LATERAL LOADING ON VERTICAL STRUCTURAL ELEMENTS DUE TO A TSUNAMI BORE

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and

Ian N. Robertson

Research Report UHM/CEE/10-02

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ABSTRACT

In order to investigate forces on coastal structures caused by tsunamis, numerous experiments were carried out. The focus of this study was the loading induced by a tsunami bore on vertical structural elements such as columns and walls. Experiments were performed at the Oregon State University O.H. Hinsdale Wave Research Laboratory in two phases. The first series of tests were carried out in the Tsunami Wave Basin, and the second series were performed in the Large Wave Flume which was at least double the scale of the first phase. In the first phase, tests were performed on individual columns, multiple columns, a solid wall, and symmetric and offset shielded columns. Various column sizes and different wave heights in combination with different downstream water levels were investigated. The second phase of tests were performed in the Large Wave Flume and focused on bore loading on a solid wall. A combination of wave heights and water levels were tested.

To analyze the effect of having multiple columns versus a single column in the flow path, the forces on the middle column of the multiple column configuration were compared to the forces on an individual column. It was found that for multiple column configurations that blocked the flow at least 40%, the force on the column increased by about 25%. Similarly, to investigate column symmetric and offset shielding effects, the forces on the shielded column were compared to the forces on the individual column. It was found that symmetric shielded columns generally experienced a decrease in force, but for offset shielded columns, forces generally increased. In order to determine the percentage increase, more research is required.

For analysis of forces on the solid wall, previously developed equations and newly developed approaches from this study were tested and compared to the experimental forces. It was found that the newly developed approaches known in this study as “Momentum approaches” were the most effective in estimating experimental forces for both small and large scale tests. These formulas require bore height and bore velocity. If the bore height is determined from tsunami modeling, the bore velocity can be estimated from hydraulic jump theory and used in the formulas.
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1 INTRODUCTION

In recent years, natural disasters have resulted in numerous fatalities and enormous economic consequences. Tsunamis are one type of natural phenomenon that have earned a great amount of attention within the past several years due to one of the deadliest natural disasters in history, the 2004 Indian Ocean Tsunami. This tsunami was triggered by a massive undersea earthquake off the coast of the island of Sumatra (CAEE, 2005). The earthquake had a magnitude of 9.0 on the Richter scale according to U.S. Geological Survey, one of the strongest earthquakes recorded within the past century (CAEE, 2005). The tsunami waves generated by the devastating quake took over 230,000 lives and affected a large number of countries including Indonesia, Sri Lanka, India, Thailand, Somalia, Myanmar, Maldives, Malaysia, and several others. Another more recent tsunami occurred on September 29, 2009, affecting the islands of Samoa. It was caused by the largest earthquake in 2009, a magnitude 8.0 south of Samoa, resulting in 34 deaths in American Samoa and 130 deaths in Western Samoa (U.S. Geological Survey, 2009). The most recent tsunami was triggered by an 8.8 magnitude earthquake just west of Chile (U.S. Geological Survey, 2010). Many people living in the small coastal towns lost their lives, and many buildings were destroyed from tsunamis up to 30 feet high (National Geographic, 2010). The towns were not prepared for the giant waves. These recent tsunamis are a grave reminder of the reality and importance of knowledge and preparation regarding these natural disasters.

All three of these tsunamis were generated by subduction zone earthquakes. When two convergent tectonic plates suddenly slip and displace large amounts of sea water, waves are generated with wavelengths hundreds of kilometers long in deep ocean that travel at speeds faster than 800 kilometers per hour. A tsunami in deep ocean has an amplitude less than 1 meter so is not easily detectable. When the tsunami approaches shallow waters, due to wave shoaling, its speed and wavelength decrease drastically while its amplitude increases resulting in a very large wave. This wave will normally break offshore and come on land as a broken bore. Depending on the coastal topography, the bore can travel many kilometers inland due to the enormous amount of energy, causing
mass destruction of vegetation, damage to structures, and injury and death to humans or animals caught in its path.

In structural engineering, tsunami loads are comparable to seismic and wind loads. These loads are infrequent but can lead to disaster if a structure is not adequately designed for such events. Somewhat different from seismic and wind loads, tsunamis will usually have the most impact on the ground floor of a structure. In past investigations of structural damage caused by tsunamis, failures often occurred in ground floor walls, columns, and first floor slabs. It is crucial that these elements, on the ground floor of a structure in particular, are capable of withstanding a tsunami bore for the safety of the remaining structure.

Currently, the International Building Code does not address tsunami loads. The recently published FEMA P646 (Applied Technology Council, 2008) guidelines were based on experiments that may not be applicable to tsunami bore loading. A number of research projects have focused on determining forces due to tsunami bores impacting structures. However, most of these laboratory experiments were performed on small scale specimens. Until the effects of tsunamis on structures are better understood, escaping to high ground remains the safest evacuation method, and the design of structures for vertical evacuation is still uncertain.

The focus of this research is to develop a more accurate understanding of tsunami bore interaction with structural vertical elements and formulate design equations in hopes of improving current building codes.
2 LITERATURE REVIEW

Ramsden and Raichlen (1990) researched the effects of incident bores on a vertical wall using a tilted tank with a constant bottom slope up to the wall. Bore heights ranged from 2.4 to 4.9 centimeters. Ramsden et al. (1990) used an equation that incorporated both the hydrostatic and hydrodynamic forces and were able to match the experimental force values within 5% for the 4 largest bores tested. Fujima et al. (2009) investigated tsunami forces on rectangular structures using a 7 meter wide, 11 meter long, and 1.5 meter deep wave flume. The structures of interest were 10 and 20 cm wide by 10 cm long by 10 cm high, and the distance from the water varied from 20 to 150 cm. A piston-type wave paddle was used to create waves, and a beach slope of 1:3 followed by a flat run was used to simulate a beach and create a breaking bore which impinged on the structure. Fujima et al. (2009) found that hydrodynamic forms of equations estimated the experimental findings fairly well for all cases while hydrostatic forms were found to perform poorly for certain cases. Asakura et al. (2000) also investigated forces due to tsunamis on on-shore structures. Tests were performed using solitary waves and a dry reef where the structure stood. An equation in hydrostatic form was developed relating the force on a structure to the bore height. The estimated force on a structure is given by:

\[ F = \frac{1}{2} (3h_b)^2 \rho b \]  

(1)

where, \( h_b \) is the height of the bore, \( \rho \) is the density of water, and \( b \) is the width of the wall or structure. The equation assumes the force on the structure is hydrostatic and does not take into account any dynamic effects of the wave. Only the bore height is needed to estimate the force. Arnason (2005) investigated the forces of a bore on different shape columns. By normalizing the force time-history plots for all size bores by the traditional drag coefficient formulation, it was found that for square columns the coefficient was around 2.1 plus or minus 0.1 which was slightly higher than that reported in standard hydraulics texts. Wybro (1976) researched both the case of a bore and a non-breaking wave hitting a long wall. For the bore problem, he used the Cross (1966) formula for the force on a wall and modified it for a flat bore. By conservation of momentum, he was
able to find the ratio between total height of water at the wall and total bore depth, and by making the substitution into the Cross equation, he found the force on the wall to be a combination of momentum and hydrostatic forces. The height and velocity of the bore are needed to estimate the force on a wall. The total force on a wall is given by:

\[ F = \frac{1}{2} \rho gh_b^2 b \left( 1 + 2 \left( \frac{u^2}{gh_b} \right) \right) \]  

where, \( g \) is the gravitational acceleration, and \( u \) is the velocity of the bore.
3 EXPERIMENTS AND SETUP

Looking back on previous work done, the experiments were performed on relatively small scales, and specific research on determining the forces due to a tsunami bore on vertical elements is scarce. The research reported here has greatly increased the experiment scale and focused specifically on loading on vertical structural elements versus structures as a whole. For the purpose of understanding and developing design loads due to a tsunami bore on these elements, mainly columns and walls, two separate sets of experiments consisting of different waves heights and different vertical structural configurations were carried out at the O.H. Hinsdale Wave Research Laboratory of Oregon State University (OSU). The first set of tests was performed in the Tsunami Wave Basin (TWB) in Fall 2007, and the second set was performed in the Large Wave Flume (LWF) in Summer 2009.

3.1 OSU PHASE I TESTING

For the first set of experiments, two flumes were constructed within the TWB by building concrete masonry walls. These flumes were 2.133 m (7 ft) wide by 48.8 m (160 ft) long with built-up run-up reefs consisting of constant slope reefs or fringing reefs with varying beach slopes.

Figure 1: OSU Tsunami Wave Basin flumes

Structural loading tests on vertical columns and a wall were performed in the flume with a 1:5 beach slope and fringing reef.
Figure 2: OSU phase I wave flume configuration and dimensions (1m = 3.28 ft)

The beach slope terminated at 1 m above the tank bottom. The test specimens were placed along the flat portion of the flume past the fringing reef, 35 m (115 ft) from the piston-type wave maker. Different types of waves could be created by the wave maker, but for the structural loading tests, a soliton was used to create a tsunami bore. Once the soliton reached the transition zone between beach slope and fringing reef, it would become a turbulent bore which loads the structural specimen. Immediately before the bore reached the specimen, a laser surface profiler recorded the bore profile, and bore characteristics were analyzed. In-depth research on bore characteristics was performed by Mohamed (2008). Each test specimen was supported by two load cells that supported the specimen which were braced by a support post with two horizontal braces extending in a “V” shape back to the flume sidewalls. The top load cell was a single degree of freedom sensor, measuring only axial force. The lower load cell was a 6 degree of freedom sensor which measured force and moments in all three directions. Load cell readings were recorded at 1000 Hz. For each trial, the time-history of the forces and moments were recorded. A combination of different wave heights and downstream water levels were used for each vertical structural specimen configuration. Three wave heights of 20, 40, and 60 cm were considered, and two main downstream water levels of 0 and 10 cm were used. A water level of 5 cm was used for certain configurations.

3.1.1 Individual Columns

The vertical structural specimens tested in this series consisted of individual columns of the following sizes:

1) 50 mm square (2”x2”)
2) 100 mm square (4”x4”)
3) 150 mm x 50 mm rectangle (6”x2”)

4) 300 mm x 50 mm rectangle (12”x2”)

Rectangular columns were positioned with the wide face normal to the incoming bore. Figure 4 shows how these column specimens were supported by two load cells attached to a rigid frame spanning across the top of the flume. The load cells were recorded at 1000 Hz to provide a time history record of the horizontal force applied to the column.

Figure 3: Column specimen configuration

Figure 4: Individual columns (Left – 50x50mm, Center – 150x50mm, Right – 300x50mm)

3.1.2 Multiple Columns (Perforated Wall)

To simulate a typical building ground floor, configurations consisting of three of the same size column placed side-by-side across the width of the flume at 711 mm (28 in) on center were tested. This multiple column, or perforated wall, configuration consisted of
the same four column sizes considered as individual columns. In each experiment, the center column was supported by the same rigid frame and dual load cells used for the individual columns. The other two columns were rigidly secured to a steel frame spanning the top of the flume.

Figure 5: Plan view of multiple column arrangement in flume

Figure 6: Multiple columns (Left – (3) 50x50mm columns, Right – (3) 300x50mm columns)
3.1.3 Solid Wall

A solid wall which completely covered the full width of the flume was also tested for the dry reef case and standing water of 10 cm. Figure 8 shows the back of the wall viewed from the end of the flume. The wall was constructed using three sections, each 711 mm (28 in) wide by 610 mm (24 in) high. Placed side by side, the three wall sections covered the full flume width of 2133 mm (7 ft).

Figure 7: OSU phase I plan view of solid wall in flume and wall specimen configuration

Figure 8: OSU Phase I solid wall setup from behind
### 3.1.4 Shielding

In order to evaluate the effect of shielding provided by the front line of building columns, a number of symmetric and offset shielding experiments were performed.

#### 3.1.4.1 Symmetric Shielding

As shown in Figure 9, it is assumed that the tsunami bore hits exactly perpendicular to the building face in order to simulate maximum shielding of the back column.

![Figure 9: Symmetric shielding in multi-story reinforced concrete building](image)

Symmetric shielding configurations:

- 3 – 50x50mm (2x2 inch) columns shielding individual:
  - a. 50x50mm (2x2 inch)  (Figure 10 and Figure 12)
  - b. 150x50mm (6x2 inch)
  - c. 300x50mm (12x2 inch) column.

- 3 – 150x50mm (6x2 inch) columns shielding individual:
  - a. 50x50mm (2x2 inch)  (Figure 11)
  - b. 150x50mm (6x2 inch)
  - c. 300x50mm (12x2 inch) column.
Figure 10: Plan view of (3) 50x50mm columns shielding individual 50x50mm column

Figure 11: Plan view of (3) 150x50mm columns shielding individual 50x50mm column
3.1.4.2 Offset Shielding

A case more likely to occur than symmetric shielding is offset shielding. Figure 13 shows the case where the front columns do not directly shield the interior wall. Another applicable case is when the tsunami bore strikes the building at an angle, and the front columns do not directly shield the interior column.
Offset shielding configurations:

- 2 – 50x50mm (2x2 inch) columns offset shielding individual:
  a. 50x50mm (2x2 inch) (Figure 14 and Figure 15)
  b. 150x50mm (6x2 inch) column (Figure 16)

- 2 – 150x50mm (6x2 inch) columns offset shielding individual:
  a. 50x50mm (2x2 inch)
  b. 150x50mm (6x2 inch)

Figure 14: Plan view of (2) 50x50mm columns offset shielding individual 50x50mm column
3.2 OSU PHASE II TESTING

The second set of experiments was performed in the Large Wave Flume (LWF) at OSU. During these tests, the focus was on obtaining more information concerning forces on a solid wall. The flume utilized a similar set-up to the Phase I testing consisting of a sloping beach followed by a flat reef with a wall installed at some distance from the wave breaking point shown below in Figure 17.

![Figure 17: OSU phase II wave flume configuration and dimensions (1m = 3.28 ft)](image-url)
The wave maker was a piston driven panel. The wall that was installed was the width of the flume, 3.66 m (12 ft), by 1.83 m (6 ft) tall. Four load cells were installed behind the wall, with one load cell close to each corner of the wall. Thirteen FPG pressure sensors and four Druck gauges were installed along the vertical center line of the wall.

The druck gauges were installed for the purpose of validating the FPG gauges readings. All load cells and pressure sensors were recorded at 1000 Hz.
The different wave heights used in the LWF were determined based on chosen ratios of wave height (above standing water) to standing water height (from the bottom of the tank). Downstream water levels on the reef were chosen to be 0, 5, 10, 20, and 30 cm (or 2.36, 2.41, 2.46, 2.56, and 2.66 m with respect to the bottom of tank). The selected wave ratios were 0.1, 0.2, 0.3, 0.4, and 0.5. Secondary wave ratios were 0.05, 0.15, 0.25, and 0.45. Computed wave heights for the different downstream water levels were as follows.

Table 1: Soliton wave heights in cm

<table>
<thead>
<tr>
<th>Water Level on Reef</th>
<th>Water depth in flume</th>
<th>Wave height to Water height Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>0.05</td>
</tr>
<tr>
<td>0 cm</td>
<td>2.36 m</td>
<td>11.8</td>
</tr>
<tr>
<td>5 cm</td>
<td>2.41 m</td>
<td>12.05</td>
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<tr>
<td>10 cm</td>
<td>2.46 m</td>
<td>12.3</td>
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<tr>
<td>20 cm</td>
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<td>12.8</td>
</tr>
<tr>
<td>30 cm</td>
<td>2.66 m</td>
<td>13.3</td>
</tr>
</tbody>
</table>
4 RESULTS AND ANALYSIS

4.1 INDIVIDUAL COLUMNS

The first set of tests performed during Phase I at OSU was on the individual columns. The time-history of the hydrodynamic load on the columns were plotted. A typical plot for the dry bed case is shown in Figure 20 and for the wet bed case in Figure 21 and Figure 22.

![Dry Bed: Resultant Force vs. Time](image)

**Figure 20:** Force time history for 300x50mm individual column on dry bed
The peak forces are of interest for design and are used for comparison with the multiple columns and shielded columns in subsequent sections of this report.

4.2 MULTIPLE COLUMNS (PERFORATED WALL)

The OSU phase I tests on multiple columns were performed to investigate the effect of adjacent columns on the hydrodynamic load on an individual column in a row of columns blocking the flow of a bore. This case is common in many buildings. In the
The experimental setup, the center column was instrumented with load cells, and two identical columns were placed on each side, spaced at 711 mm center-to-center distance, as shown in Figure 6. The wider side of the columns was faced towards the flow.

### 4.2.1 Results of Multiple Columns

The peak force on the center column was compared to the peak force for the individual column case in order to see the effect of the adjacent columns. Figure 23 and Figure 24 show example plots of the individual and multiple column force time histories for the dry reef condition with the peak forces indicated by a dot.

![Dry Reef: Resultant Force vs. Time](image)

**Figure 23:** Individual 50x50mm column force time history for two 60cm waves on dry bed
In order to compare the forces on the individual column to the forces on the center column in the multiple column arrangement, the average peak forces are compared and plotted as a column force ratio below in Figure 25 and Figure 28. The column force ratio is the average peak force on the center column of the multiple column arrangement divided by the average peak force for the identical size single column setup. The ratio is plotted against the flume percentage closure, which is the total width of the three columns divided by the total width of the flume. Three 50 mm wide columns had a 7% closure, three 100 mm columns had a 14% closure, three 150 mm columns had a 21% closure, and three 300 mm columns had a 42% closure.
Similarly, for the wet reef condition, example plots of the individual and multiple column forces are shown below in Figure 26 and Figure 27.

Figure 26: Individual 300x50mm column force time history for two 40cm waves with 10cm standing water
Water 10cm: Resultant Force vs. Time

- Average peak force = 252.5 N
- Column force ratio = 252.5 / 203 = 1.24

Figure 27: Multiple 300x50mm column force time history for two 40cm waves with 10cm standing water

Figure 28: Effect of multiple columns for wet reef condition
4.2.2 Analysis of Multiple Columns

The peak forces showed significant scatter. The effect of multiple columns was not always consistent. The 100 mm square column showed a slight reduction in the peak force with multiple columns, whereas for the other sized columns the force typically increased by an average of around 15%. Forces were generally larger particularly for the largest column, both at the peak force and shortly after. The force on the largest column due to the 60 cm wave on the dry reef increased by about 30% due to the multiple column effect. For the wet reef case with 10 cm of standing water, the force on the largest column due to the 40 cm and 60 cm waves increased by about 25%. The increase in force was due to the increase in closure of the flume due to the adjacent columns. However, the force on the smallest column due to the largest wave also increased by about 25% for the dry reef case. Due to the limited amount of data collected and contrasting results, it was difficult to draw any firm conclusions. However, it is recommended that the effect of multiple columns be considered to increase the peak hydrodynamic force by 25% for cases with percentage closure greater than 40%. More experiments with different closure percentages are necessary to understand the effect better.

4.3 COLUMN SHIELDING

4.3.1 Column Symmetric Shielding

The configurations consisted of three 50 mm or 150 mm wide by 50 mm columns shielding a single 50, 150, or 300 mm wide by 50 mm columns. Three 50 mm wide columns had a 7% closure and 21% for three 150 mm columns. Similar to the multiple column analysis, the force on a symmetrically shielded column was compared to the force on the same individual column. Figure 29 to Figure 31 show example plots of the force time histories for each case with the peak forces indicated by a dot.
Figure 29: Individual 150x50mm column force time history for two 60cm waves with 10cm standing water

Figure 30: 150x50mm column symmetrically shielded by (3) 50x50mm columns force time history for two 60cm waves with 10cm standing water
Figure 31: 150x50mm column symmetrically shielded by (3) 150x50mm columns force time history for two 60cm waves on 10cm standing water

Similar to the multiple column analysis, the column force ratio was plotted for the comparison of the force on the individual column and the force on the column shielded by either (3) 50x50mm columns or (3) 150x50mm columns. Figure 32 to Figure 34 show the effect of symmetric shielding.

Figure 32: Effect of symmetric shielding on 300x50mm interior column
Figure 33: Effect of symmetric shielding on 150x50mm interior column

Figure 34: Effect of symmetric shielding on 50x50mm interior column
4.3.2 Column Offset Shielding

Column offset shielding effects were investigated by placing two 50x50mm or 150x50mm columns at 711 mm (28 in) on center in front of one 50x50mm or 150x50mm column centered behind (Figure 14).

The typical force time history plots for a column offset shielded by either (2) 50x50mm columns or (2) 150x50mm columns is shown below in Figure 35 and Figure 36. The plot for the individual column is the same as in Figure 29.

![Water 10cm: Resultant Force vs. Time](image)

Figure 35: 150x50mm column offset shielded by (2) 50x50mm columns force time history for two 60cm waves on 10cm standing water
Figure 36: 150x50mm column offset shielded by (2) 150x50mm columns force time history for two 60cm waves on 10cm standing water

Same as for symmetric shielding, the column force ratios are plotted below in Figure 37 and Figure 38.

Figure 37: Effect of offset shielding on 150x50mm column
4.3.3 Shielding Analysis

It was shown in Figure 32 to Figure 34 that symmetric shielding can reduce the peak force by about 10% to 20% when the shielded column is smaller than or equal to the front columns. When the front columns are smaller than the shielded column, the reduction of peak force becomes on average less than 10%. This reduction of peak force only applies when the bore hits exactly perpendicular to the front columns, and the shielded column is directly behind a front column.

Figure 37 and Figure 38 show that offset shielding tends to focus the flow on the back column instead of shield it, resulting in a fairly significant increase in the peak force. The increase in force is shown to be greater when the offset front columns are smaller due to less flow blockage. The force also tends to increase slightly when there is a larger column at the back. With the 50x50mm columns at the front, the peak force increased up to 25%. With the larger 150x50mm columns at the front, the peak force increased by only 10%.

Due to the possibility of tsunami bore impact at an oblique angle, the effect on the interior columns is uncertain whether it is positive or negative. Because the case of
symmetric shielding is an ideal situation, it is not recommended to take any reduction in force. Instead, it is recommended that the force be increased by some percentage due to the likely possibility of offset shielding. More tests need to be performed before this percentage can be more precisely estimated.

4.4 SOLID WALL

The last scenario investigated was the force of the tsunami bore on a solid wall. A building’s walls would be the first element hit by a tsunami bore. Therefore, exterior walls facing the coast should be designed for the tsunami force. In the tests, the wall spanned the width of the flume, perpendicular to the flow.

4.4.1 OSU Phase I Solid Wall Results

The same three wave heights of 20, 40, and 60 cm were run. However, the wall was not tall enough to catch all the run-up from the 60 cm wave; therefore water splashed over the top of the wall, and the peak force was missed. It was found by synching the video of the wave hitting the wall and the force time-history that the peak force occurred when the jet of water that ran up the wall collapsed down on top itself. Therefore, since the 60 cm wave trials missed the peak force due to loss of water over the wall, they were omitted from results. By losing the 60 cm wave trials, only a total of four trials remained for the solid wall results. Because of this problem, for the OSU Phase II testing, more combinations of wave heights and downstream water levels were run.

The force time history for Phase I is shown in Figure 39 and Figure 40. Two force estimation equations were developed, and equations from previous work were tested. Formulations by Asakura et al. (2000) and Wybro (1976) were investigated.
Figure 39: OSU Phase I solid wall force-time history for dry reef

Figure 40: OSU phase I solid wall force-time history for wet reef

Figure 41 shows the notation used for the bore in the force estimation equations. In the figure, $u$ is the velocity of the bore, $L$ is the length of the leading edge of the bore, $h_s$ is the height of the standing water, $h_j$ is the height of the jump or the bore height above the standing water, and $h_b$ is the total height of the bore.
4.4.1.1 Asakura Formulation

The equation by Asakura et al. (2000), shown below in Equation (3), was compared to the measured forces. The plot showing predicted force vs. experimental force using this equation is shown in Figure 42. As stated earlier, this equation is based only on the height of the bore.

\[ F = \frac{1}{2} (3h_b)^2 \rho b \]  

(3)
4.4.1.2 Bore Velocity

The remaining formulations include a hydrodynamic term; therefore the velocity of the bore is required. The velocity of the bore could either be measured or estimated theoretically with a given bore height. The bore velocity was measured by analyzing the high-speed video captures. From hydraulic theory, the theoretical bore velocity could be calculated by Equation (4) for the dry reef condition and by Equation (5) for the wet reef (Mohamed, 2008).

\[ u = \sqrt{gh_b \left[ \frac{1}{2KC_D} - 1 \right]} \] (4)

\[ KC_D = 0.05 \]

The velocity for the dry reef condition required a frictional coefficient, \( KC_D \) which was determined specifically for the OSU TWB flume.

\[ u = \sqrt{gh_s \left[ \frac{1}{2} \left( \frac{h_b}{h_s} \right)^2 + \frac{1}{2} \left( \frac{h_b}{h_s} \right) \right]} \] (5)

Both measured and theoretical velocities were used in the formulations and compared. The advantage to being able to use theoretical velocities is that it only requires the bore height to be estimated.

4.4.1.3 Wybro Formulation

Wybro’s (1976) formulation is shown in Equation (6), and the plot of predicted force vs. experimental force is shown below in Figure 43. His formulation was developed under a dry reef condition.

\[ F = \frac{1}{2} \rho gh_b^2 b \left( 1 + 2 \left( \frac{u}{gh_b} \right) \right) \] (6)
Figure 43: Comparison of OSU phase I solid wall experimental forces to Wybro predicted forces using measured bore velocities

Figure 44: Comparison of OSU phase I solid wall experimental forces to Wybro predicted forces using theoretical bore velocities
The following two approaches were developed in a similar way to Wybro’s formulation. Both approaches include a hydrodynamic and hydrostatic term, but the derivations differ. The hydrodynamic term is the force contributed due to the momentum of the bore. These approaches will be referred to as “Momentum approach #1” and “Momentum approach #2.”

4.4.1.4 Momentum Approach #1

Momentum approach #1 was developed by Paczkowski (2010) using the conservation of mass to set flow in equal to flow out. The additional notation used in the derivation is shown in Figure 45. In the figure, \( v_r \) is the velocity of the reverse flow, \( h_r \) is the height of the reverse flow, and \( h_p \) is known as the ponding height.

![Diagram of Momentum Approach #1](image)

Figure 45: Momentum approach #1 notation (courtesy of Krystian Paczkowski, 2010)

The standing water velocity is expressed as a percentage of the bore velocity:

\[
v_s = \alpha u
\]  

(7)
The reverse flow velocity is assumed to have a Froude number, \( \alpha_0 \):

\[
v_r = \alpha_0 \sqrt{gh_r}
\]  

(8)

By writing the conservation of mass for flow in being equal to flow out, the reverse flow height can be solved for and expressed by:

\[
h_r = \left[ \frac{(\alpha_r h_r + h_j)u}{\alpha_0 \sqrt{g}} \right]^{2/3}
\]  

(9)

The total force can be expressed in terms of a momentum term and a hydrostatic term as follows:

\[
F = (\rho bh_j u^2 + \rho bh_j v_j^2) + \frac{1}{2} \rho g h_p^2
\]  

(10)

The coefficients \( \alpha_0 \) and \( \alpha_i \) were fit to the OSU phase II solid wall data to obtain the best results. From this, it is assumed \( \alpha_0 = 2 \) and \( \alpha_i = 0 \). Therefore, the force equation becomes:

\[
F = \rho bh_j u^2 + \frac{1}{2} \rho g h_p^2
\]  

(11)

where

\[
h_p = h_r + h_b = \left[ \frac{h_j u}{2 \sqrt{g}} \right]^{2/3} + h_b
\]  

(12)

This formulation assumes that the standing water is not moving, and only the jump contributes to the momentum force. Then it assumes that the full height of water backed up contributes to the hydrostatic portion. The results for this approach are shown below in Figure 46 and Figure 47.
4.4.1.5 Momentum Approach #2

The last approach investigated was Momentum approach #2. This second approach results in a formula very similar to the first. Instead of solving for a ponding height by
conservation of mass, a returning jump height, \( y_2 \), is solved for by hydraulic jump theory and expressed as:

\[
y_2 = \frac{y_1}{2} \left( \frac{1 + 8 \frac{u^2}{gy_1}}{\sqrt{1 + 8 \frac{u^2}{gy_1}} - 1} \right)
\]  

(13)

where, \( y_1 = h_b \). Then the hydrostatic portion of the force is assumed to be due to the height \( y_2 - h_s \). The hydrodynamic portion is assumed to be due to the momentum force of the total bore height, \( h_b \). It is assumed that the standing water is accelerated to the same speed as the moving jump. Therefore, the force can be expressed by:

\[
F = \rho bh_b u^2 + \frac{1}{2} \rho gb(y_2 - h_1)^2
\]  

(14)

The results for this approach are shown below in Figure 48 and Figure 49.

---

**Figure 48:** Comparison of OSU phase I solid wall experimental forces to Momentum approach #2 predicted forces using measured bore velocities
4.4.2 OSU Phase II Solid Wall Results

Due to the limited amount of data collected in Phase I, it was decided that many more combinations of wave heights and standing water would be tested at the OSU LWF during Phase II. In contrast with Phase I, the wall height was extended to prevent overtopping. In Phase I, pressure sensor readings were unreliable because the FPG sensors that were originally out of water were affected by the drop in temperature when submerged. For Phase II testing these sensors were protected from thermal shock using high density grease and insulation tape. More reliable Druck pressure gages were also used to confirm the FPG sensor readings. All load cell and pressure sensor readings were monitored at 1000 Hz during each wave run.

The force-time histories for the various standing water heights and waves are shown in Figure 50 through Figure 59. Similar to Phase I, the four different formulations for estimating the peak force on the wall were investigated.
Dry Bed: Resultant Force vs. Time

Figure 50: OSU phase II solid wall force-time history for 23.6cm wave (ratio of 0.1) on dry reef

Dry Bed: Resultant Force vs. Time

Figure 51: OSU phase II solid wall force-time history for 118cm wave (ratio of 0.5) on dry reef
Figure 52: OSU phase II solid wall force-time history for 24.1 cm wave (ratio of 0.1) on 5 cm water

Figure 53: OSU phase II solid wall force-time history for 120.5 cm wave (ratio of 0.5) on 5 cm water
Figure 54: OSU phase II solid wall force-time history for 24.6cm wave (ratio of 0.1) on 10cm water

Figure 55: OSU phase II solid wall force-time history for 123cm wave (ratio of 0.5) on 10cm water
Figure 56: OSU phase II solid wall force-time history for 25.6cm wave (ratio of 0.1) on 20cm water

Figure 57: OSU phase II solid wall force-time history for 128cm wave (ratio of 0.5) on 20cm water
Figure 58: OSU phase II solid wall force-time history for 53.2cm wave (ratio of 0.2) on 30cm water

Figure 59: OSU phase II solid wall force-time history for 119.7cm wave (ratio of 0.45) on 30cm water
The results for the Asakura et al. formulation are shown below in Figure 60 through Figure 64.

**Figure 60:** Comparison of OSU phase II solid wall experimental forces to Asakura et al. predicted forces (dry reef)

**Figure 61:** Comparison of OSU phase II solid wall experimental forces to Asakura et al. predicted forces (5 cm water on reef)
Figure 62: Comparison of OSU phase II solid wall experimental forces to Asakura et al. predicted forces (10 cm water on reef)

Figure 63: Comparison of OSU phase II solid wall experimental forces to Asakura et al. predicted forces (20 cm water on reef)
Figure 64: Comparison of OSU phase II solid wall experimental forces to Asakura et al. predicted forces (30 cm water on reef)

For the remaining formulations, the dry reef condition is not shown because more research is required to better understand the hydraulics of the bore in this situation. The results using Wybro’s formulation for measured and theoretical velocities are shown in Figure 65 to Figure 72.
Figure 65: Comparison of OSU phase II solid wall experimental forces to Wybro predicted forces using measured velocities (5cm water on reef)

Figure 66: Comparison of OSU phase II solid wall experimental forces to Wybro predicted forces using theoretical velocities (5cm water on reef)
Figure 67: Comparison of OSU phase II solid wall experimental forces to Wybro predicted forces using measured velocities (10cm water on reef)

Figure 68: Comparison of OSU phase II solid wall experimental forces to Wybro predicted forces using theoretical velocities (10cm water on reef)
Figure 69: Comparison of OSU phase II solid wall experimental forces to Wybro predicted forces using measured velocities (20cm water on reef)

Figure 70: Comparison of OSU phase II solid wall experimental forces to Wybro predicted forces using theoretical velocities (20cm water on reef)
Figure 71: Comparison of OSU phase II solid wall experimental forces to Wybro predicted forces using measured velocities (30cm water on reef)

Figure 72: Comparison of OSU phase II solid wall experimental forces to Wybro predicted forces using theoretical velocities (30cm water on reef)
The results using Momentum approach #1 are shown in Figure 73 to Figure 80.

**Figure 73:** Comparison of OSU phase II solid wall experimental forces to Momentum approach #1 predicted forces using measured velocities (5cm water on reef)

**Figure 74:** Comparison of OSU phase II solid wall experimental forces to Momentum approach #1 predicted forces using theoretical velocities (5cm water on reef)
Figure 75: Comparison of OSU phase II solid wall experimental forces to Momentum approach #1 predicted forces using measured velocities (10cm water on reef)

Figure 76: Comparison of OSU phase II solid wall experimental forces to Momentum approach #1 predicted forces using theoretical velocities (10cm water on reef)
Figure 77: Comparison of OSU phase II solid wall experimental forces to Momentum approach #1 predicted forces using measured velocities (20cm water on reef)

Figure 78: Comparison of OSU phase II solid wall experimental forces to Momentum approach #1 predicted forces using theoretical velocities (20cm water on reef)
Figure 79: Comparison of OSU phase II solid wall experimental forces to Momentum approach #1 predicted forces using measured velocities (30cm water on reef)

Figure 80: Comparison of OSU phase II solid wall experimental forces to Momentum approach #1 predicted forces using theoretical velocities (30cm water on reef)
The results for the Momentum approach #2 are shown in Figure 81 to Figure 88.

**Figure 81:** Comparison of OSU phase II solid wall experimental forces to Momentum approach #2 predicted forces using measured velocities (5cm water on reef)

**Figure 82:** Comparison of OSU phase II solid wall experimental forces to Momentum approach #2 predicted forces using theoretical velocities (5cm water on reef)
Figure 83: Comparison of OSU phase II solid wall experimental forces to Momentum approach #2 predicted forces using measured velocities (10cm water on reef)

Figure 84: Comparison of OSU phase II solid wall experimental forces to Momentum approach #2 predicted forces using theoretical velocities (10cm water on reef)
Figure 85: Comparison of OSU phase II solid wall experimental forces to Momentum approach #2 predicted forces using measured velocities (20cm water on reef)

Figure 86: Comparison of OSU phase II solid wall experimental forces to Momentum approach #2 predicted forces using theoretical velocities (20cm water on reef)
Figure 87: Comparison of OSU phase II solid wall experimental forces to Momentum approach #2 predicted forces using measured velocities (30cm water on reef)

Figure 88: Comparison of OSU phase II solid wall experimental forces to Momentum approach #2 predicted forces using theoretical velocities (30cm water on reef)
4.4.3 Analysis of Solid Wall

The hydrostatic formulation by Asakura et al. (2000) did not work well for the dry reef or the wet reef case. Although it was developed based on dry reef experiments, the formula consistently underpredicted for both the OSU small and large scale dry reef experiments as shown in Figure 42 and Figure 60. For the wet reef cases, it overpredicted in the small scale experiments. For the large scale, it came close for the 5cm and 10cm standing water level conditions but started to overpredict more and more as the water level was increased. Overall it did not perform well and demonstrated the weakness in using a purely hydrostatic form. However, a positive aspect of the formula was that the predicted values showed less scatter than other methods, though they were unable to accurately predict the experimental results.

The hydrodynamic formulations all tended to follow similar trends and in general performed a lot better than the hydrostatic form. In the deeper standing water level conditions, the plots showed more scatter, which could be partially attributed to the variation in measured bore velocities between identical wave trials. On the other hand, when theoretical bore velocities were estimated using hydraulic jump theory and the bore height, there was less scatter in the predicted results. The use of theoretical velocities would be more desirable for design purposes, since bore height would be the only input parameter required in the hydrodynamic analysis.

When comparing the three formulations, the one that was overall the most consistent between small and large scale experiments was the Momentum approach #1. This formulation performed second most accurate after Wybro in the small scale and best overall in the large scale for both measured and theoretical velocities. The only negative point was that it underpredicted for the 5cm and 10cm water levels with theoretical velocities. The 20cm and 30cm plots were quite consistent along the 45 degree line. Furthermore, this approach showed less scatter when compared to Wybro and Momentum approach #2 when using the measured velocities.

The Momentum approach #2 performed very well in the small scale except for the two dry bed points that were greatly overpredicted. For use of theoretical velocities in the large scale experiments, the Momentum approach #2 performed most consistently for all
water levels. It underpredicted slightly for the 10cm and 20cm conditions but not as poorly as Momentum approach #1 did for the 5cm and 10cm levels. The Wybro formulation performed just below both Momentum approaches. However, in the small scale, Wybro performed the best out of the three with close to exact predictions and the smallest standard average error.

After comparing the performance of these formulations, it was found that either of the Momentum approaches performed consistently well at both small and large scales. This provides a measure of confidence that these approaches can be used for real life load estimation.
5 CONCLUSIONS AND RECOMMENDATIONS

The forces on structural vertical elements caused by a tsunami bore were investigated by scaled experiments and studied in depth in hopes of providing the engineering community with design guidelines and equations for these types of forces.

For the multiple column effect, it was found that forces generally increase due to the blockage of flow by adjacent columns. Firm conclusions could not be drawn due to the limited amount of data, but it is recommended that the effect of multiple columns be considered to increase the hydrodynamic load by about 25% for cases with percentage closure greater than 40%. More experiments with different closure percentages are necessary to understand the effect better.

For symmetric and offset shielding effects, it was found that although symmetric shielding generally decreases the force on the shielded column, it assumes that the tsunami bore is perfectly perpendicular to the building. Offset shielding occurs when the tsunami bore impacts the building at an oblique angle. In this case, it was found that forces generally increase due to focusing effects from front columns. Therefore, it is recommended that the force on interior columns be increased due to the potential of offset shielding. More tests need to be performed to establish the magnitude of this effect more precisely.

For the solid wall experiments, four different force estimation approaches were compared – Asakura et al. (2000), Wybro (1976), Momentum approach #1 (Paczkowski, 2010), and Momentum approach #2. An equation to estimate the force on a wall with a dry reef condition was not developed in this study. For the wet reef case, it was found that the Momentum approaches performed well at both small and large scales. Because of variability in the measured bore velocities, it is recommended that a theoretical bore velocity be computed using hydraulic jump theory with bore height and standing water height as the input parameters. This resulted in less scatter in the predicted forces on the wall. Tsunami inundation models are generally more accurate at predicting the bore height than the velocity. Therefore, using the bore height as the only input parameter, and predicting the velocity using jump theory, is a more desirable design approach.
6 REFERENCES


