Run-up related to onshore tsunami flows

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RUN-UP RELATED TO ONSHORE TSUNAMI
FLOWS

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ASIAN SCHOOL OF THE ENVIRONMENT

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Abstract

The focus of the thesis is on onshore tsunami flows with special attention on tsunami run-up. In the aftermath of a tsunami, the most evident clue on the extent of a run-up is the water marks left behind. The trim-line recorded by surveyors is one of the most valuable pieces of information needed to determine the hydrodynamics parameters of the tsunami. However, it is also known that the most commonly used 2-D hydrodynamic models that attempt to reproduce the tsunami event do not produce good results during the onshore propagation phase. To bridge this gap, the use of an amplification factor proposed by Satake (1994) is investigated in this study. Experimentally, we used solitary waves to impinge on a model scaled coastal cliff to examine the flow depth at different onshore locations. The measured data are compared with the numerical results using the Delft3D-FLOW model. Besides, we also used dam break waves to evaluate the run-up both experimentally and numerically. The reason for doing so was two folds. First, dam break waves can provide us with a comparison to the often used solitary waves. Second, due to the discovery of a cave that has well preserved tsunami deposits off the coast of Banda Aceh, Indonesia, we found that experimental dam break waves have the potential not only to bring the sediments in, but also allow ample time for deposition to occur. Numerically, the sedimentation time within the cave is not easy to obtain using 2-D numerical models. Hence, based on the trim-line time series recorded, we provided a hypothetical sedimentation time that can be obtained using non-sediment laden flow tests.

Chapter 1

Introduction and Objectives

1.1 Introduction

The formation of most tsunamis are the result of fault movements in coastal regions. As the denser oceanic plate subducts beneath the continental plate, strain energy will be stored and accumulated until it exceeds the frictional forces that exist between the plates. When this occurs, an earthquake will first be triggered followed by a displacement of a massive amount of water that propagates as a wave train at rapid speed from the epicenter. The earthquakes that normally precedes the tsunami event are usually huge and shallow. These events often occur along the boundaries of the Pacific Ocean; aptly named the Pacific ring of fire. One such example was the 1960 Chilean tsunami event. The event was caused by a magnitude 9.6 earthquake off the coast of Chile. Lander and Lockridge (1989) stated that the local run-up was around 25m and that in Japan was around 6m. This tsunami propagated through the Pacific Ocean; severely affecting Hawaii and was even recorded as far as The Philippines. Dudley and Min (1988) noted that it took about 15 hours after the first earthquake for the first waves to reach Hawaii with recorded heights of around 10m. Carroll et al. (1960) estimated that the Chilean tsunami incurred around USD 650 million in damages, killed 2200 people, injured 3000 people and made 2,000,000 homeless.

Besides this conventional trigger mechanism, tsunamis can also arise through other causes. When a huge mass move towards the water body and lead to water displacement, a tsunami will also form. These tsunamis are normally termed as landslide tsunamis. One classic example of such a tsunami is the 1958 Lituya Bay tsunami in Alaska. Miller (1960) stated that a magnitude 7.9 earthquake had caused a rock filled landslide at Lituya Bay, Alaska. This landslide generated a wave that reached a maximum height of 524 m on the opposite

shore and created a 30m high wave that swept through Lituya Bay. In the process, it sank two fishing boats and killed two people.

In more recent years, the tsunamis that garnered most attention were the 2004 Indian Ocean tsunami and the 2011 Tohoku earthquake-tsunami events. The 2004 Indian Ocean tsunami resulted in up to 150,000 people missing and most were feared dead. Further investigations and estimates elevated the number closer to 220,000 with the maximum waves reaching 30m (USGS, Tectonics Observatory at the California Institute of Technology). Although the economic impact caused by the 2004 Indian Ocean tsunami is relatively small as the sectors affected are primarily fishing and tourism (Gunatillake, 2007), the same cannot be said of the 2011 Tohoku earthquake. According to a report from the International Business Times (2011), the damage caused by the tsunami and the resulting nuclear power breakdown has been estimated to be about USD 235 billion dollars by the World Bank and the Japanese government own estimate was at USD 300 billion. Moreover, up to 20,000 people were either reported dead or missing.

To most people, the most dangerous aspect of a tsunami is the height that it is capable of reaching. But in fact, its onshore speed is also another factor that contributes to the loss of lives in many of these events. Besides destroying nearshore structures (Yao et al., 2014) and claiming lives, tsunami flows can potentially uproot trees and generate debris that might obstruct evacuation passages or collide into building and humans, thus worsening the damage. Titov and Synolakis (1997) mentioned that as the tsunami reaches shallow water depth, it shoals and its wave height amplifies. This amplified tsunami can potentially inundate the coastline with flow velocities as high as 14 ms^{-1} , destroying everything in its path.

The run-up of a tsunami is more complex than perceived by masses. Firstly, there is the area of hydrodynamics where the parameters of interest are the maximum flow depth the tsunami can attain, the maximum inundation distance (which can be determined by knowing the maximum tsunami flow depth and the onshore topography) and the onshore flow field. Next is the area on sediment transport. To understand the sediment transport phenomenon, useful information needed are the flow velocity, bed shear stress and the particle's dimensions and characteristics. This study approach is more of a transient method. This is because to find out sediment concentration using this method, real-time data of the velocity flow field is needed. Thus researchers who focus on this area tend to invest their time in conducting mainly physical experiments and probably to a certain extent backed up by some numerical simulated results. Then, there is another group of researchers who go for field visits or conduct post tsunami surveys. These are the sedimentologists. They are interested to 1) trace the origin of the event that led to the deposition of the sediments onshore and 2) use the sediment characteristics to determine the hydrodynamic flow patterns that caused them to be deposited. Finally there is the research area on debris-laden flow. These are tsunami flows that carry with it a large amount of debris such as fallen trees, vehicles and even overturned houses.

The effect of debris on the maximum run-up has not been explored much. In fact, the maximum inundation distance would be affected if the tsunami needs to flow past objects as it propagates. This is due to the presence of drag, which reduces the flow energy. Huang et al (2011) did a study on the effect of emergent mangrove forests on solitary waves. In their study, they found that as the wave flows past a prototype mangrove forest, the transmitted wave height reduces as the solid volume density of the forest increases. Debris

flow is much more complicated as the objects are not rooted to the ground and they can be of all forms of shapes and sizes. To consider the importance of debris to the maximum run-up, one has to first consider the study location. For areas that are densely populated with buildings spread along the coast, the effect of drag on the incoming flow would certainly be more prominent. However, the study area of this thesis (Pulau Sibigau) is sparsely populated with few vegetation along the coast. Thus the likelihood of debris being formed and affecting the maximum run-up is very small.

Due to the economic losses and a high count of fatalities as a result of tsunamis inundations, researchers and the relevant government agencies are interested in two areas. The first is the mitigation of tsunamis. Bernard et al. (2006) mentioned that the gist of the tsunami mitigation lies in providing a reliable tsunami hazard assessment. Aspects that should be considered when drafting this include; the tsunami return period, maximum run-up height, forces, damages to buildings etc. Eisner (2005) came up with a list of seven principles that could make a coastal community prone to tsunami inundations be better prepared in the face of a future event. A more direct approach that most government agencies prefer to adopt is to defend their coasts with concrete structures. These coastal structures can be seawalls or breakwaters. However, constructing these structures is not only costly, it also does not guarantee absolute protection from the tsunami. Recent video recordings of the 2011 earthquake-tsunami event shows how the seawalls were being breached causing the waves to inundate to great distances inland. If coastal hardening through these structures does fail, the government agencies could actually make use of the coastal ecosystems that are present to reduce the impact of the tsunami when it attacks the coast in the future.

Cochard et al. (2008) provided a review on the effectiveness of different coastal ecosystems in mitigating tsunamis based on a case study of the 2004 Banda Aceh tsunami event. Their proposed coastal systems are separated into three groups: coral reefs and sea grass (nearshore), vegetation and forests (onshore), and sand dunes (onshore). Harada and Imamura (2005) studied the effectiveness of coastal forests in mitigating the effects of tsunamis. They concluded that coastal vegetation not only can reduce a tsunami's transmission energy but also act as buffers to stop drifting debris from being carried farther inland. One of the most common coastal ecosystems is sand dunes. Although, they are abundant along most coasts, they are not as widely studied as other ecosystems. Sand dunes are normally found very near to the coastline and if left undisturbed can be used as a first line of tsunami inundation defence. Despite an increasing interest in studying coastal ecosystems, there are few researches on sand dunes. The current research area on sand dunes is mainly on post extreme event field survey. A recent study by Mascarenhas and Jayakumar (2008) gave evidence that some areas along the Tamil Nadu coast, India were protected by sand dunes from the 2004 tsunami event. They concluded that areas where dunes had been leveled by humans to create more land space recorded much higher inundation distances.

Sim et al (2013) mentioned that for coasts with sand dunes, the two key parameters that may influence tsunami energy dissipation are the relative dune height (H/H_0) and relative dune space (L/L_0), in which H is the dune's height, H_0 is the incident wave height, L is the dune spacing and L_0 is the incident wave length. Two of the possible mechanisms to dissipate tsunami energy under such settings are wave breaking and/or vortex shedding as the wave moves over the dunes. Energy loss through vortex shedding is influenced by the

geometric properties of the dunes. Wave breaking or overtopping on the dunes is dependent on the flow depth and the height of the dune. Of the two mechanisms, energy dissipated through wave breaking is much greater than the energy loss through vortex shedding. However, this does not mean that energy loss through vortex shedding is insignificant. If there are a series of dunes, the cumulative energy loss as the wave goes over each dune would also render vortex shedding to be a crucial energy loss mechanism. In other words, it can be expected that if H/H_0 is large, both vortex shedding and wave breaking or overtopping will lead to significant energy dissipation. If H/H_0 is small, the main mechanism of energy loss will only be through vortex shedding. This implies that dunes are most efficient in dissipating wave energy if H/H_0 is large, i.e., such coastal dune system can be fully optimized as a coastal defence measure only against small tsunamis or storm surge events. They would most probably not be capable to function as an effective defence mechanism if the tsunami wave height is very large.

In reality, tsunamis have wavelength that can be up to a few hundreds of kilometres long offshore. When the waves approach the foreshore, they shoal and then break before rushing up towards the coastline and begin to inundate the onshore zone. Although the near shore tsunami wave length is shorter than when it is offshore, it is still most probably much longer than the dune spacing (if dunes are present on the coast). This would mean that the possibility of an actual real setting having a large L/L_0 is very low. On the contrary, coastal areas that have dune heights in the same order of magnitude as the incoming wave height are more readily identifiable. The onshore tsunami bore that propagate inland after the tsunami breaks normally has a depth around 3-6 m. This is about the height of sand dunes on the coast (although some can reach as high as 130 m, e.g, Dune of Pyla, France). This

would then mean that, in the real world the likelihood of having dunes that have a greater H/H_0 ratio is higher than having dunes with a greater L/L_0 ratio. Hence, H/H_0 could be the more important parameter in dissipating tsunami energy.

The second area that researchers and government agencies could be interested in with regards to tsunami is understanding the causes of each individual tsunami. There are many approaches to take when investigating tsunamis. There are now post tsunami survey teams who arrive at the site to collect sediment samples, measure water marks on buildings and also conduct interviews with survivors (e.g. Liu et al., 2005, Fritz et al., 2006). Recently there has also been researchers who used video recorded by survivors to obtain crucial flow information of the tsunami event (e.g. Foytong et al., 2013). Another real time measurement system reported by Gonzalez (2005) was the use of the DART (Deep-ocean Assessment and Reporting of Tsunamis) system in tandem with real-time bottom pressure measuring tsunameters. This system managed to successfully negate a distant warning signal from a tsunami that is potentially not serious and then relaying this message to the warning centres.

For decades, numerical models have been the go-to method to understand tsunamis for researchers. These models if thoroughly tested not only can help us understand the causes of tsunamis, but also be used as a crucial early warning system tool. Some of the numerical tools used by researchers to replicate a tsunami event include COMCOT (Cornell University, USA), TSUNAMI-N1/2 (Tohoku University, Japan) and MOST (National Center for Tsunami Research, USA). These numerical models are capable of evaluating the entire process of a tsunami; from its generation to onshore inundation. Most of these tsunami models, were governed by the non-linear shallow water equations (NLSWEs) (Imamura et al., 1988, Liu et al., 1995, Cho and Yoon, 1998) and then numerically solved

to obtain the necessary hydrodynamic information. However, the basis of these NLSWEs models is that they assume a hydrostatic approximation in the vertical direction and thus ignoring the vertical velocity fluctuation over the water depth. This assumption is justified by the fact that tsunami wave lengths are usually much larger than ocean depths. There are other tsunami models that tried to improvise on the models mentioned above by incorporating the vertical velocity variation. These models which were constructed based on Boussineq-type equations (Peregrine, 1967, Madsen and Sorensen, 1992, Nwogu, 1993, Lynett and Liu, 2004a, b), include more terms to better describe the frequency dispersion in shallow water and thus can simulate nearshore flows better than the NLSWEs based models. Examples of such models include CoulWave (Lynett and Liu, 2004a, b) and FunWave (Kennedy et al., 2000, 2001). However the main drawback of the models governed by the Boussineq-type equations is that they are computationally more expensive to run; in general, they require days or months to complete a simulation that NLSWE-based models could complete in hours (Liu, 2013).

There is also the possibility of applying a full three dimensional model to the domain of interest. Huang et al. (2012) did some work to investigate the onshore flow depths of a tsunami bore and he also tried to determine the force exerted by the tsunami bore on an onshore tree. This study comprised of experimental work done by the author of the thesis as well as numerical simulation carried out by peers from Taiwan. It should also be noted that the slope bathymetry are similar to the one used in this thesis. The numerical model used in this above mentioned study was a three dimensional volume of fluid solver. The volume of fluid (VOF) method was used to track the free surface elevation of the flow. Comparison of the numerical results to the experimental results show that the numerical

model can adequately capture the return bore motion. Besides it was also able to successfully record the instances of aerated flow near the cliff toe due to turbulence that set in the moment the tsunami bore impacted the cliff.

Figure 1.1 shows the comparison of the simulated flow depth against the experimental flow depth. G1 to G5 indicates five different onshore locations with G1 being the shoreline and G5 the toe of the cliff. From the figure, it can be clearly seen that the numerical results matches rather well with the experimental results. This goes to show that using a three dimensional model would most likely produce a much better comparison to the experimental results for this thesis.

Fig 1.1) Comparison of measured and simulated flow depths at 5 cross-shore locations for the hypothetical toe-eroded slope (Image courtesy from Huang et al. (2012))

Another approach that can be utilized in this study is a mesh-less system. A meshless system is one whereby the domain need not be gridded. One such approach is the Smoothed Particle Hydrodynamic (SPH) method. To surmise, this method makes use of a large number of particles that have pre-defined diameters to represent the entire fluid body. In the scope of

solitary waves and tsunami application using this numerical model, Shao (2005) studied the application of solitary waves and also breaking waves. Rogers and Dalrymple (2008) recorded their findings using SPH as tool for tsunami simulation. Jian et al. (2015) studied the solitary wave run-up on a coastal cliff by comparing the results from the SPH model to the numerical model. This study consists of experimental work conducted by the author of the thesis and numerical work from the paper's (Jian et al. (2015)) first author.

Figures 1.2 and 1.3 show the comparison of the simulated flow depth using SPH against the experimental flow depth. S1 to S4 indicate four different cross-shore locations with S1 and S2 offshore and S3 and S4 onshore. From the figure, it can be clearly seen that the numerical results matches rather well with the experimental results. A detailed look at the location S4 from both incident wave heights show that the SPH numerical model is capable of matching the results obtained to the experimental results.

Fig 1.2) Time history of water levels at the four monitoring points (a) S1; (b) S2; (c) S3 and (d) S4 by the SPH model and the experimental measurements for $H_0 = 0.025\text{m}$ (Image courtesy from Jian et al. (2015))

COMCOT (Liu et al., 1998) has been a useful tool for tsunami studies based on its usefulness in having different mechanisms for generating tsunamis such as earthquakes due to fault rupture and submarine landslides. There are various different fault models that can be used with the COMCOT model. Li et al. (2014) conducted a comparison of four different fault rupture models for the 2004 Indian Ocean earthquake to investigate the sediment transport process along the Khao Lak coast, Thailand using the improvised COMCOT-SED model (Li et al., 2012). These fault models were derived based on data collected from various sources such as; GPS data, tidal gauges, seismic waves etc. Besides COMCOT, other numerical models that have been used to study the run-up and inundation of the tsunami affected areas include FunWave (University of Delaware, USA), Tsunami-Claw (Washington University, USA), MIKE21 (DHI, Denmark), and 3DD (ASR Ltd., New Zealand).

Fig 1.3) Time history of water levels at the four monitoring points (a) S1; (b) S2; (c) S3 and (d) S4 by the SPH model and the experimental measurements for $H_0 = 0.055\text{m}$ (Image courtesy from Jian et al. (2015))

The advantages of using models such as COMCOT for tsunami studies, early warning system applications and probabilistic tsunami hazard assessment are that it is very quick, computationally efficient, able to incorporate different tsunami generation mechanisms and it can now even have sediment transport incorporated. However, when it comes to the onshore region, these NLSWEs based models start to reveal their flaws. Liu et al. (1991) showed that numerical models based on NLSWEs are incapable of dealing with flows over complex topography. Hamilton (1977) also mentioned that in regions where steep slopes are present onshore models based on the NLSWEs do not produce good results. In most cases, the NLSWEs can correctly predict the wave evolution up till near wave breaking. At this moment, even for a plane beach with a very mild slope the NLSWEs over predicts wave celerity and the weakly dispersive Boussinesq approximation under-predicts wave celerity. Synolakis and Skjelbriera (1993) proposed a two-zone shoaling model to better predict data from experiments with mild sloping beaches. They suggested that at the offshore zone, the Green's law asymptotic behaviour has a better fit whereas closer to the breaking zone, the Boussinesq approximation is more appropriate. Synolakis (1991) also added that for steep beaches, the breaking waves will evolve according to Green's law and then decay. Their statements are further backed by a recent study by Hill et al. (2012). In their study, they compared water levels observed onshore at Pulau Sibigau, Indonesia to their simulation results using COMCOT and found that the results produced by the model NLSWEs significantly underestimated the water levels on this island, where an important topographic feature is the presence of a very steep inland cliff.

The study conducted by Hill et al (2012) highlighted the need of an empirical correction to tsunami run-ups and heights given by NLSWEs based model in order to make a meaningful

comparison between field observations and numerical results. The practice of correcting tsunami run-ups and heights onshore for using NLSWEs to study tsunami problems has been used in past studies. After the 1992 Nicaragua earthquake and tsunami event, Satake (1994) compared the tsunami heights measured onshore with the computed tsunami heights offshore at a shallow water depth. Satake (1994) was the first to term the ratio of the measured onshore tsunami run-up/height to the computed tsunami height at a reference location in shallow water the tsunami amplification factor. Recognizing that models based on NLSWEs are incapable of producing accurate results onshore, Satake constructed a numerical domain with an offshore boundary in a shallow water region and then related the wave height at this boundary to the recorded onshore tsunami run-up/height. However, it has been noticed in the literature that this tsunami amplification factor varies with locations: for the 1992 Nicaragua earthquake and tsunami event, Satake (1994), Imamura et al. (1993), and Piatanesi et al. (1996) all provided different values of the tsunami amplification factor for the same event. Although their study areas are the same, the actual locations in which they computed the nearshore tsunami heights are different. This shows that the effect of local topography on the tsunami amplification factor cannot be ignored.

The best way to determine the tsunami amplification factor is to use a NLSWEs based model to first obtain a numerical surface elevation at shallow water depth. With the known onshore run-up height, one can then correct the computed surface elevation with an empirical factor or formula. The main issue here is that there has to be clear indicator of a run-up mark or trim line so that any inversion can take place. Even with the presence of such an indicator, it means that the entire study would only be useful in reconstructing the individual tsunami event but does not necessary lead to better understanding of past events

or preparing for future events. The next alternative is then to have a method to produce multiple scenarios of tsunami propagation and run-up under a controlled environment and then compare with the numerical model based on NLSWEs. Laboratory experiments, though unrealistic due to its scale effects and incapability of actually producing a tsunami-like wave form; can partially resolve this issue.

For most physical experiments done to investigate tsunamis, most people have used solitary waves to replicate the tsunami wave front in their studies (Synolakis (1986, 1987), Zelt (1991a, 1991b), Tadepalli and Synolakis (1994), Hsiao and Lin (2010)). Other than being easy to generate, solitary waves have been frequently used in past experimental studies as most of the wave front of observed tsunamis is very similar to that of solitary waves (An and Liu., 2014). However, solitary waves are actually very different from tsunami waves. Madsen et al., (2008) presented a thorough review on the applicability of using solitary waves as a benchmark for tsunami studies. They mentioned that in reality it is almost impossible for tsunami waves to develop into solitons as it propagates into the shallow water depth region. Moreover, due to geographical constraints, there is also not enough space and time for the leading undular bores to fully develop into solitons. Additionally due to laboratory limitations, most experiments were conducted with an incident wave height to water depth ratio in the region of 0.05 to 0.5 which corresponds more to wind waves than long waves. Nonetheless, solitary waves are still frontrunners when it comes to physically studying tsunamis on a laboratory scale.

Another wave form that can be considered for physical tests are the dam break waves. From the experimental approach, some of the work done using dam break waves in relation to studying tsunami include: Aguíñiga et al.'s (2013) work on investigating the force of the

dam break flow on coastal structures, Liu et al.'s (2014) study on boulder movement using dam break waves and Bremm et al.'s (2015) experimental study using dam break waves to study flow-structure interactions.

To analyze instantaneous dam failure, the standard non-linear shallow water equations can be used. In the simplified one dimensional horizontal mode, where a horizontal bottom surface is assumed, the shallow water equations are;

$$\frac{\partial \eta}{\partial t} + \eta \frac{\partial U}{\partial x} + U \frac{\partial \eta}{\partial x} = 0 \quad (1.1)$$

$$\frac{\partial U}{\partial t} + U \frac{\partial U}{\partial x} + g \frac{\partial \eta}{\partial x} + g \frac{n^2}{\eta^{\frac{4}{3}}} U |U| = 0 \quad (1.2)$$

Here, η is the surface elevation or in this case the water depth at a point after the dam failure, U is the depth averaged velocity, g is the gravitational acceleration and n is the Manning's roughness coefficient.

For ideal flow cases where the bottom roughness can be ignored, Ritter (1892) stated that the wave front moves with a celerity $= 2\sqrt{gD}$, where D is the water depth and it moves away from the wall in the opposite direction with celerity $C = \sqrt{gD}$, between these two regions the flow velocity and water depth can be given as,

$$U = \frac{2}{3}(xt + \sqrt{gD}) \quad (1.3)$$

$$\sqrt{g\eta} = \frac{1}{3}\left(2\sqrt{gD} - \frac{x}{t}\right) \quad (1.4)$$

These solutions are able to provide a good approximation to the dam failure case but fails to consider flow resistance. To take into account of the wave tip which is not present when using the equations from Ritter (1892), the flow resistance have to be considered. Without this consideration, the extent of flow propagation, flow velocity and flow depth will be overestimated. Dressler (1952) proposed a perturbation of the velocity and wave celerity using Ritter's equations to observe how modifications in the final desired parameters vary with changes to bottom roughness effects.

Witham (1954) used an integral approach to address this challenge. For the wave-tip region he recommended a force-balance approach. In his approach, the flow resistance acts as a balance to the forces acting on it. Hogg and Pritchard (2004) criticized Witham's evaluation of the wave front velocity. They noted that Witham (1954) had failed to consider the spatial variation of the horizontal velocity in the wave tip region. Using an asymptotic expansion of the flow depth, flow velocity and wave front, they found that Witham's equation underpredicts the flow velocity in the wave tip region by up to 20%. For their analytical approach; rather than implementing a direct formulation of the flow velocity, they conducted a numerical integration of the governing equation with a drag coefficient that is a function of the Reynold's number.

The equations that Dressler (1952) and Witham (1954) proposed can provide the analytical solutions of dam failure cases in idealized scenarios. However, they failed in more complex cases. The assumptions under which these equations can be used are,

1. Constant roughness factor
2. Flat bed topography
3. Instantaneous dam break

Since then, more analytical work has been done so that the theories need not be restricted by these assumptions. Chanson (2006a) provided simplified analytical solutions to the dam failure problems by considering a varying roughness factor and sloping bed profile. Prior to solving for the hydraulic characteristics of the dam failure he made several assumptions. First, according to Dressler (1954), Estrade (1967) and Liem and Kongeter (1999), the spatial variation of the wave front velocity is minimal and thus inertial terms in the governing equations could be ignored. Second, the dam failure is assumed to be instantaneous. Third, the flow velocity in the wave front is assumed to be the wave front celerity. Adopting these assumptions meant that the momentum equation can be simplified into the diffusive wave type system where,

$$\frac{\partial \eta}{\partial x} + \frac{fU^2}{8g\eta} = 0 \quad (1.5)$$

Here, the Darcy-Weisbach friction coefficient (f) is used instead of the Manning's roughness coefficient as the flow will be turbulent and unsteady and the Manning's roughness coefficient is deemed unsuitable. The Darcy-Weisbach friction coefficient can be expressed by the Altsul formula (Genic, 2011) as

$$f = 0.1 \left(1.46 \frac{k_s}{4\eta} + \frac{100}{4Re} \right) \quad (1.6)$$

In reality, tsunami surges are often the main cause of damages and casualties (Borrero, 2005; Tsuji, 2005). As the surge travel inland, its front is characterized by quick fluctuations in the flow depth and velocity. These surge waves are reminiscent of surge waves as a result

of a dam breach. Chanson (2006b) compared the water depths from a video recording of the 2004 Indian Ocean tsunami event to the results obtained using his analytical approach of dam break waves. The results were quite comparable. This shows that to a certain cross-shore extent, the surge created by dam break waves do resemble that of the nearshore to onshore tsunami surge.

1.2 Objectives

Figure 1.4 shows the main research area of interest for this thesis. The figure shows a bat cave etched onto a steep cliff that has well-preserved tsunami deposits which have remained undisturbed over thousands of years. This bat cave has an opening of about 10m with the cave extending about 200m into the cliff. At the rear of the cave, there is a hole at the ceiling.

The preservation of tsunami deposits could be an important data in determining the incoming-wave characteristics of historical tsunamis. For example, Paris et al. (2007) described the erosional and depositional process of sediments along the coast of Lhok Nga Bay, pointing out that if there were successive tsunami waves, the erosion and deposition processes of each wave would be interlinked. Although it might be possible to reveal the number of waves that impinge the coast from the number of distinct depositional layers, the onshore wave parameters derived from simple models or numerical modeling are unlikely to be accurate. In addition, a very distinct tsunami trim-line was found on the cliff (about 20m above mean sea level), which indicates the extent of the most recent tsunami run-up (2004, Indian Ocean tsunami) on the cliff. Reconstruction of these events may be done through post tsunami field surveys or studying tsunami deposits with the help of numerical

modeling. The observed data can potentially be used to derive the tsunami source parameters for the 2004 tsunami event.

Fig 1.4) Photograph of a coastal cliff with a bat cave found along the coast of Banda Aceh, Northern Sumatra, Indonesia. Tsunami deposits were found in the cave. (Photograph courtesy of Professor Kerry Sieh and Professor Charles M. Rubin, both at the Earth Observatory of Singapore.)

Physical experiments on sediment transport due to solitary waves impinging on the coast have been conducted by some researchers with almost similar observations. Kobayashi and Lawrence (2004) did their tests using incoming breaking solitary waves. In their experiments, they observed that as the solitary wave broke, it suspended a significant amount of sand from the sand bed. As the wave propagated further inland, it led to a high run-up. Once the water body reaches its peak, it converts most of its kinetic energy into potential energy before moving back into the offshore region as a sheet flow. This sheet flow has a very high velocity and it can push the sand seawards, leading to offshore deposition of the sand. Young et al. (2010) conducted a study to better understand the mechanisms behind the sediment movement when a solitary wave moves up and down a

plane sand beach. They observed that during the run-up, turbulence aided in entraining the sand from the foreshore and the wave's landwards momentum brought the sand farther inland. As the wave retreats, it forms a hydraulic jump that collided with the still water. Within this region, the flow velocity drastically reduces and deposition will occur. Chen et al. (2012) also performed experiments to examine coastal morphological changes and mean grain size distribution using solitary waves. Their tests results showed that erosion on the foreshore was caused by the strong drawdown of waves and this process created an offshore sandbar. The formation of offshore sandbar was also observed in both Kobayashi and Lawrence (2004) and Young et al. (2010). These experimental studies exhibited similar bed morphological trends to the aftermath of the 2011 Tohoku earthquake and tsunami event. The field study done by Goto et al. (2012) revealed that eroded sediments not only can be transported and deposited landwards; it can also be transported back into the sea and be deposited due to the return tsunami flows.

Song (2013), conducted tests on bed-form evolution due to the run-up and run-down phase of a solitary wave impinging on a composite beach coastal slope. The main difference between her study and those done by the previous studies was that there was a steep coastal cliff onshore. In her study she found that the inflow phase did not suspend much of the sand; rather a significant amount was suspended by the amplified return flow in the drawdown phase. Her findings (refer to figure 1.5) was similar to that of Chen et al (2012) and Young et al (2010), who noticed that the amount of suspended sediments in the breaking region was insignificant.

Fig 1.5) Inflow and outflow process of a solitary wave impinging on a composite bed slope. (Fig 14 from Song, 2013)

Song's (2013) experimental setup was similar to ours. Compared to other studies done, the sediment transport mechanisms differ vastly due to the presence of the cliff. During the run-up phase, the presence of the cliff impedes the flow and prevented it from reaching a farther location. So looking at the run-up alone, sediments that are picked up during this phase are not pushed all the way up the beach slope but remains entrained in the flow until the wave hits the steep cliff. During the drawdown phase, Song (2013) observed an amplified return flow instead of a sheet flow which the other studies have noticed. The amplified flow is capable of entraining more sediments due to turbulence caused by the breaking of the return flow upon impact on the cliff. Despite the major differences in the mechanisms of sediment transport due to the presence of the steep onshore cliff, the erosion and deposition trends were similar to that observed by that from Kobayashi and Lawrence (2004), Young et al. (2010) and Chen et al. (2012).

It is true that the vertically-integrated shallow water model does not accurately represent the effect of a complex topography within the cave. In fact, when the tsunami reaches most onshore parts, the application of the shallow water equation model as a solver for such cases is an approximation at best. Hence there are other more viable models that can be used. However, a non-linear shallow water equation model was still adopted instead. This is because the scope of this thesis is to adopt an approach to accurately match the numerical results to the experimental results using the shallow water equation model. The persistence of using the shallow water model boils down to the fact that despite its inaccuracy especially in the onshore region, the shallow water equation model is the least computational intensive and as a result it takes a much lesser time for each simulation to complete compared to the other approaches. To sum up, the core of this thesis is not to find a numerical model that can best match the experimental results. Instead, this thesis explores a method which can be implemented in the more commonly used shallow water equation model to improve its output accuracy in the onshore region.

The first objective is then to obtain a run-up amplification factor through conducting both physical experiments and numerical simulations based on the NLSWEs based models. The run-up trim-line shown may be the result of 1) splashing water as the tsunami reaches the area or 2) extreme nearshore flow amplification due to wave reflection. As mentioned in section 1.1, NLSWEs models are the most frequently used approach in the study of tsunamis at present but its main discrepancy occurs during the onshore propagation phase of the tsunami. By doing both the physical and numerical experiments, we can better understand the nearshore flow amplification and also any nearshore flow hydrodynamics and at the same time propose an initial correction factor that can possibly be incorporated in these

numerical models. The next objective is then to evaluate the differences between the different wave forms used in the physical and numerical experiments. Both solitary waves and dam break waves are tested to investigate their respective effects on the onshore flow depths and run-up on the cliff. Reverting to the research area of interest, it is important to find out how the sediments are able to enter the cave. However, the investigation of sediment transport is beyond the scope of this thesis. Besides the sediment source and sediment characteristics, one other key parameter that should be determined is the time interval allowed for sedimentation to occur. The knowledge of this parameter can help to determine the onshore characteristics of the wave that arrives onshore. Finding out the sedimentation time of the cave under different wave forms and offshore flow characteristics will then be the last objective of this thesis.

Chapter 2

Methodology

2.1.1 Physical experiments equipment

Solitary waves were used to simulate tsunami-like waves in a wave flume of 34 m in length and 0.54 m in width. These waves were generated by using a piston-type wave generator located at one end of the wave flume. According to HR Wallingford (2015), the main advantage of a piston type wave maker is that the paddle stroke is not restricted, enabling the wave paddle to extend back and forth horizontally. The HR Wallingford wave maker that we used has a wave paddle attached to an overhanging electric actuator that comprises an extruded beam. The movement of the beam is driven by an AC servo motor, which has a drive system that provides the signal gain and damp to ensure that the wave paddle follows the initial position signal. Moreover, the system has a dynamic absorption mode where wave reflection from the paddle itself can be minimized. For the solitary wave generation, the wave generator was designed to generate waves based on the Goring (1978) method. Based on his method, to obtain a propagating solitary wave that has a surface elevation expression of $\eta(x,t)$ with an incident wave height H_0 where,

$$\eta(x,t) = H_0 \text{sech}^2[k_0(ct - x)] \quad (2.1)$$

The piston has been programmed to move based on the following displacement function $\xi(t)$, where,

$$\xi(t) = \frac{H_0}{K_0 h_0} \tanh[k_0(ct - \xi)] \quad (2.2)$$

k_0 is a wave number, h_0 is the water depth and c is the wave celerity.

Theoretically, a solitary wave has an infinite length. The solitary wave number k provided by Goring (1978) has been widely accepted to be

$$k = \frac{1}{h_0} \sqrt{\frac{3H_0}{4h_0}} \quad (2.3)$$

where H_0 is the wave height and h_0 is the water depth

Madsen et al (2008) mentioned that it is more purposeful in practice to define the solitary wavelength as the wave length in effective space and time. This means that actual limits of a solitary wave can be imposed for a finite wavelength to be given. To do so, he first define a solitary wave length L_0 as the ratio of 2π to the solitary wave number which is similar to most other waves.

$$L_0 = \frac{2\pi}{k} \quad (2.4)$$

To check the validity of using this assumption to represent the solitary wavelength, an aerial integral of the solitary wave surface elevation equation can be done. Using equation (2.1) and imposing the limits of $+L_0/2$ to $-L_0/2$, when in theory it should have been from $+\infty$ to $-\infty$, the aerial integral can be expressed as,

$$A = \int_{-L_0/2}^{L_0/2} H_0 \text{sech}^2 [k(x - ct)] dx = \frac{2H}{k} \tanh(\pi) \quad (2.5)$$

From equation (2.4) it can be seen that the non-unitary term $\tanh(\pi)$ is approximately equals to 0.9963. This means that the solitary wavelength proposed covers almost all of the wave

except the extreme ends. Thus it is justifiable to simply define the solitary wavelength as in equation (2.4). Combining equation (2.3) and (2.4),

$$L_0 = 2\pi \sqrt{\frac{4h_0^3}{3H_0}} \quad (2.6)$$

Equation (2.6) is then the simplified definition of a solitary wave wavelength.

Acoustic Ultralab ULS sensors (USS20130, General Acoustics, Kiel, Germany), (s1-s4) were used to record water surface elevations at the cross-shore profiles. The acoustic sensors have an effective range of 200 mm to 1300 mm. Taking into account the working range of these sensors, they are mounted at a height of around 600 mm from the top of the still water level. These sensors have a resolution of 0.2 mm. Figure 2.1a shows the Ultralab sensor that was used and figure 2.1b shows the control box that could accommodate up to four Ultralab sensors.

Fig 2.1a) Ultralab sensor used and b) Ultralab sensor connected to the control box. Images

courtesy of General Acoustics

In addition to the measurements taken from the Ultralab sensors, web cameras were also used to capture the flow patterns: the cameras were used either on the side or from the plan view, primarily for capturing the onshore flow field. The web cameras used are corded web cameras from Logitech (model: HD Pro Webcam c920). These web cameras have a maximum frame rate of 30 fps at a capture resolution of 640 x 380 pixels. Figure 2.2 shows the webcam that was used in the experiments.

Fig 2.2) Logitech HD Pro Webcam c920. Image courtesy of Logitech

The time series of the surface elevations at various locations can be extracted from the video recordings. To do so, a single recording has to be broken down into individual jpeg frames. This can be done easily using any video to image converter. The number of frames extracted from each video recording is dependent on the duration of the video recording as well as the frame rate of the video. For the test runs conducted for this study, the recorded duration has been pre-set to 1 minute from the start of the forward movement of the wave paddle. The principle of the image digitizing process is to scale-up the size of the image coordinates to the actual laboratory sizing. This can be done by pre-determining the actual laboratory scale size on the image coordinates. The WebPlotDigitizer software was used in this study.

2.1.2 Dam break wave tests equipment

For the dam break tests cases, the equipment used is the same except that the wave generation mechanism is different. A reservoir was made within the wave flume and was filled with water. A mobile gate was then installed at the other end of the reservoir.

The dam break release mechanism is controlled by using a pulley mechanism as shown in figure 2.3. A Perspex gate was placed 8.84 m away from the toe of the beach profile. This gate was then placed in an aluminum frame to keep it in place. Rubber stoppers were fitted on the sides of the frame to prevent water from seeping through. To lift the Perspex wall, a polypropylene rope was attached to it on one end, roped through a pulley system and the other end fitted to a carabineer which will hold the weights require to raise the Perspex gate.

The weights used were four concrete cuboid bricks. These four bricks were secured together using a nylon rope and a loop was made so that it can be hooked onto the carabineer. A helper held onto the Polypropylene rope as a manual support until the commencement of the test so that the Perspex gate will not be abruptly lifted beforehand. We had earlier determined that four bricks were needed to pull the Perspex gate instantaneously. However, for this study, it would also be helpful if we were to investigate how different gate opening speeds will affect the incident wave profile. The most important factor that affects the gate speed is the mass of the weights used. From simple mechanics, the following equation describes the physical system;

$$\frac{m_{weight}}{m_{gate}} = \frac{g + a}{g - a} \quad (2.7)$$

Fig 2.3) Frontal view of dam break gate mechanism

Where g is the gravitational acceleration and a is the acceleration of the entire system. It may be inferred from this simple equation that to have an instantaneous release, i.e. higher a , the mass of the weight used have to be much greater than the mass of the gate. However, it is not possible to continually increase the mass of the weight used. This is because the helper cannot continually support the additional weights. To obtain tests with of slower gate release speed, the easiest method is to use less bricks. This approach did not prove successful as using less bricks resulted in the gate not being lifted at all. Hence, we had to manually control the descent of the four bricks so that it will not instantly fall to the ground at the time of release. Due to the fact that it would be difficult to quantify the factors that affect the gate release speed using this method, a qualitative analysis on how different gate release speeds influence the incident wave profile was described instead. This part of the study can be found in chapter 4 of the thesis.

2.1.3 Comparison of the Ultralab sensor recordings with those digitized from video recordings

Since two different methods were used to obtain the values of the surface elevation at the onshore locations for our experimental results, a comparison between both sets of data was necessary. We picked out one example to validate the accuracy of both methods. From the study of the solitary wave amplification factor (refer to figure 3.8 for setup), at location s4, we compared the Ultralab sensor readings to the readings digitized from videos. Figure 2.4 shows an example of this comparison for the case of cliff slope $\theta = 14^\circ$, and incident wave height $H_0 = 2.5$ cm, and figure 2.5 shows another example for the case of $\theta = 79^\circ$ and $H_0 = 5.5$ cm. The data reveal that the Ultralab sensor recordings show a slightly higher maximum water level than the one obtained by digitizing video recordings. At this location, the discrepancy is less than 7%, which could be attributed to the following factors: (1) there could be a variation of water level in the longshore direction; (2) whitecaps could have been formed on the wave crests; and (3) the low frame rate of the web camera could have caused the video recording to slightly miss the maximum water level. If the flow is not uniform in the longshore direction, using either Ultralab sensors or video recordings to obtain the surface elevation is fundamentally inaccurate, since both the sensor and the web camera only take measurements in a single cross-shore plane. In the rest of the study, the reported results of the water levels at various locations were obtained from the video recordings, and thus about a 7% under-estimation of tsunami height may be expected.

Fig 2.4) Comparison of sensor readings to digitized video recording for s4; $H_0 = 2.5$ cm, $\theta = 14^\circ$

Fig 2.5) Comparison of sensor readings to digitized video recording for s4; $H_0 = 5.5$ cm, $\theta = 79^\circ$

2.2.1 Introduction to the numerical model

For this study, the following nonlinear shallow water equations are used to model the propagation of a solitary wave on a beach profile:

$$\frac{\partial \eta}{\partial t} + \nabla \cdot (D\vec{u}) = 0 \quad (2.8)$$

$$\frac{\partial \vec{u}}{\partial t} + \vec{u} \cdot \nabla \vec{u} + g \nabla \eta + \frac{\vec{\tau}_b}{\rho D} = 0 \quad (2.9)$$

where $\nabla = (\partial/\partial x, \partial/\partial y)$, η is the surface elevation, $D = \eta + h$, is the instantaneous flow depth, $\vec{u}$ is the depth averaged velocity vector, and $\bar{\tau}_b$ is the bottom shear stress given by

$$\bar{\tau}_b = \frac{\rho g \vec{u} |\vec{u}| n^2}{D^{1/3}} \quad (2.10)$$

with n being the Manning's roughness coefficient having units of $(\text{s/m}^{1/3})$. The above NLSWEs are solved using the Delft3D-FLOW software package with appropriate initial and boundary conditions corresponding to the experimental setup. Delft3D-FLOW has 3 modules, and we used only the module for depth-integrated flows.

Delft3D-FLOW is an easy to use numerical model that is capable of resolving multi-layered flow fields in the vertical direction. The software package has a GUI (graphical user interface) that provides the user with a clear idea on the files that have to be prepared before running the simulation. In general, the user only has to prepare the bathymetric data file, an initial condition file (if necessary) and a boundary file. Other physical parameters that need to be specified like water density etc, can be keyed in the GUI. Numerical parameters like the number of vertical layers and the ratio of each layer height to the water depth can also be entered once the above-mentioned files have been created. The drying and flooding scheme applied in Delft3D-FLOW is suitable to be used for coastal regions as in our case (Delft-3D Flow Manual, 2011). When used with the Flooding scheme for advection in the momentum equation, it is also effective and accurate for rapidly varying flows due to the presence of hydraulic jumps or the occurrence of bores as a result of dam breaks.

The post data processing procedure is also relatively straight forward for this software package. Once the simulation is complete, a trim-file will be produced. This file contains

all the necessary hydrodynamic information required such as water level, depth averaged velocity etc. A compatible MATLAB GUI script named QUICKPLOT (that comes with the software package) allows the desired parameter to be extracted for further analysis.

Besides modelling hydrodynamic quantities, the Delft3D-FLOW model is also capable of modelling sediment transport and bed morphological changes. Li and Huang (2013), noted that in the Delft3D-FLOW model suspended sediment transport is calculated by solving a three-dimensional advection-diffusion equation and the bed load is calculated using Van Rijn's (1993) sediment transport formula. The bed morphology will be updated at each computational time-step based on each time step iteration of the sediment transport.

In our numerical simulations, the numerical grid size was 0.005 m and the time step was 0.006 seconds, which satisfies the Courant–Friedrichs–Lewy condition. For the solitary wave simulations, Riemann forcing was applied on the offshore boundary for generation of the solitary wave. For solitary waves running up a gentle slope, most wave energy is dissipated by wave breaking and bottom friction. However, when an onshore coastal cliff is present, strong reflection by the cliff can significantly increase the water level in front of the cliff, and the effects of energy dissipation due to wave breaking and bottom friction are minor. In our experiment, no significant wave breaking was observed, and thus no background horizontal viscosity was used in the simulations for turbulence generated by wave breaking. The Manning's roughness coefficient of $n = 0.009 \text{ m}^{-1/3} \cdot \text{s}$, selected for polyvinyl chloride plates (Neale and Price, 1964). For the dam break experiments, an initial condition file was set up specifying the initial water depth and initial water velocity at each cell. The implementation of the Riemann forcing boundary was not required for the dam break case. The release motion used for the dam break scenarios are instantaneous dam

break cases. Other parameters such as the grid size, time step and bottom roughness coefficient are identical to the solitary wave cases.

2.2.2 Sensitivity Tests

Sensitivity tests were used to determine the values of the parameters that should be used for the numerical simulations for the Delft-3D model. The main parameters for interest were that of the grid size and the roughness coefficient which will be important for onshore measurements. The benchmark case that we will use for this sensitivity test is the solitary wave test case with an onshore cliff of 79° .

Figure 2.6 shows the numerical domain setup for the Delft-3D simulation. The composite beach profile is made of three different slope profiles. The three angles used; θ_1 , θ_2 and θ_3 are, 1:0.68, 1:15 and 1:30 respectively. The onshore cliff has an angle of 79° and is situated 0.21m from the shoreline. The incident wave has a maximum amplitude of around 0.055m.

The grid size sensitivity tests were carried out by extracting the maximum surface elevation at four locations; s3, shoreline, s4 and at the toe of the cliff

Figure 2.7 shows the sensitivity grid test conducted for a test case using solitary waves. The height ratio term on the y-axis is defined as the ratio of the maximum surface elevation recorded in the experiments at the pre-determined cross shore locations against the numerical simulated results at the same location. If the ratio is unity, it means that the numerical results are able to reproduce the experimental results. From figure 2.7, it can be seen that the effect of grid sizes are only more apparent when the location is taken very close to the toe of the cliff. The other locations exhibit a constant height ratio value despite

changes in the grid size used. Based on this, the numerical simulations from here on will be using a fixed grid size of 5mm.

Fig 2.6) Numerical test case setup and domain

Fig 2.7) Sensitivity grid test for solitary wave test case

The other parameter that might also affect the numerical results is the bottom roughness. Different values in bottom roughness could lead to difference in the wave's surface elevation during the wave's onshore propagation. Neale and Price (1964) stated that for polyvinyl chloride plates the Manning's roughness coefficient used is $n = 0.009 \text{ m}^{-1/3} \cdot \text{s}$.

Figure 2.8 shows the sensitivity test conducted for the Manning's roughness coefficient using two different test cases. Both cases has the same domain settings as that of figure 2.6 but an additional case of $H_0 = 0.035\text{m}$, and onshore cliff angle of 14° was added. This particular case was chosen for testing because in contrast to the benchmark test case, it was at the other end of the spectrum, i.e., with a very gentles slope and the smallest incident wave height.

Similar to the case of the grid sensitivity test, the y-axis also defines the ratio of the maximum surface elevation measured experimentally against the numerically simulated results.

Figure 2.8 shows that the values of this height ratio remains relatively constant despite varying bottom roughness coefficients. This means that for the onshore locations, varying the roughness coefficient does not significantly vary the numerically simulated water depth results. Although it can also be seen that further onshore and closer to the cliff, the height ratio increases, this observation will be explained in more details in section 3.3.3. In summary, for the numerical simulations in this thesis, we will be using a standard grid size of 0.005m and a bottom roughness coefficient of $0.009\text{m}^{-1/3} \cdot \text{s}$.

Fig 2.8) Sensitivity roughness test for solitary wave test cases

2.2.3 Trial cases using Delft3D model

The numerical work conducted in this thesis focuses on two different wave forms; solitary waves and dam break waves. To show that the numerical model is able to provide a valid comparison with our experimental results, we begin by comparing published data against the numerical results obtained from our Delft-3D model.

Young et al (2010) conducted an experimental study on solitary wave impinging on a plane beach. They were interested in analyzing the hydrodynamics evolution of the solitary wave as well as the morpho-dynamic changes of the beach during the run-up and drawdown process. For our analysis, we only consider the hydrodynamic findings that they have made.

Their experiment was conducted in a wave basin of 48.8 m long $\times$ 26.5 m long and 2.1 m deep. Figure 2.9 shows the detailed experimental setup.

Fig 2.9) Young et. al's (2010) experimental setup. (figure obtained from Young et al (2010))

Figure 2.10 shows the numerical setup used for our numerical simulation. To prevent numerical dissipation of the incident wave, we moved our numerical wave boundary 10m shoreward thus using WG1 as the incident wave location. Because of this, the beach slope started at $x = 2\text{m}$. The incident wave height used in Young et al's (2010) experiment was 0.6m and we used this value for our numerical simulation. In total, Young et al's (2010) experiment had 12 wave gauges, but only the data of WG1 and WG8 were provided. Hence for the numerical simulation, only two artificial wave gauges were installed.

Figure 2.10) Numerical setup for Young et al (2010) experiment (drawing not to scale)

The total simulation time allocated for the simulation is 30s, the time step is 0.3s. The grid size in the cross-shore direction is 0.05m. The Manning's coefficient used is $0.009\text{m}^{-1/3}.\text{s}$ and the horizontal viscosity is set to $0.0005\text{m}^2/\text{s}$. The Flooding momentum advection scheme was chosen.

Figure 2.11) Comparison for Young et al (2010) experiment with the numerical results

Figure 2.11 shows the comparison of the surface elevation experimental results obtained at WG1 and WG8 against the numerical simulated data. From WG1 it can be seen that the numerical simulated incident wave height magnitude is about the same as the result obtained

from Young et al. (2010). However, the reflected wave that occurs around 21s was not apparent in the numerical results. Only a slight hump that represents the reflected wave from the plane beach was visible at around 23s. WG8 shows the wave time series at a shallower depth; the numerical wave profile shows that the surface elevation of the experimental data matches well with the numerical results. However, the undulating tails which the experimental data exhibits could not be adequately captured by the numerical model. This simple solitary wave height comparison of the experimental data from Young et al. (2010) with the numerical results produced by the Delft-3D model shows that in general the non-linear shallow water equation governed models are capable to reproduce the experimental results. This comparison validates the usage of the Delft-3D model for the numerical works on solitary waves in this thesis.

Besides solitary waves, the other wave form that is discussed in this thesis is the dam break waves. Unlike solitary waves, dam break waves are often due to large and abrupt changes in the water depth in the cross-shore direction. This sudden change in water depth could lead to turbulence when the extra body of water mass comes crashing down. Hence, the additional effect of turbulence may not be neglected in dam break cases. To justify the usage of the Delft-3D model for the reproduction of dam break cases, the simulated results are compared with the experimental results of Stelling and Duinmeijer (2003) to the results produced by the Delft-3D model.

Figure 2.12) Numerical setup for Stelling and Duinmeijer (2003) experiment

Stelling and Duinmeijer (2003) conducted experiments to determine the propagation of a dam break flow after its release. Figure 2.12 is an exact replicate of the setup used by them. The experimental setup consists of two portions. On one end is a reservoir filled with water up to 0.6m and the other end is a long basin which is filled to 0.05m depth. In the experiments, the gate moved at a speed of 16m/s. However, this movable partial boundary could not be implemented into the numerical model. Thus, for the numerical simulations, an instantaneous dam break scenario was used. The locations where the wave gauges were installed during the experiments are at; 1m, 6m, 9m, 13m, 17m, 21m and 23m from the gate opening. However, for the numerical simulations we only selected the following cross-shore locations; 1, 6 and 21m for comparison.

An analytical approach can be used to discuss and evaluate the results of Stelling and Duimeijer (2003) experiments. The Stoker's formula is a modified version wet bed of Ritter's dry bed solution to the dam break problem. Stoker's formula for flow depth are given as,

$$h(t, x) = \frac{4}{9g} \left(\sqrt{gh_l} - \frac{x-x_o}{2t} \right)^2 \quad \text{if } x_A(t) \leq x \leq x_B(t) \quad \text{and} \quad (2.11)$$

$$h(t, x) = h_m \quad \text{if } x_B(t) \leq x \leq x_C(t) \quad (2.12)$$

h_m is an surface elevation caused by the release of the dam break. This value is a constant and can usually be found out through experiment tests. For simplicity, we refer back to Stelling and Duinmeijer (2003) experiment and made use of the parameters to obtain the final flow depth at rest. This value is 0.094m. x_A , x_B and x_C are time dependent variable locations of the wave front right-wards and left-wards of the flume. x_o is the position of the dam which is at 2.5 m. x is then the position of s1, which is at 3.5 m. The time dependent equations for x_A to x_C are,

$$x_A(t) = x_o - t\sqrt{gh_l} \quad (2.13)$$

$$x_B(t) = x_o + t(2\sqrt{gh_l} - 3c_m) \quad (2.14)$$

$$x_C(t) = x_o + t \frac{2c_m^2(\sqrt{gh_l} - c_m)}{c_m^2 - gh_r} \quad (2.15)$$

h_l is the reservoir depth which is 0.6 m, h_r is the wet bed initial flow depth at 0.05 m and c_m is defined as $(gh_m)^{1/2}$.

For the location of s1 at $x = 3.5$ m, the reference locations of particular importance are x_B and x_C . The first step in determining the time series plot of s1 would then be to check if s1

lies within the range of x_B and x_C transiently. After finding that out, the suitable equation should then be used to determine the flow depth at each time interval.

Fig 2.13) Analytical solution using Stoker's equation at s1 for Stelling and Duinmeijer experiment

Figure 2.13 shows the analytical solution for the dam break case at s1. As can be seen, the solution shows a very steep increase in flow depth followed by a quick drop. This tooth edged shape of the incident profile of the dam break wave can be attributed to the Stoker's formula. Looking back at equation 1, the only variable is time. This means that the flow depth is proportional to $(1/t)$. When the value of t is small, $h(t)$ will be large and if t gets larger, $h(t)$ will start decreasing. This explains why the curve slopes downward from about 1.5s to 2.5s.

For the numerical experiments, the total simulation time allocated for the simulation is 20s and the time step is 0.006s. The grid size in the cross-shore and long-shore direction is 0.1m. The Manning's roughness coefficient used is $0.009\text{m}^{-1/3}\cdot\text{s}$ and the horizontal viscosity is set to $0.0005\text{m}^2/\text{s}$. The flooding momentum advection was chosen.

Figure 2.14) Comparison for Stelling and Duinmeijer (2003) wet bed with the numerical results

Figure 2.14 shows the wave time series comparison for the three mentioned locations. Here, s1 refers to the location 1m downstream of the gate, s2 is 6m downstream from the gate and s3 is 21m downstream from the gate. Location s1 shows the incident wave profile of the dam break wave. The maximum wave amplitude recorded from the numerical simulation was around 5cm lower than that measured in the experiments. Moreover, the experiment results do depict a slight trough in the tail of the incident wave which is represented by the slight drop in the water level. The water depth from locations s2 and s3, do show that the magnitude and profiles of the waves measured at these locations from Delft-3D do match quite well with the experimental data. Reflections from the boundaries were also captured as shown from the second peaks (around 9s in s2 and around 16s in s3). These comparisons show that the Delft-3D numerical model is capable of reproducing dam break waves.

The above comparison was made with the release of a dam break wave into a partially filled basin of water. However, our main study area of interest is in fact the onshore region. Hence,

it would be more appropriate if further validation of the numerical model is made on dam break waves propagation into a dry basin.

Figure 2.15 shows the comparison of the numerical results computed using Delft-3D against the data from Stelling and Duimeijer (2003) for a dry bed computation. The incident wave profile s1 for both the numerical model and experimental model show a sharp increase in the wave height. This particular profile is rather characteristic of dam break waves when the wave fronts passes a particular location. This sharp increase is followed by an essentially constant water depth as shown from around 1s onwards in s1. The water depth at s2 also show similar profiles for the numerical results and experimental data. However, the water depth at s3 illustrates a visible difference for both methods. The experimental data shows a sharp increase in the water depth which is not present in the numerical data. Rather, the numerical model reveals a much smoother water depth increase. It may be surmised that the Delft-3D model is unable to accurately capture such surface elevation spikes. However, the overall trends of surface elevation variations obtained for the numerical model data compares well with the experimental results.

This sub-section shows the comparison of the numerically simulated data using Delft-3D against published experimental test results using both solitary waves and dam break waves. In general, the numerical results simulated with a solitary wave compares well with the experimental results of Young et al (2010). For the dam break cases, the wet bed experimental data do match better to the numerical results compared to the dry bed scenarios. But on the whole, the comparison between the dam break experimental data and numerical results is quite good.

The comparison conducted in this section justifies the usage of the Delft-3D model for the test cases we will have for comparison with our experimental data.

Figure 2.15) Comparison for Stelling and Duinmeijer (2003) dry bed with the numerical results
from Delft-3D

Chapter 3

Solitary Waves

- *Physical and Numerical
Experiments*

3.1 Introduction

As discussed in section 1.1, solitary wave is a good benchmark representation of the leading wave front of a tsunami wave. Moreover, solitary waves are also rather easy to be reproduced in a laboratory environment. Besides, tests conducted using such waves can be easily repeated under a closed environment given that they are simply generated by using a wave paddle system. Hence, in this chapter, the onshore flow of varying solitary waves under different onshore topographical settings with a focus on the run-up is discussed. We will first look into the experimental and numerical discrepancies for solitary waves run-up. Next, we will be looking into understanding the need for a tsunami amplification factor in numerical simulations. We will be using the Delft-3D FLOW model to conduct the numerical simulations and compare it with results of physical experiments conducted in the hydraulic laboratory. Because of the presence of the coastal cliff in this study, the topography we will be using differs much from other studies that investigate flow patterns on a plane beach especially on the onshore region and the run-up. Hence we will first discuss the run-up comparison between the physical and numerical obtained results. The next topic that will be touched upon is how the tsunami amplification factor is defined and the importance of selecting a reference location to be used when calculating the tsunami amplification factor. After which we will then discuss the parameters that will most likely affect the amplification factor. This includes the cliff slope angle and the onshore distance from the shoreline to the toe of the cliff.

Potentially, it would be ideal to develop a suitable empirical formula that can be used seamlessly with NLSWEs based models. However, after doing this comparison, it seems that this task is not as straightforward. The main reason is that the onshore flow patterns are

highly influenced by bathymetry and topographical changes. This set of physical and numerical experiments only considered one benchmark bathymetry albeit with a varying cliff angle. More data would be required for comparison if any empirical formula can be developed. To do so, more experiments have to be conducted. Hence, in this chapter, the key message to be delivered is the requirement of an amplification factor and how it could be potentially affected by other parameters.

3.2.1 Experimental setup for solitary wave run-up

Figure 3.1 shows the test case for a solitary wave run-up on a onshore coastal cliff. For this set of experiments, we covered the domain of interest with a dark cloth and illuminated the domain of interest with a spotlight. This was done so that the video recordings captured will not turn out to be that blurry. The onshore cliff had an angle of 79° . There was a cave with a frontal dimension of $0.05 \times 0.05\text{m}$ with a length of 0.3m tunneling through the cliff. The purpose of this cliff was to replicate the scenario of the cave discovered recently in Banda Aceh, Indonesia (see section 1.2). For our experiments, we carried out tests when the cave ceiling has a hole and when it is sealed. Three sensors (s1-3) were used to measure the real-time surface elevation changes. The three sensor locations were chosen because of the dam break wave's setup in chapter 4. The time-series results obtained here will be used to compare with those obtained in chapter 4 for the dam break waves. The onshore surface profiles; including the run-up on the cliff and flow inside the cave were recorded using web-cameras. Two cameras were used to capture the run-up trim line and onshore flow patterns (one from the side view another from the plan), the third camera was used to record the flow patterns in the cave.

The main parameter of interest for this study is the run-up. To do that we have one web camera mounted from the top to capture the wave time-series of the flow on the cliff. For the rest of this thesis, this variable will be termed as the trim-line. Figure 3.2 illustrates how the trim-line is defined. Assuming a plane beach case where the flow depth is at h_0 at time t_0 ; there are certain offshore forcing mechanisms that causes the water line to rush up; this is represented by h_1 and h_2 at t_1 and t_2 respectively. By knowing the values of h_0 to h_n for each time step, we can then capture the maximum flow depth over the entire domain for each time step. This recorded time-series is the trim-line.

Fig 3.1) Experimental model for solitary wave run-up

Fig 3.2) Trim-line definition

3.2.2 Solitary waves run-up

The maximum run-up is the easiest found visual indicator during the aftermath of a tsunami. Here, the maximum run-up for solitary waves were successfully recorded and compared with the numerically simulated results. The experimental and numerical setup used will be identical to that of figure 3.1 for the study of solitary wave run-up on the steep onshore cliff. The incident solitary wave heights used is tabulated in Table 3.1.

Table 3.1) Solitary wave run-up test cases incident wave heights

Solitary Wave Cases	
Test Case	Incident Wave Height (cm)
SW1	5.0
SW2	5.5
SW3	6.1
SW4	6.5
SW5	7.2

It should be noted that the presence of the cave on the face of the cliff could affect the run-up value. However, constructing the cave that tunnels through the cliff is not plausible in the numerical setup. Thus the numerical topography does not include the cave. Because of this, an assumption made is that the cave does not really affect the run-up value in the experiments. As the wave approaches the cave, the wave front will rush up the cliff and reach the highest possible point. At the same time, part of the water mass that reaches the cliff face will rush into the cave. The main reason why most of the water will rush up is because the cross-sectional area of the flow is much greater than the cross-section opening area of the cave.

Figure 3.3 shows the time series comparison of the experimental data and numerical results for the test case SW2. From figure 3.3, the time series of s1 to s3 show that the numerically

results match rather well with the experimental results. Other than the absence of the undulating tails for the three sensor locations, the only other visible difference is the time at which the peak wave occurs in s2 and s3. This small time difference could be because wave dispersion is neglected in the numerical model. Locations s1 and s2 clearly illustrate the effect of dispersion which is present in the experimental results but not considered in the numerical results. Location s2 clearly shows that the wave profile still maintains its steep wave front for the numerical case but for the experimental case, the wave shape has already started to elongate. By definition, dispersion is a wave phenomenon that states that waves of different wave lengths travel at different phase speeds. In terms of energy conversation, it means that the wave energy is spread over the wave envelop due to the different wave celerity of waves with different wavelengths. Because of this, the most visible observation of dispersion is then a change in the shape of the wave as it propagates towards shore. Non-dispersive waves are idealized and does not occur in reality. However for most numerical models, dispersive codes are not included due to the additional computational time requirements.

Figure 3.4 shows an illustration of a solitary wave propagation. Due to dispersion, after travelling a distance L , the wavelength increases and the surface profile starts to elongate.

According to Glimsdal et al. (2013), dispersion

could influence the incident wave front most during the beginning of the wave propagation, but as the wave continues to travel, dispersion becomes more significant in the undulating tails of the incident waves. This also means that dispersion is more dominant to the wave crest in the initial period of wave propagation but its effect starts to diminish as the wave starts to propagate shoreward.

Fig 3.3) Time series comparison of experimental and numerical results using test case SW2

Fig 3.4) Illustration of wave surface profile evolution

The trim line of the experimental and numerical results show that the run-up of solitary waves can be adequately captured and represented using the numerical model. The difference between the measured run-up and numerically simulated run-up is generally minimal. Figures ii to iv in the appendix show the other comparisons for different incident wave heights.

Figure 3.5 shows the trim-line comparison for three different incoming solitary waves. From the figure, three distinct observations can be made. 1) the experimental maximum recorded run-up is clearly much greater than the numerical results, 2) the numerical results all depict similar trim-line profile despite having different incoming wave heights and 3) the experimental run-up increases with increasing incident wave height.

The first observation is a rather known fact as sudden spike in water levels and nearshore turbulence cannot be reproduced in the numerical model. The second observations seems to hint that there is a certain cut-off level at which the numerical model can capture any water level changes for this particular topography. It seems that beyond this elevation level, the numerical model cannot record any water level changes. From our results, this elevation level is around 16cm.

The Ursell number (U_r) (Ursell, 1953) provides an indicator on the importance of non-linearity and dispersion when it comes to long waves. Ursell's number (U_r) is expressed as,

$$U_r = \frac{H}{h} \left(\frac{L}{h} \right)^2 \quad (3.1)$$

where the term H/h is widely regarded as the non-linear factor. The importance of non-linearity comes into play when U_r is large. From the equation above, for U_r to be large, the flow depth h has to be much smaller compared to the surface elevation.

Liu et al. (2009) compared the difference between linear shallow water equation models and non-linear ones. Figure 5 from his paper clearly shows the discrepancy of the linear model at shallow water depths. The wave height measured at a depth of 60 m was about five times less in the linear model compared to the non-linear model. This would mean that

if the tsunami run-up is the parameter intended to be studied, a non-linear numerical model would be more appropriate.

However, that is not often the case for run-up and even more so for the case study in this thesis. Onshore wave propagation is much more complex due to other important wave processes such as diffraction, refraction, shoaling and harmonic interaction. In this thesis, due to the onshore topography, even wave reflection has to be accounted. For example, Glimsdal et al. (2013) mentioned that at the nearshore and onshore region, the incoming wave might separate into undular bores which are can be aptly described as a train of solitary waves that could have wave heights more than twice the incoming wave. Currently, these undular bores could not be simulated in non-linear shallow water equation models as the interaction between dispersion and non-linearity have to be considered to do so.

Figure 3.5 then shows that even when the non-linear shallow water model is used, the discrepancy between the results computed and that from the experiments are still fairly large. Although it can be argued that the non-linear shallow water model would produce a much better result than a linear model, it still does not cover the fact that for the case in this thesis even considering non-linearity is insufficient. To surmise, the effect of non-linearity on run-up is important but for this case, it might not be of utmost importance.

Fig 3.5) Trim line comparison for solitary waves using experimental and numerical results

Figure 3.6 shows the variation of the solitary wave run up with the incident wave height. The circles represents the experimental data and the squares the numerical data. It can be observed that as the incident wave heights increase, the recorded experimental run-up also increases. However, the numerical simulated run-up does not increase as quickly. Another observation made is that as the incident wave height increases, the difference between the run-up measured by both methods also increases. For SW1, the experimental result is around 15% greater than the numerical result. For SW5, this difference is around 60%. With increasing incident wave height, more splashing of the wave front as it hits against the cliff will occur. These water splashes that hit the face of the cliff would most likely not be represented in the numerical simulation, thus the difference. Figure 3.7 shows the side view images captured for the respective maximum run-up. Figure 3.7b shows the water splashing for the case of SW5, but this is absent in SW1.

Fig 3.6) Trim line comparison for solitary waves using experimental and numerical results

Besides the incident wave height which could affect the maximum run-up value, another parameter that can also influence the maximum run-up is the onshore wave celerity. The analytical solution for the run-up of a single crest solitary wave provided by Madsen and Schäffer (2010) is,

$$R = 3.043H\sqrt{kct_0} \quad (3.2)$$

in which R is the run-up, H is the wave height, k is the wave number of the solitary wave which can be expressed as,

$$k = \sqrt{\frac{3H}{4h^3}} \quad (3.3)$$

where h is the water depth. c is the solitary wave celerity which can be expressed as,

$$c = \sqrt{g(H + h)} \quad (3.4)$$

and g is the gravitational acceleration, t_0 can be expressed as,

$$t_0 = \frac{h \cot \theta}{\sqrt{gh}} \quad (3.5)$$

where θ is the slope of a plane beach.

Equation 3.2 shows that the run-up is affected by the solitary wave celerity. A faster wave will lead to a greater run-up.

Fig 3.7a) side view maximum run-up for SW1, b) side view maximum run-up for SW5

Fig 3.8) Onshore horizontal displacement for test cases SW1-SW5

Figure 3.8 shows the onshore wave front displacement for the five solitary waves tested. The onshore horizontal displacement is measured using the frames captured from the side view camera. Each marker on the figure represents the location of the wave front at each frame. The time step is derived based on the frame rate of the web camera used. By making linear plots of these two sets of data and then retrieving the gradient, the onshore flow velocity can be deduced.

Using this approach as a means for approximating the onshore flow celerity; for SW1, the onshore flow celerity is 1.67m/s while the flow celerity for SW5 is 2.05m/s. The respective r-squared values for the linearization of the data from both sets of points are 0.95 and 0.98.

The faster onshore speed for SW5 would mean that more energy would be available for the water to climb to a higher elevation and thus record a greater run-up value compared to the test case for SW1. This supports our observations for the run-up as shown in figure 3.5. This finding then shows that the onshore speed of the incoming wave could be a contributing factor that led to the increase in the run-up value.

3.3.1 Experimental setup for tsunami amplification factor test

Figure 3.9 shows the experimental setup for the solitary onshore flow depth amplification factor study. The cross-shore profile of the experiment was modeled using a composite slope, made of three sections with three different slope profiles (from upstream: 1:0.68, 1:15 and 1:30), with the gentlest onshore to replicate the beach. A cliff model, with adjustable steepness/slope, was mounted onto the beach section to reproduce various topography scenarios. In the experiment, the water depth used was fixed at 0.35m in the wave flume, and the foot of the cliff was 0.21m away from the shoreline, with $x = 0.00\text{m}$ as the shoreline and $x = 0.21\text{m}$, as the foot of the onshore cliff. The distance from the wave generator to the toe of the model was 17.34m. The main differences in this setup compared to figure 3.1 are 1) the location of the sensors, 2) the location of the composite slope from the wave generator and 3) the position of the cliff from the shoreline. The sensors s1 and s2 were placed at their locations to record the incident wave height of the solitary waves. Sensor s3 was placed at the nearshore region as it is the location for the reference wave

height. Sensor s4 was placed at an onshore location to capture the onshore flow which was then used to compare with the results obtained from the video recordings (see section 2.1.3). Table 3.2 shows the four incident solitary wave height used. Five onshore cliff slopes (angles) were tested: $\theta = 14, 21.67, 39.33, 49$ and 79° . Six onshore locations were designated to measure the onshore flow depth and the flow depths at the following six locations were extracted from the video recordings: $x = 0.00, 0.06, 0.11, 0.13, 0.16$ and 0.21 m. In addition the run-up trim line was also captured.

Fig 3.9) Experimental setup for onshore flow amplification factor

Table 3.2) Incident solitary wave characteristics used for the amplification factor study

H_0 [m]	0.025	0.035	0.045	0.055
L_0 [m]	9.50	8.03	7.08	6.41
L_0/L_B	45.24	38.24	33.73	30.30
L_B/L_0	0.022	0.026	0.030	0.033

The relative distance of the cliff location can be defined as L_B/L_0 , where L_B is the distance from the shoreline to the toe of the cliff and L_0 is the solitary wave wavelength ($L_B = 0.21$ m in this study).

Scaling is always an issue for any experimental study of tsunami problems due to the extremely long period and length of tsunami waves (see Chen et al., 2013). Even though it is possible to generate a wave whose wave front is qualitatively similar to that of typical tsunami waves, both the length and time scales of actual tsunami waves cannot be downscaled in wave flume experiments geometrically according to Froude's similarity law. Distorted models have to be used when studying tsunamis in wave flumes.

As suggested by Satake (1994), an offshore location, called reference location, needs to be selected to define the tsunami amplification factor. The following order-of-magnitude estimates are intended to show that it is unrealistic to work on a geometrically undistorted model when experimentally studying tsunamis in wave flumes. If we assume that our reference location (at still water depth of 0.058 m) corresponds to a tidal gauge installed nearshore in a water depth of 10 m, which is typical for most tidal gauges, then we have a geometric scale factor of about 172 for this experiment, which means that in field conditions the distance between the shoreline and the toe of the cliff is about 36 m and the incident wave height H_0 is specified at a water depth of about 60 m. Corresponding to the test conditions mentioned in table 3.2, the tsunami wave length in water of 60 m varies from 1092 m to 1629 m, which is almost 10-20 times shorter than typical tsunami wave lengths in this water depth for most of tsunamis. If we want to get the tsunami length scale correct, the wave height we use in the experiment must be less than 0.2 mm, which is too small to make any meaningful observation, and the wave flume that can be used for such length of

solitary waves must be at least 300 m long, which is not available in most hydraulics and coastal laboratories.

Fortunately, the relative distance between the shoreline and the toe of the cliff, L_B/L_0 , is much less than 1 in our experiments, which is consistent with that found in field conditions. The amplification of the water level in front of a cliff is due mainly to the reflection of the incoming wave by the cliff. Similar to standing-wave patterns due to long water waves reflected by a vertical wall, as long as the distance between the cliff and the location for water-level measurement is much smaller than the typical tsunami wave length, the amplification factor obtained from the geometrically distorted models is expected to be a good approximation to reality. Therefore, it is reasonable to expect that the amplification factors obtained from our hydraulic tests are not significantly affected by the distortion in tsunami wave length.

3.3.2 Definition of tsunami amplification factor

The tsunami amplification factor is defined in terms of tsunami heights (i.e. the maximum water-level rise measured from mean sea level; see Hill et al., 2012) by

$$Tsunami\ Amplification\ Factor = \frac{H(x)}{H_r} \quad (3.6)$$

where H_r is the tsunami height at an offshore location (reference location) x_r and $H(x)$ is the onshore flow depth at an onshore location x . Both H_r and $H(x)$ are measured from the mean sea level (MSL). The selection of the reference location x_r takes into consideration of the following factors: (1) typical locations of tidal gauges and (2) the shoreward boundary beyond which shallow water equations produce significant errors.

According to the definition, the value of the tsunami amplification factor is affected by the selection of the reference location. Piatanesi (1996) assumed that the reference point was at a water depth of 10 m, close to the coastal boundary. Imamura (1993) included vertical impermeable walls along the coastline in his numerical computation and used the computed wave heights there as the reference for his amplification factor. Satake (1994) also used the computed wave height along the coast as the reference location to determine his amplification factor. The two main reasons for the selection of reference location close to the coastline are: (1) the limitations of existing numerical computation capacities for onshore tsunami flows and (2) the availability of tidal gauges in shallow water. The real-time measured information obtained from these tidal gauges can be used to check the computed measurements at these locations.

In this study, we selected our reference location at a distance of 0.87 m from the experimental shoreline (i.e., $x_r = -0.87$ m) and the water depth at the location was 0.058 m. The reference location s_3 acts as a separation point between two domains; to the left (seawards), hydrodynamic parameters can be reproduced to decent accuracy using existing numerical models as mentioned in Satake (1994), to the right (shoreward), the non-linear shallow water numerical model often under-predicts the onshore flow depth and run-up. This then means that the validity of the non-linear shallow water model is better offshore. Under real time tsunami warning periods, researchers would then be able to rely on their numerical models to at least obtain surface elevation data nearshore. After which, more sophisticated models that require longer computational time could be implemented to determine the onshore flow depth or run-up. Another alternative to determine the onshore flow depth is to use the data gathered from that nearshore location and input it into an

empirical formula for the given topography so as derive the onshore parameters. One of the methods that could be used is by finding out the amplification factor. Hence, it would defeat the purpose of proposing an amplification factor if the reference location is too far offshore (for example, near the source). Besides, tidal gauges are less often found too far offshore thus in real time, the most reliable data are often conveyed through these devices which are located closer to the shore.

In the appendix of this thesis, figure ii, iii and iv show the time-series surface elevation for the three cases SW3, SW4 and SW5. The time series of the surface elevation at s3 for these three figures show two distinct peaks. The first is around 10s and the second at around 12.5s. The first peak can easily be attributed to the incident wave. The second peak on the hand is attributed to the reflected wave from the cliff superimposing on the still incoming incident wave. Due to the presence of two distinct peaks at location s3, with the earlier one due to the incoming wave, it is safe to conclude that at this location the first peak surface elevation is not affected by reflection and thus is suitable to be used as the reference location for this thesis.

Wave heights at this location were measured experimentally and computed numerically using the nonlinear shallow water equation solver provided by Delft3D-FLOW. We have computed the ratios of the computed wave heights to the measured wave heights at various locations, and found that (1) there was a significant difference between the computed and measured wave forms and heights at the onshore location s4, and (2) the agreement between the measures and computed wave heights at locations s1-s3 were very good. In particular, the ratio between the measured and numerical results and standard deviation for the reference location s3 are 1.039 and 0.0713, respectively. This means the values of the

computed and measured wave heights at the reference location are almost identical with a rather small discrepancy, confirming the validity of nonlinear shallow water equations at this location. It must be stated that we have used the wave heights obtained at the location s3 ($x_r = -0.87$ m) to compute the measured tsunami amplification factors. If a different reference location is used in the calculation of amplification factor, a new amplification factor can be calculated by correcting the reference location using the following shoaling relationship based on linear long waves (Green's Law),

$$\frac{H_1}{H_2} = \left(\frac{h_1}{h_2} \right)^{-1/4} \quad (3.7)$$

where H_1 and H_2 are the wave heights in water depths h_1 and h_2 , respectively. It is pertinent to stress that Green's law is not valid in the onshore location where the land is initially dry. Equation 3.7 implies that a small change in the reference location will not significantly affect the amplification factor, i.e., the amplification factor is not very sensitive to small changes in the reference location. To show this, we used the test case of $H = 5.5$ cm, and $\theta = 79^\circ$, to carry a simple sensitivity test. We first derived the amplification factor at the six aforementioned onshore locations with the reference location being at 0.87 m from the shoreline. We then shifted the reference location to 0.57 m from the shoreline and 1.17 m from the shoreline. Based on a scale factor of 172, this means that the distance between the reference location and the shoreline in field condition varies from 98 m to 201 m (or the water depth in field condition varies from 6.5 m to 13.4 m). Using the linear shoaling relationship to estimate the wave height at these two new locations, we found that the amplification factor did not vary too much for small changes in the reference location. Hence, our choice of reference location at 0.87 m from the shoreline is appropriate to be

used in further derivation of the amplification factor for the other test cases. This could prove to be helpful if near-shore flow records are absent; it can then be predicted by the shoaling model and further used to determine the amplification factor.

3.3.3 The effect of cliff slopes on tsunami amplification factor

The presence of an onshore cliff not only impedes the inundation of the tsunami onshore but also changes the dynamics of the run-up and drawdown processes. Contrary to plane beaches where the wave runs up the beach to reach its run-up limit before flowing back to the sea as a sheet flow, the presence of the cliff alters the onshore flow patterns. As the water reaches the shoreline, it moves towards the toe of the onshore cliff as a moving bore. Upon impact with the cliff, the water mass builds up in front of the cliff, amplifying the flow depth in front of the cliff by several times compared to plane beaches.

Figure 3.10 shows the effect of different cliff slopes on the amplification factor defined by equation 3.6 at different onshore locations. The wave height at the reference location used the result obtained from the Delft3D-FLOW simulation. Figure (i) in the appendix shows a similar result when the physical experiment obtained wave height was used instead. The following conclusions can be drawn:

- The presence of a coastal cliff can greatly increase the water depth in front of the cliff.
- At each onshore location, the amplification factor generally increases with increasing cliff slope for a given incoming wave height.
- At each onshore location, the amplification factor is affected by the incoming wave

height for a given cliff slope. Smaller incoming waves in general tend to have larger amplification factor.

- The amplification factor generally increases towards the cliff for a given incoming wave height and a cliff slope.
- At the toe of the cliff and for the maximum runup, the amplification factor corresponding to the largest wave ($H_0=5.5$ cm) is smaller than others, possibly due to the energy loss due to wave breaking.

When the cliff slope is greater than some critical value, the effect of the cliff slope on the amplification factor becomes insignificant, and the effects of incident wave amplitude dominates. In our experiment, this critical cliff slope is found at about $\tan \theta \approx 1$, i.e. $\theta \approx 45^\circ$. Before this critical slope, the amplification factor increases as the cliff slope increases, but beyond this critical slope, the amplification factor somehow remains constant with further increasing cliff slope.

3.3.4 The effect of onshore distance on tsunami amplification factor

Another factor that could potentially affect the tsunami amplification factor is the onshore flow distance. Experiments conducted using the setup in figure 3.9 has an onshore distance of 0.21m compared to 0.3m using the setup in figure 3.1. The run-up amplification factor is determined using the water level obtained from a reference location at 0.87m from the shoreline.

Fig 3.10) Variation of numerical amplification factors with cliff slope and incident wave height at different onshore locations. (a) $x = 0.0$ cm; (b) $x = 6.0$ cm; (c) $x = 11.0$ cm; (d) $x = 13.0$ cm; (e) $x = 16.0$ cm; (f) $x = 21.0$ cm (g) run-up. The four wave heights are 2.5 cm (circles), 3.5 cm (crosses), 4.5 cm (asterisks), and 5.5 cm (squares).

Fig 3.11) Run-up amplification factor for test cases SW1-SW5

Figure 3.11 shows the run-up amplification factor for the test cases of SW1 to SW5. On average, the run-up amplification factor for these five cases is around 2.3 with a standard deviation of 0.16. These values indicate that the run-up amplification factor for each case falls within a smaller range and thus it also means that the incident solitary wave height has little effect on the value of the amplification factor.

Wave breaking is a very interesting phenomenon that is very important to onshore flow and its dynamics. Despite its importance, there has not been a standard guideline to define the wave breaking criteria. The most commonly accepted criteria for wave breaking at constant water depth is the McCowan's (1894) criterion which is,

$$\frac{H}{h} = H^* > 0.78 \quad (3.8)$$

However, as a wave shoals when it reaches shallow water due to bathymetry changes, the shoaling criteria is different. Camfield and Street (1969) concluded based on experiments that there were no observations on wave breaking if the slope angle is larger than 12° , Synolakis (1987) evaluated that all solitary waves would eventually break during run-up if the following was met,

$$H^* > 0.818s^{10/9} \quad (3.9)$$

where s is the gradient of the slope that should be expressed as the ratio of the vertical elevation to the horizontal length. Based on their studies, Grilli et al. (1997) determined that the most important parameter that influences wave breaking is the slope angle followed by the initial wave height. They also reaffirmed the findings by Camfield and Street (1969) where stable waves do not break if $s > 1:4.7$ (which is 12°).

Hence, we will define wave breaking based on two criteria; 1) H^* has to be greater than 0.78 at constant water depth for wave breaking to occur and 2) the slope (s) has to be less

than 1:4.7. In this thesis, the incident wave heights ranged from about 0.05 m to 0.07 m and the still water depth is fixed at 0.35 m. Based on this input, the maximum value of H^* is 0.14 which is much lesser than 0.78. The first portion of the beach model has a gradient of $s = 1:0.68$ which is greater than the threshold verified by Grilli et al. (1997). Thus, in the cases in this thesis belong to solitary waves that do not break.

We can come to a few preliminary findings on the run-up amplification factor based on an incident solitary wave; 1) the effect of incident solitary wave has negligible effect on the run-up amplification factor, 2) based on the results obtained from figure 3.10, for the case of the onshore cliff angle of 79° , the run-up amplification factor for an incident wave of 0.055m is 2.3. This is similar to the value we obtained for SW2 under a similar incident wave height and onshore cliff angle except that the onshore distance is 0.3m. This would most probably mean that the distance from the shoreline to the cliff angle does not really affect the run-up amplification factor. Even though the presence of an additional travel distance (by around 1.4 times) might serve to retard the onshore flow velocity, it seems that the added friction due to contact with the bottom surface does not influence the maximum run-up value by much. This would then mean that adjusting the onshore distance by these amounts does not really affect the tsunami amplification factor. However, if the onshore distance would be much longer; in the order of a few offshore wavelengths, the run-up on the cliff would most probably be reduced significantly. But such a real-life scenario would be unlikely as tsunami offshore wavelength are usually in the order of a few hundred kilometers. Under this condition, any onshore cliffs situated hundreds of kilometers would have lost its significance as the incoming wave would have already inundated most of the onshore area.

3.3.5 Discussion on the amplification factor

Based on the study of the 1992 Nicaragua earthquake and tsunami event, Satake (1994) obtained an average approximate amplification factor of about 3 for Corinto and Puerto Sandino, Piatanesi et al. (1996) used an average representative factor of 3.45 for the whole Nicaraguan coast, and Imamura et al. (1993) found that the average amplification factor to be in the range from 5.6 to 10 for the following six locations; Corinto, Puerto Sandino, El Transito, Masachapa, Casares and San Juan der Sur. Except the one used in Imamura et al. (1993), the other two amplification factors are close to what we found in this study. Very different amplification factors used in these studies can be attributed to several reasons to be discussed below.

From our results shown in figure 3.10, we can see that the onshore flow depths at several distances from the shoreline vary with the distance from the cliff, and the amplification factor for the maximum run-up on the cliff should be larger than the amplification factor for the flow depth in front of the cliff. The survey locations used in the aforementioned studies to define their amplification factors may have contributed to the different amplification factors used in these studies. Real-time data on tsunami flow depth onshore are rare and the limited data available in the literature were mainly derived from survivor videos (Fritz, et al., 2012). Majority of the data about tsunami flow depth and runup onshore are collected from post-tsunami surveys. According to the post-tsunami survey field guide published by UNESCO (2014), tsunami traces can include lines of debris, denuded hillsides, debris left on stairs, or broken branches. However, no detailed guideline was stated to distinguish the tsunami traces due to real flow depth from the traces caused by sources such as splashing water.

The post tsunami survey conducted by Satake (1993) yielded the data based on the following measurement procedures: 1) eyewitness accounts, 2) water-marks on walls, 3) debris caught on trees, and 4) amount of barks stripped from trees. The most debatable measurement process would be the last one, which in reality could be related to the actual tsunami flow depth at the location or due to splashing water generated by impact on trees. Numerical simulations that are based on linear or non-linear shallow water equations are unable to replicate the phenomenon of the splashing water. Hence to provide a better match to the observation data, most of such results have to be multiplied by a much larger amplification factor. If splashing water was taken into consideration in our experiment, the largest amplification factor measured for our study would be around 4.7 based on our video recordings, which is now closer to the one used by Imamura et al. (1993). However, this is in fact a large deviation from the onshore flow depth which is the parameter we are interested in and calculated by using the nonlinear shallow water equations. These examples emphasize the importance of conscientiously detailing the entire measurement process of a post tsunami survey. Moreover, by doing so, it would better help us distinguish whether the recorded measurement is in fact the actual tsunami flow depth or marks left by the tsunami splashing water.

One example related to splashing water would be the 1946 Aleutian tsunami event. The tsunami wave that hit the coast reached a wave height of 42 m and destroyed the Scotch Cap lighthouse killing everyone present at that moment (Lopez and Okal, 2006). Okal et al. (2003) theorized that some other mechanisms had to be in play that led to such a run-up value. Although they proposed that it could be due to a slope failure, which essentially means a tsunami superimposed by earthquake and submarine landslide, the recorded

measurement might also be due to splashing water as the incoming wave splashes up the lighthouse.

When using post-tsunami survey data, the importance of onshore topography cannot be neglected. At the three sites where post-tsunami surveys were conducted after the 1992 Nicaragua earthquake and tsunami event, and studied by Satake (1994), Piatanesi et al. (1996), and Imamura et al. (1993), the topography can be constructed using Shuttle Radar Tomography Mission (SRTM) data with an onshore resolution up to 90m (USGS). The topography at El Transito has a much hillier landscape than at the other two sites: the maximum run-ups at El Transito was 10 m, but it was 4 m at the other two sites. This inland feature at El Transito most likely have stopped the incoming wave from inundating further inland causing a wall of water to be formed near its vicinity due to a strong blockage of flows by the still hills, resulting in the observed run-up/flow depth being much higher at this site compared to the other two sites.

The selection of the reference location may also contribute to the different values of tsunami amplification factor used in the literature. Satake (1994) chose his reference location as the locations of the near shore tidal gauges. Corinto's tidal gauge was located at 10 m water depth while Puerto Sandino's was located at 4m water depth. The incoming tsunami wave form recorded at Puerto Sandino's tidal gauge might have a larger amplitude than at Corinto. Satake (1994) stated that the amplification factor at Corinto was around 5 and less than 3 for Puerto Sandino. This could be because 1) the incoming tsunami was partially dissipated by the presence of a natural sea dike at Puerto Sandino, resulting in a much smaller onshore run-up, and 2) the choice of the reference location at Puerto Sandino is

nearer to the shoreline compared to that of Corinto (i.e., the wave at Puerto Sandino would have shoaled to a much higher amplitude compared to that of Corinto).

The tsunami amplification factor can be potentially used for aiding in tsunami forecasts for early warning, probabilistic tsunami hazard assessment, and understanding tsunami deposits. Accurate early warning forecasts are crucial as not only can it provide ample time for human evacuation, it can also prevent over-warning. One example of the latter is in 1986 when the Waikiki Beach in Hawaii was evacuated; the estimated economic loss was in the range of US\$40 million (Bernard, 2005). The entire tsunami early warning forecast can be broken up into two spatial regions; offshore and near shore. In the offshore region, Titov et al (2001) have showed through a series of sensitivity tests that the two most important parameters are the earthquake source magnitude and its location. Once these parameters are known, the particular fault zone can be segmented into discrete unit sources, and these unit sources can then be linearly summed or scaled to provide the closest tsunami amplitude for different earthquake scenarios (Burbridge et al, 2008, Titov et al, 2005). Using the information obtained, tsunami wave propagation can be modelled using a linear shallow water model to the near shore region. Rather than using numerical models to simulate the wave propagation process in shallow water or onshore, simple methods such as tsunami amplification factor can be handy tools to provide a fast estimation of flow depths/runups; for example, Mofjeld et al (2000), proposed a semi-empirical statistical method to find the amplitudes of later waves in shallow water. These simple methods can potentially be much quicker than running the near shore inundation simulation, in view of the uncertainty involved in specifications of the source parameters and the unavailability of high-resolution bathymetry and topography for detailed numerical computation.

3.4 Chapter Summary

In this chapter, the concept of the tsunami amplification factor, which has been used in previous studies, was studied experimentally in a wave flume using solitary waves. We studied the effects of the slope of a coastal cliff on tsunami amplification. We found that closer to the onshore cliff, the amplification was stronger. A steeper cliff also resulted in a greater amplification compared with a gentler one. However, below a critical slope of about 45° , the amplification factor increased as the cliff slope increased; beyond this critical slope, the amplification factor remained more or less constant as the cliff slope further increased.

If water splashing is considered, the amplification factor would be even larger. Depending on the topography of the coast, the wave run-up might lead to splashing if it is too steep like the one shown in the thesis. The importance of water splashing is not that well understood at present. It is very possible that water splashes could break branches of high lying coastal vegetation if they induce enough force. On the other hand, it is also very possible that water splashing is not destructive at all. Hence, the first step that should be taken is to differentiate the actual trim-line from the water splashes. The next step would be then to determine the force induced by the water splashes. It would be quite difficult to investigate these two unknowns using real time data. Physical and numerical experiments should still be done. As numerical models are very likely not able to reproduce even the run-up well, physical experiments should still take precedence.

High-speed cameras with good resolution should be used and synchronized with the experimental setup. These cameras should be placed from the side and must be able to record the run-up phase. With the help of this visual enhancer, it would be much easier to distinguish the actual trim-line from water splashing. To determine the impact force due to

water splashing, force sensors can be installed in arrays mounted onto the cliff. After an experimental run is carried out, post analysis of the data can be taken and its significance can then be found out.

The amplification factors reported in this study in principle can be used for a different reference location by a simple shoaling calculation. Moreover, we also showed the comparison of the numerical run-up with the experimental results. Our analysis revealed that the using the 2D NLSWEs based numerical models, the run-up will be underestimated. Hence the amplification factor may be a useful technique to correct this discrepancy.

It is very crucial to look forward so as to come up with preventive measures or have some form of protective mechanisms that can help to defend the coast. At the same time, we should not neglect the present as we are not able to accurately predict when the next big event will happen. Thus there should be two different approaches. The first would be to methodically plan on how to protect the coasts in the long run. The first step that should be taken here is to first identify coastlines that are prone to tsunami attacks. This can be done by analyzing historical data of sediments on the coastlines and determine whether there are any sediments that are carried onshore by tsunamis. The next step would then be to digitally map out the nearshore bathymetry and topography of the area. Then, scaled analogue experiments can be carried out in hydraulic laboratories. By doing so, scaled down hydrodynamic parameters can be recorded especially within the onshore domain. Concurrently, numerical experiments can also be carried out and its results can be compared with those obtained from the physical experiments. By analyzing these results, an amplification formula could then be formulated and then implemented into a real-world scenario. It should be noted that a large amount of initial conditions have to be tested

sufficiently so that a robust enough databank can be produced so as to cover a wide range of scenarios for a particular site.

The second approach is to plan for the present. To do so, a quick and efficient method should be used. At present the closest thing to a real time system is the tsunami early warning system. This system comprises of offshore buoys that are able to detect tsunamis through signal changes in their real time recorded data and simple tsunami models that require minimal computation time to generate outputs of run-up and inundation distance. These models often under-predict both the actual results so it would be more feasible to apply a safety or amplification factor to the numerical results. Hence, till enough research is done for to protect coastlines susceptible to tsunami attacks, we can only rely on early warning systems and find ways to narrow the range of the amplification factor that should be used for each area.

The idea behind the amplification factor is to predict the maximum run-up on the coast before tsunami impact. After impact, the amplification factor is not as important anymore. This is because the maximum inundation distances and run-up can be easily identified. According to the post-tsunami survey field guide published by UNESCO (2014), tsunami traces can include lines of debris, denuded hillsides, debris left on stairs, or broken branches. Besides, the guide also recommend conducting interviews on survivors of the disaster as their accounts might provide further insight into the characteristics of the tsunamis. Although for the ‘now’ the amplification factor might not be as important, it is still very crucial to extract data (such as run-up and inundation distance) onsite to be used to predict future tsunamis for the future. The data collected can be compared to numerical simulated

results and through further fine-tuning using a more robust database, the amplification factor for that particular site can be more accurate.

At present the most reliable real-time measurement system is the DART (Deep-ocean Assessment and Reporting of Tsunamis) system developed by NOAA (National Oceanic and Atmospheric Administration). This system was first introduced in 1995 and now due to continual development it is already in its fourth generation. Figure 3.12 shows the first generation DART system developed by NOAA. The two main components of the system are the sea bottom pressure recorder and the buoy that houses all the important electronics of the system. The buoy is moored to the seabed by a four ton weight. This ensures that the buoy would not be displaced under extreme wave conditions.

Fig 3.12) First generation DART system. Image taken from Bernard and Meinig (2011)

Figure 3.13 shows the network of the current DART system. At present there are about forty systems deployed worldwide with the majority placed around the infamous Pacific Ring of Fire. However, only two stations are owned by Asian countries. One in Indonesia and the

second in Thailand. At the time of the 2004 Indonesian tsunami, this system was not available yet in Indonesia. Hence, there are no reliable real-time data for that event that can be derived from the buoys.

Fig 3.13) A global map showing the locations of the DART network. Image taken from Bernard and Meinig (2011)

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Chapter 4

Dam Break Waves

- *Physical and Numerical
Experiments*

4.1 Introduction

In chapter 3, the onshore characteristics of a solitary wave impinging on a composite beach with a steep onshore cliff was discussed. In this chapter, we will be introducing another wave form; dam break waves. Dam break waves differ significantly from solitary waves. As mentioned in Chapter 1, dam break wave surges are rather similar to that of the tsunami wave onshore surge. The applicability of the analytical solutions for dam break waves proved to be quite reasonable when used on tsunami onshore surges (Chanson, 2006b). This means that for the onshore region, tsunami surges behave rather similarly to dam break waves surges. Hence it would be useful to use dam break waves. Moreover, based on the findings from the study area of interest as shown in figure 1.5, where only a long enough wave has the potential of filling up the cave to ensure that deposition occurs, dam break waves being longer than solitary waves, have the potential of filling up the cave from a laboratory perspective . However, the main drawback of using dam break waves is that it cannot be easily repeated. Due to the manual release mechanism of the gate, repeatability is a problem. Although there are other problems that might hamper the experiment, such as time delay needed to fill up and empty the wave flume after each run, the main issue is still the repeatability. To overcome this problem, we have to ensure that the gate release happens quickly. This means that once the equipment are ready to be synchronized, the previously mentioned helper had to release his grip on the rope instantly (see section 2.1.2).

By incorporating dam break waves in our experiments, it could potentially affect the maximum run-up measured on the steep cliff compared to the results obtained for the solitary waves experiment. Besides the different wave form used that will lead to different run-up values, there could also be other reasons for the observed differences. These

differences are the steepness of the incoming wave front and its onshore celerity. For the dam break waves, other mechanisms that could lead to varying run-up heights are the dam reservoir's height and length, as well as the dam break gate release speed. These issues are discussed in this chapter. The numerical portion of this chapter is also simulated by the Delft3D-FLOW package. Comparison between the experimental and numerical results is also conducted. A dimensionless parameter (which is the ratio between the physical run-up and the simulated run-up) is presented and its applicability to real scenarios is also discussed.

4.2.1 Experimental model for dam break wave run-up

Fig 4.1) Experimental setup for dam break wave run-up

Figure 4.1 shows the experimental setup used for the dam break flow. The beach setup is made up of the similar composite type slope shown in figure 3.1. The still water depth was

fixed at 0.35m. The distance between the toe of the beach profile and the dam break gate is 8.84m. All cases were ran twice to ensure its repeatability.

Fig 4.2) Comparison of experimental results from two separate runs for test case DB4

Figure 4.2 shows the comparison of measured wave profile time series at the three offshore locations (s1, s2 and s3) from the raw recorded data. Other than a slight shift in time due to the time difference for both the runs between the gate release and the start of recording, the key characteristics of the wave profile measured like the maximum wave height are almost similar. For each test case, at least two separate runs were conducted and the results were found to be acceptable. In terms of the number of runs required for each test case when conducting wave experiments, other researchers e.g. Li (2000) also made two runs.

These tests were conducted with the same equipment as the solitary wave run-up cases. However there is the addition of a fourth camera that was placed at the side to capture the gate release speed. The locations of the acoustic sensors are the same as that of the solitary wave tests. Sensor s1 is placed close to the mobile gate to record the wave profile the instant

the gate is released. Sensor s2 is placed closer to the composite slope 6.37m from s1 to record the dam break wave's propagation and the wave reflection phenomenon. Sensor s3 was placed at the nearshore depth region 1.69m from s2 to look into the shoaling or amplification from the incident wave. It should also be highlighted that the sensor locations for the solitary wave run-up experiments (refer to figure 3.1) are identical to that for this set of experiments. Moreover, we ensured that the spectrum of test cases carried out for the dam break waves did cover those done using solitary waves; i.e. the solitary waves tests have a similar counterpart in terms of incident wave magnitude for the dam break waves.

Table 4.1 shows the different configurations of cases tested. The solitary wave experiments were also included in this table as results obtained from these two different wave forms will be compared to examine how different wave forms affects the run-up on the cliff.

4.2.2 Dam break waves and solitary waves - wave form comparison

We first compared the cases between DB5 and SW1. The respective incident wave heights measured for the two cases were; 4.98cm and 4.97 cm respectively. By referring to figure 4.3, the measurement for s1 shows that the dam break wave has a constant wave height that remains for around 10s compared to that of the solitary wave which is around 1s. The measurement for s2 indicates that there is superposition of waves around the 15s mark for the dam break waves. This is due to wave reflection from the composite beach slope. This superposition phenomenon is absent for the solitary waves. The main reason for this is that the dam break wave is much longer than the solitary waves. Hence, its wave front reaches

the beach quicker and it gets reflected first compared to the solitary wave. The measurement of s3 simply further emphasizes the point made above on s2.

Table 4.1) Configurations for physical experiments on dam break waves and solitary waves

Dam Break Cases					
Test Case	L (m)	D(cm)	Gate Release	Avg Gate Speed (cm/s)	Incident Wave Height (cm)
DB1	6	10	Instantaneous	117.7	5.68
DB2		13	Instantaneous	119.4	7.23
DB3		16	Instantaneous	120.35	8.55
DB4	9	10	Instantaneous	102.41	6.09
DB5			Slow_1	30.09	4.98
DB6			Slow_2	17.13	4.63
DB7	10	10	Instantaneous	105.36	6.53
Solitary Wave Cases					
Test Case	Incident Wave Height (cm)				
SW1	4.97				
SW2	5.52				
SW3	6.06				
SW4	6.51				
SW5	7.18				

By looking at the run-up trim line measurement in figure 4.3 the maximum run-up for DB5 is 24.77 cm compared to 15.44 cm for SW1. Both these waves have almost identical incident wave heights, however the recorded run up actually differs by about 1.6 times. This means that there has to be other factors that have affected the maximum run-up other than the incident wave height. The time at which the maximum run-up was recorded for DB5 and SW1 were 10.91s and 11.27s respectively. Table 4.2 shows the comparisons of all the test cases done. Figures (v to viii) in the appendix show the other comparisons.

Table 4.2) Test cases for wave form comparison with their respective incident wave and run-up

Solitary wave cases	Incident wave height (cm)	Run-up (cm)	Dam break wave cases	Incident wave height (cm)	Run-up (cm)
SW1	4.97	15.44	DB5	4.98	24.8
SW2	5.52	16.24	DB1	5.68	23.24
SW3	6.06	18.19	DB4	6.09	25.01
SW4	6.51	24.75	DB7	6.53	24.68
SW5	7.18	26.06	DB2	7.23	55.11

Fig 4.3) Time series for test case DB5 and SW1

From table 4.2, it can be seen clearly that despite having almost similar incident wave heights, the maximum run-up actually differs appreciably. On average the difference between the first four comparisons is around 35%. Only the last comparison has a much larger difference. The reason why this is so will be elaborated later. This comparison between the solitary waves and dam break waves demonstrates that even though the

magnitude of the incident wave height does have an effect on the maximum run-up, there should be other factors that affect the maximum run-up. This is so because, given the similar incident wave heights, dam break waves are able to reach a greater run-up value.

Other factors that could potentially affect the maximum run-up value is the onshore flow celerity as well as the wave steepness. For the offshore section of the experiment, this can be approximated to 1.97m/s using equation 3.4. Alternatively, we can also deduce the wave celerity from the time-series in figure 4.3 for s1 and s2. By extracting the time when the peak occurs, we can also calculate the wave speed. Using this approach, the wave celerity would then be 1.79m/s. This would mean a 9% difference from the first approach. Utilizing this method, we can then also determine a representative wave celerity for the dam break wave. From figure 4.3 for s1 and s2, the computed wave celerity is 1.89m/s. This represents about a 6% speed increase. This difference in flow celerity would have resulted in more kinetic energy at the wave front of the dam break wave resulting in a much higher run-up.

Another factor that could influence the run-up is the wave-front steepness (refer to figure 4.3). The onshore wave front steepness as defined in figure 4.4 would be 0.319 for SW1. For the DB5, the wave front steepness is 1.05 (refer to figure 4.5). The steeper wave profile from the dam break wave would probably result in a much larger run-up value.

Fig 4.4) Definition of wave front steepness, V/H

Fig 4.5a) wave front of SW1, b) wave front of DB5

4.2.3 Dam break gate release speed effect on run-up

It has already been discussed in chapter 2 that the dam break gate release speed varies. The gate speed for each release is recorded in table 4.1. In the table, we classified any gate movement speed greater than 100 cm/s as an instantaneous release. Due to the varying gate release speed that we had for each set of dam break case run, it might mean that potentially the wave form produced and the resulting run-up could differ. Besides, a quicker gate release would most likely lead to the resulting surge wave having a greater flow celerity compared to that with a slower release speed. A greater flow celerity could lead to a much greater run-up on the cliff. Hence, it is important to investigate the effect of the gate release speed on the wave forms produced and the resulting run-up.

For the scenario of a reservoir length of 9 m and reservoir height of 45 cm (DB4-DB6) we have tried three different gate release speeds. The first trial was deemed instantaneous. This is because the gate speed was 102 cm/s; the height of the mobile gate was around 60 cm. Given the gate speed, the time in which the gate will be completely lifted took only 0.6s. In contrast, the second trial's gate release speed was 30 cm/s and the third was 17 cm/s. The

gate release speed is varied by using less weights and more resistance to retard the quick upward motion.

Figure 4.6 shows how the gate release speed affects the incident wave profile and the run-up trim line on the cliff. The first graph shows the variation of the gate speed, the second the incident wave profile and the third the run-up trim line. Although the gate release has been retarded, the incident wave form shown does not vary too much from the instantaneous gate release case. It seems that a quicker gate release speed would lead to a slightly steeper wave profile compared to those with a slower gate release speed. It can be seen that the run-up trim line for these three cases are almost identical. For this study, since the most important parameter that we are looking into is the run-up trim line, it seems that varying the gate release speed does little to affect the run-up magnitude on the cliff.

Fig 4.6) Comparison of instantaneous gate release v.s. two different slow gate release for 9m reservoir depth and 45cm height

4.2.4 Reservoir height and length on run-up

In terms of dam break waves, there are two easily controllable parameters that can affect the run-up. The first is the reservoir height. By having different reservoir heights for our initial conditions, the incident wave profile will most likely be different, thus leading to different run-up heights on the cliff. The second controllable parameter is the reservoir length. By increasing the reservoir length, the wavelength of the surge wave will increase. This will lead to a difference in the amount of time the water body remains at the front of the cliff.

Hence, in this section, we first will be looking at how different reservoir heights will influence the run-up height on the cliff. Three different heights were chosen in this study; 45cm, 48cm and 51cm (DB1-DB3). All the test cases were conducted for a reservoir length of 6m with instantaneous gate release speeds.

Figure 4.7 shows the time series for sensor location s1-s3 of DB1-DB3 with the trim-line. By looking at the first sensor location s1, differences in the incident wave height were observed. The respective incident dam break wave heights were 5.68cm, 7.23cm and 8.55cm. With reference to the reservoir height of 45cm, the increment of the other two incident waves are 27% and 50.5%. In comparison to the percentage difference in the reservoir height (6.6% and 13.3%), the incident wave heights increased much more significantly.

Fig 4.7) Time series of the dam break waves for 6m reservoir length and 45cm height

Figure 4.7 shows that the maximum recorded run-up for the case of the 45cm reservoir height was around 23cm from the mean water level. However, the maximum recorded run-up for the 48cm and 51cm reservoir height cases were 55cm and 80cm respectively. A plausible cause for this large increment given the smaller increase in the incident wave heights could be attributed to the onshore flow celerity. A quicker flow speed would lead to the wave front climbing up to a higher point. This is true for solitary waves and most likely for dam break waves. Ritter (1852) provided an analytical solution for the instantaneous dam break wave celerity.

$$U = 2\sqrt{gD} ; U = 6.264D^{\frac{1}{2}} \quad (4.1)$$

U is the wave front celerity in m/s and D is the reservoir depth in m. From the above equation, the wave front celerities upon the gate release for the three cases are 4.2m/s, 4.34m/s and 4.47m/s (3.33% and 6.43% increment from reference). Chanson (2006a)

provided an analytical relationship for the instantaneous dam break wave height (D) at a fixed cross-shore location.

$$d = f(D^{7/9}, U^{2/3}, G^{1/9}) \quad (4.2)$$

G is a friction factor. From this relationship, it can be shown that the incident wave height not only varies non-linearly with the initial reservoir height but also the wave front celerity.

Figure 4.8 shows the snapshot wave profiles for the three cases. Each wave profile in each subplot was taken from a single video frame with the next profile obtained from the next video frame. The frame rate for the acquisition system used was 30 fps as mentioned in chapter 2. Hence the time between each wave profile plotted in all the subplots is about 0.03 s. Using these information, the onshore wave front displacement can be determined. As the frame acquisition rate of the cameras is also known, the onshore wave celerity can also be determined. Figure 4.8 gives a clearer understanding of the onshore propagation speed of the wave profiles shown in figure 4.7. The respective wave front celerity are 1.32m/s, 1.78m/s and 3.78m/s.

From figure 4.8, the wave front locations and time instances that it occurs can be extracted from each test case. With reference to the 45cm wave height case (DB1), the corresponding increase in the onshore wave front celerity for the other two cases are 35% (DB2) and 186% (DB3) respectively. Compared to the ratios for the wave front celerity upon the gate release, these values are much greater. This goes to show that increasing the reservoir height does increase the maximum run-up measured. However, a more important parameter that is affected by increasing the reservoir height is the onshore wave front celerity. It seems that increasing the reservoir height serves to further increase the wave front celerity. Despite

only a slight increase in the reservoir height this huge increase in the wave front celerity plays an important role in influencing the maximum run-up value.

Fig 4.8) Spatial wave profiles evolution for the three cases; DB1-DB3

We also investigated how the reservoir length would affect the subsequent wave profiles and run-up for cases DB4 and DB7. The time series of s1 in figure 4.9 shows the recorded incident wave profile. As shown for the dam break case with a reservoir length of 10m (DB7), no trough was observed at around 15s. Instead, another peak can be identified. This is due to the length of the reservoir with respect to the flume length. For this situation, this peak was most likely due to the reflected wave that bounces off the rear stationary wall. At the location of s1, before the dam break wave can be fully developed, the reflected wave has already arrived, resulting in an increase in water depth at that time instant ($t = 15s$). If a longer wave flume was used to conduct the experiments, the time series wave profile for the 10m reservoir length case for location s1 should be similar to that of the 9m reservoir

length case with the exception that the constant water depth time interval section ($t = 6\text{s}$ to 15s) will be longer due to the longer expected wave length.

The time series for the wave profile for s2 and s3 show that both DB4 and DB7 are rather similar. The main visible difference lies around the 20s mark. At this instant, it can be observed that the longer reservoir length case led to a greater amplification of the tail end of the wave profile. This increase can be due to both the longer wavelength of DB7 and also because of the additional water input due to the increased reservoir length.

The run-up trim line profile for the 9m reservoir length case and the 10m reservoir length case elucidates two findings; 1) the maximum run-up recorded is not affected by the reservoir length. This can be shown in the time series of T_L in figure 4.8. As observed, the maximum run-up for both cases are almost the same. This means that for dam break cases, the main variable that actually influences the maximum run-up height is the initial reservoir height. With the reservoir height remaining constant, increasing the reservoir length does little to change the maximum run-up recorded. 2) the duration at which the water body stays right at the front of the cliff is longer if the reservoir length is longer. From figure 4.9, this duration is longer for the 10m reservoir length case by about 1.5 s. As mentioned before, this increased duration is due to the increased wavelength of the dam break wave as well as the additional mass of water that was added into the system.

It can also be argued that in the previous discussion where the reservoir height was varied, additional water mass was also added. For the previous section, the added volume of water from the benchmark case (DB1) was around 0.1m^3 and 0.2m^3 respectively. In this discussion about the reservoir length, the additional volume of water added to DB7 from DB4 was around 0.24m^3 . These comparisons show that it is important to find out the main

area of study area of interest when it comes to run-up due to dam break waves. Although, additional water mass was added to both benchmark studies it yielded different results with respect to the resulting run-up. Given that similar amounts of water was added to DB3 from DB1 and to DB7 from DB4 ($\sim 0.2\text{m}^3$), both show contrasting results in the trim-line profile. It seems that having more water in the reservoir in the vertical direction will contribute to increasing the maximum run-up. However, having more water in the cross-shore direction would instead lead to a greater duration at which water body will remain in front of the cliff.

Fig 4.9) Time series of the dam break waves for 9 m and 10 m reservoir length and 45 cm reservoir height (DB4 and DB7)

4.2.5 Experimental and numerical comparison of dam break waves

Figure 4.10 shows the time series comparison between the experiment and numerical simulation results of test case DB2. Plots s1-s3 in figure 4.10 shows that the incident wave generated due to the instantaneous dam release in the numerical simulation created a rather steep wave front as compared to that measured experimentally. This could be due to: 1) the grid size is not small enough to capture the small incremental changes. 2) the current numerical tool that we use Delft-3D does not have the capability to capture such shock waves, as it is still a 2-D package based on the SWE rather than a full 3-D model.

Looking at s1, the experimental results show a period (5.5-10.5s) of rather constant water level. This period is longer than that of the numerical simulation's period of around 3.5s. In addition, the presence of the undulating tails of the waves from the experiments is not present in the numerical simulation. Moreover, s2 and s3 show the reflected wave front (between 20 -25s). This is not apparent in the experiments except for the wave peak in s3 at around 30s. This meant that the numerical model generated a wave with a shorter wave period and a larger wave celerity. In the numerical model, there is the neglect of important wave interactions nearshore and onshore such as wave breaking. Besides, dispersion is also not considered in the simulation. A combination of initial water level at 0 m, and wave forcing at the start of the run would cause a shock in water movement in the onshore region. This shock in water movement can be identified by a sudden spike in the water surface elevation which has a very steep wave front. Because of the presence of the steep cliff, onshore wave propagation is halted and the wave is being forced to move back in the opposite direction. Because dispersion is not considered here, the reflected wave would have the similar steep crests as it propagates back.

The trim-line profile clearly shows that the numerical model failed to capture the maximum value of the run-up on the cliff. In the numerical simulation, there is a cut-off maximum reading of the run-up height at around 20cm. In reality there is the presence of a return bore and splashing water; such phenomena could not be captured using the numerical model. Besides, the trim-line profile also shows that the water mass remains at the front of the cliff for a much shorter time compared to the actual experimental results. Figures ix to xi in the appendix show the other comparisons of different incident wave heights.

Fig 4.10) Time series comparison of experimental and numerical results using test case DB2

It has already been shown that dam break waves can potentially record a greater run-up height than solitary waves. For solitary waves, one parameter that affects the maximum run-up is the incident wave height. This same parameter was also used as a factor to determine its influence on the dam break wave run-up. Figure 4.11 shows the maximum recorded run-up plotted as a function of the incident wave height and reservoir length for both the

experimental tests and the numerical simulations. We observed that the numerical results fail to capture the actual maximum run-up from the experimental results. This could be due to two reasons; 1) the maximum run-up occurs rather quickly, it reaches its peak value like a sudden spike. Also this can be attributed to the time-step given in the numerical simulation. However, reducing the time-step would then mean that the grid sizing would have to be reduced as well, increasing the computational time. 2) The time period in which the wave moves up the cliff and then comes crashing down is highly turbulent. This nearshore region has flows that could be multi-directional. However, the numerical model used is a 2-D non-linear shallow water equation solver. The solution provided is merely in the depth-averaged form and this would underestimate the run-up measured on the cliff.

The three runs used to plot the results in figure 4.11a were that of DB1, DB2 and DB3. These three runs have the same reservoir length but the difference lie in the initial reservoir height. If we were to only look at the experimental results of the maximum run-up height, it can be seen that the maximum run-up increases with increasing incident wave height. The numerical simulations results show that the maximum run-up does not vary much with increasing wave height.

For the experiments and numerical simulations, another factor that was changed was the reservoir length used. Figure 4.11b shows the effect of increasing reservoir length on the maximum run-up. The three runs used were DB1, DB4 and DB7. These three runs have similar initial reservoir height but different reservoir length. It can be seen that apart from the discrepancy in values between the experimental and numerical results, both seem to exhibit a constant value with increasing reservoir length. This would probably mean that the length of the reservoir does not really affect the maximum run-up for dam break waves.

Determining the amplification factor of the actual run-up compared to the numerical run-up can be a useful approach to estimate the run-up under other conditions. From a numerical standpoint of view, it would be much more accurate to predict the maximum run-up using 3-D models. But if for example an early warning system is adopted and being put into place, there would definitely be insufficient time to complete the simulation. Under such circumstances, using the results from 2-D shallow water equation models coupled with an amplification factor could be a plausible solution.

Fig 4.11a) Incident wave height and b) reservoir length variation with maximum run-up (experimental and numerical)

As the maximum run-up height is not affected by the reservoir length for the dam break cases, it should suffice by taking a simple average of both the experimental runs and numerical simulations to obtain a difference ratio. To illustrate this, assume a tsunami had occurred offshore and the fault has a rupture length of L_o . Since offshore buoys with tidal gauges installed had recorded the time series of the water elevation changes, the incident

wave height H_o can be obtained. Moreover, the maximum recorded run-up $R_{o, real}$ was also captured after post tsunami surveys. Scientists might then use these same parameters to obtain a numerical run-up value, $R_{o, num}$. From our results, we find that the ratio $R_{o, real} / R_{o, num}$ could be used by scientists to predict the maximum run-up caused by a future earthquake generated tsunami of a different rupture length L_I but has similar incident wave height H_o . In other words, for such cases

$$R_{1, real} = \frac{R_{0, real}}{R_{0, num}} \times R_{1, num} \quad (4.3)$$

The offshore incident wave generated by the fault rupture is the main parameter that influences the maximum run-up. By obtaining the ratio of the points from figure 4.10a and then using linear regression, we are able to obtain a simple linear relationship between the ratio and the incident wave height (D is the offshore water depth). For our experimental cases and simulation runs, the minimum (H_o / D) used was 0.163. This would mean that the applicability of this result can only be used at regions of deep to medium water depth.

$$\frac{R_{real}}{R_{num}} = 34.53 \left(\frac{H_o}{D} \right) - 3.93 \quad (4.4)$$

To put into perspective, if for example the wave height measured at 12 m water depth is about 2 m, the ratio obtained from the linear equation is 1.8. This means that the actual run-up would be around 1.8 times the result obtained from numerical simulations using the 2-D shallow water equations.

4.3 Dam break waves amplification factor

Table 4.3) Test cases for amplification factor study

Dam break wave cases	Incident wave height (cm)	Run-up (cm)	A factor	Solitary wave cases	Incident wave height (cm)	Run-up (cm)	A factor
DB5	4.98	24.8	1.877	SW1	4.97	15.44	2.803
DB1	5.68	23.24	1.855	SW2	5.52	16.24	2.805
DB4	6.09	25.01	1.998	SW3	6.06	18.19	2.899
DB7	6.53	24.68	1.965	SW4	6.51	24.75	3.722
DB2	7.23	55.11	2.644	SW5	7.18	26.06	3.602

Table 4.3 shows the cases used for the amplification study for the dam break cases and the solitary wave cases. As only tests on the cliff angle of 79° was studied for dam break waves, we could only obtain the data shown in figure 4.12. Moreover, we could only be discussing the amplification factor for the maximum measured run-up. For this comparison, we will be using location s3 (refer to figure 4.1) as the reference location and the measured incident wave height at that point as the reference wave height.

Fig 4.12) Amplification factor study for dam break and solitary waves for the maximum run-up at a cliff angle of 79°

The results in figure 4.12 show that the amplification factor of the dam break wave tests is smaller than that of the solitary wave tests. This means that despite the measured run-up using dam break waves is higher than the solitary waves, the reference wave height obtained for location s3 is much smaller for the solitary waves compared to the dam break waves. Similar to the trend observed from the solitary wave amplification factor test study, increasing the incident wave height would result in an increase in the amplification factor for the dam break case.

4.4 Chapter Summary

This chapter introduces dam break waves as an alternative test wave for tsunami studies to solitary waves. Given a similar incident wave setting, dam break waves will produce a greater run-up height and also the duration in which the trim-line stays in front of the cliff will also be longer. This is because dam break waves have a faster flow celerity which will potentially lead to a much higher run-up than solitary waves. Besides, the trim line initiated by a dam break wave will be over a longer time interval as dam break waves are longer than solitary waves.

The release speed of the dam break gate could have an impact on the run-up of the cliff. However, from our observations, we saw that the run-up was not severely affected by the difference in the gate release speed. The major difference noted was the incident wave form. It seemed that a faster release would lead to a steeper incident wave.

For the experimental dam break waves tests, the parameters that can be easily controlled are the reservoir height and the reservoir length. Increasing the reservoir height will lead to

an increase in the run-up height while increasing the reservoir length will lead to an increase in the duration the water body that remains in front of the cliff.

The comparison of the experimental and numerical results of the dam break waves reveal that the numerical results underestimate the experimental run-up height. The duration in which the water body remains in front of the cliff is also shorter from the numerical results.

Chapter 5

Hypothetical Sedimentation Time

5.1 Introduction

To really determine the importance of the sediment motion during a tsunami run-up event, it is important to first understand the mechanics of the sediment motion during both the run-up phase and also the drawdown phase. For this thesis, the main sediment type that we are interested in is fine sand (~ 0.2 mm in diameter).

Figure 5.1 shows the illustration of a typical tsunami run-up and drawdown process on a plane beach. In figure 5.1a, as the tsunami bore moves onshore, it carries with it sediment as bed load (Young et al. (2010)). During the onshore shoaling and wave breaking process, the tsunami would also entrain sediments and carry them towards the shore as suspended load. In the drawdown process, the retreating flow is like a sheet flow with high velocity. Sediments carried away is usually through bed load.

Fig 5.1) Illustration of the run-up and drawdown process of a tsunami on a plane beach

Figure 5.2 shows the illustration for a tsunami run-up and drawdown process for the setup of this thesis. The key difference here is represented in figure 5.2e. It can be clearly seen that the cliff halted the propagation of the tsunami bore and this led to an abrupt return flow. Besides, the flow depth just in front of the cliff was also amplified. Additional phenomenon that were observed during this flow depth amplification process was an additional wave

breaking. In terms of sediment transport, this meant few things. Firstly, there would be sediments accumulated at the foot of the cliff during the run-up process. During the return flow duration, wave breaking occurs and the sediments at the foot of the cliff would get entrained and be suspended in the flow body. Before the sediment can settle down on the beach, the return flow would have transported these sediment back into open waters.

Figure.5.2) Illustration of the run-up and drawdown process of a tsunami on a plane beach with a steep inland cliff

In addition, it should be noted that figure 5.2 only illustrates the run-up and drawdown of a single tsunami bore. In reality, a tsunami event would result in multiple inundation by multiple waves spread out over a certain time period. This would mean that according to figure 5.2, subsequent incoming waves would have carried much more suspended sediment in its body. Thus it would be very likely that the sediments in the cave near Pulau Sibigau are transported through suspended load.

Based on the numerical model used in this thesis, it would be hard to incorporate a cave model into the domain. Hence, we have to think of other ways to deduce the sedimentation

time allowed in the cave model. A simple approach that possibly could allow us to compare our experimental results with the numerical results obtained from Delft3D-FLOW is to define the sedimentation time as the amount of time the run-up trim-line remains above the cave opening. This definition is based on the assumption that if the water mass remains above the cave ceiling, the cave is filled. Moreover, the horizontal cross-shore flow within the cave is likely to be very small and may be considered to having zero velocity. If the horizontal flow within the cave is indeed zero, this meant that if the flow does carry any sediments, the sediments will have this amount of time, during which the water is stagnant, to be deposited inside the cave. By making this hypothesis, we would then be able to make two different comparisons; 1) experimentally compare the result obtained by adopting this hypothesis to time the water actually remains in the cave to determine the accuracy of this assumption and 2) compare the result obtained from this hypothesis to the trim-line duration obtained through the Delft3D-FLOW model.

Fig 5.3) Schematic of the hypothetical definition of sedimentation time

Figure 5.3 shows a schematic on how the hypothetical sedimentation time is defined. t_0 marks the time instant when the water mass first hits the cliff. At this point, the height of the water body just in front of the cliff is shown. t_1 marks the moment at which the maximum run-up was recorded. From t_0 to t_1 the water level in front of the cave is increasing. During

this time interval, the water continue to fill the cave, and it should be completely filled as the water level in front of the cliff begins rising to its maximum level. The water level within the cave should not drop until the water level in front of the cliff fall below the height of the cave opening (which is 5cm in the experiment). So, from t_1 to t_2 , the water level in front of the cave is decreasing. The time interval at which the water body in front of the cave first hits the cliff, then begin to increase to its maximum height, before eventually falling to the level of the cave ceiling (t_0 to t_2) marks our definition of the hypothetical sedimentation time in which the cave is completely filled.

5.2.1 Comparison of experimental results with hypothetical sedimentation time

Here, we compare the actual time in which the water remains in the cave to the time the water body remains in front of the cliff. Due to the fact that some time has to be taken to fill up the cave as well as for the water to drain out, this initial assumption may not be exact. The extent of the accuracy of this hypothesis will be shown below. It should also be pointed out that for this experiment, tests using solitary waves were carried out but they failed to completely fill the cave. This likely meant that there is insufficient time for any sediments carried by the wave to be deposited. Hence for this discussion, only the tests conducted using dam break waves were considered. For the flow inside the cave, web cameras were placed by the side to record the flow movement.

Table 5.1 shows the configurations used for the hypothetical sedimentation time tests. The experimental setup is identical to the one shown in figure 4.1.

Table 5.1. Configurations of dam break tests

Dam Break Cases			
Test Case	L (m)	D(cm)	Incident Wave Height (cm)
DB1	6	10	5.68
DB2		13	7.23
DB3		16	8.55
DB4	9	10	6.09
DB5			4.98
DB6			4.63
DB7	10	10	6.53

Figure 5.4 shows the case of DB4. The top figure shows the time in which the water stays in the cave and the bottom figure the trim-line. The horizontal line drawn across the bottom figure at the 5cm mark indicates the cave ceiling. From figure 5.4, the duration in which the cave is completely filled is about 9s whereas the duration in which the trim line is above the cave ceiling is around 10s. The cave flooding process can be described as follows. Initially when the trim line exceeds the cave ceiling, water has already started filling the cave. However, it takes around 1.5 seconds for the cave to be completely filled. Once the cave is completely filled, the trim line was above the cave ceiling for about 8s. During this duration, no visual interpretation of flow movement can be observed by the cameras. When the trim line is at the cave ceiling after 8s, the cave remains completely filled and water only starts to flow out about 0.5 seconds later. One thing to note is that the total duration in which water remains in the cave is almost the same as the total duration in which water stays at the front of the cave entrance.

When sediment transport within the cave is considered, the time series profile of the flow depth inside the cave can provide us with a preliminary understanding. The water body first brings sediment (from offshore or near shore) into the cave. It then began to completely fill

the cave in about 1s. For the next 9s, the cave remain completely filled up. During this duration, it is expected that sediment carried by the water body will start to settle down. This is because the water mass right in front of the cave exerts a pressure gradient on the water body in the cave, thus cancelling out any return flow out of the cave. This would result in an almost negligible cross-shore velocity scenario within the cave hence allowing the sediment to settle. The drawdown period begins after 10s and lasts for around 2s. This drawdown period should coincide with the erosion of any deposited sediment or removal of any sediment still suspended in the flow body.

Fig 5.4) Comparison of flow time-series in the cave and trim line time series above cave ceiling
for 9m reservoir length, 45 cm reservoir height

We would expect that a longer reservoir length will mean a longer dam break wave length which will result in the water body being able to stay in front of the cave entrance for a longer duration. To reaffirm the initial hypothesis, a comparison is made using the case for the 10m reservoir length and 45cm reservoir height. From figure 5.5, the duration in which

the cave is completely filled is around 10s. The total duration in which the water body stays at the front of the cave entrance is around 13.5s.

Using the cave height (5cm) as a cutoff, the amount of the time that the water body was over the cave ceiling was around 13s for the 10m reservoir length case. For the 9m reservoir length case, this amount of time was around 10s. Intuitively, this would mean that a longer wave length would result in the cave being filled for a longer duration.

Fig 5.5) Comparison of flow time-series in the cave and trim line time series above cave ceiling
for 10m reservoir length, 45 cm reservoir height

5.2.2 Comparison of experimental results with hypothetical sedimentation time for the tests with a hole at the rear of the cave's ceiling

In section 1.2, it was stated that an objective of this study is to investigate the deposition of sediments inside the cave in Banda Aceh. It was reported that there was a hole at the ceiling

of the cave in Banda Aceh. This could lead to certain differences in the flow field within the cave. With the presence of the hole, air can be forced out and hence the air pressure that may be retarding the flow will be negligible. This means that the water can easily enter the cave and it will be filled up more rapidly. To replicate the scenario of what was seen in Banda Aceh, we conducted experiments with a hole at the back of the cave ceiling and compare the results with those without a hole.

Fig 5.6 shows that the maximum run-up measured for the case with a hole at the ceiling of the cave is similar to that without the hole. On the other hand the flow patterns in the cave may be different as the effect of air pressure is now removed. However, the recorded data show that the duration in which the cave is completely filled was about 10s. A slight difference to be noted was that for the case with the hole on the cave ceiling, the cave was filled slightly faster by about 0.25s. This difference could be due to the effect the air pressure has when the cave is sealed at the ceiling leading to this pressure slowing down the water that is entering the cave.

Figures 5.7 and 5.8 show the flow pattern of the water mass as it flows into the cave. The right hand side is the cave entrance and the left hand side the rear of the cave. In other words, water enters from the right hand side of the image. The spacing between two yellow vertical stripes in each time frame image from figure 5.7 indicate a 5cm gap. The positions 10, 15 and 20cm inside the cave are shown in the time frame image. When the hole in the ceiling is sealed, water is unable to escape through any outlet, hence it bounces off the back wall and is reflected almost immediately. For the other case, water can escape through the hole at the back of the cave ceiling. This means that for the return flow to reach the same flow depth at a particular location, more time will be needed. Thus the delay.

Fig 5.6) Comparison of flow time-series in the cave and trim line time series above cave ceiling
for 9m reservoir length, 45 cm reservoir height with a hole at the ceiling of the cave

The flow exit patterns also vary for both cases. Figures 5.9 and 5.10 show that for the case without the hole at the cave ceiling, the water depth first dropped near the cave entrance, this leads to an induced flow movement towards the end of the cave. After which, the water is reflected from the rear of the cave before exiting the cave. On the contrary, for the case with the hole on the cave ceiling, the water level dropped from the rear of the cave first. This leads to a reduced flow depth towards the entrance of the cave. However, this drop in water depth also coincides with the drop in water depth in front of the cave thus resulting in a superposition of waves near the cave entrance. The exiting flow has a much higher horizontal flow velocity than that created by the drop in water depth at the cave entrance resulting in the water body exiting the cave without being reflected back to the rear of the cave. If sediment transport is considered, the location at which superposition of the waves occurred would have resulted in a very small cross shore velocity. This would mean that

additional deposition (if the flow still has suspended sediment) would likely take place at that location.

Fig 5.7) Snapshots of the time of entrance of the water mass till completely filled for the 9m reservoir length, 45 cm reservoir height case

Fig 5.8) Snapshots of the time of entrance of the water mass till completely filled for the 9m reservoir length, 45 cm reservoir height case with a hole at the ceiling

Fig 5.9) Snapshots of the of exit of the water mass for the 9m reservoir length, 45 cm reservoir height case

Fig 5.10) Snapshots of the of exit of the water mass for the 9m reservoir length, 45 cm reservoir height case with a hole at the ceiling

5.3 Comparison of trim-line duration with experimental and numerical results

Fig 5.11) Incident wave height and reservoir length variation with hypothetical cave submergence time (experimental and numerical)

Although the Delft3D-FLOW model is not capable of producing any results pertaining to the flow within the cave, it can still tell us the trim-line duration in which the water mass stays in front of the cliff. Figure 5.11 shows how the hypothetical sedimentation time varies under different incident wave height and reservoir lengths conditions. The test runs used for the variation in incident wave height are DB1, DB2 and DB3; the test runs used for the variation in reservoir length are DB1, DB4 and DB7 (refer to table 5.1).

Figure 5.11 shows that differences in incident wave heights do not significantly affect the hypothetical sedimentation time. However, there is a slight difference in the time interval between the experimental results and the numerical results. The average time difference in the sedimentation time is around 2.45s. This time difference can be attributed to the discrepancy in the maximum run-up height that was recorded for the numerical cases compared to the experimental results. As discussed previously, the value of the maximum

run-up height for the experimental results is greater than the numerical results. This means that for the experimental cases, the trim-line right in front of the cave is higher than the numerical cases and hence the time interval in which the cave is hypothetically submerged is also longer.

However, when observing the variation in the reservoir length with the hypothetical sedimentation time, it can be seen that increasing the reservoir length do increase the sedimentation time. The duration in which the water mass stays in front of the cliff is mainly due to 1) the incoming wave's wavelength and 2) the additional amount of water mass added to the entire system. The longer the wavelength and the greater the amount of water added to the system increases the time during which the water mass stay right in front of the cliff and as a result, the cave would also be submerged for a longer period of time.

Similar to the maximum run-up, an amplification factor of the numerical hypothetical sedimentation time to the experimental hypothetical sedimentation time can also be found. From our results, we found that,

$$\frac{T_{exp}}{T_{num}} \approx 1.5 \quad (5.1)$$

where T is the hypothetical sedimentation time. Using this as reference guideline, we can then correct the numerical hypothetical sedimentation time.

5.4 Scaling to the real scale to estimate the sedimentation time and justification of using dam break waves for this study

Using the Froude scaling laws, we can provide a first estimate of the duration in which the cave in Aceh was filled after the first tsunami wave had impacted the coast (assuming all other parameters are the same). The Froude scaling law for time can be expressed as,

$$T_r = T_p \sqrt{\frac{L_r}{L_p}} \quad (5.2)$$

In which T_r is the real/actual time; and T_p is the laboratory or model time. Here we used the value obtained from the case of DB4 as an illustration. L_r is the length of the actual cave, which is around 200m; and L_p is the laboratory cave length, which is 0.3m. Based on these numbers, the approximate time needed for the real cave to be completely filled is 4.3 mins, recognizing that it requires 10 seconds to fill up the model cave.

The Froude similarity law has often been used for hydraulic and sediment transport studies. For a perfectly undistorted model, equation 5.2 would provide a correct estimate of the sedimentation time. However, due to the constraints of the equipment used in most model studies, including those in this study, the model is distorted. This means that if proper scaling were to be adopted, the ratio of the onshore wave wavelength, water depth, etc. must also have the same ratio as that of the cave dimension. However, this is not possible because the wavelength of the onshore tsunami bore simply was not known and instead, it is a property that the study wishes to obtain. Moreover, the actual topography and the actual water depth during the tsunami at the site also are not known. Consequently, the present approach provides a first estimate of the sedimentation time under the given constraint.

Besides the dam break waves, the duration in which solitary waves fill up the cave was also investigated. However, as the water body carried by the solitary wave into the cave is much smaller, the duration in which the water body remains in the cave is much shorter. Hence we will use the time series of the run-up trim-line to determine the duration of which the water body stays in the cave for the solitary wave cases instead.

From figure 5.12, the duration in which the trim line stays above the cave ceiling is 10s for DB4 whereas the duration in which the trim line stays above the cave ceiling is 1.5s for SW3. Noting that the actual time span in which the cave is completely filled is about 90% that of the initial assumption for DB4, the actual time span in which the cave is completely filled can be estimated to be 1.3s. Using the equation for Froude scaling, the approximate time in which the actual cave would be completely filled if the incoming wave is similar to SW3 would be about 33.6s.

However, it was observed that for the case of DB4, the cave needed 1.5s to be completely filled. Based on the discussion in chapter 3.3.2, SW3, which had a slower onshore wave celerity would need a much longer time to fill the cave up. However, the duration in which the trim line stayed above the cave ceiling was only 1.5s. This means that the cave was not filled. If sediment transport was to be considered, this meant that there would be very little time in which the water mass in the cave remained still. Thus very little sediment could settle. This could imply that it would be less probable to have an onshore wave similar to SW3 in the actual case.

Fig 5.12) Comparison of the surface elevation height measured for experimental cases of DB4 and SW3

Fig 5.13) Comparison of the surface elevation height measured for experimental cases of DB7 and SW4

To further illustrate that the onshore wave forms that occurred were unlike what laboratory scale solitary waves could have generated, another comparison can be made. With reference to figure 5.13, for DB7, the total duration in which the water body stays in front the cave entrance is 13.5s and that for SW4 is 1.7s. However, the actual duration in which the water body stays is only around 1.3s. Taking into consideration the time needed for the cave to be filled up completely, this time span would be insufficient for the cave to be fully filled.

In brief, utilizing solitary waves to understand the flow mechanisms within the cave would not be accurate. Dam break waves may be a better choice as the duration in which the water mass generated by such waves could remain much longer within the cave.

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5.5 Chapter Summary

This chapter incorporates a hypothetical sedimentation time that can be easily integrated into numerical models and used for scenario studies such as the one that we did. The main idea is to track the time-series of the trim-line of the incoming wave and use the information obtained to determine the hypothetical sedimentation time.

The hypothetical sedimentation time can be defined as the maximum allowable time for any suspended sediments in the flow to be deposited. Assuming an ideal scenario whereby the thickness of the deposit is caused by an individual tsunami attack only and the sediment particles are homogenous. We can then determine the concentration of sediments in the flow and the settling velocities of the particles which are based on their dimensions and densities. With these values, the settling time needed to settle out the amount of particles needed to achieve the measured deposit thickness can then be calculated. The next step is

then to approximate this sedimentation time to the actual time needed to fill up the cave which can be estimated to be the period of the tsunami wave. From chapter five in this thesis, we have shown that the actual time needed to fill up the cave is actually longer than the allowed sedimentation time due to the time needed for the water to enter the cave and for it to be drained out. With knowledge of the wave period (T), the tsunami wavelength can then be expressed as,

$$T = \frac{L}{c} \quad (5.3)$$

where L is the tsunami wave length and c the wave celerity. Assuming that wave form onshore is similar to that of a solitary wave, the wave period can be further simplified in terms of the wave height (H) and flow depth (h) into,

$$T = \frac{2\pi\sqrt{\frac{4h^3}{3H}}}{\sqrt{g(h+H)}} \quad (5.4)$$

This method of inversion can only be extended till the onshore region. To characterize the tsunami in further detail, forward simulation have to be carried out from the tsunami source till the location where the parameters of H and h can be obtained.

Another approach of obtaining tsunami parameters from sediment deposits is to have a sediment transport model so as to determine the onshore flow field. Jaffe and Gelfenbaum (2002) mentioned that sediment thickness and grain size distribution is caused by the mode and strength of the sediment transport during the tsunami which in turn is related to the flow velocity. During stagnation, particles in suspension will start to be deposit. Larger particles which have higher settling velocities will settle out first followed by the finer particles. This then means that if the layer of sediment deposit exhibit a upward fining sequence, it is very

likely that the sediments have been suspended in the flow. However, Jaffe et al. (2003) also pointed out that a single layer of sediment deposit does not equate to a single wave event. There could be a series of waves, run-up and backwash within a single wave event. Besides, unsteady flow in either the run-up or backwash could potentially affect the sediment transport and as a result lead to multiple layers of deposit forming.

Experimentally, there are two ways to determine the sedimentation time. The first is the direct approach, whereby the duration in which the water body completely filled the cave is recorded. The indirect approach was the method adopting the trim-line tracking approach. We found that the indirect approach yields a longer duration than the direct approach due the filling up and drawdown stage. However, the time difference is not significant. Expectedly, a longer reservoir would also lead to the hypothetical sedimentation time being longer than one with a shorter reservoir length.

In order to apply the trim-line tracking method to the cave in Banda Aceh which has an opening at the top of the cave near the rear, physical tests were then experimentally conducted with this opening sealed and unsealed. The run-up value does not vary for a fixed set of initial conditions. However, the flow conditions within the cave do differ. When the hole is sealed, air pressure serves to retard the onshore motion. Hence, it takes longer for the cave to be filled under such scenarios.

Potentially, the hypothetical sedimentation time approach can be utilized in 2-D NLSWEs models. We compared the hypothetical sedimentation through both experimental and numerical means. The difference which is obvious and consistent can be attributed to the difference in the run-up produced by both methods.

Chapter 6

Conclusions and Recommendations

6.1 Conclusions

This thesis discusses several key issues. The first is the need for an amplification factor to supplement most NLSWEs based numerical models especially if the region of interest is the onshore region. The concept of the tsunami amplification factor, which has been used in previous studies, was studied experimentally in a wave flume using solitary waves; the results are compared with those obtained from the Delft3D-FLOW software package. Parameters that were studied included the slope angle of a coastal cliff and the onshore distance effects on the onshore flow depth amplification. We found that closer to the onshore cliff, the amplification was stronger. A steeper cliff also resulted in a greater amplification compared with a gentler one. With regards to the onshore distance, the results reveal that the distance from the shoreline to the cliff angle does not affect the run-up amplification factor. Moreover, the amplification factors reported in this study can principally be used for a different reference location by a simple shoaling calculation.

The second point that was investigated relates to the use of dam break flows to understand tsunami run-up. Laboratory generated dam break waves are found to induce flows with much higher velocity and hence are able to reach greater onshore heights compared to solitary waves. The influence on the height and length of the reservoir was also studied. We found that a reservoir with a higher initial depth will result in a greater run-up but a reservoir with an initial longer length will not. Instead, the longer initial reservoir length will make the water body remain in front of the cliff for a longer duration. This phenomenon was not observed during the solitary wave tests as solitary waves are much shorter. Based on the results a formula is obtained for dam break waves which may be used to compute projected real-life tsunami run-up.

The main reason why the dam break waves were chosen as the initial wave form is discussed. Due to the evidence of sediments found in a cave in Banda Aceh which have been proven to be deposits from the 2004 Indian Ocean tsunami event, one may reasonably surmise that the sediments that were deposited must have been brought in by waves that inundated the area. Moreover, the waves have to be long enough so that 1) the cave will be fully submerged and 2) the water that filled the cave must have remained quiescent over a certain time interval, allowing ample time for sedimentation to occur. Using these constraints, physical tests on solitary waves were not realistic as the water that flows into the cave does not stay for a long enough time. Under similar test conditions, applying dam break waves resulted in the submergence and filling up of the cave for a significantly longer time period.

Due to the fact that 1) no sediments were used and 2) the incapability of the Delft3D-FLOW software to simulate the cave interior in the numerical domain, a hypothesis to evaluate the sedimentation time was proposed. The hypothetical sedimentation time is the time interval at which the trim line first goes over the cave entrance ceiling to the time at which the trim line falls below the cave entrance ceiling. Using this approach, the physical and numerical hypothetical sedimentation time is obtained.

To apply the key points from this study to the case of Banda Aceh, we can first propose a run-up amplification factor based on the numerical wave height obtained from a location at shallow water depth. However, this amplification factor is affected by the location's topography. Replicating the area's topography into laboratory settings could be a way to determine the amplification factor. If the amplification factor is known, the actual tsunami run-up can then be calculated from the numerical simulation run-up height. After the run-

up is obtained, the trim line time series could be determined. Using the trim line time series, the hypothetical sedimentation time can also be obtained.

6.2 Recommendations

Moving forward, more tests should be conducted for the amplification factor study. For a start, more variations of the beach topography should be tested. It would also be better if a wider spectrum of incident wave heights were tested. For the dam break waves tests, it would be better if wider wave basins were used instead. An electronic mobile gate release would also be a vast improvement from the one used in the current study.

Another area of research interest is to have a methods to utilize both wave forms mentioned. Solitary waves have often been proclaimed to have their wave fronts similar to tsunami waves. This is mostly valid in the offshore region. Onshore, there are others who feel that the surge created by dam break waves are similar to that of the tsunami waves. Having a way to combine these two different wave forms would possibly be a much better representation of a tsunami's propagation from its offshore source to the onshore region.

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Appendix

Appendix

Fig i) Variation of physical amplification factors with cliff slope and incident wave height at different onshore locations. (a) $x = 0.0$ cm; (b) $x = 6.0$ cm; (c) $x = 11.0$ cm; (d) $x = 13.0$ cm; (e) $x = 16.0$ cm; (f) $x = 21.0$ cm (g) run-up. The four wave heights are 2.5 cm (circles), 3.5 cm (crosses), 4.5 cm (asterisks), and 5.5 cm (squares).

Fig ii) Comparison of experimental and numerical results for test case SW3

Fig iii) Comparison of experimental and numerical results for test case SW4

Fig iv) Comparison of experimental and numerical results for test case SW5

Fig v) Time series for test case DB1 and SW2

Fig vi) Time series for test case DB2 and SW5

Fig vii) Time series for test case DB4 and SW3

Fig viii) Comparison of experimental and numerical results for test case DB1

Fig ix) Comparison of experimental and numerical results for test case DB3

Fig x) Comparison of experimental and numerical results for test case DB4