# EXTREME WAVES IN SHALLOW WATER AND DREDGED CHANNELS FOR DIRECTIONAL SEAS

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## INTRODUCTION

Extreme wave conditions at coastal structures located offshore of natural coastlines and the zone of wave breaking are influenced by the deepwater wave conditions, the coastal bathymetry between deepwater and the structure, and the ongoing growth, generation and decay of waves across the coastal shelf. At locations where the coastal shelf is relatively flat, the gradually reducing water depths can limit the significant and maximum wave heights and crest elevations that may be encountered. If a coastal structure is located in deeper water than the bathymetry of approaching waves, for example port or navigation structures located in dredged channels or basins, coastal engineers need to consider wave conditions in the shallower approach waters and the transformation and propagation of waves across the dredged area. The directional characteristics of wave spectra in this situation can have a significant impact on extreme wave conditions that occur within the dredged basin.

Forristall (2004) concluded that for shallow water with a flat seabed, the commonly adopted wave breaking coefficient (H/depth) of 0.78 was unrealistic and that wave breaking coefficients that large could not be observed in many physical model data sets. Forristall (2004) also concluded based on the information available at that time, that it was uncertain if Nelson's (1994) depth limiting breaking coefficient of 0.55 was realistic for conditions other than the relatively broad and flat reef conditions examined in that paper.

This paper presents insights into extreme wave conditions for gently sloping natural seabed profiles and also within a dredged basin, for directional seas which occur within the direct path of severe storms; for example, tropical cyclones. The random nature of directional seas has a significant influence on the observed maximum wave heights on the natural seabed and within a dredged basin. Data from a large 3D physical model, in conjunction with numerical modelling and literature, has been used to gain insight for practitioners into maximum wave heights for directional seas in finite water depth.

## PHYSICAL MODEL DESCRIPTION

The physical model data set described in Baker *et al.* (2019) details a large 3D 1:35 scale physical model with varying bathymetry and directional wave generation. Figure 1 presents a plan view of the wave basin as set up for these model tests. The data set from the model included non-directional wave measurements from 20 locations, and directional wave data from two locations, for over 60 tests representing wave conditions between typical annual wave heights (1-year ARI) up to cyclonic waves with an Average Recurrence Interval (ARI) of 1,000-years. By varying directional spread parameters, various directional wave conditions ranging from long-crested waves up to cyclonic seas with a directional spreading ($\sigma$, wrapped-normal) of 20-degrees were generated and measured. Baker *et al.* (2019) provides details on the wave absorption included in the model which was verified to absorb and dissipate over 95% of the incident wave energy over a broad range of water depths and wave periods. Partial wave absorption was also included along the side-walls of the model as shown in Figure 1.

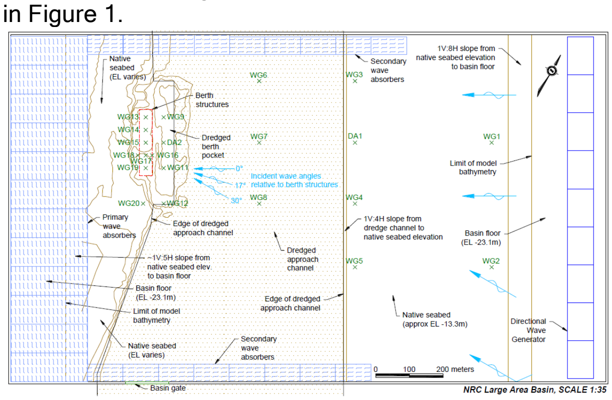

Figure 1 - Physical model basin layout (Baker *et al.*, 2019).

The model bathymetry indicated on Figure 1 featured a near constant depth seabed after the wave generation zone representing the study area where seabed slopes are in the order 1V:1000H. The model then transitioned over 1V:4H batter slopes into a near constant depth dredged navigation channel. Between the navigation and the wharf structure locations, a deeper berth pocket was represented in the model. Beyond the location of the wharf structures, the bathymetry transitioned to natural seabed depths and then into the wave energy absorption zone.

## WAVE TEST DATA

This paper presents model data from a series of tests that represented extreme cyclonic sea state conditions at the site for Average Recurrence Intervals (ARI) between 50 and 1000 years. The site is located in a coastal environment with a large tide range (up to 7 m) and there is a strong positive correlation between extreme wave conditions and water level. For this analysis, only wave directions incident (head-on) to the wharf structure and where the mean wave direction is normal to the wave generators on the right boundary of the model (see

Figure 1) have been presented.

A summary of the wave test conditions is presented in Table 1. The wave test data was obtained from a comprehensive cyclonic sea state study. The adopted directional spread was obtained from the analysis of modeled wave spectra and is consistent with the assessment of Young (2006) for directional spreading in cyclonic seas. Each sea state was generated for a 2-hours (prototype scale) duration and 2 repeat tests were completed to generate 6 hours of irregular, random wave data for each return period. In total, 15 tests have been analyzed to inform the results presented in the following section. The wave data has been analyzed based on their position in the model with respect to the bathymetry and the wharf structure. Table 2 summarizes the wave analysis locations and details the measurement locations (points) analyzed for each location.

Table 1 – Summary of wave test conditions.

| ARI (yr) | $H_{m0}$ (m) | WL (m) | $T_p$ (s) | Dir. Spr $\sigma_\theta$ (deg) |
|---|---|---|---|---|
| 50 | 5.2 | 6.7 | 8.5 | 20 |
| 100 | 5.6 | 7.0 | 9.0 | 20 |
| 200 | 6.0 | 7.2 | 9.4 | 20 |
| 500 | 6.7 | 7.4 | 10.0 | 20 |
| 1000 | 6.9 | 7.9 | 10.2 | 20 |

Table 2 – Summary of wave measurement locations.

| Location | Number of Pts | Point IDs | Depth (m) |
|---|---|---|---|
| Natural Seabed | 3 | WG3, DA1, WG4 | ≈13.6 |
| Dredged Channel | 3 | WG6, WG7, WG8 | ≈15.4 |
| Berth | 3 | WG9, DA2, WG11 | ≈16.6 |
| Wharf | 3-5 | WG13, WG15, WG19 (WG14, WG18) | ≈15 - 16.5 |

The wave conditions in all model tests were moderately nonlinear. Based on Goda's nonlinear wave parameter (Goda, 2000, Equation 9.132) all the wave data analyzed had a nonlinearity parameter value between 0.085 and 0.205, and the measured wave crest ratio ($\eta$/H) varied between 0.50 and 0.77.

## DATA ANALYSIS

A range of analyses were completed on the data set to provide insight into maximum wave conditions, and the relationship between maximum wave height and significant wave height for the locations identified in Table 2.

The variation in maximum wave height for each analysis region and each individual model test is presented in Figure 2. The random nature of the maximum wave heights is illustrated by the variance between each location in the model for individual tests, and between the various test data sets. Figure 3 presents a similar plot but with the maximum wave height on depth ($H_{max}$/depth) for each model test. There is no obvious correlation between the output locations and there is significant variation between model tests. During the experiments, significant wave breaking was observed in the natural seabed area between the wave generation zone and the dredged channel for the 500 and 1000 year ARI wave conditions. This is indicated by a general flatting of the $H_{max}$/depth ratio for the 500 and 1000 year ARI wave tests. The maximum $H_{max}$/depth ratios observed at all locations were between 0.59 and 0.62.

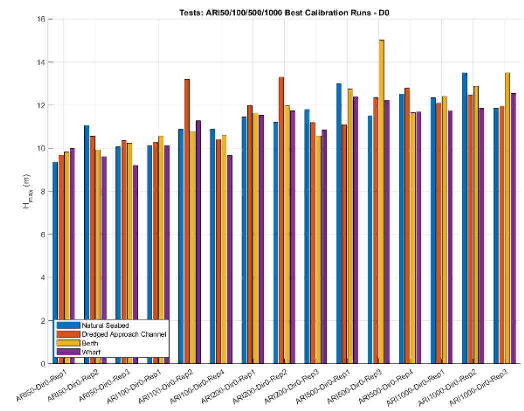


Figure 2 – Maximum measured wave height for each model test between 50 and 1000 years ARI.

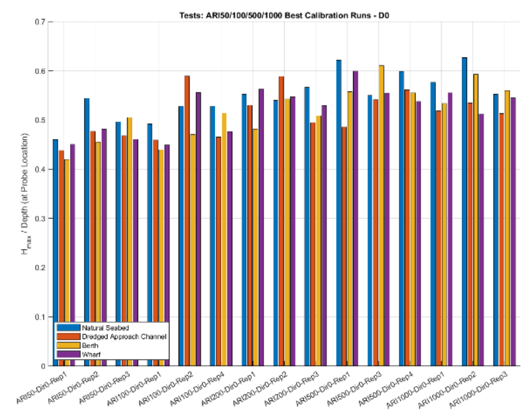


Figure 3 – Maximum wave height on depth ratio ($H_{max}$/depth) for each model test between 50 and 1000 years ARI.

The test data has been analyzed to calculate maximum wave height probability as a function of significant wave height ($H_{max}/H_{m0}$). Figure 4 presents cumulative probability functions using all of the test data, and also analyzed for each of the output locations defined in Table 2. The solid lines in Figure 4 represent all the data from the directional wave tests and the maximum wave height probability functions are similar for all sites. As a comparison between directional sea state conditions as defined in Table 1, with long crested wave conditions, Figure 5 includes a data sample from one test completed for 100 year ARI wave conditions with no directional spreading. The maximum wave height ratio for long crested waves is significantly lower than all the directional wave data which is consistent with established literature, including Goda (2000).

Further analysis of the wave height ratio was completed for the 5 measurement points located in the wharf location array. Figure 5 presents the maximum wave height probability from analysis of the peak maximum wave height from a combination of 1 to 5 measurement locations along the 152 m distance between points WG13 and WG18 (Figure 1). The results indicate that the maximum wave height probability curve for all the data collected at the 5 locations is consistent with the data measured at location WG13. As the number of locations used to calculate the maximum wave height in each 30 minute period of data increases, the maximum wave height probability function increases in likelihood of larger maximum wave heights, particularly between the 0 and 0.8 (0% to 80%) probability range.

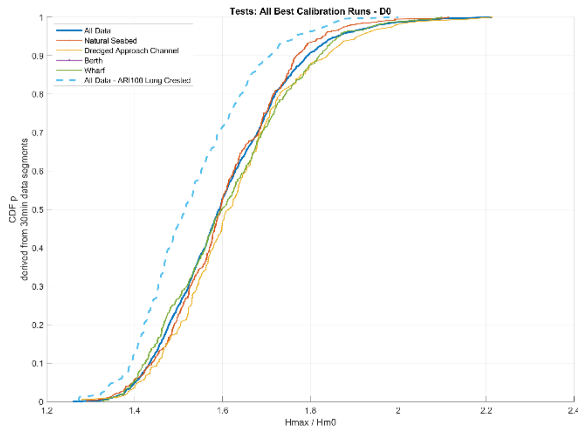


Figure 4 - Cumulative Probability Function (CDF) for maximum wave height to significant wave height ratio ($H_{max}$/ $H_{m0}$) for combined data between 50 and 1000 years.

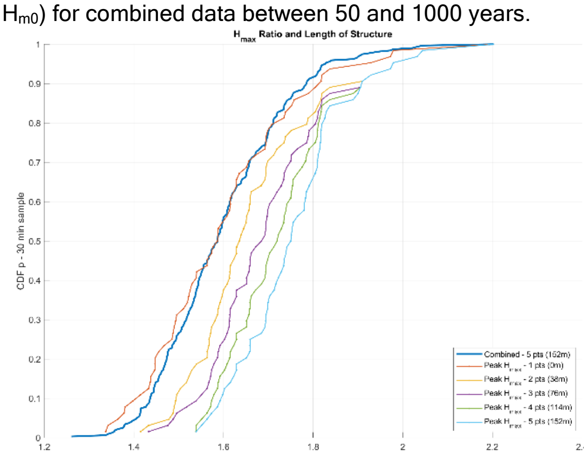


Figure 5 - Cumulative Probability Function (CDF) for maximum wave height to significant wave height ratio ($H_{max}$/$H_{m0}$) analyzed at a single location, and for combined locations along a nominal 150 m wharf structure.

## DISCUSSION

The wave data measured in this large scale 3D physical model provides significant insight into the characteristics of maximum wave conditions for directional seas in water depths of 15 to 25 m. The maximum wave heights observed in the Berth and Wharf locations are not significantly correlated with the largest waves occurring on the shallower seabed approaches. Results from the physical model were supplemented with numerical random sea state simulations undertaken using the WAFO wave analysis toolbox presented in Brodtkorb (2000) to confirm that there was no significant correlation between maximum wave heights in the natural seabed approach, and within the dredged areas near the berth and wharf structures for directional sea states.

The test data indicates that the maximum wave heights inside the dredged basin were limited by depth and wave steepness based on water depths within the dredged basin, rather than the wave height limitations associated with the shallower channel approaches. Based on all model tests, the peak H/depth ratios were between 0.59 and 0.62 of the local water depth, which is consistent with guidance provided in Forristall (2004) and data presented in Babinin *et al.* (2001) for shallow water directional waves. The data presented in this paper clearly indicates that for directional seas in shallow water, adopting the findings of Nelson (1994), based on a flat top reef where the H/depth limit for those wave conditions was 0.55, is non-conservative.

When the test data in this paper was applied to define cyclonic wave criteria for design, the maximum individual wave height along 150 m of wharf length was approximately 0.8 m (6 to 7%) larger at the 500-year ARI return period than calculated for a single location. A similar trend was observed for wave crest elevation. For structures that are sensitive to wave loads at specific elevations (above still water) at any location along the structure, the maximum wave height and wave crest level should consider the integrated probability of extreme waves along the length of structure.

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