The Assessment of Harbour Protection Schemes for Long Period Waves

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Abstract
The Royal Yacht Club of Victoria based in Hobsons Bay is at the northern end of Port Phillip, Victoria, Australia. The site is exposed to waves generated by container ships transiting the adjacent Port of Melbourne shipping channel. These long period waves are known as Bernoulli waves and they have caused damage to moorings and boats within the marina. The Club wishes to develop its site and provide greater protection against these waves.

An investigation has been made of the protection offered by a proposed harbour protection scheme consisting of full and partial depth wave screens. The investigation was made in two phases, the first of which involved the numerical modelling of propagation of ship generated and wind generated waves and their impact on the site and testing a number of harbor protection concepts. Due to the constraints at the site, partial depth structures were essential, rather than structures over the full depth. This necessitated the development of numerical models to assess wave interaction with the partial depth structures and wave transmission into the harbour.

The second phase of the development involved physical model testing of the partial depth wave screen. The testing regime was designed to enable an assessment of wave transmission through the partial depth wave screen, including testing for both the long period ship generated Bernoulli waves and the long period wind generated waves.

This paper presents the results of the investigation including the initial numerical model set up and validation, the methods used to model wave transmission through the partial depth wave screen, and a comparison of predicted transmission with that measured in physical model tests.

Keywords: Wave transmission, ship generated waves, marina development.

1. Introduction

The Royal Yacht Club of Victoria (RYCV) is located in Williamstown, Australia. The site faces Hobson's Bay, which is in the northern portion of Port Phillip Bay in Melbourne, Australia. Figure 1 shows the proximity of the site to the Williamstown Channel, an approximately 14m deep dredged shipping channel which provides access to the nearby Port of Melbourne.

Figure 1 Aerial photo of Hobson's Bay, with key locations and shipping channels. Note the RYCV Yacht Club adjacent to Williamstown Channel.

The Port of Melbourne is Australia's busiest container and generalised cargo port. In 2008 the Port commenced a major channel deepening project, a precursor of which was an extensive environmental impact assessment. As part of this assessment the Port commissioned a study to assess the effects of ship generated waves on RYCV, which included the deployment of an Acoustic Doppler Current Profiler (ADCP) offshore of the marina and the matching of ship generated waves against passing ship traffic.

Vessels transiting the Williamstown Channel are restricted to an 8 knot speed limit, which generally minimises the effects of ship generated waves at RYCV. However, under certain wind conditions it is necessary for pilots to exceed this 8 knot limit to maintain steerage when exiting the Port. Such situations, particularly large southward bound container ships heading into southerly winds, often result in the generation of larger ship generated waves which cause damage to boats and moorings at RYCV.

2. Bernoulli Waves

Long period waves related to the passage of a deep draft vessel are a result of the variation in pressure distribution around the vessel's hull [5]. With the atmospheric pressure being constant, the pressure distribution in the water column is balanced by a velocity pattern. Water accelerates from the bow through the midships causing a local depression, and decelerates past the stern [6]. The lowering and rising of the water level as the ship passes is referred to as a Bernoulli wave, which may have a period of tens of seconds.

Bernoulli waves are often more difficult to see than archetypal Kelvin wakes due to their long period and lack of reference frame [6]. While any moving vessel is theoretically capable of producing these long period Bernoulli waves, large vessels in confined waters are the most common source of Bernoulli waves of substantial amplitude [7].

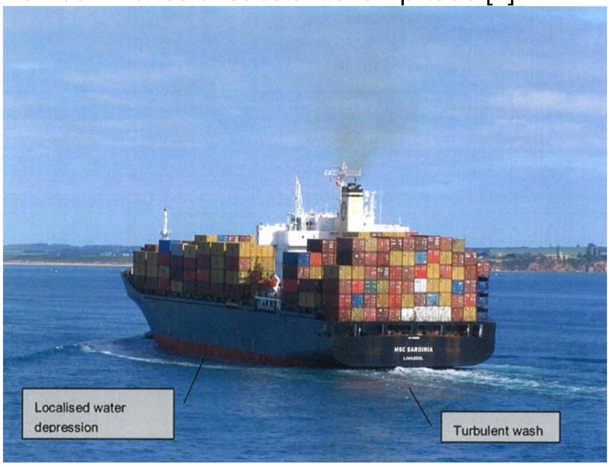
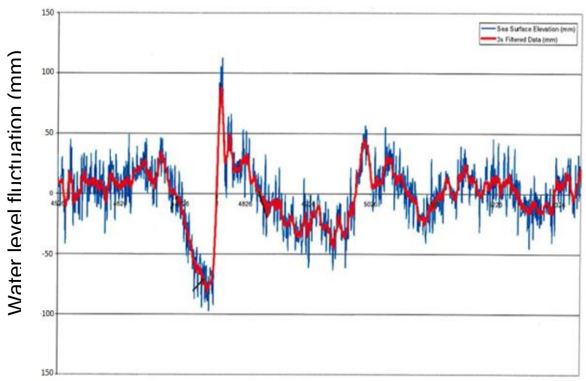


Figure 2 Bernoulli wave evident on side of tanker transiting the entrance to Port Phillip Bay (Source: [6]).

Bernoulli waves are characterized by a long drawdown followed by a sharp surge and a series of oscillations. The initial drawdown and surge contains the majority of the wave's energy.

Figure 3 shows a typical Bernoulli wave recorded at Hobson's Bay including the characteristic draw down followed by a very sharp surge and a series of oscillations typically lasting up to approximately 90 seconds [6].



Time (0.1s)

Figure 3: Characteristic Bernoulli wave in Hobsons Bay resulting from ships transiting the nearby Williamstown Channel (Source: [6]).

Previous works on wave transmissions through partial depth screens such as Pierson and Cox [6] have focused on shorter period waves which are more commonly encountered, while long period waves have been largely omitted from studies. Hence it was necessary to consider them in detail, as outlined in the following sections.

3. Numerical Modelling

3.1 CGWAVE model selection

CGWAVE is a general purpose, state-of-the-art phase-resolving wave prediction model based on the elliptic mild-slope equation (Demirbilek et. al. [2]). The model is most widely and commonly used to simulate wave patterns in and around harbours, open coasts, inlets, islands and around fixed and floating structures. The model was developed at the University of Maine (USA) under a contract by the U.S. Army Corps of Engineers (USACE) and interfaced with the Surface-Water Modeling System (SMS) for pre- and post-processing (Briggs et. al. [1]).

Unlike full depth structures – which are simply excluded from the model computational domain – the incorporation of partial depth structures (e.g., floating attenuators, partial depth wave screens, etc) in CGWAVE is more complicated and does not yield an exact solution. Instead an approximate solution following Tsay and Liu [9] and later refined further by Dongcheng [3] is suggested by CGWAVE to handle the treatment of floating docks and partial depth structures. Referred to as the "rigid lid" approximation, this approach initially considered a local modification to the wave number as a function of the "under-keel" clearance in place of the total depth. Dongcheng [3] further refined the approximation by modifying the model grid depth (h) beneath the structure as equal to the under-keel clearance (d1) multiplied by a correction factor (α). The correction factor (α) is calculated as:

$$\alpha = A \ln (ka) + B \qquad (1)$$

where k is the wave number, a is the half-width of the structure, and A and B are given in Figure 4 as a function of the wave number, the local depth (h), and the draft of the structure (d). When compared to exact solutions, Dongcheng [3] demonstrated this modified approach yields very good results.

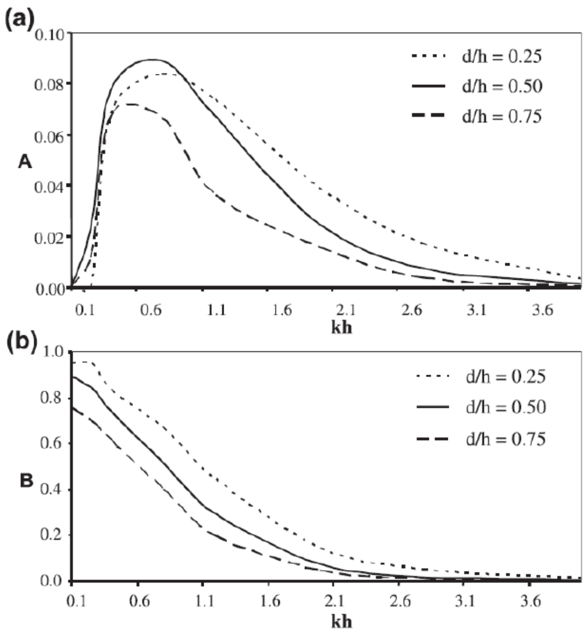


Figure 4 Values of A and B for determining α (Source [3])

3.2 Longwave agitation modelling

3.2.1 Setup and calibration

Long wave agitation modeling was carried out to numerically replicate the agitation and wave amplification phenomenon periodically occurring within the existing marina basin and confirm the effectiveness of coastal protection schemes (namely, the fixed and partial depth wave screens) associated with the proposed marina expansion plans. Incident wave conditions were selected based on a review of the 2006 Channel Deepening study findings [4]. In all, 30 individual cases were performed with wave periods ranging from 17s to 35s and wave directions ranging from 0degN, 10degN and 20degN.

The primary focus on calibration for the long wave modeling effort was focused on replicating the wave amplification patterns documented at the existing RYCV site. In the absence of recorded data within the marina, calibration relied on the direct experience of RYCV staff and findings from the Maunsell Australia Pty Ltd study [6]. Wave simulations were performed for waves incident from different directions with entire coastline boundaries assigned 0% reflection coefficients for the existing marina grid. This allowed inspection of the incident transformation processes through the model domain. Subsequently, the same set of cases were simulated on the same model grid but with entire coastline boundaries assigned 100% reflection coefficients to generate a worst scenario in terms of amplification through the model domain. Following a number of iterations, including mapping of the shoreline and assignment of representative reflection coefficients, long wave amplification patterns within the existing marina were successfully replicated to the overall consensus of the Club. As such the model was considered calibrated as far as possible with the information available. Figure 5 shows wave height contours for the worst incident wave conditions for the existing marina (T_p = 17s, Direction = 10°N)

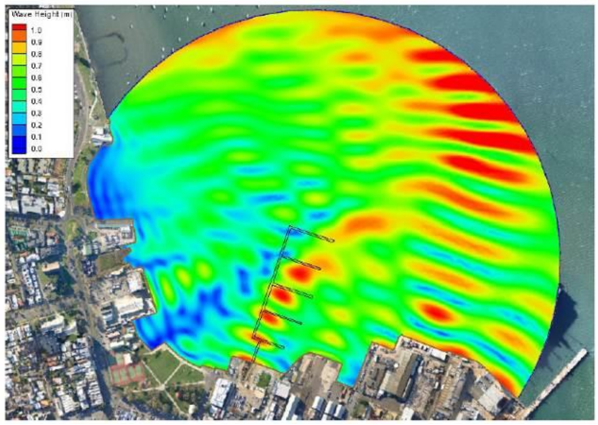


Figure 5 Model output showing wave height contours for 17s, 10°N, existing marina. Note contours are relative to a 1m incident wave height for ease of comparison of amplification factors.

3.2.2 Assessment of harbour protection scheme

In comparison to Figure 5, Figure 6 illustrates the wave height contour results of study case (17s, 10°N) inclusive of the impact of the full and partial depth structures. The introduction of these structures minimizes the amplification patterns within the RYCV basin, as compared to the same case without protection structures in place. Other cases showed similar trends for the proposed marina model grid. In general, the wave protection schemes were predicted to be successful at minimizing wave amplification within the proposed marina basin.

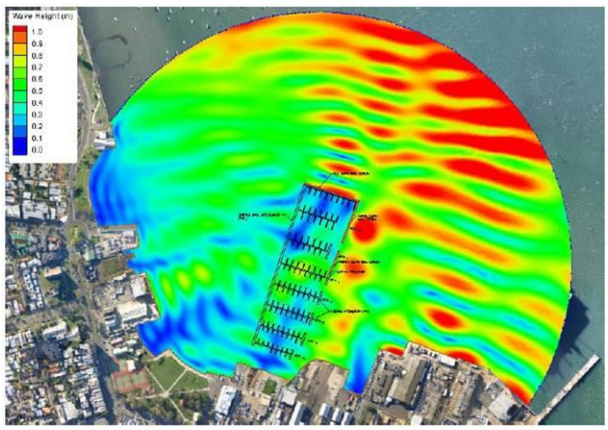


Figure 6 Model output showing wave height contours for 17s, 10°N, proposed marina.

3.2.3 Additional assessment of wave transmission

Additional modelling was also carried out to enable a comparison of predicted long wave transmissions against the results of physical model testing. This additional modelling was necessary so as to directly compare the results from the wave flume study with CGWAVE long wave results presented in Section 3.2.2 for the following reasons:

- The wave flume excludes wave reflection whereas the long wave findings in CGWAVE are partially a result of wave agitation due to variable reflection along the coastline;
- The wave flume tests consider waves approaching perpendicular to the partial depth wave screen, whereas the long wave model cases in CGWAVE consider waves approaching from the 0°, 10°N and 20°N directions, which do not impact the partial depth wave screen perpendicularly.

Instead, the wave flume was numerically recreated in CGWAVE at full scale in order to simulate the five test cases performed in Water Research Laboratory (2014) and allow a direct comparison between the physical and numerical model results. The five test cases are summarized below in Table 1.

Table 1 CGWAVE computer modelling long period wave transmission results

Test №	T (s)	$H_{offshore}$ (m)	$H_{transmitted}$ (m)	Coefficient of Transmission (K_t)
13	18.7	0.4	0.29	0.71
14	19.9	0.4	0.29	0.71
15	21.8	0.4	0.27	0.66
16	22.1	0.4	0.27	0.68
17	24.3	0.4	0.27	0.68

4. Physical modelling

4.1 Experimental Set-up

A 35m long two dimensional (2DV) wave flume was used to test a 1:10 linear scale model of the proposed partial depth screen.

Waves were generated by a 35kW hydraulic piston driving a full width wave paddle. Paddle time-displacement movements were pre-programmed to generate the desired wave conditions.

Free surface levels were recorded using an array of capacitance wave probes (CWP) as shown in Figure 7.

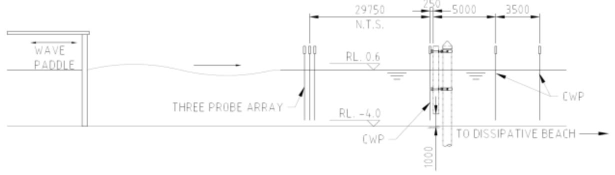


Figure 7 Wave flume set-up

Wave reflections from the rear of the flume were minimized by use of a dissipative beach. Incoming wave heights were determined during the calibration testing with no model structure in the flume.

The partial depth wave screen consisted of GG-95 FRP sheet piles (modelled in aluminium) connected to 700 Ø tubular steel piles spaced at 2.4m (scale) by two walers and pile clamps. The gap below the wall was held at 1.0m (scale) and the water depth at 4.6m (scale). Refer to figure 8.

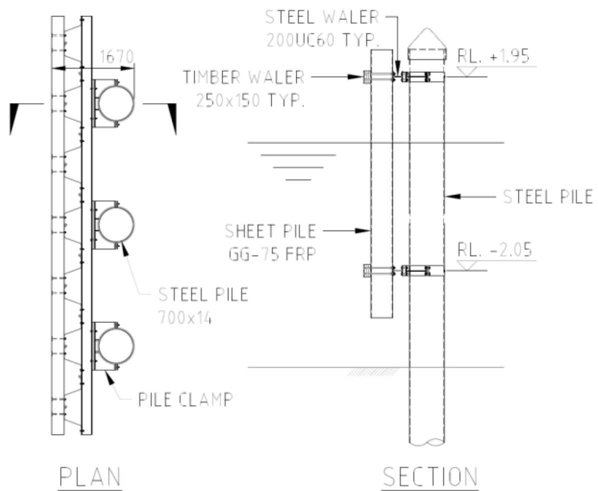


Figure 8 Partial depth wave screen to be tested by wave physical modelling

The model structure was braced against movement by large brackets (as seen in Figure 9) positioned so as not to influence the experiment.

Figure 9 Geometrically similar partial depth wave screen model in 0.9m wide wave flume. Note the load cells placed between the wall and piles. Gap below wall not visible.

4.2 Bernoulli Wave Generation

In each run of the experiment, a single long period wave (ranging from 18.7s to 24.3s) was generated in the wave flume with pre-programmed paddle time-displacements. The paddle started at rest in the forward position and was moved backward to create an initial drawdown followed by a quick forward movement to produce a sharp surge. This wave form is characteristic of ship generated Bernoulli waves.

Figure 10 shows the three offshore probe outputs during testing. The noise in the second half of the record is due to reflection from the structure.

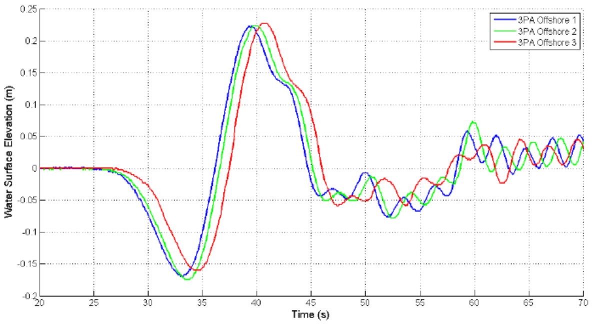


Figure 10 Wave probe record of 19.9 second Bernoulli wave test. Note the strong initial draw-down and sharp surge.

4.3 Transmission Test Results

Short period wave statistics were derived using zero-crossing analysis over the entire records, however for the long-period wave tests, transmitted wave height was determined by manually picking the peak and trough from the probe records.

Table 2: Long period wave transmission results

Test №	T (s)	$H_{offshore}$ (m)	$H_{transmitted}$ (m)	Coefficient of Transmission (K_t)
13	18.7	0.390	0.270	0.692
14	19.9	0.390	0.275	0.705
15	21.8	0.390	0.295	0.756
16	22.1	0.40	0.285	0.713
17	24.3	0.40	0.285	0.713

5. Discussion

A comparison of the physical modelling and numerical modelling results is provided in Table 3.

Table 3: Comparison of long period wave transmission results

Test №	T (s)	K_t – Physical Modelling	K_t – CGWAVE	% Diff.
13	18.7	0.69	0.71	3%
14	19.9	0.71	0.71	1%
15	21.8	0.76	0.66	-12%
16	22.1	0.71	0.68	-5%
17	24.3	0.71	0.68	-5%

The comparison indicates that CGWAVE performs well in comparison to the wave flume physical model results. Transmitted wave heights deviated by up to 12% for a single measurement with an aggregate average absolute difference of 4%. The wave transmission coefficients predicted by CGWAVE varied between 66% and 71%, whereas transmission coefficients measured during the physical model study varied between 69% and 71%.

Given the nature of the study it is considered that the results from the two investigations compare very well, giving confidence in the overall harbour protection predicted by CGWAVE. In addition the ability of CGWAVE to simulate the wave flume model study and results reveals the model's reliability and numerical accuracy to handle wave phenomenon in the case of partial depth structures.

Of relevance to designers, it is important to note that both means of investigation determined very high transmission results for long period waves. Given the partial depth screen terminated one meter from the seabed, it can be seen that the use of partial depth structures in the presence of long period waves needs careful consideration.

6. Conclusion

This study has investigated the protection provided by a proposed partial depth wave screen against long period wave transmission. The investigation was carried out in two phases, viz. a numerical model study using CGWAVE, and a physical model study in a wave flume, which reveals the following:

- Transmission coefficients predicted by CGWAVE varies between 66% and 71%, while transmission coefficients measured by physical model varying between 69% and 71%.
- The results of both studies compare well and the ability of CGWAVE to closely replicate the transmission phenomenon with wave flume model study has been validated.
- Results also demonstrate the model's reliability and accuracy to determine transmitted wave height for partial depth coastal protection structures.
- In all cases wave transmission was significant, highlighting the caution needed while considering the use of partial depth structures in harbour protection schemes at sites exposed to long period waves.

7. Acknowledgements

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8. References

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