Design of a partial depth wave screen to protect the Tawau Ferry Terminal, Malaysia

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Abstract
This paper describes the design and construction in early 2019 of a precast concrete partial depth wave screen to protect the Tawau Ferry Terminal in Sabah, Malaysia. Extensive numerical modelling was completed in order to determine the hydrodynamics and design wave climate at the site, which showed exposure to rough seas particularly during the monsoon season. A geotechnical investigation found poor ground conditions including up to 10m of soft silt, and this, combined with water depths of approximately 10m, made traditional rubble mound breakwater options impractical. Floating attenuators were considered but wave periods at the site were assessed as being too long for floating attenuation to be viable. The solution adopted was a pre-cast concrete partial depth wave screen supported by tubular steel piles.

Keywords: harbour protection, wave screens, coastal structures.

1. Introduction

The Tawau Ferry Terminal, which is the region's gateway between Malaysia and Indonesia, is a pontoon-based terminal capable of handling eight ferries with a total capacity of approximately 1000 passengers at any one time. The site location and existing terminal is shown in Figure 1.

Despite the distance from the site to the open sea and the sheltering from Pulau Sebatic and the mainland, relatively rough seas and high swell have been reported during the southwest (SW) monsoon season. Since the opening of the terminal in 2016 these conditions were found to affect passenger safety and result in operational down time.

Some form of wave protection was therefore required, and the solution ultimately adopted was a partial depth wave screen, sometimes known as a skirted breakwater. This innovative approach was the result of a successful design and construct approach and is thought to be the first wave screen of its type in Malaysia and the region generally [1].

The purpose of this paper is to describe the key design and construct considerations influencing the project. The objective is to share the lessons learned during the design and construction of the wave screen in the hope that they may assist the reader in resolving similar issues at other sites.

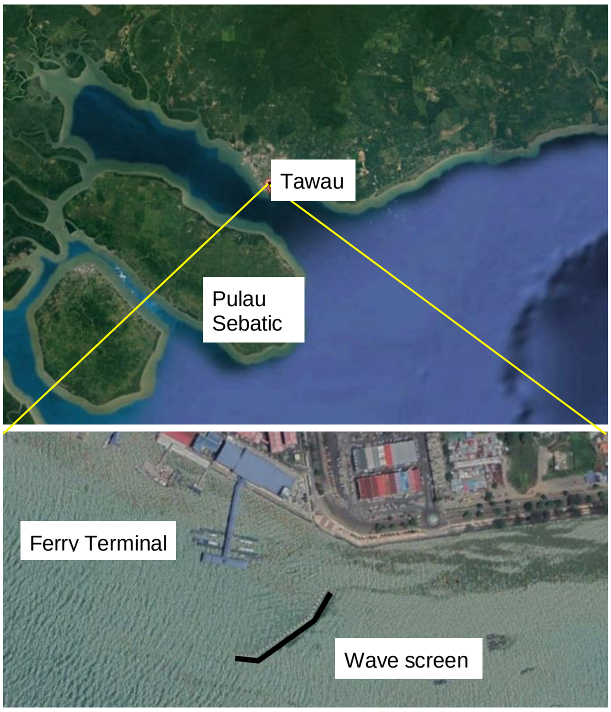

Figure 1 Site location. Above: Located in Sabah, Malaysia the site is partially sheltered by the island Pulau Sebatic. Below: Ferry terminal and wave screen.

2. Site conditions

2.1 Hydrodynamics

In order to determine the design conditions at the site, DHI [2] completed an extensive numerical modelling exercise. This included the deployment of three ADCPs and a self-recording water level recorder to capture approximately 3 weeks of wave, current, and water level data. Transect measurements were also used to record the flows between Tawau and the nearby island Pulau Sebatic. This information was used to validate a Mike21 SW, with the model showing good agreement with recorded site data [2]. The model setup is shown in Figure 2.

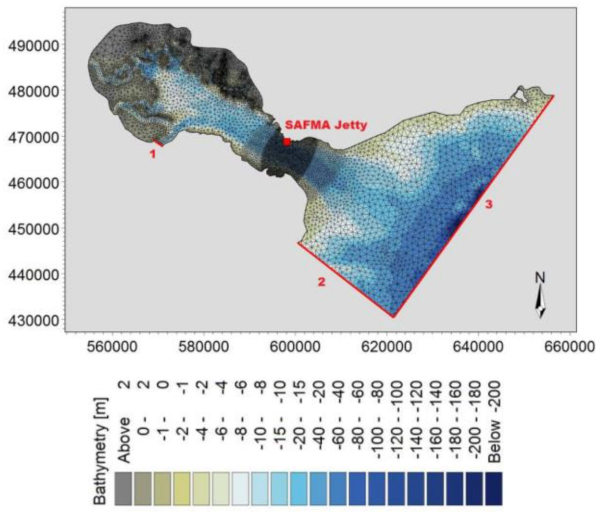

Figure 2 - Unstructured flexible mesh used for local 2D hydrodynamic model (Source: [2]). Red lines indicate open boundaries of the model. Boundary 1 uses predicted tidal elevation from Wallace Bay while boundaries 2 and 3 use predicted tidal elevation from Tawau as input.

Of relevance to this paper, key design criteria determined by the wave study included:

- 1%AEP Significant Wave Height, Hs (m): 0.52
- 1%AEP Peak Period, Tp (s): 5s
- 1%AEP Current velocity: 1.9ms^{-1}

In addition to extreme design conditions, operational conditions were also assessed using nine years of hindcast wave data based on 2003 to 2012 wind records.

The assessment showed that the incident waves at the site are predominantly from the SE and SSE directions throughout the whole year, corresponding to the orientation of the channel between the mainland and Pulau Sebatic. Wave conditions were found to be calmer towards the end of the year, during the NE monsoon, while rougher wave conditions were found during the SW monsoon period.

In terms of operational downtime, the assessment showed that significant wave heights greater than 0.2m were found to be associated with peak wave periods typically less than 6s. At these periods the limiting acceptable significant wave height for the pontoons was 0.3m, as specified by the pontoon supplier. The all-year wave rose at the site is shown in Figure 3, showing that these heights are exceeded at the site and that some form of wave protection is required.

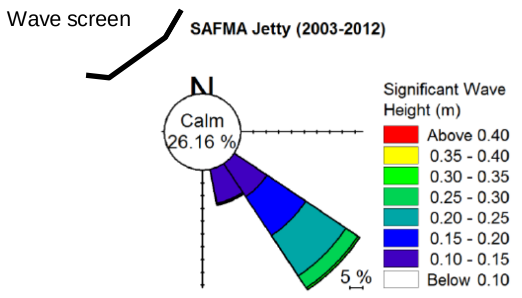

Figure 3 – All-year wave rose between 2003 and 2012 (Source: [2]). Waves were found to occur predominantly from the SSE, and to exceed the 0.3m significant wave height design criteria of the terminal pontoons.

A similar exercise was also completed for current direction and speed, based on hindcast data for the years 2004-2012. This showed that the flood and ebb currents are predominantly from the WNW and ESE directions, again corresponding to the orientation of the channel between the mainland and Pulau Sebatic. Current speeds are stronger at ebb tide in comparison to flood tide because of tidal asymmetry. The tide range at the site is provided in Table 1 and the all-year current rose is shown in Figure 4.

Table 1 Tidal water levels (m) at Tawau Port relative to Chart Datum (source: [2])

MSL	MHHW	MLLW	HAT	LAT
2.01	3.40	0.61	3.86	0.00

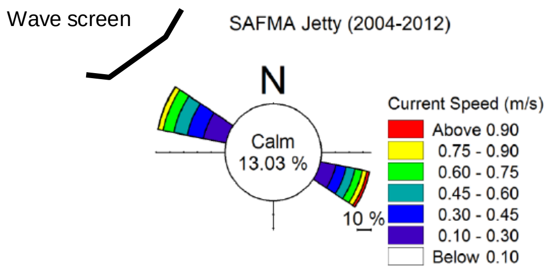

Figure 4 – All-year current rose between 2004 and 2012 (Source: [2]). Currents were found to be strongest on the ebb spring tides and exceed 0.9m/s.

It should also be noted that the 1% AEP current velocity (1.9ms^{-1}) was significantly greater than the findings of the operational assessment (which was based on tidal constituents). This was due to a significant non-tidal residual current that results from storm surge, which was estimated separately using an extreme value analysis of the variation of predicted water levels and currents versus recorded data.

2.2 Bathymetry and geotechnical conditions

Although not a focus of this paper, it is important when providing the context of the wave screen design to note that a hydrographic survey showed the seabed to be at approximately -10.0mCD.

Geotechnical conditions were also investigated at the site through the drilling of three (3) offshore boreholes. These confirmed the presence of 8-12m of very soft silty clays with zero end bearing capacity, overlying stiffer clays. Rock was not encountered in any boreholes, which were drilled to a total depth of 30m.

3. Wave screen concept

On the completion of the wave and hydrodynamics study it was concluded that some form of wave protection was required to ensure that the design criteria of the pontoons was not exceeded, and to improve the safety of the ferry operations.

Traditional rubble mound breakwaters were considered, however this was quickly discounted given:

- The combination of soft clays and significant water depths requiring a significant volume of material
- The effects of a full depth structure on the strong currents at the site
- The difficulty of constructing a detached breakwater at this site, and the consequences on shoreline dynamics if building out from the shore.

Floating attenuation was also considered, however the wave periods at the site were considered to be too high for floating breakwaters to be effective. Both physical model tests from suppliers and available literature show that the performance of attenuators drop significantly once wave periods are greater than 4s ([3],[4]).

A wave screen, or skirted breakwater, was therefore selected as the preferred breakwater concept. Initially a concept following that proposed by Suh [5] was considered, which consisted of a row of tubular steel piles driven closely together with partial depth infill panels between the piles, as shown in Figure 5.

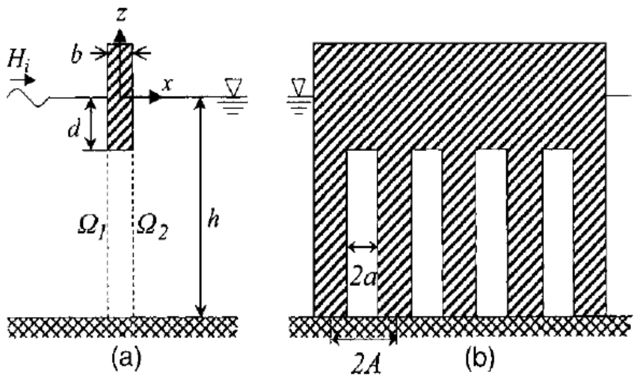

Figure 5 Definition sketch of wave screen concept after Suh (Source: [5])

Numerical and physical modelling of such a concept showed that the closely spaced piles offer some additional reduction in wave transmission when compared to traditional curtain wall breakwater [5]. For this to be true, however, the piles need to be very closely spaced. Gous [6] for example, demonstrated that for a piled breakwater (without panels), wave transmission roughly doubled when pile porosity was increased from 0.04 to 0.2, and that wave transmission was over 80% for some test cases with 0.2 pile porosity. At this site the ground conditions were problematic and early concept design showed that a piling arrangement using such closely spaced piles was inefficient in terms of the resistance of lateral wave loads.

An alternative concept was therefore developed that consisted of a precast concrete wave screen supported by piles arranged in pairs of one vertical and one rear raking pile. A typical section of such a concept is provided in Figure 6.

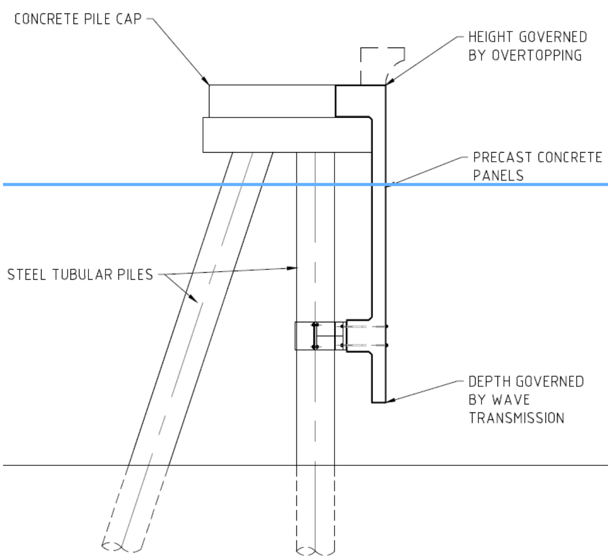

Figure 6 Wavescreen concept developed for Tawau

4. Key dimensions

4.1 General arrangement

The plan dimensions of the wave screen were based on assuming an overall wave transmission of no more than 50% in extreme design cases and using the numerical model to determine the plan arrangement required to achieve the desired wave climate at the ferry terminal. The safe navigation of ferries to and from the terminal was also a consideration when determining the offset of the wave screen from the ferry terminal, as was limiting the effects that the wave screen may have on the currents at the site. Figure 7 shows example model plots comparing different wave screen arrangements.

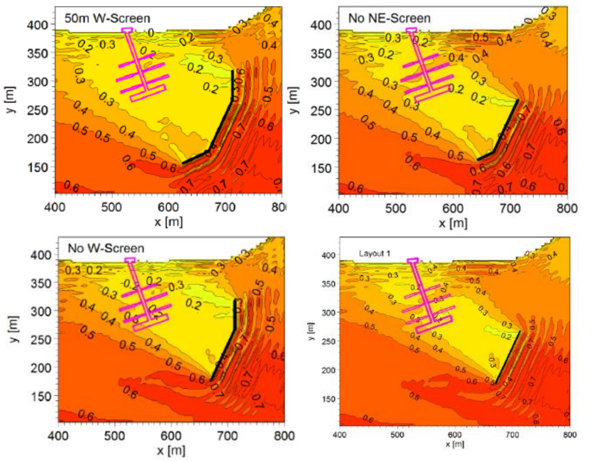

Figure 7 Example model plots showing predicted wave heights at the terminal for various wave screen arrangements (source: [2]).

A number of arrangements were trialled in the model and ultimately it was concluded that the wave screen should consist of:

- A 100 m-long main screen, orientated at 55 degrees
- A 30 m-long western screen, orientated at 275 degrees
- A 50 m-long north-eastern screen, orientated at 30 degrees

4.2 Panel depth

In the absence of the opportunity to complete physical model testing, the depth of the wave screen required in order to minimise wave transmission to acceptable levels was based on an extensive literature review, acknowledging that a cautionary approach was needed in the absence of project specific tests.

References applicable to the wave screen concept being considered include Peirson [6] and physical model test results from other relevant projects [7].

CFD models can also be used to predict transmission. Peirson [6], for example, compares physical model tests of wave transmission under skirted breakwaters (i.e. wave screens on widely spaced pile supports) against transmission predicted in accordance with Wiegel [7]. This demonstrated that in some cases for waves with a water depth (d) divided by wavelength (L) greater than 0.25, as is the case for this project, the transmission prediction by Wiegel significantly over estimated measured transmission. The relevant figure, annotated to show the relevance of the design conditions at Tawau, is reproduced in Figure 8. Based on this approach a panel depth of -2.5mCD was adopted.

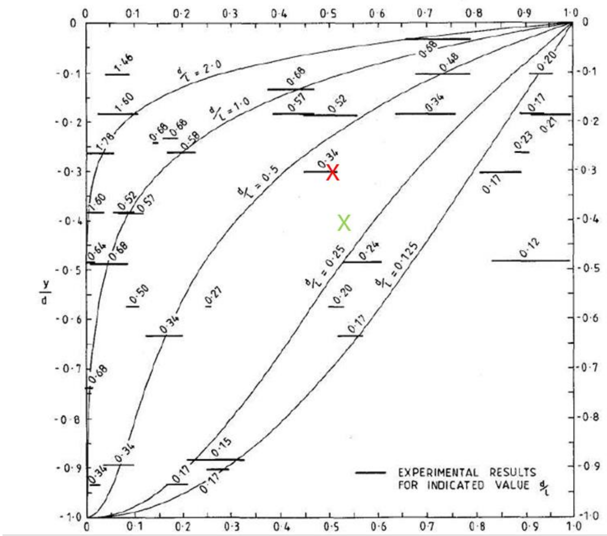

Figure 8 Transmission coefficients as estimated by Wiegel [7] and as measured in available physical model studies (source: [6]). The red mark represents main and western screen sections, and the green mark the northern screen.

4.3 Panel height

Overtopping was estimated using the empirical methods summarised in the 2007 European Overtopping Manual [8], which has shown good correlation with physical model test results from other similar projects. This assessment was based on a 1:100yr storm occurring at MHHW, which was considered to be suitably conservative combination of extreme waves and water levels. Estimated overtopping was limited to 10/l/s/m, this being a limit recommended to avoid the sinking of small boats in the lee of the wave screen [9]. The resulting crest heights were approximately +5.0mCD.

5. Constructability

A key consideration in this design was the lifting capacity of locally available plant, with the maximum permissible weight governing the wave screen panel width. The design developed specifically for the Tawau wave screen consisted of approximately

3.1m wide precast concrete panels, sized with this lifting constraint in mind. The panels are supported at the top by a precast "L" shaped beam, and at the bottom by a steel square hollow section waler beam.

The beams were sized to span approximately 6.2m between pile bents, with each pile bent consisting of one vertical pile and one raked pile. The pile cap that connects each vertical and raking pile was formed using a precast concrete shell with an insitu concrete pour.

The advantages of this approach in terms of constructability included:

- A reduction in the number of piles required, which was particularly important at this site given the depth to competent founding material;
- Maximised use of off-site prefabrication, which improved the quality and safety aspects of the project
- The ability to take up piling tolerances within the insitu pile cap

The most difficult aspect of construction was the accurate installation of the lower waler assembly. The strong currents and poor visibility made all dive work difficult, and the installation of the panels was limited to slack tide only. There was therefore little margin for error as the panels needed to be fixed before the outgoing tide.

In order to resolve this a dummy wave screen panel was made up that consisted of a steel template accurately replicating the fixing locations of the wave screen panels. A trial install of each panel was then completed, and any final adjustments made to the steel waler and clamp assembly. This then enabled each panel to be installed successfully. Example photos are provided in Figure 9 and Figure 10.

Figure 9 Dummy panel installation

Figure 10 – Wave screen panel installation in progress

6. Conclusion

This paper has provided a brief summary of the design and construction of a precast concrete wave screen in Tawau, Malaysia.

The design included extensive numerical modelling to determine the design wave condition at the site, and in the absence of the opportunity to complete physical model testing used empirical formula and existing model test data to determine the key dimensional requirements of crest height and overtopping.

Anecdotal from the Sabah Port Authority, operators of the ferry terminal, is that downtime has been significantly reduced since completion of the wave screen. A site monitoring program is currently being developed to assess the performance of the wave screen in detail, the results of which will help inform the design of future waves screens in the region.

Constructability was a key consideration during design, particularly the lifting capabilities of locally available plant. The most challenging aspect during construction was the installation of the precast panels in the tight slack tide window, and this was achieved by first trialling the installation at each location with a steel dummy panel.

7. References

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