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Publisher’s version / Version de l'éditeur:

Proceedings of the 6th International Conference on the Application of Physical Modelling in Coastal and Port Engineering and Science (Coastlab16), 2016-05

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Ottawa, Canada, May 10-13, 2016

PHYSICAL MODELLING OF THE LAKEVIEW WATERFRONT CONNECTION PROJECT

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ABSTRACT

The Lakeview Waterfront Connection (LWC) project seeks to create a new natural park that will establish ecological habitat and public linkages on the eastern Mississauga waterfront of Lake Ontario through the innovative use of clean fill from regional infrastructure upgrades. This paper describes a 1:35 scale physical modelling study that was commissioned to investigate the interaction of moderate and extreme waves with the proposed LWC conceptual design, to help optimize the design of the project including the beach fill and the rubble-mound structures, and verify the stability and response of the headland-beach system under a range of design conditions, including storms approaching from easterly and southerly directions, at average and high lake levels.

KEYWORDS: physical modelling, shoreline protection, headland-beach system, cobble beach, design optimization

1 PROJECT BACKGROUND

Current and past land utilization has degraded the quality of terrestrial and aquatic habitat along the shoreline of eastern Mississauga, located on Lake Ontario approximately 15 km west of Toronto. Additionally, public access to and along the waterfront has been impeded by security restrictions associated with the regional wastewater treatment plant and a former power generating station. The Lakeview Waterfront Connection (LWC) project seeks to create a new natural park that will establish ecological habitat and public linkages on the eastern Mississauga waterfront through the innovative use of clean fill from regional infrastructure upgrades. Key objectives for the project include establishing a diverse range of native terrestrial and aquatic ecosystem habitats, safe and accessible public linkages for access to and along the waterfront, ensuring the project is compatible with existing infrastructure, coordination with other local planning and development initiatives, and an innovative funding approach that maximizes public benefit and value by reusing locally generated fill from existing regional capital works projects.

1.1 SITE CONDITIONS

Short-term water level fluctuations typically last from less than an hour up to several days, and are usually caused by local storm events where barometric pressure differences and surface wind stresses cause temporary imbalances in water levels at different locations on the lake. These storm surges, or wind-setup, are most noticeable at the ends of Lake Ontario, particularly when the wind blows down the length of the Lake. Seasonal fluctuations reflect the annual hydrologic cycle which is characterized by higher lake levels during the spring and early part of summer, and lower levels during the remainder of the year.

Measured wave data on Lake Ontario is very limited and generally covers only short periods of time. A 36-year wave hindcast was conducted by Shoreplan using wind data from the nearby Toronto Islands to produce deep water wave conditions offshore of the LWC site. Wind data records for the years 1973 – 2008 were used to produce hourly estimates of the deep-water significant wave height, peak mean period, and mean wave directions. The offshore wave climate features waves approaching predominantly from the east and southwest. The vast majority (~72%) of total wave power comes from the east, while a lesser portion (~22%) comes from the southwest, with the remainder (~6%) distributed over all other directions. The results of an extreme value analysis for easterly storm events indicated the design 100-year return period wave condition has an offshore wave height of 5.8 m with a peak period of 10.5 s. A similar analysis for southwesterly storms yielded a 100-year return period wave condition with an offshore wave height of 4.5 m and peak period of 8.0 s.

The nearshore wave climate, also developed by Shoreplan, was determined by transferring the offshore wave conditions in to the LWC project site using CMS-Wave, a two-dimensional spectral wave model with energy dissipation and diffraction terms. The model simulates a steady-state spectral transformation of directional random waves co-existing with ambient currents in the coastal zone. A large number of representative offshore wave conditions were used to interpolate nearshore waves for each wave in the 36-year hindcast. The predicted 100-year return period design wave condition has a nearshore wave height of approximately 3.2 m.

1.2 CONCEPTUAL DESIGN

Five alternative LWC project configurations were designed, each including a similar division of habitat varieties, and accommodations for flows from Applewood and Serson Creeks. In particular, all but one design made use of headland-bay configurations. A headland-bay beach is defined as a beach lying in the lee of a headland subjected to a predominant direction of wave attack (Yasso, 1965). Waves arriving at the shoreline of crenulate-shaped bays both diffract and refract into the shadow zone of the upcoast headland, so producing the log-spiral shape of this zone of the bay. Wave propagation results in a normal approach to the beach at the breaker line when the bay is in complete equilibrium, so preventing a littoral current (Silvester and Ho, 1972). Based on the key objectives outlined in the Environmental Assessment, a comparative evaluation was undertaken, and the *Island Beach C* configuration featuring crenulate-shaped bays was selected as the preferred alternative (see Figure 1).

The shoreline protection features of the selected LWC conceptual design include an armour stone revetment, cobble beaches, a terminal groyne structure, and offshore islands with two creek outlets (Serson and Applewood) incorporated into the footprint. The new beach will be roughly 1 km long and is comprised of cobble with a mean diameter of 150 mm. The new cobble beach will be located between a terminal groyne at the northern end and a revetment headland at the southern end. It will be anchored and partially sheltered by three low-crested islands, each roughly 140 m long. The offshore islands are designed for frequent overtopping to discourage access by terrestrial flora and fauna, including cormorants, with their main coastal function being beach stabilization. The islands are expected to be separated from the shore under average lake levels. All the structures will be armoured with locally procured quarry-stone.

Figure 1. Lakeview Waterfront Connection conceptual design.

2 PHYSICAL MODELLING

The National Research Council (NRC) was commissioned to conduct a physical model study to investigate the interaction of moderate and extreme waves with the proposed headland-beach system, to help optimize the design of the project including the beach fill and the rubble-mound structures, and verify the stability and response of the headland-beach system under a range of design conditions, including storms approaching from easterly and southerly directions, at average and high lake levels.

The 3D model study of beach response and armour stability for the Lakeview Waterfront Connection project was conducted in NRC's 50.4 m by 29.5 m Large Area Basin, as seen in Figure 2. The facility can accommodate water depths up to 1.5 m, is easily accessed by heavy equipment used to construct physical models, and can be equipped with up to three computer-controlled portable wave machines. During testing, the depth of water in the basin was held steady by means of an automatic water level controller connected to a sump pump. The system maintained a constant water level with a tolerance of roughly ± 1 mm at model scale (MS).

Figure 2. Overview of the Lakeview Waterfront Connection model in NRC's Large Area Basin.

2.1 MODEL SCALE EFFECTS

These model studies have been conducted at a geometric scale of 1:35, which means that all lengths in the model were 35 times smaller than the corresponding prototype length. This scale was selected so that the proposed cobble beach and a portion of the adjacent shorelines and surrounding bathymetry could be included in the physical model. Froude scaling has been used to convert quantities measured in the model to full scale values. Since wave motion, wave-structure interactions and hydraulic stability are governed by balances between the gravitational and inertial forces acting on water particles and armour units, similitude of the Froude number in the model and at full scale, together with geometric similitude, ensures that the model provides a realistic simulation of these processes. A relatively large model scale was used in this study to minimize, as much as possible, scale effects due to the distortions in surface tension and viscous forces. Hence, the model was expected to provide reliable predictions of armour stone stability at full scale under equivalent conditions.

Studying coastal sediment transport problems requires a model where all or part of the model bed is composed of granular material that can be transported by hydrodynamic forcing imposed by waves and currents. Bedload sediment transport is caused primarily by fluid shear stresses initiating sediment particle motion and moving the particles along the bottom (Kamphuis, 1991). Given the relatively large material sizes proposed for prototype construction, it is safe to assume that bedload will be the dominant mode of transport. Therefore, it is appropriate that the material chosen for construction of the physical model was scaled by the length scale.

2.2 MODEL BATHYMETRY

A three-dimensional physical model of the proposed LWC project and the surrounding seabed bathymetry was constructed for this study. The bathymetry was fabricated as a fixed bed made of smooth concrete, and was a faithful representation of the existing seabed, down to approximately the 69 m depth contour (mean lake level is ~75 m). The offshore edge of the bathymetry featured a gentle slope (1:10) down to the basin floor, which represented the 63 m depth contour. The model layout was designed such that waves approaching the site from easterly directions (specifically 105°) would be very well represented over the majority of the proposed shoreline. Some modifications were made to the southern portion of the bathymetry to accommodate waves approaching from 180° that could be moderately well represented without greatly affecting the ability to generate easterly waves.

The model bathymetry was designed using CAD software and constructed using a network of fibreboard templates placed at 1 m (MS) intervals. The templates were drawn and cut with a ±2 mm (MS) tolerance, and were erected on a grid of pads that had been levelled to within ±1 mm (MS) using precise surveying methods. Once the templates were erected, they were backfilled with fine gravel, which was then compacted to a level approximately 5 cm (MS) below the top of the templates. The 5 cm (MS) gap was filled with a skin of concrete grout that was screeded to the desired elevation. Finally, a smooth finish was achieved through a combination of hand and mechanical trowelling.

2.3 WAVE GENERATION

In the model, irregular long-crested waves were generated using a set of computer-controlled WM7 wave machines, located in a water depth of 12 m (at mean lake level, full scale). Irregular (random) waves were used because they provide a more authentic simulation of wind-generated waves in nature. The portable WM7 wave machines were relocated during the study to generate waves from three different directions. The predefined wave machine positions were marked on the basin floor so that the machines could be relocated with good precision, even without draining the basin. Portable walls (wave guides) were used to direct the waves from the wave machines up towards the project site. The wave guides prevented the loss of wave energy through diffraction effects and helped to compensate for the finite length of the wave machines. The perimeter of the basin was lined with wave-absorbing beaches to minimize unwanted wave reflections. These beaches, made of porous rock, were placed at a slope of approximately 1:10 and absorbed most of the spurious wave energy along the boundaries of the model domain.

2.4 CONSTRUCTION OF COASTAL STRUCTURES

Careful attention was given to the location, dimensions, composition, and methods of construction of the coastal structures (e.g. cobble beaches, island breakwaters, revetments, etc.) to ensure that they replicated the proposed designs accurately and faithfully (see Figure 3). The alignment of each structure was precisely laid out using a robotic total station, ensuring the overall positioning and curvature of each coastal structure was well reproduced.

Working from CAD cross-sectional drawings provided by Shoreplan, a large number of fibreboard templates were prepared and used to guide the placement of the core, filter, and armour materials for each of the various coastal structures (see Figure 3b). Elevations were controlled by carefully surveying the templates using an optical level, and were verified after construction for quality assurance. The templates were removed prior to testing.

A widely-graded gravel material (Granular A) was used to build the impermeable core berms for all structures. The core was covered with a filter cloth material to keep the core material separated from the filter and armour materials, and permitted faster rebuilding during later stages of the test program. The filter cloth also served to keep finer materials in the core from seeping through the armour layers and creating murky water conditions. This filter cloth material had no bearing on the overall performance of the coastal structures.

The underlayers and different rock armour gradations of the coastal structures were painted different colours to assist in visualizing stone motion and damage. Cobble materials were bulk placed by shovel, and then gently hand-packed and shaped to match the beach template profiles using steel trowels (see Figure 3a).

The placement methods of the model armour stone attempted to mimic those used in the prototype so that the rock behaved similarly and displayed the same kind of interlocking characteristics. In many areas, a bulk placement method was utilized in the model to investigate the performance of this type of placement in terms of stability. In this case, rocks were placed by shovel or bucket in the model, and then gently re-shaped by hand to conform to the desired cross-section shape. However, certain sections of the various structures (typically over/near the structure crest) were placed by hand, unit-by-unit, to match the prototype techniques of placement of each rock by crane/excavator (see Figure 3c & d).

Figure 3. Construction of coastal structures.

2.5 CONSTRUCTION MATERIALS

Thirteen different classes of rock material were specified for use in construction of the various prototype structures. This included seven classes of filter and armour stone ranging in mass from 350 – 7,000 kg (M_{50}), and six classes of rip-rap and cobble materials ranging in size from 10 – 340 mm (D_{50}). These specifications were reduced from model scale using the scaling relationships described above. The filter and armour materials were scaled by weight to maintain an equivalent submerged stability in the model, while the rip-rap and cobble materials were scaled by diameter. The rock materials and gradations were prepared, as much as possible, to replicate the characteristics of the materials under consideration for use in prototype construction.

2.6 INSTRUMENTATION

Wave conditions were measured in the model using twenty-four capacitance-wire wave gauges. Four of the gauges were arranged in a compact array in order to collect the information necessary to resolve the directional properties of the wave field at one location. Wave-induced flow velocities were measured using three 2-axis (u,v) electromagnetic current meters. Two simple, accurate, and reliable overtopping systems were developed for use in the model. Each system consisted of a water storage reservoir, a capacitance wave gauge to measure the level of the water in the reservoir, and a metal tray to convey the overtopping flow from the crest of the breakwater into the reservoir. A photographic damage analysis system comprising six remotely-operated digital cameras was used in this study to monitor the movement of armour stone on the surface on the revetments and breakwater islands. Since each camera remained fixed throughout a test series, the movement of individual stones could be detected by comparing photographs taken at different times. Four high-quality video cameras, with remote pan, tilt, and zoom capabilities, were used to document and digitally record all tests during the model study.

Profiles of the cobble beach were taken at eight locations prior to, during, and at the completion of each test series. In this way, the amount of reshaping at each location could be used as a metric of relative performance. The profiling technique used a range pole that was installed in the concrete walkway behind each profile location, with a collar nut secured to the pole and surveyed to a known elevation. An aluminum beam was lowered onto the pole to rest on the nut, and this beam was levelled using a small tripod at the far end. Holes were pre-drilled in the beam at specific positions (e.g. slope breaks on the cross-section), and slender rods were lowered through the holes until coming in contact with the cobble material on the structure (see Figure 4). The length of the rods hanging below the beam was measured, and the distances were subtracted from the known elevation of the beam to give an elevation of the structure. These elevations and offsets were used to develop the profiles of the cobble beach sections.

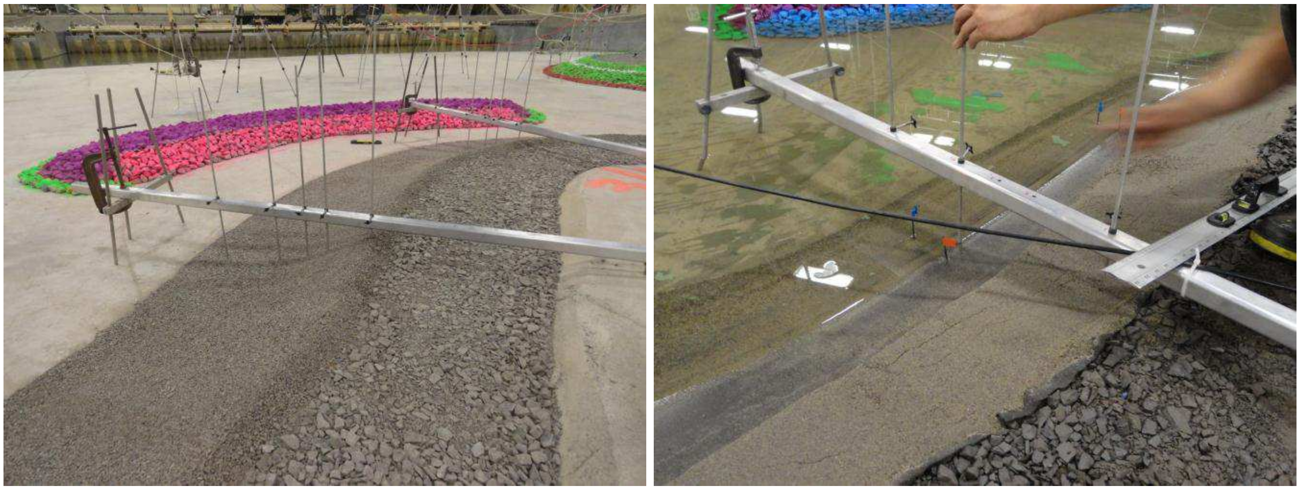

Figure 4. Cobble beach cross-section profilers.

2.7 INCIDENT WAVE CALIBRATIONS

A temporary gravel beach at approximately 1:10 slope was placed along the future beach shoreline to absorb virtually all of the incident wave energy and prevent reflected wave energy from contaminating the wave height measurements during the wave calibration phase. A series of wave probes were laid out just offshore of the proposed beach-headland site. These probes were located approximately along the 70 m depth contour (mean lake level is ~75 m) and with an average horizontal spacing of 160 m (full scale). Three additional wave probes were laid out near the centre of the basin and roughly perpendicular to the easterly wave direction in order to measure cross-wave profiles.

During the wave calibration phase, one-hour long versions of each seastate were generated in the basin, and the resulting wave conditions were measured at 15 different locations throughout the model domain. Two different sets of wave gauges, named ‘control groups’ were established for easterly and southerly incident waves. An iterative procedure was followed to obtain, for each prescribed seastate, a set of wave machine command signals which produced measured wave conditions (for the appropriate control group of wave probes) that were in close agreement with specifications. Three-hour duration realizations of each tuned seastate were also generated and measured to verify that a change in the duration did not substantially affect the measured statistics of the wave field.

3 TEST PROGRAM

The testing program involved several series of waves that modelled moderate to extreme conditions and their effects on the island breakwaters, revetment headland, and cobble beaches (see Figure 5). The stability of all coastal structures was observed, and optimizations to improve their performance, constructability, and cost effectiveness were undertaken. Four distinct test series were undertaken in this study. The main objectives of the testing program were to:

- Determine the natural beach planform for representative annual conditions;
- Determine the response/performance of the beach to moderate and severe easterly storms;
- Determine the response/performance of the beach to a severe southerly storm;
- Determine the response/performance of the beach under “overload” conditions that exceed design conditions;
- Assess the duration of extreme easterly waves required to expose core material beneath the beach cobbles;
- Assess the response/performance of a smaller cobble size in the northern beach cell;
- Confirm the location and layout of the revetments, islands, and the terminal groyne;
- Confirm the stability of the rubble-mound structures under extreme wave conditions;
- Assess overtopping volumes under design conditions;
- Assess nearshore wave conditions, wave-driven currents, and circulation patterns under design conditions; and,
- Assess the relative response/performance of alternative initial beach fill placements

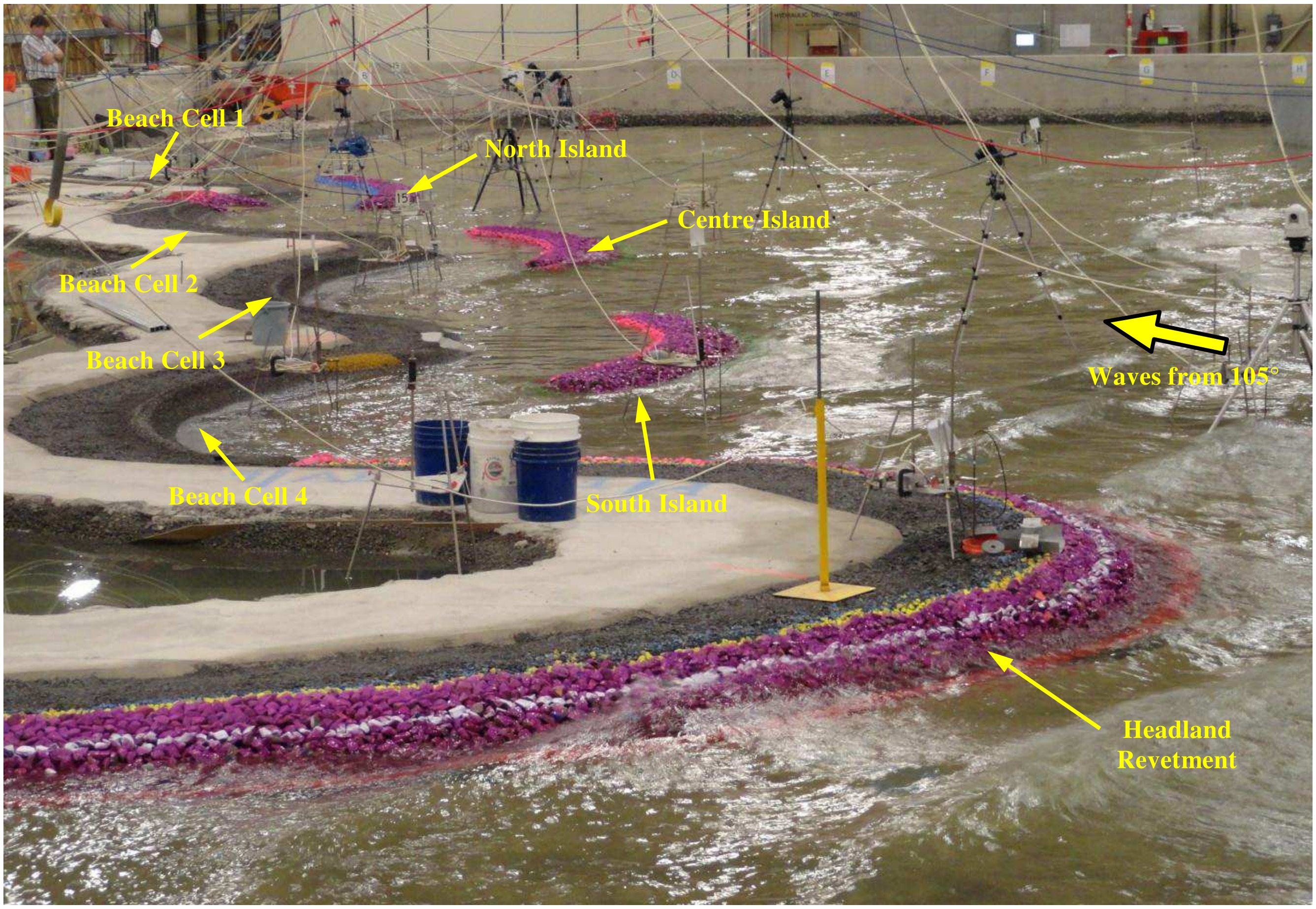

Figure 5. Testing of the Lakeview Waterfront Connection project in the NRC Large Area Basin.

4 ASSESSMENT OF THE COBBLE BEACHES

The characteristics of a cobble beach are very different from those of a sand beach, and despite the large grain size, the material used in the model was highly mobile. In this study, exposed portions of the cobble beaches (those areas not directly sheltered by the island breakwaters) had a tendency to form a steep berm towards the high water mark, below which was a beach face of fairly uniform slope. The active swash zone continuously re-worked the beach face throughout the storm cycle (see Figure 6). The first few waves in each new storm segment quickly altered the beach profile and moved the cobble berm up to the new high water mark. In several instances when a less energetic seastate was tested after the storm peak, smaller secondary berms were formed below the main ridge which had been formed during the more energetic seastate. Based on its performance for the range of waves and water levels tested in the model, the 150 mm (D_{50}) cobble stone appears suitably sized for the beach cells.

Vertical segregation by particle size was also observed on the cobble beaches throughout the study. In general, the coarser grains migrated onshore as bed load and subsequently formed into berms. Finer cobble materials were transported offshore and typically settled below the waterline near to the structure toe. This process is well known to occur in nature.

Small coloured marker flags were placed at the waterline before and after many test segments. These flags helped provide a visual comparison of where the initial waterline had been, and how the shape of the cobble beaches progressed during testing. Dye tracer was also injected into the water and the individual cobble stones were closely observed to determine the predominant direction of the wave-induced currents and the direction of longshore transport. It was observed that much of the material eroded from the edges of the beach cells was being transported towards the cell centre, leading to increased cobble thickness and greater build-up of the berms. Ideally, the plan shape of the beach cells should be such that little to no longshore transport of cobble material occurs. The progressive beach erosion near the edges of the beach cells could indicate the alignment (plan shape) changes too drastically, and the erosion/deposition signifies the beach cell was attempting to come to equilibrium at a new orientation.

Based on detailed shoreline alignment surveys and observations of areas with loss of cobble material, the overall alignment of the beach cells was adjusted during detailed design. Notably, the headlands behind the island breakwaters were brought slightly onshore, while the centre of the south beach cell was brought slightly offshore in order to provide a more gentle transition to the centre beach cell and better alignment with the crests of the incoming waves. The South Island was extended slightly southwards (as tested in the latter stages of this study) to provide additional sheltering for the north end of Beach Cell 4 where consistent erosion issues were identified.

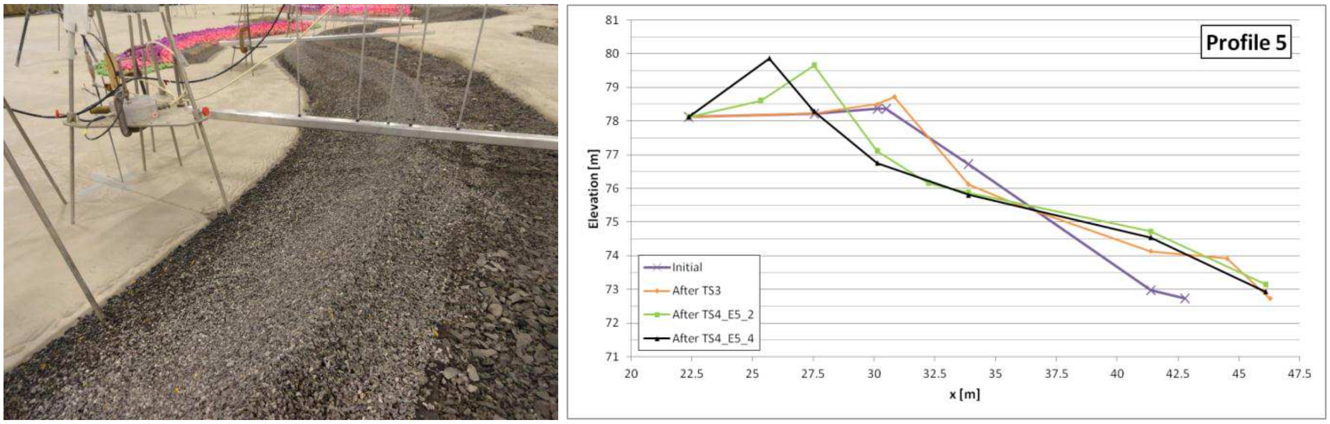

Figure 6. Typical reshaping and cross-shore profile evolution at the centre of the beach cells.

5 ASSESSMENT OF THE ISLAND BREAKWATERS

The first design for the island breakwaters featured 3-5 tonne armour on the crest and front slope. The breakwaters suffered effectively no damage in the first test series, and overtopping only reached ½ to ¾ of the way over the crest. Based on these observations, the island breakwaters were optimized and rebuilt with a number of modifications, including slightly lowered crest elevations and with added submerged extensions to each roundhead to provide increased sheltering to the leeside of the breakwaters. The revised designs performed well, with only minor damage observed in severe storm conditions. Although significant 'wash-over' events on the islands will mostly occur on the rare occasions of high water levels with large storms (see Figure 7), these events are not expected to cause any significant damage to the structures, but will help to discourage access by terrestrial flora and fauna. It should be expected that occasional maintenance may be required to repair minor damage.

Figure 7. Overtopping of the South Island during severe conditions.

6 ASSESSMENT OF THE REVETMENT HEADLAND

The first design for the revetment headland featured 6 tonne (M_{50}) armour stone with a relatively high crest elevation. Observations from the first test series indicated effectively no damage and very little overtopping even under the most severe conditions. A scour hole developed at the interface between the revetment and Beach Cell 4, caused by waves rushing along the end of the revetment and crashing directly onto the smaller cobble material at the edge of the beach cell. Based on these observations, the revetment design was optimized and rebuilt with a number of modifications, including smaller rock armouring (M_{50} 4 tonne), lowered crest elevations, and included a small end groyne at the interface with Beach Cell 4. The revised revetment designs investigated in the second test series performed adequately (see Figure 8). A number of potential further optimizations were identified for consideration in the final design:

- It was possible to slightly decrease the size of the toe armour stone all along revetment;
- The crest elevation of the Type A1 revetment near the end groyne was reduced further;
- The crest elevation of the Type B revetment near Serson Creek was reduced further;

At average lake water levels, even during very energetic storm conditions, overtopping rates were low (<0.05 L/s/m at the eastern tip of the headland). However, flowrates did increase significantly during higher water levels. Some minor damage behind the crest was observed (see Figure 8b), particularly just to the south of the easternmost point of the headland. This level of damage would only be anticipated to occur during particularly severe conditions; however it should be expected that occasional maintenance may be required to repair minor damage. The end groyne suffered effectively no damage, and considerably reduced the formation of the scour hole at the interface with Beach Cell 4.

Figure 8. Initial and final condition of the revetment headland.

7 CONCLUSIONS

A physical model study was conducted to investigate the interaction of moderate and extreme waves with a proposed headland-beach system, in order to optimize the design of the beach fill and rubble-mound structures, and also to verify the stability and response of the headland-beach system under a range of design conditions, including storms approaching from the east and south directions, at average and high lake levels. This study generated a large quantity of good information that was used by the design engineers to further optimize and finalize the design plans. This optimization lead to significant cost savings for the overall project. This study demonstrates the valuable role that physical modelling can play in assessing the performance and optimizing the design of shoreline protection schemes.

ACKNOWLEDGEMENT

The authors wish to thank the Credit Valley Conservation Authority, the Toronto and Region Conservation Authority, and the Region of Peel for funding this study.

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