Empirical modelling of run-up and overtopping at a boulder beach, Raglan, New Zealand

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Abstract
Boulder and cobble beaches constitute a significant portion of New Zealand's coastline. While general aspects of their morphodynamics have been well documented, quantitative analysis of run-up and overtopping characteristics has been less forthcoming. Analysis using established run-up criteria developed for fine sediment beaches or artificial rock structures yields run-up and overtopping values significantly in excess of observed values. Additionally, predicted values exceed those expected based on geomorphological development processes of such beaches. A site-specific study has been carried out at Whale Bay, New Zealand to provide calibration data for existing generic run-up and overtopping models. The models were then verified using run-up elevations and overtopping discharges observed during a recent significant (≥100yr return period) storm event. Finally, the model has been used to assess run-up and overtopping hazard at the study site. Results show significant impediment of run-up within the swash zone compared to that predicted by the models for plane slopes. This is attributed to the extreme permeability and roughness of the poorly sorted boulder beach. The calibrated roughness coefficient was found to be lower than that currently suggested by design manuals for either fine sediment beaches or artificial rock structures. Additionally, run-up amplitudes at the site of interest were found to be greatly influenced by wave groups as well as incident waves breaking seaward of the run-up slope. This last observation infers that the spectral properties of incident wave fields may have substantial influence on run-up and overtopping potential
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1 Introduction

The study site is located on a barrier spit which encloses an intertidal lagoon at Whale Bay, approximately 8 km southwest of Raglan Township on the west coast the New Zealand North Island. The barrier spit protrudes east from the base of Mt Karioi and comprises moderate to large boulders. The barrier spit is of Holocene age (last 10,000 years) and narrows from west (oldest) to east (youngest) (refer Figure 1). Within the lagoon, fine to medium sand overlies boulder formations and small dunes have formed in the backshore.

Surveys and analysis of water levels at the site by Skyworks Waikato and Coastal Systems (NZ) Ltd. give a mean high water spring (MHWS) tidal level of 1.45 m above mean water level (MWL). The boulder bank crest lies at around 4.7 m above MWL, lowering slightly from the basal to the distal end of the spit.

Figure 1 Boulder bank at Whale Bay, Raglan (Source: Skyworks Waikato, 2005).

The barrier spit is comprised of boulders ranging from 300mm to over 1m in diameter. The intertidal seaward face of the barrier spit has slopes between 5(H):1(V) and 7(H):1(V). This is a fairly typical slope for materials of this size. The slope then decreases to near horizontal on a berm located at around the low tide mark before steepening again to around 10(H):1(V) to 15(H):1(V). This slope continues fairly consistently offshore. The backshore region is near horizontal and heavily vegetated with shrubs and grasses.

While the site is located on the New Zealand west coast, an area subjected to very high wave energy, Whale Bay lies in the lee of Mt Karioi, a volcanic headland. Substantial reduction of wave height during refraction of the predominant west to south-west waves appears to occur.

The decreasing crest elevation from west to east along the spit indicates a decrease in wave energy at the spit face. This is consistent with offshore bathymetry (Hutt, 1997) showing the seabed becoming slightly flatter to the east. This will typically induce wave breaking further offshore, with more potential for wave energy to dissipate before interacting with the barrier spit.

2 Study Objective

The objective of the study was to assess run-up and overtopping hazard at the study site. This was achieved by calibrating existing run-up and overtopping models with site-specific data and verifying the model with a known extreme event.

3 Site data collection

3.1 Methods

Key variables which were assessed included significant wave height (H_{sig}), the run-up exceeded by the largest 2% of waves ($R_{2\%}$) and the still water level (SWL) at the time of recording. Collection methodologies for acquiring these data were applied using a combination of observational and video imaging techniques.

First a transect line in the direction of incident wave run-up was surveyed down the boulder bank and wooden stakes placed every 0.5m vertical elevation change (refer Figure 2). This transect line was selected as being the critical flow path for water overtopping the boulder bank and posing a risk to the future dwelling. The placement of the stakes enabled the maximum extent of run-up to be accurately determined.

As the run-up elevation corresponding to the level exceeded by the largest 2% of waves was required, data was analysed over a predetermined period of time based on the mean wave period. For example, if a mean wave period of 12s was determined on-site, the average time taken for 50 waves to arrive is 600s or 10min. By selecting the largest run-up elevation collected every 10min and averaging subsequent data-sets, a mean value of $R_{2\%}$ could be extracted. A video camera setup aimed along the transect line was used to record run-up elevations using a 'time-stack' method.

Wave measurements were based on the 'line-or-sight' methods developed by the Beach Protection Authority of Queensland (Patterson and Blair, 1983). The line-of-sight method measures the height of the wave-crest above the still water level (H_{los}) and is based on the assumption that rays emanating from the horizon are parallel in the vicinity of the shoreline. Breaking wave height equals line-of-sight wave height, plus wave setup, plus trough height. A graduated staff placed a known elevation above the SWL was used to determine line-of-sight wave height. Based on these line-of-sight wave heights, corrections could be applied to account for wave setup, wave trough height and earth curvature. From the resultant wave heights, a significant wave height could be determined and this was used for analysis purposes.

Wave period was determined from the video by measuring the time taken for a group of sequential waves to reach the break-point after an initial wave had broken and then dividing this time by (n-1) where n is the number of waves in the group. This was repeated several times and the results averaged to give a statistically stable mean wave period. The time taken for waves to travel across the width of the surf zone, required for line-of-sight wave height corrections, was determined from the video by timing a relatively large wave from break point until its bore reached the shore. This was repeated several times and the results averaged.

The static water level at the time of recording was acquired based on NIWA's numerical tide model at the time of survey and the NIWA water level gauges at Anawhata on the West Coast of Auckland and at Westgate in New Plymouth. These gauges indicated whether there was deviation from the tides predicted by the model due to atmospheric or environmental factors.

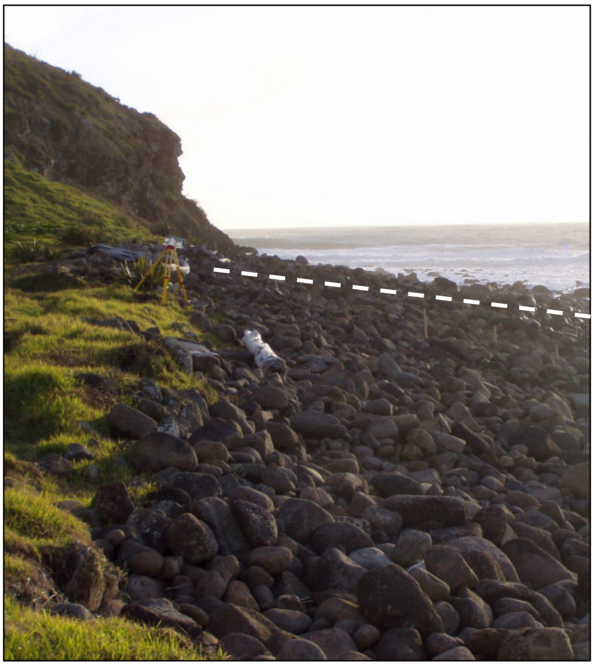

Figure 2 Video camera and run-up marker setup

3.2 Results and analysis

Raw data was processed to acquire a single significant wave height, water level and 2% run-up elevation for each survey set. These results are presented in Table 1.

Table 1 Results of site specific wave and run-up analysis

Date	SWL (MWL)	H_{sig} (m)	T_p (s)	$R_{2\%}$ (m)
10-Jul-06	0.9	4.8	16.5	1.76
22-Sep-06	1.2	2.3	10	1.05
23-Sep-06	1.2	2.7	10	1.28
25-Sep-06	1.2	3.0	15	1.43
10-Oct-06	1.6	4.5	13.5	1.53
11-Oct-06	1.5	4.5	15	1.76

Good agreement occurs between significant breaking wave height and run-up. Linear regression analysis shows a goodness of fit (correlation) coefficient of $r = 0.91$. Using Pearson correlation analysis for 6 observations (n=6), the correlation becomes significant at the 10% level when $r > 0.729$, and significant at the 5% level when $r > 0.811$ meaning that the observed relationship ($r = 0.91$) is statistically significant.

However, additional factors such as wave period and wave breaking offshore from the boulder bank being dependent on the water level, will influence the final relationship.

4 Run-up assessment

4.1 Methodology

The model developed by Hughes (2005) was employed in run-up calculations (Equation 1). This model, incorporating a wave momentum flux parameter, is presented in a US Army Corps of Engineers technical report titled 'Estimating Irregular Wave Run-up on Rough, Impermeable Slopes'.

$$\frac{R_{u2\%}}{h} = 4.4(\tan\alpha)^{0.7}\left[\frac{M_F}{\rho g h^2}\right]^{1/2} \qquad (1)$$

for $1.5 \leq \cot\varphi \leq 30$

Where:

$R_{u2\%}$ = the vertical distance between still water level and the elevation exceeded by 2% of run-up values (m)

h = water depth (m)

φ = structure slope (degrees)

ρ = fluid mass density (kg/m^3)

g = gravitational acceleration (m/s^2)

M_F = depth-integrated wave momentum flux, having units of force per unit wave crest length and being based on wave height, period, depth at the structure toe and a roughness parameter.

Two options were available to calibrate the model using the observed run-up data:

1. Introducing an additional factor to take into account site-specific influences such as permeability and offshore features

2. Adjust the existing roughness coefficient.

As the existing roughness coefficient is simply an empirical parameter adjusted according to laboratory results, it was deemed appropriate to simply modify this and not to add a further parameter. The roughness was therefore adjusted until the $R_{2\%}$ value predicted by the model matched the observed $R_{2\%}$.

4.2 Calibration and verification

Roughness factors used to match model run-up with that observed during field surveys are presented in Table 2.

Results of the analysis show roughness reduction factors varying from 0.35 to 0.5, with a mean factor of 0.4. This is a relatively narrow range, with values being within 30% of each other. The upper end value (r = 0.5) is also in reasonable agreement with values suggested by the Coastal Engineering Manual for similar materials (r = 0.5 to 0.6).

Table 2 Results of roughness factor calibration for run-up model

Date	Measured $R_{2\%}$ (m)	Best-fit roughness factor (r)	Modelled $R_{2\%}$ (m)
10-Jul-06	1.76	0.5	1.79
22-Sep-06	1.05	0.35	1.08
23-Sep-06	1.28	0.41	1.30
25-Sep-06	1.43	0.41	1.44
10-Oct-06	1.53	0.37	1.54
11-Oct-06	1.6	0.38	15

In order to verify the applicability of the proposed run-up model, a hindcast of run-up levels occurring during a particularly significant recent storm at Whale Bay has been carried out. This storm occurred over the 18-19th September 2005 and widely affected the west coast of the North Island. The storm had characteristics of: a still water level at Whale Bay of 2.3m above MSL; a significant wave height of 4.1 m and peak period of 12s and a return period of at least 100 years (Shand, 2007).

Incorporating a wave setup of 17% of the significant wave height gives a still water level at the boulder bank of 3.0m above MSL. Based on a roughness reduction factor of 0.5 as determined above, run-up exceeded by 2% of waves is calculated at 2.4m, giving a run-up elevation of 5.4m above MSL. This exceeds the boulder bank crest level by 0.7 m. An assessment has also been carried out using the mean calibrated roughness reduction factor of 0.4. This gives a 2% run-up of 1.9m, resulting in a total elevation of 4.9m above MSL. This level exceeds the boulder bank crest level by 0.2 m.

Observation at the site following the September 2005 event suggested that reasonably significant overtopping occurred, with debris being transported up to 15 to 20 m back from the crest (Eco Nomos, 2005). While the landward extent of flooding cannot be determined using the above run-up model, an overtopping height of 0.7 m appears to better fit the site observation. This will be further discussed in the overtopping assessment (refer Section 4.3).

4.3 Results and analysis

Based on the calibrated roughness reduction factor of 0.5, a set of run-up relationships for various offshore wave heights and a range of still water levels have been produced. These relationships are presented in Figure 3.

Results show that overtopping of the boulder bank crest, which lies at around 4.7m above MSL, is possible with a very high water level and relatively common wave heights, or with very large waves during a spring high tide. Such an event is likely to have a low probability of occurrence, such as the September 2005 event. These results are in agreement with generally accepted morphological models of equilibrium gravel/cobble beaches. Such models state that gravel and cobble beaches are generally built to the level dictated by significant storm events.

We note that should sea level rise increase as predicted with climate change scenarios, the overtopping could also increase, although in time, the boulder spit would adjust upward and possibly translate landward in response to the increase in level.

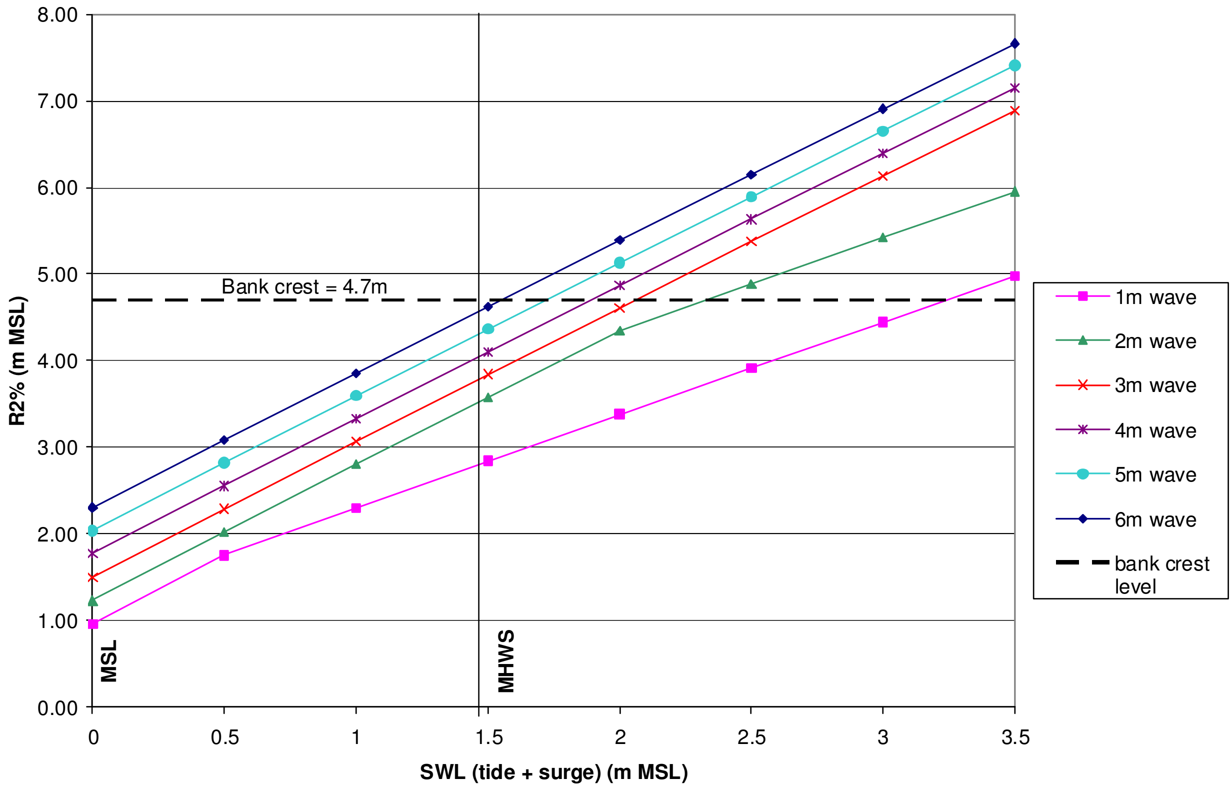

Figure 3 Run-up elevations exceeded by 2% of waves according to still water level and significant wave height

5 Overtopping

5.1 Methodology

Overtopping discharges have been determined using methods described in *Wave Overtopping of Seawall: Design and Assessment Manual*, an R&D technical report by HR Wallingford Ltd. (1999). This method determines the mean overtopping discharge rate per linear meter along the slope and is able to include the effects of a crest berm width. This crest width has been defined as the landward inundation from the boulder bank crest. The method is based on the following relationships:

$$Q = Q_* T_m g H_s C_w \qquad (2)$$

Where:

- Q = the mean overtopping discharge rate per meter length of crest ($m^3/s/m$)
- Q_* = a non-dimensionalised parameter $= Ae^{(-BR*/r)}$
- A,B = empirical coefficients dependent on the sloe geometry
- R_* = a non-dimensionalised parameter $= R_c/(T_m(gH_s)^{0.5})$
- R_c = the freeboard of the crest above still water level (m)
- H_s = significant wave height at the structure toe (m)
- T_m = mean wave period (s)
- r = roughness coefficient
- g = acceleration due to gravity (m/s^2)
- C_r = a reduction factor to take into account a permeable crest berm $= 3.06e^{(-1.5Cw/Hs)}$
- C_w = the crest berm width in meters

A roughness reduction coefficient of 0.5 based on the results of the site-specific run-up assessment has been assumed.

There are limitations with the use of the above method for calculating overtopping rates. The Wallingford report states that *a study by Douglass (1985) concluded that calculated overtopping rates, using empirically derived equations, should only be regarded as being within, at best, a factor of 3 of the actual overtopping rates.* Furthermore, the Wallingford report stated that *it is generally reasonable to assume that the overtopping rates calculated using the methods contained in this report are accurate only to within one order of magnitude*. Based on this caution, an assessment incorporating an order of magnitude (x10) factor of safety to overtopping rates has also been calculated with the above method to provide an indication of model sensitivity.

5.2 Results and analysis

Overtopping discharge rates as a function of distance landward from the berm crest have been calculated for a range of water levels from MSL to MHWS + 2m and significant wave heights from 3 to 6m. In all cases, a mean wave period of 12s has been assumed. This period is an upper limit for storm events from the critical (NW) wave approach direction.

For a building to suffer no damage during an overtopping event, a tolerable mean discharge rate has been determined through empirical studies (HR Wallingford Ltd, 1999). This tolerable mean discharge rate is given at 0.000001m^3/s/m or 0.001 l/s/m. A summary of these distances is presented in Table 3 and Figure 4.

For the case of still water level of 2.3m above MSL and significant wave height of 4.1 m, which corresponds roughly to the September 2005 event, a mean overtopping discharge at the crest of 144 l/s/m was obtained. This corresponds to a zero building damage distance of 21.0 m unfactored and 24.5 m factored from the beach crest (refer Figure 5).

These values agree reasonably with the observed distance that material was transported by overtopping during the storm event as described by Eco Nomos, 2005 (refer Figure 6).

Table 3 Distance from crest (m) to achieve mean tolerable discharge for zero building damage

	Significant wave height			
Water level scenario	**3m**	**4m**	**5m**	**6m**
1.45m (MHWS)	12	13.5	15	17
1.45m (10 times FOS)	15	17	18	20
2.5m	21.5	22.5	24	26
2.5m (10 times FOS)	24.5	26	28	30
3.5m RL	31	32.5	34	36
3.5m (10 times FOS)	35	37	39	41

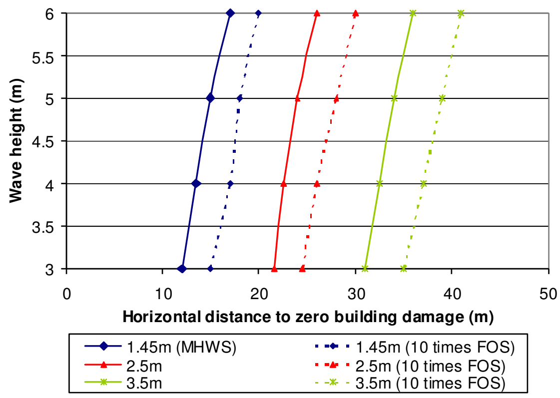

Figure 4 Horizontal distance from crest to achieve mean tolerable discharge corresponding to zero building damage for a range of water level scenarios.

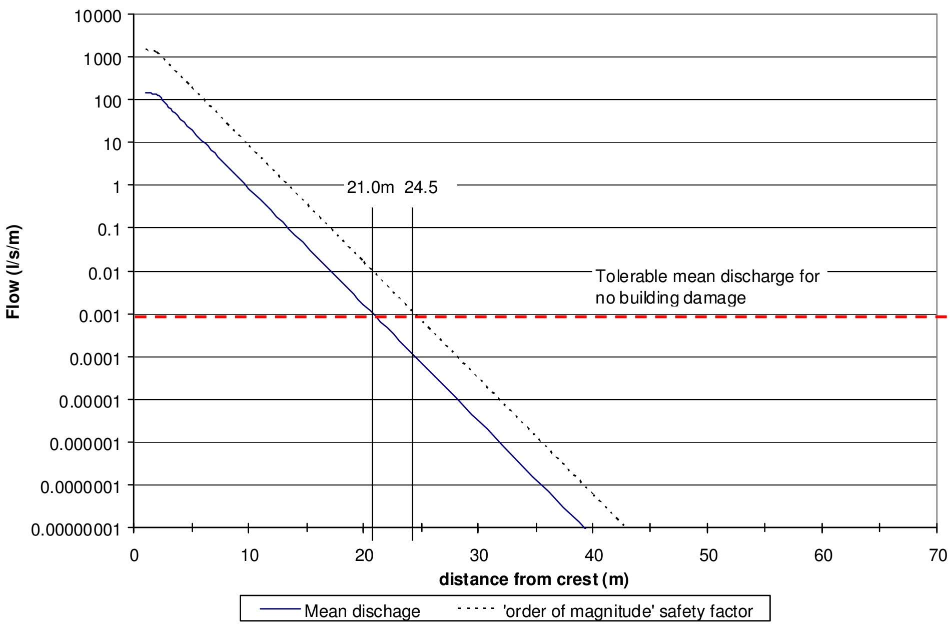

Figure 5 Mean overtopping discharge calculated for the September 2005 storm event

Figure 6 Distance from crest to over-wash debris from the September 2005 storm event. Photo: Eco Nomos Ltd

6 Conclusions

A site-specific investigation was undertaken by Tonkin & Taylor Ltd in conjunction with Coastal Systems (NZ) Ltd and Skyworks Waikato to calibrate run-up and overtopping models for a boulder beach at Whale Bay, Raglan. The models were calibrated by adjusting the roughness coefficient. This coefficient takes into account the friction and impedance to flow of material on the beach, and also beach porosity. The mean value of this roughness coefficient was 0.4 (0.35 – 0.5). These results reflect the very rough and highly permeable nature of the boulder bank and are at the low end of the range suggested by the Coastal Engineering Manual. A roughness coefficient of 0.5 was found to produce results consistent with observed storm effects so was adopted for use in this study.

Using the calibrated run-up mode, run-up elevations were produced as a function of still water level (tide + storm surge) and significant wave height. These elevations show that overtopping of the boulder bank crest, which lies at around 4.7m above MSL is possible with a very high water level and relatively small waves, or with a common high tide level and very large waves.

An assessment of overtopping was then carried out to determine overtopping discharges as a function of onshore distance from the boulder bank crest. The overtopping model used was capable of satisfactorily reproducing the characteristics of the 2005 event. An assessment was carried out for a range of still water levels up to 3.5m above MSL (MHWS + 2 m) and significant wave heights of 3 to 6 m. A tolerable mean discharge of 0.001 l/s/m was established as corresponding to no building damage. Results show discharge capable of causing damage to buildings occurring up to 36m from the beach crest, or 41m incorporating the factor of safety.

This study has quantitatively shown that gravel and boulder beach systems are particularly effective wave energy dissipaters, and as such present substantially less property hazard than sandy beaches with similar energy regimes.

7 References

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