WAVE TRANSFORMATION AND OVERTOPPING CHALLENGES SOLVED BY MULTI-DISCIPLINARY MODELLING

P. Sloth\textsuperscript{1}, J. Fuchs\textsuperscript{2}, H. Kofoed-Hansen\textsuperscript{3}, S.B. Mortensen\textsuperscript{4} and D. Mühlestein\textsuperscript{5}

Abstract: Wave transformation and wave overtopping challenges for a new cruise terminal in Jamaica were solved by multi-disciplinary numerical and physical modelling approach. The main objective of the study was to document the overtopping and, if required, mitigate the overtopping reaching the buildings on the terminal pier.

The cruise terminal is located at Falmouth, on the northern coast of Jamaica and therefore potentially exposed to tropical hurricanes. The terminal is located on a shallow reef fronting the shoreline at Falmouth. At the reef edge, approximately 1km from the shoreline, the water depth drops from a few meters very steeply to several hundreds of meters. An access channel (depth of 12.6m) is dredged from the reef edge to the terminal, which consists of berths on either side of the pier, see Figures 1 and 2.

The main challenges studied were:

- The complex transformation of extreme waves from deep water, over the reef and through the dredged channel, to the terminal pier.
- The complex wave overtopping of the pier and the associated wave forces on walls protecting the buildings on the pier.

Figure 1. Perspective view of the project site.

Keywords: Falmouth; Jamaica; wave breaking; MIKE 21; CFD; reef; hurricanes; overtopping; wave forces.

\textsuperscript{1} Senior Engineer, DHI, Agern Allé 5, DK-2970 Horsholm, Denmark, Email: prs@dhigroup.com
\textsuperscript{2} Head of Projects, Ports & Offshore Technology, DHI, Agern Allé 5, DK-2970 Horsholm, Denmark, Email: juf@dhigroup.com
\textsuperscript{3} Head of Dept, Ports & Offshore Technology, DHI, Agern Allé 5, DK-2970 Horsholm, Denmark, Email: hkh@dhigroup.com
\textsuperscript{4} Senior Engineer, DHI, Gold Coast Office, Level 2, 12 Short Street Southport, QLD 4215, Australia, Email: sbm@dhigroup.com
\textsuperscript{5} Senior Engineer, DHI, Agern Allé 5, DK-2970 Horsholm, Denmark, Email: dm@dhigroup.com
INTRODUCTION

The new Falmouth Cruise Terminal on Jamaica is shown in Figure 2 below.

The investigations included the following elements:

- Numerical simulations of the transformation of extreme (hurricane) waves from deep water to the terminal pier.
- Physical model tests to investigate wave overtopping conditions at the pier; the tests included also to study the effect of alternative structural measures aimed to reduce the wave overtopping and the impact of this on the main terminal building.
- Measurements in the physical model of wave loads exerted on the wave screens aimed at mitigating overtopping at the pier.

The extreme offshore wave conditions were studied by others prior to the present study; this study established the extreme (up to 1 in 100yrs) wave conditions off the reef edge.
Wave conditions at the terminal

With the objective of determining the extreme wave conditions at the terminal pier, numerical wave transformation modelling was carried out.

Figures 1 and 3 present the bathymetry at the project site. In front of the reef edge, the water depth is several hundreds of meters. Along the outer part of the dredged channel leading to the cruise ship terminal, the depth is 12.6mMSL.

The transformation of extreme waves from deep water towards the terminal is governed by wave breaking on the reef edge and in the outer part of the dredged channel and by wave diffraction and refraction along the channel sides. The latter implies that wave energy will be spread from the channel towards the shallower areas along the channel where the energy is dissipated. On the other hand, some wave energy will reach the channel from the adjacent shallow areas.

Modelling Approach

As no single numerical wave model is capable of or practically able to reproduce all the relevant wave transformation processes without calibration and tuning, an approach including three different types of numerical wave models in combination with findings from literature was adopted. Finally the model was verified against physical model results.

DHI’s computationally efficient wave model, MIKE 21 PMS [parabolic mild-slope type model, DHI(2011a)], was applied for the final simulations after tuning the model parameters based on simulations with other numerical models and on findings from literature. MIKE 21 PMS is capable of reproducing phenomena such as shoaling, refraction, energy dissipation due to wave breaking and bed friction, forward scattering and partial diffraction of directional irregular waves.
As the MIKE 21 PMS model only partly includes diffraction and as restrictions apply to the wave directions (waves can only run forward in the model grid, ie reflections cannot be included), simulations were also carried out with the more comprehensive wave model, **MIKE 21 BW** [Boussinesq-type model, DHI (2011b)], to verify the results of the MIKE 21 PMS model as regards wave refraction and diffraction. The MIKE 21 BW model is the most accurate model as regards modelling of refraction and diffraction, but it was not feasible to use the model’s wave breaking facilities in the present case – hence the choice of MIKE 21 PMS for the final investigation. MIKE 21 BW is based on the numerical solution of Boussinesq type equations applicable for non-linear waves. MIKE 21 PMS simulates linear waves.

Owing to the very steep seabed slope right in front of the reef, the very shallow water along the channel, and the large design wave height, the wave transformation over the reef edge and along the outer part of the channel is not trivial, and model settings normally applied for typical coastal applications can therefore not be assumed. This applies especially to the description of the reduction in wave height due to wave breaking. Therefore, the wave breaking was studied separately through two-dimensional (2D) CFD modelling and from literature. In order to study the wave breaking numerically, DHI’s fully non-linear CFD model, **NS3** (Christensen et al (2009)) was applied. The model applies a fully non-linear 3D Navier-Stokes equation solver with a Volume of Fluid (VOF) treatment of the free surface. Furthermore, the wave breaking was studied through literature and physical modelling.

**Verification of Refraction and Diffraction in MIKE 21 PMS**

As described above, the MIKE 21 BW model was applied to verify the wave refraction and diffraction of the MIKE 21 PMS model. In order to allow the comparison of the models, the same model bathymetry was applied in both models. The model bathymetry is shown in Figures 1 and 3. The model grid size was 5m.

Simulations were carried out with a wave period corresponding to the 100-year design wave conditions, being \( T_p = 14.4s \). The incident wave direction was NE. To avoid wave breaking in the MIKE 21 BW model, an incident wave height less than the 100-year wave height was applied in both models (MIKE 21 BW and MIKE 21 PMS) for verification of refraction and diffraction in MIKE 21 PMS. Figure 4 shows a snapshot from one of the simulations with the MIKE 21 BW model. It can be seen that the waves become very steep on the side slopes of the channel, where water depths become very shallow and where the wave field becomes very complex due to the highly irregular bathymetry.

![Figure 4. Snapshot of surface elevations from MIKE 21 BW simulations. Dashed lines indicate channel edge.](image-url)
Figure 5 shows examples of computed relative wave heights from the two models. Though some differences are seen when comparing the results, the modelling showed acceptable agreement, with the MIKE 21 PMS giving the larger wave heights at the pier. It is thus judged that MIKE 21 PMS gives slightly conservative (ie on the high side) wave heights at the pier as regard wave refraction and diffraction effect. However, wave breaking will be by far the governing factor on wave heights at the terminal, wherefore the slight conservatism on the refraction/diffraction effects was concluded to be of little importance.

Wave Breaking Parameters

In order to be able to define the wave breaking parameters for the MIKE 21 PMS model to be applied for the final/production simulations, wave breaking over the steep reef edge was studied through a combination of numerical modelling, literature and physical modelling.
**CFD Model NS3**

Simulations were carried out with DHI’s in-house CFD model, NS3, being capable of resolving highly complex wave breaking. Only 2D simulations were carried out as a full 3D model was not feasible due to its high CPU demand.

The offshore water depth was 300m. A multi-block mesh consisting of 9 blocks and a total of 1.7 million grid cells was used to discretise the domain.

The wave condition applied on the offshore model boundary consisted of a wave spectrum corresponding to the 100-year event. A water level of +1mMSL was assumed. An active wave absorbing offshore boundary condition was applied to absorb reflected energy from the reef shelf and assure a zero net wave-driven mass flux into the domain.

The model was used to simulate 55 minutes of wave propagation in real-time. Figure 6 shows a snapshot from the CFD modelling.

![Figure 6. Snapshot from NS3 modelling. Y-axis: meters. Close-up at reef edge.](image)

Wave spectral analyses were carried out based on surface elevation time series extracted with 100m spacing along the reef shelf for a 25-minute period, starting 30 minutes into the simulation. Figure 7 presents the calculated significant wave height as function of the distance from the reef edge.

![Figure 7. Significant wave height (Hₘ₀) along the reef shelf; based on NS3 simulations.](image)
Literature Review

Wave transformation across reefs has been studied by numerous researchers; see e.g. Jensen (2002), Gourlay (1994 and 1996) and Massel (1999), where results from both physical and numerical models are published. For a horizontal reef-shelf, both field and laboratory data show that the maximum wave height does not exceed 0.6 times the water depth – as opposed to a value of around 0.8 often assumed for sloping seabeds. In Jensen (2002), an empirical formula for estimating wave transformation on reefs is proposed. This formula was applied by this study for estimating the wave height decay on the reef shelf. The results are presented in Figure 8 for the 100-year wave conditions and a water level of +1mMSL.

A 2D model of the wave transformation across the reef as modelled by the NS3 model was set up also in the MIKE 21 PMS model, and the model parameters were tuned to fit with the findings of the NS3 model and the empirical formula (Jensen (2002)). Figure 9 shows the comparison of the three methods after tuning of the MIKE 21 PMS model. The terminal pier head is located approximately 600m from the reef edge. The resulting model parameters determined through this 2D calibration were subsequently applied in the final 3D model set-up in MIKE 21 PMS.

Verification against Physical Model Tests

After having set up the physical model (see below), it was possible to compare results obtained from the numerical model also with recordings from that physical model. A numerical model flume (2D model) was set up in the MIKE 21 PMS model with depth conditions similar to those in the physical model in order to allow for a comparison of wave conditions simulated in the numerical model with wave conditions recorded in the physical model. The numerical wave simulation in this numerical 2D model was carried out with the model parameters used for the final 3D simulations presented above. Figure 9 compares simulated and recorded wave heights. The comparison shows almost identical results of the numerical and the physical model.
Wave transformation and overtopping challenges solved by multi-disciplinary modelling

Figure 9. Comparison of wave height decay from 2D numerical (MIKE 21 PMS) model and physical model.

**Final Simulations of Wave Transformation from Deep Water to the Pier**

Based on the findings presented in the preceding sections, the MIKE 21 PMS model was finalised and the design wave conditions at the pier were established in the model, including effects of wave breaking. Figure 10 shows an example of a wave height contour plot from the simulations as well as the wave height from deep water to the pier as a profile plot. The wave height was found to be reduced to around 1/3 of the incident wave height in deep water.

Figure 10. Modelled wave conditions. 100-year event.

Applying this numerical model, all relevant design wave conditions were derived at the pier and applied as input to the physical modelling of wave overtopping and wave forces.
PHYSICAL MODELLING OF WAVE OVERTOPPING AND WAVE FORCES

Wave overtopping of the pier head and wave impact loads were studied by tests carried out in a physical model at scale 1:35 at DHI, Denmark.

The main objective of the model tests was to optimise the layout of protective walls in front of the main buildings on the pier to prevent wave overtopping from causing flooding or other damage to the buildings.

The physical model did not attempt to reproduce the actual settings in front of the terminal such as the full three-dimensional bathymetry with features such as the steep reef edge, the reef proper and the slope aligned channel from the reef edge to the terminal pier. Instead, a simplified model bathymetry was built, which allowed specifically the overtopping of the pier head to be investigated. The approach adopted was to reproduce the wave heights estimated by the numerical model simulations at the head of the terminal pier in the physical model and then by the physical model to study the wave overtopping and wave impact.

The model of the outer part of the pier was reproduced in a 5.5m wide wave flume. The width of the wave flume corresponded hence to 192.5m at full scale. Figures 11 and 12 show a plan of the wave flume in which the model tests were carried out. At the offshore end of the flume, a wave generator was placed to produce the required incident waves. The model seabed level in front and alongside the pier corresponded to -11.6mMSL. In front of the wave generator, the seabed level corresponded to -22.8mMSL. This latter depth was required in order to allow for generation of sufficiently high waves offshore needed to create the required wave heights at the pier. The slope of the reef edge was 1:1.2, ie corresponding to the average slope of the reef edge in nature.

**Figure 11. Plan of the model set-up of the wave flume (note that the sketch is not to scale).**

**Figure 12. Section of the flume model set-up.**
Waves were calibrated in the physical model after construction of the model seabed prior to placing the model of the pier in the wave flume. Wave conditions with return periods of 1, 10 and 100 years were extracted from the numerical wave simulations and reproduced.

The main terminal building was represented in the physical scale model as a simple box. Figure 13 shows a photo from the model tests.

![Figure 13. Photo from the model tests showing the pier head with wave screens.](image)

**Wave Overtopping**

A number of alternative layouts of wave screens aimed to protect the main building against flooding and wave impact were investigated in the model. Figure 14 gives an overview of the investigated layouts. Different configurations, lengths and heights of the screen were investigated.

![Figure 14. Overview of the tested wave screen layouts.](image)

The model tests concluded that the preferred layout was Layout D (see Figure 14) with three lines of screens. With this layout, wave overtopping of the second wave screen occurred regularly during the design event; approximately 20% of the individual waves gave rise to overtopping, and water reached
the area protected by the third wall. The model tests, however, indicated that overtopping water as such is unlikely to hit the building, and therefore no structural damage to the building is to be expected even during extreme events.

It was determined that the following main features are important for the efficient performance of the second wave screen:

- minimum height: 1.5m
- wall extent along the quays shall be sufficient to protect the building
- wall shall be inclined or feature a recurved cap.

**Wave Forces on Wave Screens**

The same physical model used for the wave overtopping tests was used for tests in which wave forces and pressures on the wave screens were measured.

Wave forces were recorded at selected sections on the front and second wave screens. Wave pressures were recorded at two locations on the second wave screen by pressure cells fitted on the wave screen. The tests in which wave forces and pressures were recorded had duration corresponding to 3.5 hours at full scale in order to include at least 1000 individual waves in each test run. The logging frequency of load and pressure cells was 1000Hz (in the model) corresponding to 169Hz at full scale.

Figure 15 shows examples of ranked distributions of recorded maximum forces and pressures. These values result from independent analyses of each signal (i.e. $F_H$, $F_V$ and $p$) and are hence not simultaneous values.

The results of a test corresponding to the 100-year event were analysed in order to estimate $F_{H,0.1\%}$, $F_{V,0.1\%}$ and $p_{0.1\%}$. $F_{H,0.1\%}$, $F_{V,0.1\%}$ and $p_{0.1\%}$ are the values with estimated exceedance probabilities of 0.1% (1/1000). These values were estimated using a logarithmic regression line through the 100 highest events recorded.

![Figure 15. Distribution of recorded maximum forces [kN/m] (left) and pressures (right).](image)
CONCLUSIONS

The award winning cruise terminal at Falmouth (Port of the Year 2011) is located on the northern coast of Jamaica and therefore is potentially exposed to tropical hurricanes. Although the surrounding reefs provide good protection from extreme sea states, the access channel allows fairly large waves to propagate towards the terminal quay, which ultimately may cause severe wave overtopping and subsequent flooding of the terminal buildings.

In order to assess the amount of overtopping and risk of flooding, a multi-disciplinary modelling approach has been demonstrated. The approach was successfully used to test and optimize various protective structures to limit potential flooding.

It was shown that complex hydrodynamic challenges – that cannot be handled accurately and efficiently by either a single numerical model or by economical yet reliable physical models – can be solved by taking advantages of combining individual models’ capabilities and strengths to reach accurate, safe and cost-effective solutions.

ACKNOWLEDGEMENTS

We wish to thank contractors E. Pihl & Son A.S., Denmark, who commissioned the study to DHI, for their contributions and cooperation during the work.

REFERENCES