shows that the data is evenly distributed around the mean, within 2 standard deviations (i.e. good fit of the regression curve to the data). (b) Normal probability plot of the residuals: the normal distribution is represented by the red line, the corresponding distribution of the data is represented by the blue crosses.
Figure 5.18: Results of the regression analysis for wave group A (short elevated waves). (a) Equation (5.38) is represented in red, the 95% confidence intervals for the predicted variable are represented in blue dashes, and the 95% confidence intervals for the regression parameters are represented in green dashes (i.e. the maximum and minimum regression fit at this level of confidence is contained within the green asymptotes).(b) Power law (5.47) and corresponding 95% confidence intervals for the predicted variable (in blue dashes), as well as 95% confidence intervals for the regression parameters k and K (in green dashes).
Figure 5.19: (a) Residual plot corresponding to the best fit for wave group B (long elevated waves). This plot shows that the data is evenly distributed around the mean, within 2 standard deviations (i.e. good fit of the regression curve to the data). (b) Normal probability plot of the residuals: the normal distribution is represented by the red line, the corresponding distribution of the data is represented by the blue crosses.
Figure 5.20: Results of the regression analysis for wave group B (long elevated waves). (a) Equation (5.44) is represented in red, the 95% confidence intervals for the predicted variable are represented in blue dashes, and the 95% confidence intervals for the regression parameters are represented in green dashes (i.e. the maximum and minimum regression fit at this level of confidence is contained within the green asymptotes). (b) Power law (5.48) and corresponding 95% confidence intervals for the predicted variable (in blue dashes), as well as 95% confidence intervals for the regression parameters $k$ and $K$ (in green dashes).
Figure 5.21: (a) Residual plot corresponding to the best fit for wave group C (all elevated waves). This plot shows that the data is evenly distributed around the mean, within 2 to 2.5 standard deviations (i.e. good fit of the regression curve to the data). (b) Normal probability plot of the residuals: the normal distribution is represented by the red line, the corresponding distribution of the data is represented by the blue crosses.
Figure 5.22: Results of the regression analysis for wave group C (all elevated waves).

(a) Equation (5.45) is represented in red, the 95% confidence intervals for the predicted variable are represented in blue dashes, and the 95% confidence intervals for the regression parameters are represented in green dashes (i.e. the maximum and minimum regression fit at this level of confidence is contained within the green asymptotes).

(b) Power law (5.49) and corresponding 95% confidence intervals for the predicted variable (in blue dashes), as well as 95% confidence intervals for the regression parameters $k$ and $K$ (in green dashes).
Figure 5.23: (a) Residual plot corresponding to the best fit for wave group D (short N-waves). This plot shows that the data is evenly distributed around the mean, within approximately 2 standard deviations (i.e. good fit of the regression curve to the data). (b) Normal probability plot of the residuals: the normal distribution is represented by the red line, the corresponding distribution of the data is represented by the blue crosses.
Figure 5.24: Results of the regression analysis for wave group D (short $N$- waves). (a) Equation (5.47) is represented in red, the 95% confidence intervals for the predicted variable are represented in blue dashes, and the 95% confidence intervals for the regression parameters are represented in green dashes (i.e. the maximum and minimum regression fit at this level of confidence is contained within the green asymptotes). (b) Power law (5.50) and corresponding 95% confidence intervals for the predicted variable (in blue dashes), as well as 95% confidence intervals for the regression parameters $k$ and $K$ (in green dashes).
Figure 5.25: (a) Residual plot corresponding to the best fit for wave group E (long N-waves). This plot shows that the data is evenly distributed around the mean, within 1 standard deviation (i.e. very good fit of the regression curve to the data). (b) Normal probability plot of the residuals: the normal distribution is represented by the red line, the corresponding distribution of the data is represented by the blue crosses.
Figure 5.26: Results of the regression analysis for wave group E (long $N$-waves). (a) Equation (5.48) is represented in red, the 95% confidence intervals for the predicted variable are represented in blue dashes, and the 95% confidence intervals for the regression parameters are represented in green dashes (i.e. the maximum and minimum regression fit at this level of confidence is contained within the green asymptotes). (b) Power law (5.51) and corresponding 95% confidence intervals for the predicted variable (in blue dashes), as well as 95% confidence intervals for the regression parameters $k$ and $K$ (in green dashes).
Figure 5.27: (a) Residual plot corresponding to the best fit for wave group F (all N-waves). This plot shows that the data is evenly distributed around the mean, within 2 standard deviations (i.e. good fit of the regression curve to the data). (b) Normal probability plot of the residuals: the normal distribution is represented by the red line, the corresponding distribution of the data is represented by the blue crosses.
Figure 5.28: Results of the regression analysis for wave group F (all N-waves). (a) Equation (5.51) is represented in red, the 95% confidence intervals for the predicted variable are represented in blue dashes, and the 95% confidence intervals for the regression parameters are represented in green dashes (i.e. the maximum and minimum regression fit at this level of confidence is contained within the green asymptotes). (b) Power law (5.52) and corresponding 95% confidence intervals for the predicted variable (in blue dashes), as well as 95% confidence intervals for the regression parameters $k$ and $K$ (in green dashes).
Figure 5.29: (a) Residual plot corresponding to the best fit for wave group G (all waves). This plot shows that the data is evenly distributed around the mean, within 2 to 2.5 standard deviations (i.e. good fit of the regression curve to the data). (b) Normal probability plot of the residuals: the normal distribution is represented by the red line, the corresponding distribution of the data is represented by the blue crosses.
Figure 5.30: Results of the regression analysis for wave group G (all waves). (a) Equation (5.52) is represented in red, the 95% confidence intervals for the predicted variable are represented in blue dashes, and the 95% confidence intervals for the regression parameters are represented in green dashes (i.e. the maximum and minimum regression fit at this level of confidence is contained within the green asymptotes). (b) Power law (5.53) and corresponding 95% confidence intervals for the predicted variable (in blue dashes), as well as 95% confidence intervals for the regression parameters $k$ and $K$ (in green dashes).
Figure 5.31: (a) Linear regression fit group A, corresponding to equation (5.31): $n=97$, with a coefficient of determination of 0.96. (b) Linear regression fit for a random sub-sample of points in group A: $n=10$, with a coefficient of determination of 0.94.
Figure 5.32: Synolakis (1987) data for breaking and non breaking waves (i.e. longer and shorter waves), with long and short waves from the present experiments. The lines of best fit have been drawn for both regimes, indicating a regime corresponding to the smaller wavelengths where the runup scales as the amplitude and another regime corresponding to the relatively larger wavelengths where the runup scales as the square root of the amplitude.
Figure 5.33: Amplitude vs. wavelength for Synolakis (1987) data.
Figure 5.34: Influence of rundown on runup for a sample of short and long N-waves.
Figure 5.35: Wave recorded by Offshore1 (blue) and velocity time series recorded by p2 (green dashes) for test 02_081028. This test has been chosen to look at a velocity probe placed in the wet region of the slope close to the wave probes.
Figure 5.36: Peak velocities vs. distance recorded by the velocity probes, as shown in Figure 3.7 (a – elevated wave 135, b – long elevated wave 415, c – short N-wave 113, d – long N-wave 307). The outliers (still present on Figure 4.18) have been removed, as described in section 4.5. X-Xb represents the distance from the toe of the beach and the dashed red lines represent the position of the initial shoreline.
Figure 5.37: Evolution of convective accelerations (from peak velocities) with distance, (a) for a short elevated wave (135), (b) for a long elevated wave (415), (c) for a short N-wave (113), (d) for a long N-wave (307).
Figure 5.38: Local acceleration $\partial |u|/\partial t$ (blue) and velocity time series (green dashes) for test 02_081028. The flow periodically accelerates and decelerates, note that because the velocity probe measures $|u|$, the accelerations alternatively correspond to the flow travelling forwards and backwards.
Figure 5.39: Initial wave acceleration $\partial |u|/\partial t$ (blue) filtered with $n=3$ points centered moving average, total acceleration $Du/Dt$ (red dashes) and average value of the minimum local acceleration reached (approximately constant, over $\Delta t=2.5s$).
Figure 5.40: Video of the 2011 Japan tsunami wave hitting the coast (Russia Today, 2011). The red circle indicates the location of a building which does not fail at first impact but after 13s the water level has risen around it. First frame: first contact of the wave with the building; second frame: the water is surrounding the building but it is still standing; third frame: collapse of the building. This and other videos showing the Japan tsunami effects on structures are available from: http://www.asiantsunamivideos.com/
Figure 5.41: Evolution of average wave pressure on one wooden building placed on the flat land area (distance d1). (a) The blue circles represent the evolution of pressures with height on the front face of the building (see Figure 3.12), the green triangles represent the evolution of the average pressure with time of interaction between the flow and the transducers, on the front face of the building. (b) Schematic location of the building and main wave parameters, for test 27_081120 (wave ID 135).
Figure 5.42: (a) Evolution of the force of a short elevated wave flow travelling inland. Buoyancy forces were checked to not influence the initial motion of the object (see Figure 3.8). X-Xb represents the distance measured from the toe of the sloping beach. (b) Successive locations of weighted foam building for wave ID 135.
Figure 5.43: (a) Evolution of average pressure with distance retrieved from the wooden buildings pressure data (see location of pressure transducers on Figure 3.12). (b) Successive locations of buildings at distances \( d_1 \), \( d_2 \) and \( d_3 \), for wave ID 135.
Chapter 6. Conclusion

The objectives of this research have been to identify gaps in the understanding of the behaviour of long propagating waves at the shore, to address some of these gaps by obtaining and analysing long propagating wave data using a new pneumatic wave generator, to compare the results obtained to previous studies, and to study the influence of a range of wave parameters on runup. Another objective of this study was to test the capabilities of the new wave generator and associated experimental setup for the investigation of long propagating waves and their runup.

The literature review has revealed that while the generation and transformation of such waves from source to nearshore can be simulated by various numerical models, their propagation in the nearshore region, across the shoreline and inland is not accurately modelled. Physical models are an appropriate alternative to study these processes, however it has been found from past studies and recently developed facilities that even large scale experimental setups struggle to reproduce some key characteristics of these waves, i.e. their very long wavelength, with or without a leading depressed component. As a consequence, existing runup equations which are based on experimental results are valid for relatively short waves, and predicting functions are mainly based on parameters of the wave amplitude and a slope factor. No experimental study to date has quantified the influence on runup of other important parameters such as wavelength, wave form and potential energy.

To address these issues, an experimental approach has been taken, with a new wave generation system. Contrary to classic paddle wave generators, the new HR Wallingford pneumatic wave generator has been designed to exchange large volumes of water with the propagating region (flume), therefore it has the potential for
generating long and depressed waves. For the testing, the device was placed at the end of a 45m long wave flume, with a bathymetry composed of a constant depth region and a 1/20 sloping beach followed by a flat land area. Firstly, the wave generator was successfully tested for its ability to reproduce long and depressed waves in a controlled manner, and with the correct scaling assumptions. Subsequently, a series of tests involving elevated and N-wave forms and a range of wavelengths was designed. These tests were used in a variety of experimental setups involving measurements of surface elevations (in the constant depth region and over the slope), flow velocities (over the slope and inland), runup, and pressures on buildings. At this point it is important to note that all the data collected was not analyzed in this thesis, as the wave pressures on buildings and flow velocities around buildings are currently being processed for another study. It was found that the long wavelengths at play in the experiments triggered reflections from the slope. Sloshing in the flume was also observed.

The data processed here included wave elevation data, velocity data (without buildings), runup data and local pressure data on the front face of a model building at a fixed location, for a long elevated wave. Sources of experimental error were identified, and operations were performed to remove high frequency noise as well as low frequency sloshing on wave and velocity probe recordings. Reflections were observed to affect the second half of the positive amplitude in long wave signals, so periods, wavelengths as well as integral measures of the wave were calculated using the first half of the positive wave amplitude assuming symmetry.

The results show that long propagating wave dynamics were satisfactorily represented in the present experiments, and that shallow water conditions were respected throughout the testing. The shortest elevated waves did not change form during
propagation. N-waves also appeared stable, however, as most waves were longer than the constant depth region of the flume, their stability over distances much larger than their wavelength cannot be confirmed. Wave attenuation was shown to be negligible.

The data was used to find runup equations including parameters not studied experimentally before. A semi-empirical approach has been chosen to investigate the relationship between wave runup and the parameters characterizing the wave form (i.e. positive and negative amplitudes, wave height, wavelength, water depth, potential energy). Dimensional analysis was first used to relate these parameters to runup. The relationship identified was a power law. Secondly, simple linear regression analysis was used to find the combination of parameters resulting in the best fit to the experimental data. Expressions for runup were derived separately for short elevated waves, short N-waves, long elevated waves, long N-waves, all elevated waves, all N-waves, and all waves. In the case of short elevated waves, the present wave data was combined with the results from Synolakis (1987). The resulting expressions were consistent with previous studies, as for short waves (elevated and N-waves), the runup scaled as the positive amplitude \( R \sim a \). In addition, long waves were shown to belong to a different regime than short waves, as \( R \sim \sqrt{a} \). When expressed using a function of potential energy, the runup of the present waves was significantly different from the runup of Synolakis (1987) solitary waves. It is believed that potential energy is a useful addition to the parameters predicting runup.

The phase speed results show that the phase speed of the investigated waves was approximately 1.8 times greater than predicted by the shallow water phase speed or the solitary wave phase speed. An explanation could be the experimental waves belong to a significantly different wave regime compared to classic short solitary waves.
Flow velocity results do not correspond to an estimation of the wave front velocity, but they are representative of the overall flow velocity in the runup region. Local accelerations can be retrieved and show that the flow mainly decelerates as it climbs up the slope (which is consistent with the classical mechanics result, i.e. the total acceleration should be negative). However, convective accelerations can be positive locally.

Finally, preliminary results for wave pressures on model buildings were presented, for short elevated waves. The evolution of average pressures on the front face of a building as well as the evolution of the force as the flow travels inland both suggest that the overall flow velocity is likely to be approximately constant during the time of interaction of the flow with the structure; so flow depth would be the main parameter influencing the wave load on a structure. However, this observation has to be verified by further data processing and analysis.

The experimental work to date, the data and conclusions presented within this thesis are considered to be an invaluable addition to the framework and understanding of this complex area of tsunami research.

**Future work**

The experimental results presented in the thesis highlighted some difficulties in assessing the stability of N-waves and avoiding reflections that were associated with the experimental setup. These issues could be partially addressed in the future by using a much longer wave flume (i.e. with a propagation region twice the length of most “short” waves, as suggested in section 5.1.4). Moreover, use of a milder slope would
lower the Irbarren number and reduce the significance of reflections. Realistically, extremely long waves such as some of the long elevated and N-waves generated in this study (i.e. 200-300m long) would always be longer than any practically sized flume, and it is expected that they would still trigger some reflections. In addition to the aforementioned changes in the flume setup, a control loop could be designed to work in conjunction with the tank valve, which would allow the tank to instantly absorb any wave travelling in the direction opposite to the intended direction of wave travel.

The wave generator itself met the original design requirements of being able to reproduce long and depressed wave components. However, the wave amplitudes that could be generated were limited by the total head difference between the roof of the tank and the control water level in the flume. Improvements made to the device would need to include altering its vertical dimensions. Negative amplitudes could also be made larger by lowering the size of the outlet relative to the total height of the tank, and working with slightly larger water depths.

The runup results suggested that long wave inundation is predicted differently from short solitary / elevated wave runup. Hence, more long wave data would be needed to verify and / or refine the results obtained, as only a small number of tests were carried out due to time constraints (this is especially true for long N-waves). Moreover, it would be preferable to treat the solitary waves from Synolakis (1987) separately from the present short elevated waves, as indicated by the grouping of data into two distinct ranges of potential energies. If the runup predictions actually depend on the range of wavelengths considered, the present analysis could also be improved by grouping the waves into domains of $h/L$ instead of wave periods. Verification of the improved runup results could be performed on a range of different slopes, and using field tsunami data.
Finally, the limited analysis of wave impact on model buildings did not allow for the calculation of wave forces. Using the time evolution of flow depth and velocities around the structure in conjunction with the full dataset of wave-induced pressures on buildings would allow for the calculation of the different force components for several of building configurations and locations.

In the present setup, 2D waves were generated and propagated over a constant depth and slope, enabling a better understanding of the behaviour of long and trough-led waves. This differs from a real tsunami which propagates in a 3D environment and can interact with complex shorelines and bathymetries. The future development of a modified pneumatic wave generator set up for use with a wave basin would allow more complex interactions to be studied.
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Appendices

A - Valve Time Series

*Input Valve Profiles for short Elevated Waves*
Input Valve Profiles for short N-waves

Input Valve Profile for long N-waves
Input Valve Profiles for Long Elevated Waves

![Graph showing valve profiles over time](image-url)
**B - Glossary**

**Attenuation** (Mc Graw-Hill, 2005):

Reduction in the level of a transmitted quantity as a function of a parameter, usually distance.

**Breaking** (Le Mehaute, 1976b):

Different types of breaking have been presented by Le Mehaute (1976b). Breaking occurs:

1. When the particle velocity at the crest becomes larger than the wave velocity
2. When the pressure at the free surface given by the Bernouilli equation is incompatible with the atmospheric pressure
3. When the free surface becomes vertical

**Dispersion:**

The velocity of propagation varies with the wavelength/frequency of the wave. Tsunami are essentially dispersive in deep water and non dispersive in shallow water.

**Flow Depth** (Pomonis et al., 2006):

During post-tsunami surveys, flow depth is defined as the depth of the flow above ground level.
**Inundation** (Pomonis et al. (2006)):

From the shoreline and at mean sea water level, it is the maximum horizontal penetration of the waves in the direction normal to the beach during the flooding.

**Mach Stem Waves** (Griffith & Rossenfeld, 2008):

They are usually defined for nuclear explosions, as described below. However, analogies can be made for tsunami waves.

If the explosion occurs above the ground, when the expanding blast wave strikes the surface of the earth, it is reflected off the ground to form a second shock wave travelling behind the first. This reflected wave travels faster than the first, or incident, shock wave since it is travelling through air already moving at high speed due to the passage of the incident wave. The reflected blast wave merges with the incident shock wave to form a single wave, known as the Mach Stem. The overpressure at the front of the Mach wave is generally about twice as great as that at the direct blast wave front.

*Figure 7.0.1: A diagram of the Mach effect.*

At first the height of the Mach Stem wave is small, but as the wave front continues to move outward, the height increases steadily. At the same time, however, the
overpressure, like that in the incident wave, decreases because of the continuous loss of
energy and the ever-increasing area of the advancing front. After about 40 seconds,
when the Mach front from a 1-megaton nuclear weapon is 10 miles from ground zero,
the overpressure will have decreased to roughly 1 psi.

**Numerical Scheme** (Chapra and Canale, 2002):

It is a set of algebraic equations linking neighboring point values after discretization.

**Resonance** (Durandeau et al., 1995):

Resonance is the tendency of a system to oscillate at maximum amplitude at a certain
frequency. This frequency is known as the system's *resonance frequency* (or *resonant
frequency*). When damping is small, the resonance frequency is approximately equal to
the natural frequency of the system, which is the frequency of free vibrations.

**Runup** (Pomonis et al., 2006):

The runup is the height of the tsunami at the maximum inundation point above mean
sea level.

**Systematic error** (Turner and Hill, 1999):
It is an error causing all the readings to be biased, usually arising from unsatisfactory experimental method or measurement system design. Systematic errors can be constant or related to the actual value of the measured quantity.

**Tsunami Height (or wave height) (Pomonis et al., 2006):**

In the context of post-tsunami surveys, it is the height of the water relative to ground level (EEFIT report) if specified. The indicators are watermarks on buildings, scars on trees, and rafted debris.

**Wave amplitude:**

In a wave signal, it is the maximum (positive) or minimum (negative) water surface elevation, with reference to the mean water level.

**Wave height:**

The definition of wave height used for this study is the sum of the magnitudes of maximum and minimum amplitudes (for N-waves, periodic waves such as sine waves), it is equal to the positive amplitude for solitary waves, elevated waves. In the literature, sometimes wave height refers to wave amplitude (e.g. Lin et al. (1999)).