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Evaluation of a proposed solution to beach erosion at Negril

ELISABETH A. MONDON[1] AND PHILIP S. WARNER[2]

[1]*M.Sc Coastal Engineering, Smith Warner International Ltd. Email: elisabeth@smithwarner.com*
[2]*Principal, Smith Warner International Ltd. Email: philip@smithwarner.com*

ABSTRACT. Beach erosion has been a significant factor affecting Negril for many years and this has prompted the development of engineering solutions that address the problem. One proposed solution sought to increase the stability of the beach by constructing a series of submerged breakwaters. This paper presents an evaluation of this solution using the MIKE21 coastal process model to simulate the short-term beach response to a swell event. The model works in a morphological manner as it is able to simulate the real beach response by continuously updating the seabed topography, which has been modified by locally computed sediment transport rates. Using an observed swell event that affected the north coast of Jamaica in November 2006 the model performance was tested and it was found to correctly predict the extent and magnitude of observed beach erosion. The model was then used to find out what would have happened had the proposed breakwaters been installed. The findings demonstrate that the wave and current sheltering effects in the lee of the breakwaters result in a significant decrease in the alongshore sediment transport rates at the shoreline. The breakwaters were found to reduce the wave heights and nearshore current speeds by approximately 30% and beach erosion was reduced by up to 50% in the most vulnerable sections of Negril.

Key words: Negril, Jamaica, beach erosion, MIKE21 model.

1. INTRODUCTION

The problem of coastal erosion along the beaches of Negril has received much media exposure and has been documented in “Beach Restoration Works at Negril” (SWIL, 2007). There are believed to be several factors that have contributed to the problem, such as a series of damaging hurricanes and severe swell events, as well as the construction of seawalls and other infrastructure close to, or at, the water’s edge. Those factors have a direct impact on the beaches by moving sediment or altering the pathways. In addition, declining water quality can affect sea grass and coral reef health consequently diminishing sand production and the natural protection of the shoreline against wave attack, exacerbating beach erosion problems. While coastal zone management seeks to address these wider issues in the long term, a medium term solution to the beach erosion problem is required.

The beach is typically sheltered from the Trade Winds, but when it is exposed to passing storms, the beach exhibits a noticeable response. In a typical cycle, the beach width is significantly reduced over a one or two day period as sand is moved by waves and currents, but the lost beach width is slowly restored several weeks later. Immediately following the passage of a storm, infrastructure is exposed and becomes damaged and the normal beach activities are hampered.

In order to address this problem, extensive investigative work has been carried out for the Negril Coral Reef Preservation Society in a recent preliminary engineering study entitled “Beach Restoration Works at Negril” (SWIL, 2007). Part of the work applied detailed engineering analyses to understand the prevailing coastal processes and to develop engineering solutions that address the problem of beach erosion along the shoreline. The most technically viable, least-cost solution to the problem included a first stage of beach nourishment in tandem with the implementation of nine coastal protection structures to provide a sustainable long-term solution.

The geometric details of the proposed coastal protective structures include a crest elevation 0.7 m below MSL, a crest width of 10 m and side slopes of 1:2 (V:H). A total of 3600 m of submerged structures were proposed to be built for Long Bay, equal to approximately 50% of the total shoreline length. At the south end of Long Bay (**Figure 1D**), the breakwaters are close to the shoreline, whereas in the central and northern zones (**Figure 1B-C**), they are located farther offshore and function as reef reinforcement.

As waves and currents are the main driving force of sediment movement in the nearshore zone, any solution to the ongoing beach erosion must reduce and redirect these forces to ensure the long term stability of the beach along the shoreline. The

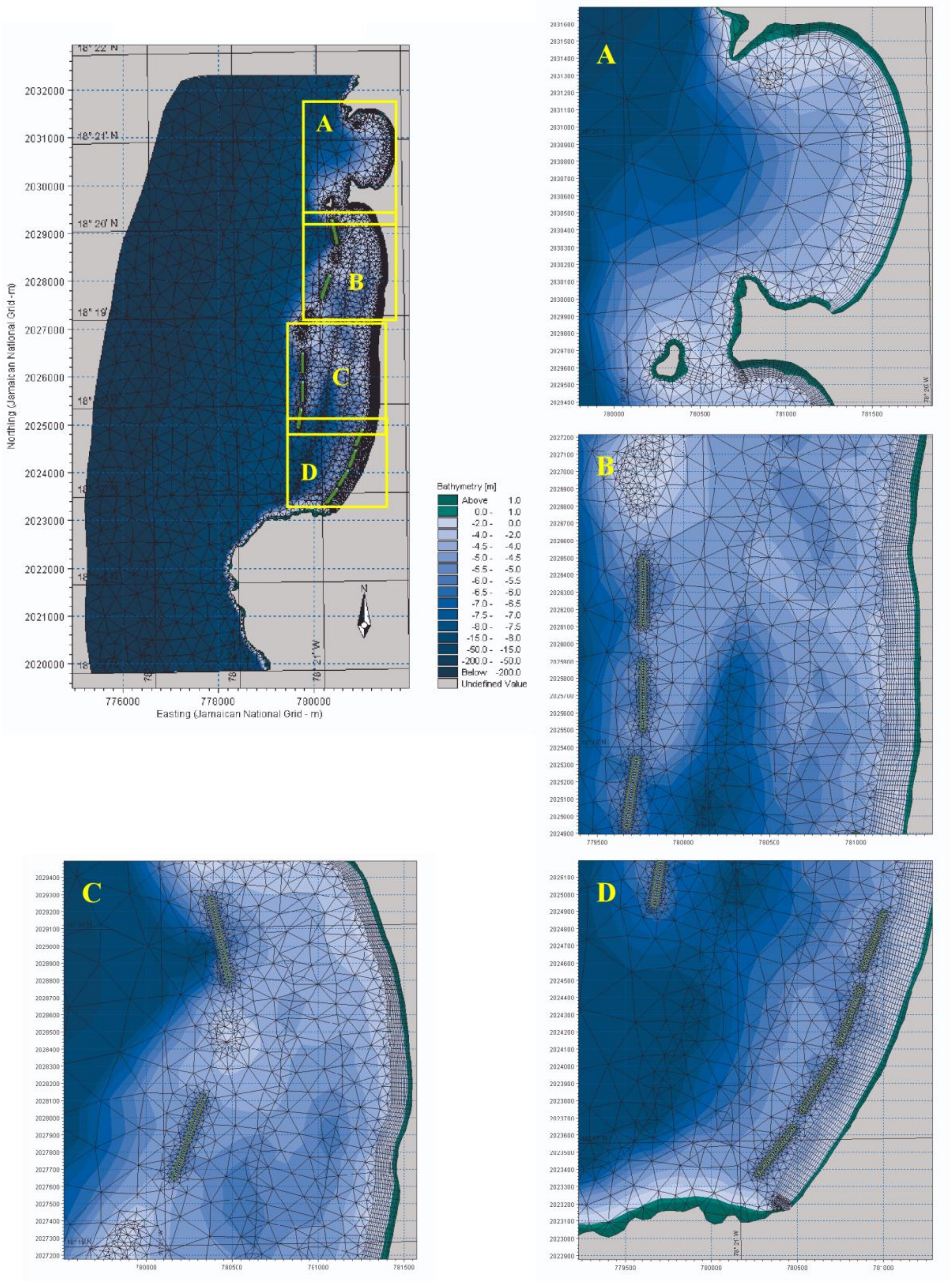

Figure 1. Flexible mesh with proposed breakwaters.

proposed coastal protection structures, or breakwaters must therefore be designed to provide shelter against wave action to the shoreline of Long Bay. This paper presents an evaluation of the proposed nearshore protective breakwaters using the MIKE21 modelling suite (DHI, 2009a, b, c). The model was used to simulate the short term beach response to a swell event in order to evaluate the effectiveness of the proposed solution in reducing fluctuations in beach width.

2. METHOD

The method used to evaluate the effectiveness of the proposed breakwaters consists of using a computer model of the study area to compare the modelled beach response to a significant storm event with and without the breakwaters. In order to determine if model was functioning properly it was tested and calibrated to ensure that the predicted and real beach responses match. After the model was verified, it was reconfigured to represent the proposed breakwaters and re-run so that comparison of the beach response between the existing conditions and proposed solution could be made.

The basic starting point of the model was the creation of a computational mesh where waves, currents, sediment transport rates and the resulting morphological changes could be determined at each simulation time step. This mesh encompasses the study area and defines the water depth and sediment thickness at a series of connected points.

Input boundary conditions, including waves and water levels, are the principal driving forces that govern sediment transport and morphology and must be derived for a beach response simulation and must also be verified.

For this application MIKE21 utilized three main functional modules; Spectral Wave (SW), Hydrodynamic (HD), and Sediment Transport (ST). These operate in a coupled mode, whereby results from one module are used by the others in order to improve the efficiency and accuracy. The SW module computes the wave conditions throughout the model domain; the (HD) module computes the water levels and current speeds, and is coupled with the SW module using radiation stress so that wave-induced currents are included. Water levels and currents affect waves and these values are therefore available to the SW module to improve its accuracy. The Sediment Transport (ST) module uses the results of the SW and HD modules to compute alongshore and cross shore transport rates. The ST model modifies the seabed depths based on the computed sediment transport rates and this modified seabed is used in subsequent time steps by the SW and HD modules to compute the wave conditions and current patterns.

2.1. Flexible Mesh

The MIKE21 model uses a flexible mesh, which represents the seabed using a series of connected triangular and quadrangular elements. Quadrangular elements are preferred in areas where significant morphological changes are expected to occur, as the gradients in sediment transport can be more accurately represented, (DHI, 2008). Along the Negril beach, the nearshore zone is known to be very active from a morphological perspective (Department of Geology and Geography, 2002). In order to adequately and accurately represent the bathymetry in this area, detailed model resolution using quadrangular elements was required, as shown in **Figure 1**.

Very small mesh elements were used in order to properly represent the breakwater structures. For comparison purposes, both the existing and the proposed solution for were modeled using the same mesh. However, for the existing conditions, the water depths in this zone were set to the existing depths; for the proposed solution, the water depths were altered to represent the correct dimensions of the breakwaters.

2.2. Boundary Conditions

Two main time-varying boundary conditions were required for this simulation; wave conditions and water levels. Archived model results from the National Oceanic and Atmospheric Administration (NOAA) WAVEWATCH III (WW3) deep-water global wave model were used to define the wave boundary values, and water levels were obtained from a global tide prediction model.

The NOAA WW3 is a third generation spectral wind-wave model developed by NOAA and is primarily used to create global wave forecasts (Tolman, 2002). In addition, a public domain hindcast archive is available, which includes the integrated spectral parameters of significant wave height (Hs), peak wave period (Tp) and mean wave direction, as well as wind speed and direction at three-hourly intervals. This archive is freely available to the public and includes data from July 1999 to November 2007. Figure 2 shows the WW3 archive results and the directional distribution of wave heights around Jamaica. Node 13 was used as the input for these investigations at Negril. Results were extracted for the period of October to November 2006 which included a swell event that occurred from the 21st to the 23rd of November 2006 and coincided with a one-month period of nearshore wave measurement that was used for verification purposes.

The global tide prediction model is part of MIKE21 and is based on the superposition of numerous sinusoidal tidal constituents, each with an amplitude and phase lag that varies depending on time and position (DHI, 2009d). The predicted tide levels for the simulation period are shown in **Figure 3**.

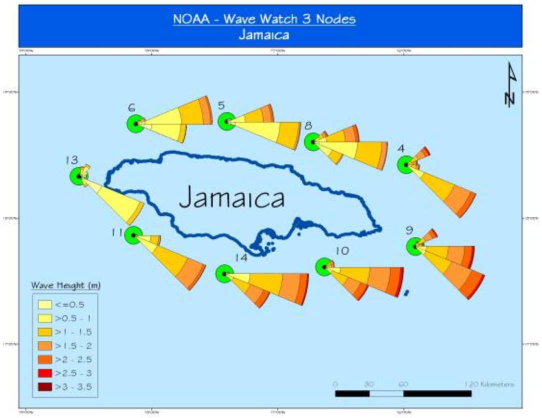

Figure 2. Deep Water Wave Distribution from NOAA WW3 historical archive.

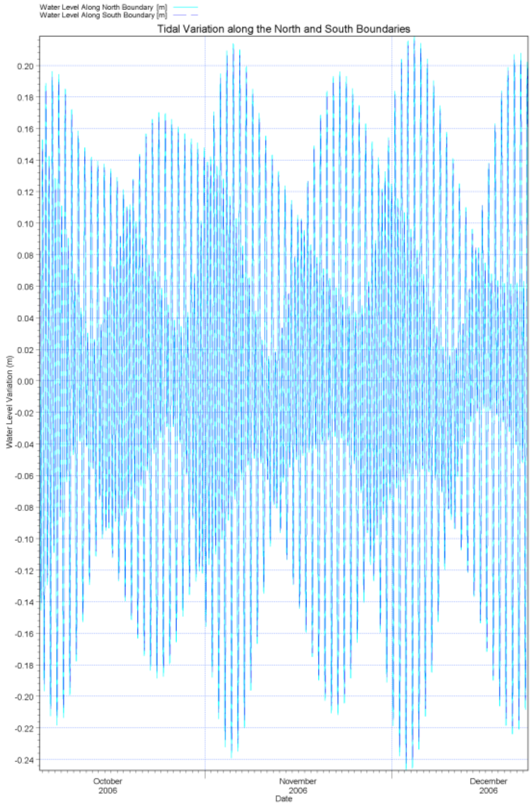

Figure 3. Tidal Levels used at the north and south boundaries of the model.

2.3 MIKE21 Model Setup

The MIKE21 SW module was run in a semi-stationary mode, with time varying inputs of integrated spectral parameters Hs, Tp and mean direction which were taken from the NOAA WW3 historical archive. These values were applied along the north, west and south model boundaries, as shown in Figure 4, with a time step of 30 minutes over the three day simulation period. Wind was not applied to the model due to the relatively small model domain and the fact that the study area lies within the predominant wind shadow zone of Jamaica. The wave breaking coefficient (Battjes and Janssen, 1978) was set to 0.8 and this produced the beach response that most closely matched what was observed on site.

The HD model computes currents that are forced by both waves and water level variations. Tidal height variations, including Coriolis corrections, were applied along the north and south boundaries of the model, as shown in **Figure 4**. Wave-induced currents, which arise as waves break and dissipate energy, are included in the HD calculations as these forces are passed from the SW module. Bed resistance was defined using a Manning coefficient (Talmon et al., 1995), and turbulence was introduced using the Smagorinsky eddy viscosity model (Smagorinsky, 1963).

The morphological model takes results from the SW and HD modules and computes the sediment transport at each point in the computational mesh. Sediment transport rates are derived by linear interpolation from a set of a pre-calculated sediment transport tables, which are used to improve the model efficiency. These tables are generated beforehand and must be configured such that all of the combinations of bathymetry, current, wave and sediment conditions possible in the simulation are within the range defined in the transport table. These transport tables were calculated using the Stokes 1st order wave theory (Fenton, 1985; Isobe and Horikawa, 1982; Doering and Bowen, 1995), which was found to be the best method to accurately reproduce the wave-induced near-bed velocities both in the shoaling and the surf zone. In the shoaling region, wave asymmetry results in onshore directed net sediment transport, which is typically small. In the surf zone, wave breaking and the associated undertow are the dominant mechanisms, which in most cases results in offshore directed cross-shore sediment transport.

A mean grain size diameter of 0.25mm and a grading coefficient of 1.1 were taken as constants for the sediment properties. An initial sediment layer thickness of 2m was used in the locations assumed to contribute to the sediment transport, such as the project shoreline and adjacent beaches. Areas outside this had an initial sediment thickness of 0.0, simulating a reef or seagrass substrate.

3. CALIBRATION

During November 2006 a moderate swell event passed north of Jamaica, resulting in noticeable beach erosion, particularly near the mouth of the Negril River. A wave recorder deployed just

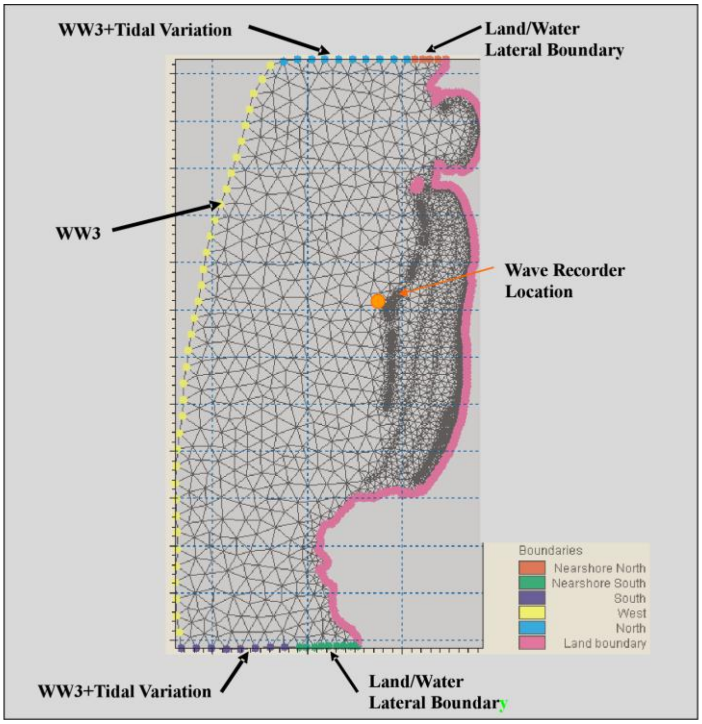

Figure 4. Model Domain and Boundary.

offshore of the existing reef at Long Bay as shown in Figure 4, was able to measure the nearshore wave conditions. Before and after photographs were used to evaluate the extent of the beach response and beach profile measurements quantified the depth of erosion.

3.1 Calibration of the spectral wave model using record of wave measurements

The first step in the calibration process involved verifying the spectral wave module. An InterOcean S4ADW current meter, which measures waves, currents and tide data, was deployed on the seabed in the nearshore area of Long Bay, Negril, in approximately 8 m water depth. The instrument was deployed on 20 October 2006 and was retrieved 28 November 2006, a period of five weeks, including the November swell event that resulted in appreciable beach erosion. The MIKE21 spectral wave model was run in a semi-stationary mode using the time-varying inputs of deepwater WW3 data along the north, west and south boundaries. Semi-stationary mode is appropriate for simulations of this spatial scale where wave conditions during each time-step are not dependant on prior wave conditions. At the location of instrument within the model, the computed time series of wave height, peak period and mean wave direction values was extracted. This allowed a comparison to be made between the model and measured values in order to validate both the model set-up and input boundary conditions. **Figure 5** shows the time-series comparisons of the measured and modeled wave heights (A) peak periods (B) and mean wave directions (C) at the location of the nearshore measurements, along with the input wave boundary values.

The figure clearly shows the increase in wave height (A) during the swell event from the November 21st to 23rd. The largest measured wave height (dark blue) at this location had a value of 2.91 m and occurred on November 21st, 2006. Figure 5 also shows that the WW3 input on November 21st is smaller (2.4 m shown in cyan) and occurs with a phase lag from the measured peak. For the peak event the modelled wave height is smaller (2.1 m shown in red) than the measured wave height. However, a second swell peak that occurs a few hours later compares better with the measured height being 2.5 m and the corresponding model result at 2.3 m. Overall the swell event appears to be well represented in the WW3 input, and the model produces reasonably accurate wave heights at the measurement site. The time signals generally appeared to be similar, with coincident periods of high and low wave heights.

The model wave periods (B) were within the range of the measured wave periods, although the overall trend was for under-prediction. The predicted wave periods range from 4 to 8 s on average compared to measured periods ranging on average from 3 to 9 s. The long period swells that arrived before the peak of the storm (18 November)

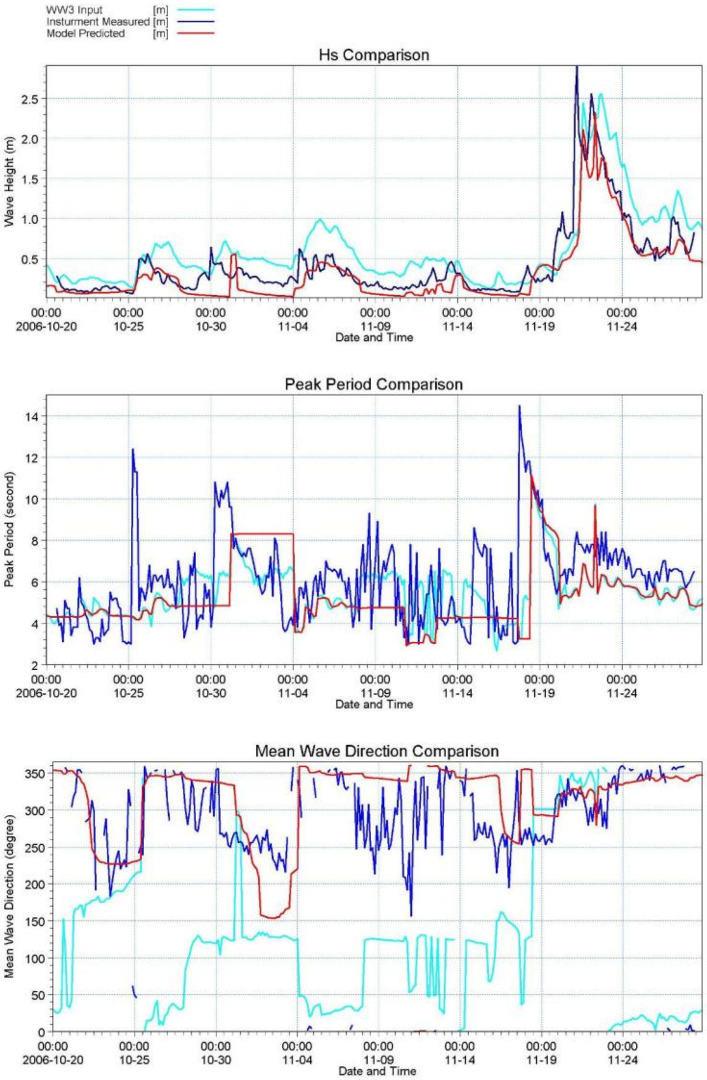

Figure 5. Time Series Comparison of Input boundary values, measured and predicted wave components.

were present in the WW3 data and were predicted by the model. The modelled waves were slightly shorter in period than the measured (11 versus 14 seconds) and arrived approximately 12 hours later. However, as with wave height, the time signals are generally similar, with coincident episodes of high and low wave periods.

Modelled and measured wave directions (C) also fell within the same quadrant (from southwest to northwest sectors). There appears to be a connection between the WW3 input wave direction and the correlation of the measured and modelled wave heights. For example, when the WW3 wave direction is between 50 and 150 degrees, the modelled wave heights are much smaller than the measured wave heights. Conversely, when the input wave direction is less than 50 degrees or greater than 200 degrees, the correlation between the measured and modelled wave heights is improved. This can be seen before and after 4th November 2006; for the two days prior, the wave angle is approximately 140 degrees and the modelled wave heights are substantially smaller than the measured heights. Immediately after 4th November 2006, the wave angle shifts to 40 degrees and the modelled and measured wave heights correlate much better. However, this only appears to be significant for the smaller wave heights, while the larger waves that affect sediment transport and beach morphology are well-represented. Overall there is reasonably good correlation between the measured and predicted results.

3.2 Calibration of the morphological model using nearshore photographs

Another aspect of the model calibration involved a comparison of the erosion observed on site with that computed by MIKE21. Observations and photographic comparisons made before, during and after the swell event from 21-24 November 2006 showed that the middle section of Long Bay did not experience much erosion during the storm, while in the northern and southern sections of the bay a significant amount of erosion occurred. **Figure 6** shows the magnitude of erosion seen at the southern end of Long Bay near the river outlet (approximately 1.5 m). However, the erosion that took place near the mouth of the Negril River was probably localized and magnified due to reflection and scour at the existing groyne located along the entrance channel of the Negril River.

Erosion observed further away from the river mouth was approximately 0.6 m in the southern end of Long Bay as shown in **Figure 7**.

The numerical model was calibrated so that the computed erosion levels were similar to that observed along different sections of the shoreline. Key model parameters, such as the wave theory used in the sediment transport tables, the sediment grain size, as well as the wave breaking coefficient were adjusted to achieve the best match between the modelled and observed erosion.

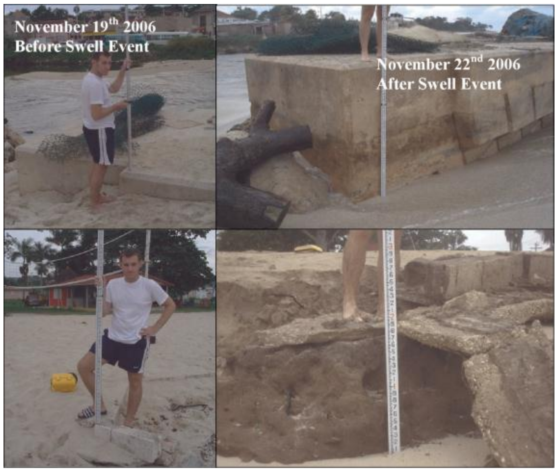

Figure 6. Localized erosion at the Negril river groyne and southern end of Long Bay.

Figure 7. Observed erosion of approximately 0.6m in the southern section of Long Bay .

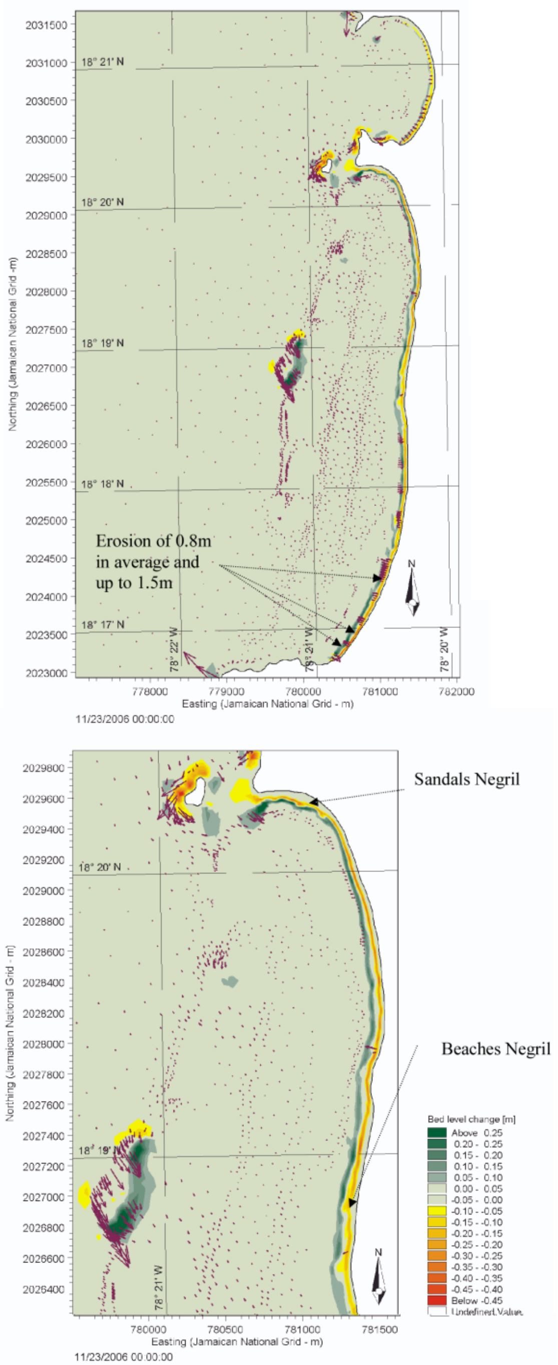

Figure 8. MIKE21 predicted bed level change with details along Beaches and Sandals Negril.

The morphological model predicted erosion up to 0.8m at the southern end of Long Bay near the river entrance as shown in **Figure 8**. In this area, erosion of up to 1.5m was observed. In the central sections and towards the north end of Long Bay, the model showed variations in beach erosion similar to what was observed. For example, at Beaches Negril, the model predicted the smallest bed level change of 0.15 m, and this corresponded to an area where only

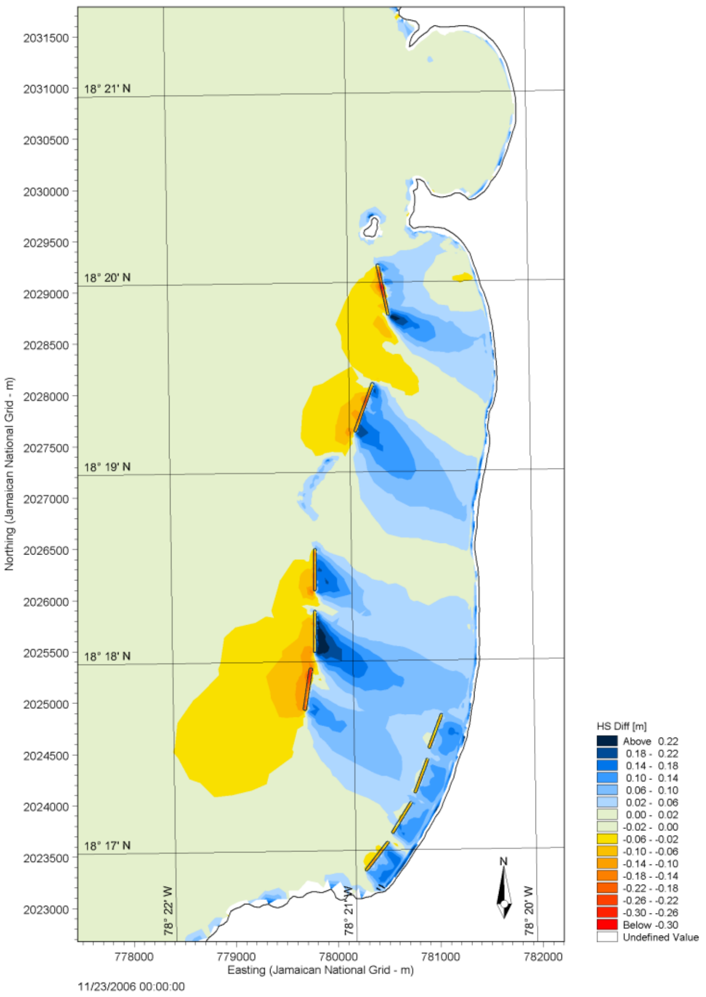

Figure 9. Wave sheltering effect of proposed breakwaters.

minor beach erosion was observed. Further north, at Sandals Negril, the predicted bed level changes were greater (0.30 m), and observations indicated that this was an area that experienced erosion during the swell event.

Only photo observations were made before and after the swell event and therefore the extent and magnitude of beach erosion had to be estimated, rather than measured. Overall, however, the predicted bed level changes were considered to be reasonably close to the observed values along the shore.

The model settings and physical parameters that were established during this calibration procedure were therefore suitable to be used in the detailed beach response modelling for the proposed mitigation strategies. Using this calibrated model, the impact of the proposed solutions on the beach during a swell event was investigated with confidence.

4. RESULTS

4.1 Wave Sheltering results

The principal function of the offshore breakwaters is to provide wave sheltering and thereby stabilize the beach. Figure 9 displays the sheltering effect of the proposed breakwaters by computing the difference between the wave heights with and without the proposed structures. It shows that the breakwaters attenuate the waves by 0.1 to 0.3m in their lee and provide a sheltered area between them and the shoreline.

Figure 10 represents the maximum computed wave height during the three day period for both existing (L) and proposed (R) scenarios. The wave height of 1.4 - 1.2 m seaward of the proposed breakwaters is reduced from 1.1 m to 0.8 m in their lee, which demonstrates the effectiveness of the proposed breakwaters in reducing wave heights by at least 30%. It can be seen that waves penetrate more in the southern section of Long Bay and reach heights from 1.6 up to 1.8 m before they interact with the southern nearshore breakwaters.

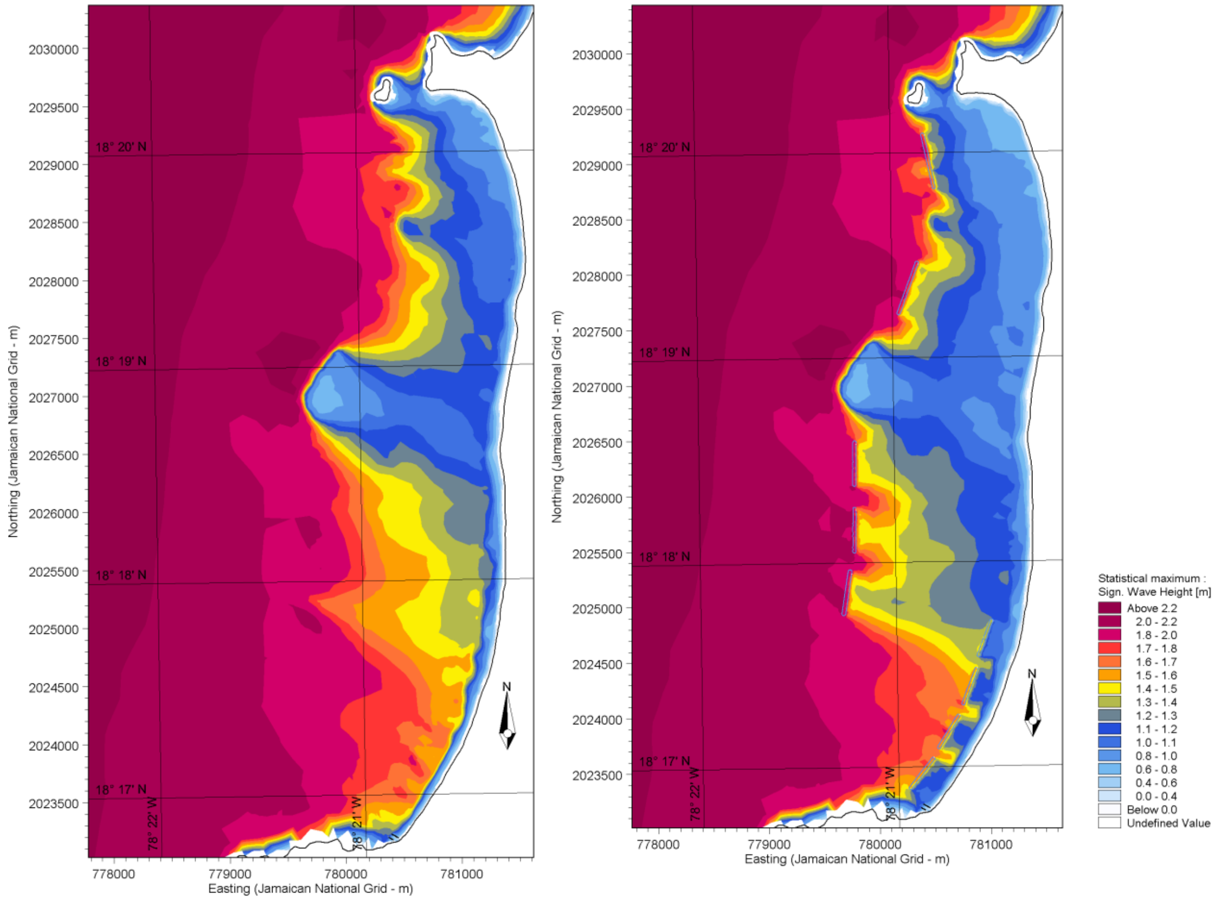

Figure 10. Maximum computed wave heights for the existing conditions (L) and proposed solution (R).

4.3 Hydrodynamic results

Figure 11 shows the resulting current speeds and directions within Long Bay for the existing (L) and proposed solution (R) on 22 November 2006 at 6pm, which corresponds to the peak of the swell event. The figure depicts the currents flowing toward the south with speeds of 0.1 to 0.4 m/s, an average of 0.2 m/s along the shore and higher than 0.4 m/s along the existing reefs at the north end on Long Bay. There appears to be high current speeds in the immediate vicinity of each proposed breakwater. At the southern end of the bay, the proposed breakwaters appear to cause the currents to increase in the gaps between the breakwaters, however, adjacent to the shoreline, the area of high current speeds (up to 0.25 m/s) has been almost eliminated.

The maximum computed currents over the three-day simulation period are presented in Figure 12 with comparisons of existing conditions (L) and proposed solution (R). Results show current speeds exceeding 0.7 m/s in the area of the existing reefs. The currents between the existing reefs and the shoreline are approximately 0.15 m/s in the northern part of Long Bay and 0.5 m/s in the south. With the breakwaters in place, maximum currents are increased by 0.2 m/s (from 0.15 to 0.35) between the breakwaters but decreased by 0.2m/s in their lee. Alongshore currents are approximately 0.1 to 0.35 m/s and with the breakwaters in place these speeds are reduced by 0.1 m/s in northern part of Long Bay and 0.15 m/s in the south. This demonstrates the effectiveness of the proposed breakwater in reducing maximum alongshore current speeds.

In the southern section of Long Bay, offshore of the proposed breakwaters, the maximum currents are predicted to increase. In this area the current speeds increase from 0.2 to 0.4 m/s for the proposed solution, when compared to the existing. The impacts of this increase in current speeds are not anticipated to be detrimental to the environment, as it is generally a decrease in current speeds that leads to stagnation and environmental degradation, whereas stronger currents are normally associated with increased biodiversity and improved coral health.

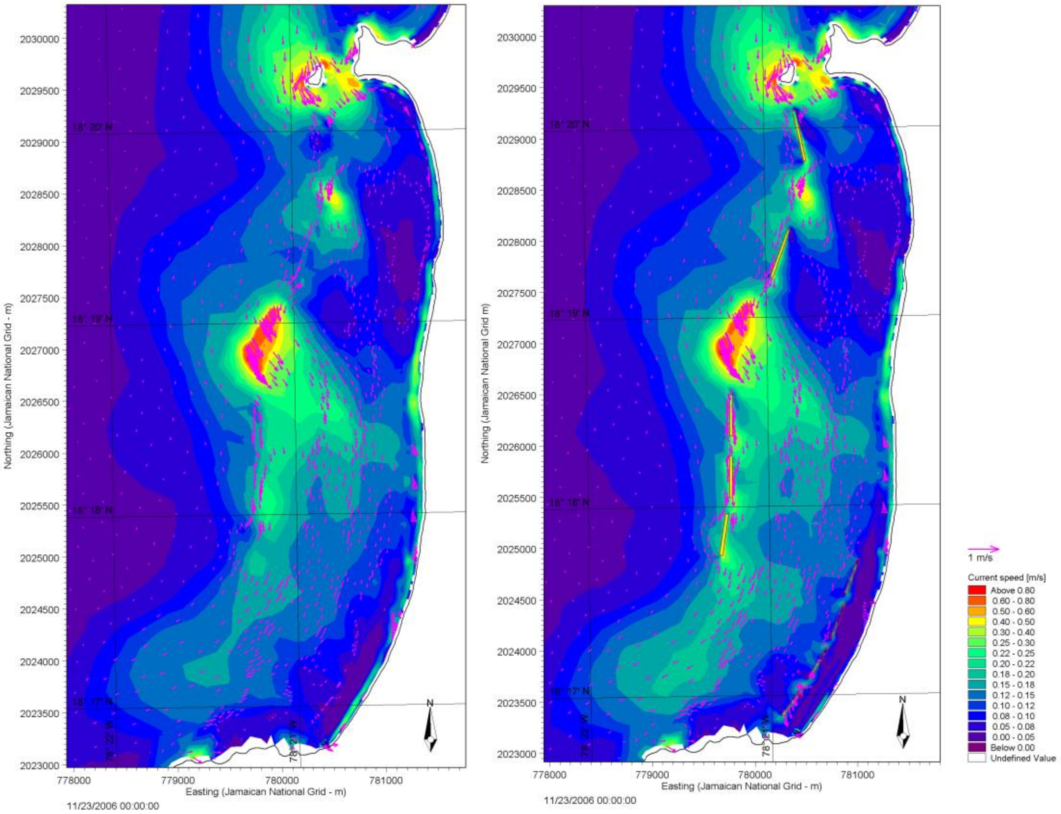

Figure 11. Current speeds and directions at the swell peak for the existing conditions (L) and proposed solution (R).

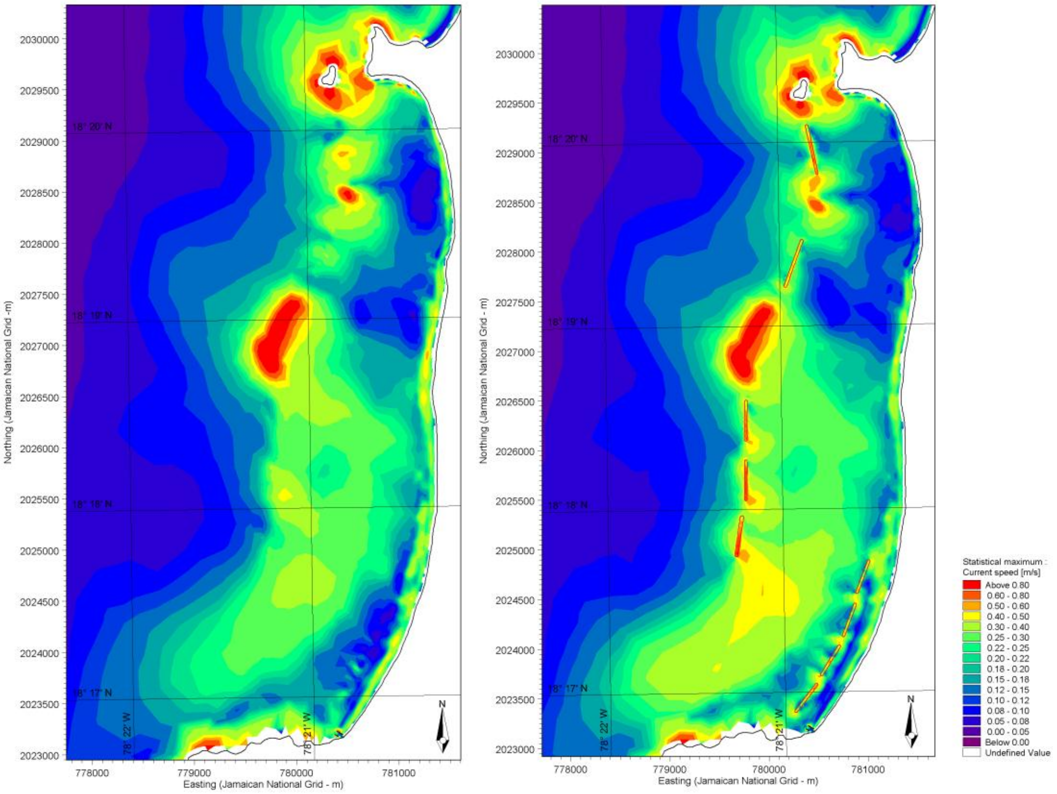

Figure 12. Maximum current speed over the 3 days simulation for the existing conditions (L) and proposed solution (R).

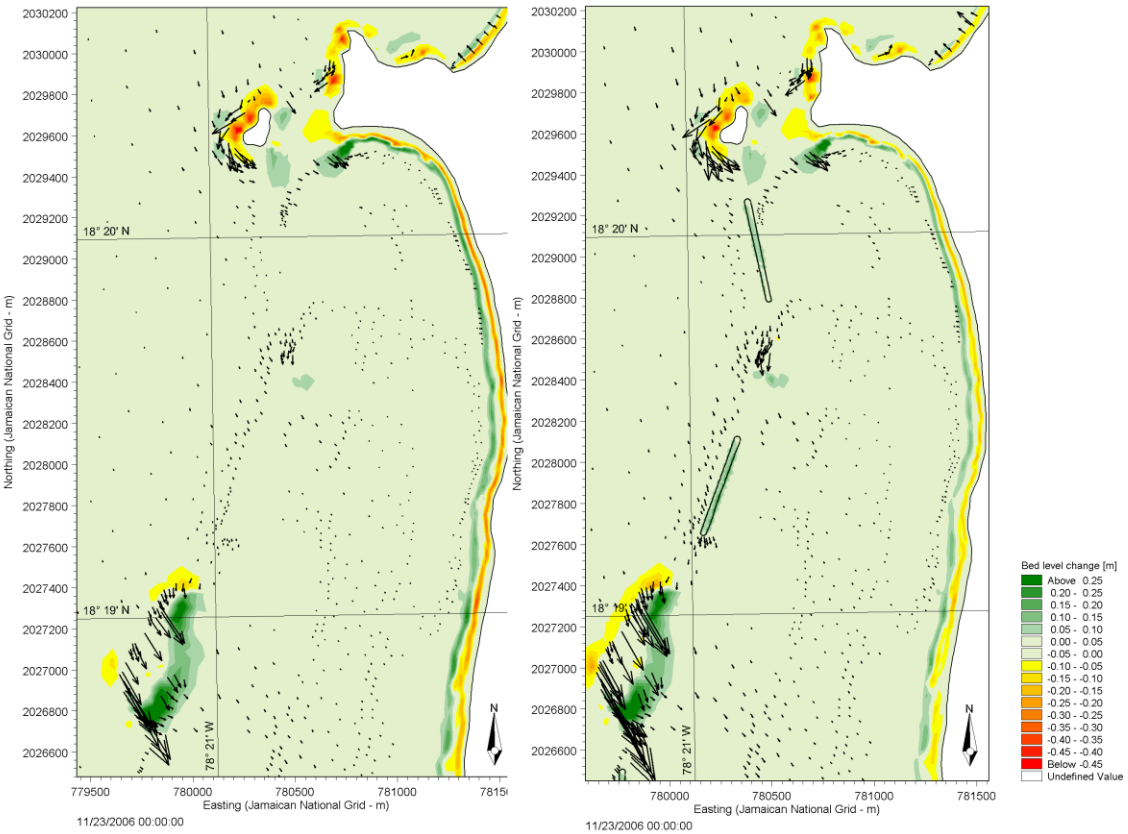

Figure 13. Bed level change and sediment transport direction north of Long Bay over the three days period, with existing conditions (L) and proposed solution (R).

4.4 Bed Level Changes

Figure 13 and **Figure 14** show the resulting bed level changes and the magnitude and direction of the sediment transport at the end of the swell event on 23 November 2006 for the north and south portions of Long Bay respectively, with comparison between the existing (L) and proposed (R) scenarios. In a general way, it can be seen that by the end of the swell event, sediment has been eroded from the high berm and has been deposited seaward of the beach slope (shown as adjacent red – erosion and green - deposition). Along almost all of Long Bay the high berm was eroded by about 0.25 m. This sediment was deposited down the beach slope further offshore, causing the entire beach profile to flatten. With the proposed breakwaters in place, the mechanism of erosion/deposition still occurs but the magnitude of erosion has been reduced by approximately half along most of the shoreline. Erosion values are no higher than 0.15 m with a maximum of 0.25 m south of Long Bay near the river entrance structures.

The magnitude and direction of sediment transport give an indication of the intensity and locations where sediment would be moving in Long Bay, and therefore identify important sediment pathways. An important source of sand appears to be the reef area at the north end of Long Bay. While the other reefs do contribute sand, the pathway of sediment to the beach does not appear to be continuous. During this swell event, the principal movement of sediment is toward the south along the shoreline with replenishment coming from offshore into the nearshore zone. The movement of sand is often seaward at the shoreline, which explains why in **Figure 13** and **Figure 14** the sediment from the high berm of the beach is being eroded and deposited further offshore forming a sand bar. With the breakwaters in place, the intensity of the potential sediment movement is decreased but the pathways and trend of sand movement remains the same.

1. DISCUSSION

The MIKE21 coupled spectral wave, hydrodynamic and morphological model was used to examine the shoreline response in existing conditions and determine the effectiveness of several shoreline protection structures, including the nearshore submerged breakwaters and offshore reef reinforcement, in reducing the beach erosion during a swell event.

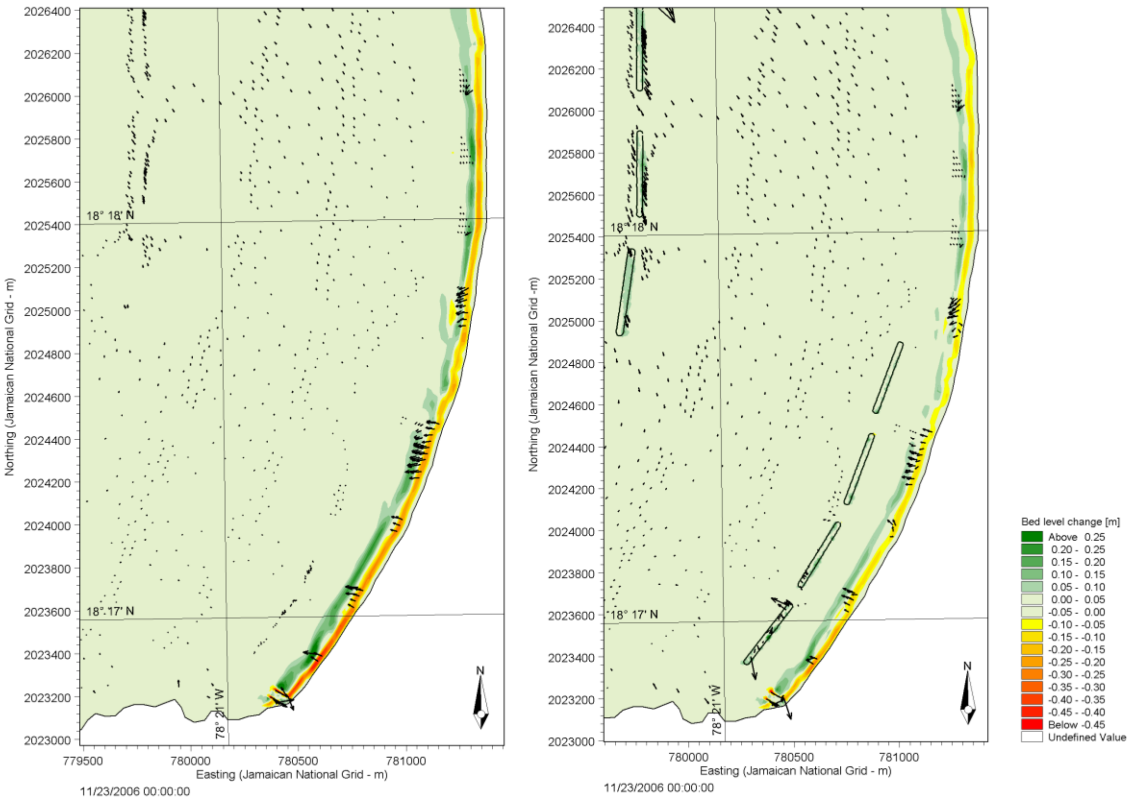

Figure 14. Bed level change and sediment transport direction south of Long Bay over the three days period, with existing conditions (L), and proposed solution (R).

The modelled event represents the erosion portion of the beach cycle, and does not include the natural beach rebuilding process. The coupled model computes wave heights, current speed and direction along the shoreline as well as the rates and direction of the sediment transport and finally calculates the bed level changes within the model domain. The bed level changes are used to update the nearshore topography, which in turn affects the wave heights, current speeds and subsequent sediment transport rates. Thus the model is able to realistically simulate the factors affecting beach morphology.

The model results have helped in determining critical spots in the design where scouring and/or deposition could be expected and these will require special attention during a final design stage. It also aided in predicting the likely impact of the proposed solution on the shoreline of Long Bay

It was found that in a general sense, the swell event caused sediment to be transported to the south and eroded from the high berm of the beach to be deposited seaward of the beach slope. The comparison of the results between the existing and proposed solution revealed that the breakwaters reduced the wave heights in their lee by 0.1 to 0.3 m, and in some instances up to a 30% reduction in wave height was achieved, as shown in **Figure 9** and **Figure 10**. The breakwaters efficiently reduced nearshore current speeds in their lee, as shown in **Figure 11** and **Figure 12**. The breakwaters reduced the erosion by up to 50% along the shore and especially in the southern sections of Long Bay where the erosion was at its maximum. As shown in **Figure 8**, **Figure 13** and **Figure 14**, the maximum erosion of 0.45 m is reduced to 0.2 m in the southern portion of Long Bay. With the breakwater in place, erosion values are no higher than 0.15 m throughout Long Bay, whereas without the breakwaters, erosion was typically 0.25 to 0.30 m.

On the other hand it was found that the proposed solution could result in wave heights and current speeds reaching higher values on the southern section of Long Bay seaward of the proposed breakwaters as shown in **Figure 9**, **Figure 10**, **Figure 11** and **Figure 12**. This is due to the fact that the breakwaters are in the very nearshore along the southern section of Long Bay and they block the nearshore currents, thereby increasing the speeds offshore.

1. CONCLUSION

In order to evaluate the effectiveness of the proposed solution to the beach erosion problems at

Negril, computer simulated beach response modelling was carried out. Using a swell event that occurred in November 2006, the waves, hydrodynamics and sediment transport were simulated using the numerical model for both the existing conditions and the proposed solution. The model was calibrated so that the predicted beach response was reasonably close to what was observed and photographed. It was found that the model was able to realistically simulate the beach morphology. The model was then configured to predict what would have happened during that same swell event, had the proposed solution been built.

In a general way, the model revealed significant decreases in the alongshore sediment transport rates at the existing shoreline in the lee of the proposed breakwaters due to wave and current sheltering effects of these structures. It can be concluded that the proposed breakwaters will be beneficial to combat beach erosion along the shoreline of Long Bay.

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