FROM DYNAMIC SAND BEACH TO STATIC RUBBLE MOUND

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Experience from construction of a self-adjusted rubble-mound breakwater at Grassy, King Island, in Bass Strait, where severe wave conditions was up to 7.6 meter incident waves, together with careful observation on large scale model tests (1:10 geometric Froude similitude without distortion) resulted in conclusion that stability of rubble mounds depend on mass stability of the mounds. Although the function of individual armor rocks is still important to cover the core layer in order to achieve static stability, use of equilibrium beach profile principles in predicting final profiles will result in more effective use of overall available material and smaller armor rocks. This study combines observation on the prototype, development of equilibrium beach profile theories, progress in reshaped berm-breakwater design, and large scale model tests to define a new procedure for rubble-mound breakwater/revetment trunk design and to predict the future of Grassy breakwater.

1. King Island’s Grassy Rubble-mound Breakwater

The following report is summarized from Rabung [1] which collects various studies prior to construction of Grassy Breakwater. It was constructed in 1970 – 1972 using overburden rock available from nearby open-cut mining. The breakwater was not built to its intended final shape, but was designed to be reshaped by storm wave action. The construction stages are shown in Figure 1. Works were done by face shovels/loaders, dump trucks and dozers. Run of quarry material less than 2 t was used to form the bulk of the mound while rocks in the range of 2-10 t were selectively stockpiled for application as armouring when required. The initial mound was advanced at a level of +4.6 m (max. tide range 1.8 m, no breaching) with an 18 m top width providing adequate room for the turning of dump trucks and dozers. After minor sorting of the outer face by
waves, armour rock 2-10 t size was fed over the edge and provided a protective parapet for plant and operators (Stage 1).

Having reached its full length (600 m), the initial mound was progressively widened to a top width of 63 m and then capped up to a level of +6.1 m with layers of selected 10 t rocks except for an 18 m wide access road along the harbour side (Stage 2). The seaward breakwater slope initially settled down to an average slope of 1:3.5; however, it was anticipated that after several years and major storm activity, the armour capping would drag down and settle to a stable slope approximating 1:7 to 1:10 in the upper regions (Stage 3). This idealised final section will, however, be subject to review in the light of future storm activity and breakwater behaviour.

The breakwater was designed to fully resist against wave of 7.6 m and subject to 10% damage to wave of 9.2 m, but the latter criterion was inappropriate for this kind of breakwater. Detailed underwater surveys were conducted in 1974 and 1987, and cross-section profile from average depth (15 m below Chart Datum) is given in Figure 2. Meanwhile, later, Australian Bureau of Meteorology made available continuous wind data of King Island from 1957 to 1986 (30 years real time). These data were used to predict maximum deep water significant wave height in Bass Strait during that period using SMB method and it was found $H_{\text{so}} = 8.9$ m. Shoaling and refraction effects was expected would resulted in less than 8 m non-breaking wave on the breakwater slope. This wave was very closed to 7.6 m design wave, so that we could use the 7.6 meter wave to study the stability of the breakwater simultaneously with verifying the result of model tests to the 1987 profile of the prototype.
The whole project (including lee breakwater, wharf and facilities) was completed for a total cost less than $2 million in twenty-six months. This should be compared with the estimated time and cost of a conventional breakwater design which could have steeper slopes but involve special quarrying of armour stones up to 45 t size. Construction time for such a structure had been estimated at forty months with a cost of $6.6 million for the breakwater alone (Burren’s report in [1]).

Some important conclusions from design stage (by Maunsell & Partners Pty. Ltd.) including preliminary model tests (by D.N. Foster in the University of New South Wales) are:

- After construction is completed the breakwater would behave essentially as a beach.
- After a stable profile has been reached, sediment would continue to be moved longshore as littoral drift. To prevent eventual breaching of the mound from this cause, the face must be armoured or the beach artificially nourished to replace the lost material. However, the latter choice may cause siltation within the harbour area.

2. Equilibrium Beach Profile

The basic equilibrium beach profile (EBP) parameters are given in Figure 3.
Bruun firstly introduced EBP equation \( h = A.x^{2/3} \), where \( A \) is ‘sediment scale parameter’ having unit \( m^{1/3} \) which depends on sediment size and falling velocity. Many formulae are given to determine \( A \), but the most popular is the chart provided by Dean based on many studies (Figure III-3-17 in [2]). Dean generalized Bruun’s equation with \( h = A.x^n \) where \( n \) is 2/3 or 0.4 depends on approaching theory. In this study we prefer to use the general form:

\[
h = A.x^n
\]  

(1)

Hallermeier ([2], p. III-3-20) determined the ‘closure depths’ \( h_c \) and \( h_d \) (\( h_d \) is not discussed here) based on field and experimental data but simplified by Birkemeier with \( h_c=1.57 H_e \). \( H_e \) is effective significant wave height which is calculated based on several years of annual mean significant wave height \( H_s \).

These data are not available in this study, thus we use Suh and Dalrymple’s formula which is based on deep water wave height:

\[
h_c = 1.6H_o
\]  

(2)

3. Development in Reshaped Berm-breakwater Design

PIANC [3] divides rubble-mound breakwaters into conventional and berm types. The berm type are divided further into statically stable non-reshaped, statically stable reshaped and dynamically stable reshaped. In dynamically stable reshaped berm breakwater, the profile is reshaped into a stable profile, but the individual stones may move up and down the front slope; thus, Grassy breakwater includes in this type at early stages, but it was expected to be static.

Van Gent et al. in [4] reanalyzed van der Meer’s data and simplified his formulae while added some new experimental data on the stability of the armor units on conventional rubble-mound breakwaters. There is no separation between plunging and surging waves. The influence of the permeability of the structures is incorporated in a direct way by using structure parameter \( D_{n50,\text{core}} \), i.e. the equivalent diameter of the core material median, compared to \( D_{n50} \) for armor stones. Van Gent’s formula (2004) is following:

\[
\frac{H_s}{\Delta D_{n50}} = 1.75\left(\frac{S}{N}\right)^{0.2}\cot\alpha \left(1 + \frac{D_{n50,\text{core}}}{D_{n50}}\right)
\]  

(3)

Value of damage level \( S \) is given in Table 1. \( H_s \) is incidental significant wave and \( N \) is number of waves in the storm, \( \Delta = (\rho_s/\rho_w) - 1 \), \( H_s/\Delta D_{n50} = N_t \) is ‘stability number’ for individual armor stones in conventional rubble-mound breakwater term which is identical to \( Ho \) for berm breakwater term, but different damage criteria.
In recent years many developments in designing and constructing berm breakwaters as presented in [3] and [4]. The most interesting is that the parameters used in analyzing berm breakwaters become closer and closer to parameters in beach profile analyses. In Figure 4, Rec is analogue to Rs recession in beach profile term, hf and hs are analogue to hc and hd closure depths, S-shape of reshaped breakwater is analogue to equilibrium beach profile (Figure 3). Thus, the idea from Grassy Breakwater becomes more realized.

Other parameters that are relevant with this study, hf and hs, are given below (from [3] and [4]):

\[
\frac{h_f}{D_{n50}} = 0.2 \cdot \frac{d}{D_{n50}} + 0.5 \quad \text{for } 12.5 < \frac{d}{D_{n50}} < 25 \quad (4)
\]

\[
h_s = 0.65 \cdot H_s \cdot S_m \quad (5)
\]

where \( S_m = \frac{2\pi H_s \cdot g \cdot T_m^2}{\Delta D_{n50}} \), \( f_N = (N/3000)^{-0.046Ho+0.3} \) for Ho < 5,

\( f_N = (N/3000)^{0.07} \) for Ho > 5, and \( f_B = \cos(\beta) \) \( \beta = \) angle between the wave direction and the breakwater trunk centerline, \( d = \) water depth, \( h_f = \) depth of intersection point, \( h_s = \) step height, \( Ho \equiv N_s = \frac{H_s}{\Delta D_{n50}} \) stability number.

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<tr>
<th>Slope angle, ( \cot \alpha )</th>
<th>Damage level, S</th>
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<tr>
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4. Large Scale Model Tests

Tests were conducted in the large wave flume at Monash University, which was equipped with a computer-driven hinged-paddle wave generator capable to produce monochromatic and spectral waves. The tank is 2.2 m wide, 52 m long and 4.0 m deep. At the model location the floor was fitted with a false floor which reduced the depth to 1.5 m in accordance to average depth in the prototype (Figure 5). This experiment was run with a 1:10 undistorted model based on Froude similitude. This choice of scale easily meets the criteria for modeling transport of sediment, including the guideline that the smallest stone size should not be less than 10 mm.

![Figure 5](image-url) Longitudinal section of the flume (dimensions and scale are approximation only).
Tests were run in monochromatic and spectral waves with 6 tests each, but only results of monochromatic waves are discussed in this paper due to limited space. Test 3 was repetition of Test 2 to confirm the stability of final profile due to the same waves. The design waves were $H_s = 40$ cm and $76$ cm associated with $4.0$ m and $7.6$ m in the prototype. Design wave frequencies were $f = 0.28$ and $0.32$ Hz associated with $T = 10$ and $11$ s in the prototype, storm durations were $14$ hrs associated with $44$ hrs in the prototype. The list of tests is given in Table 2.

Other governing variables used in this tests were materials. The materials used in the model were dense basalt of specific gravity 2.84, the same materials in the prototype. These materials were divided into three categories. Firstly, the core which was of continuous gradation with maximum size kg with a median weight of 6 kg. Thirdly, selected 10 t rocks which were stockpiled less than 2 kg. Secondly, the armour layer which was in the range of 2 - 10 evenly on top of the mound. The stones were generally rough, with very sharp edges. For mix design of the model material, it was assumed that the prototype material was normally distributed, which is common for natural material. Thus, the model comprised a mound of stone of two layers only, i.e. the core and armour layers. Total material used in the model was 21.5 m³ which is approximately 50 t. During dumping the materials by wheelbarrows, the wave paddle was running gently to simulate natural condition.

5. Results and Discussion

If we looked carefully at the process of damage of rubble mound in the experiment, it followed this path. At certain height of waves, armor rocks began to fall one by one. After sometime, core material was exposed to waves and...
began to fall down or move up and down. Suddenly, the overall mound broke down. After the first breakdown, the mound was more dynamically stable; it would require much bigger waves to break down the mound further. Thus, the functions of armor rocks are: firstly, to break the waves and absorb their energy as much as possible; secondly, to cover the core material from direct wave attacks in order to achieve static stability. The overall stability of the structure relies on the overall mound, not on the individual armor rocks, although stability of the individual armor remains important. Cooperation of stability of armor layer and core layer is required.

Figure 6 shows development of profiles in the model; these are comparable to development of profiles in the prototype of 1974 and 1987 as shown in Figure 2. Slope of the berm After Test 3B (Final Profile) is 1:5, meanwhile on the prototype 1987 profile the slope is 1:5.5. Closure depth on the model is \( h_c = 0.475 \) m, on the prototype is 7.5 m. These differences most probably are caused by small difference in the incident waves. In the model, the design wave is 0.76 m high, whereas in the prototype maximum storm between 1957 to 1986 might cause \( H_s \) up to 8.9 m which was expected would be less than 8 m on the breakwater; or, there was a bigger storm between 1986 and 1987 (but small possibility).

Curve fitting on the model berm results in power regression \( h_x = 2.4523x^{-0.454} \) with correlation index \( R^2 = 0.9322 \); meanwhile if we use Bruun’s equation with \( D_{50} = 128 \) mm of 6 kg median armour rock which has scale parameter \( A \) from Dean’s chart is 0.8, we find relation \( h_x = 0.8x^{-2/3} \). These relations are slightly different, but general form of Dean’s formulation \( h_x = Ax^n \) is well satisfied with \( n \) close to 0.4; it is needed more experiments to determine the scale parameters \( A \) for bigger stones and the power \( n \). However, the more important on practice is linier berm; curve fitting by linier regression gives correlation index \( R^2 = 0.9939 \) and \( \cot \alpha = 5 \) matching condition on the prototype.

Figure 6: Development of profiles during monochromatic wave tests.

Depth of closure in the model is 0.475 m. Using Suh and Dalrymple’s equation (2) \( h_c = 1.6H_s = 1.6 \times 0.76 = 1.216 \) m, assuming no shoaling effect
and no refraction in the wave tank, the result is quite different. Using equation (4), it is found \( h_f = 0.364 \) m which is closer to \( h_c \) in the model. Thus, in this point reshaping berm breakwater formula is better. Point \( h_s \) is not significant in the model.

For verification, conventional breakwater stability using van Gent’s formula (equation 3) with \( H_s = 0.76 \) m, \( \cot \alpha = 5 \), \( T_m = 10.5 \) s, duration = 14 h, \( N = 4800 \), \( D_{s50, \text{core}} \) of average 1 kg stone = 0.071 m, \( \rho_s = 2840 \) kg/m\(^3\), damage level \( S = 3 \) gives required armour \( D_{s50} = 0.127 \) m or \( W_{50} = 5.8 \) kg ≈ 6 kg exactly the same in the model. Thus, the berm of model is in static stability from conventional design point of view.

As final conclusion, the following extracts are presented. Firstly, in traditional rubble breakwater design, stability is relied on the stability of individual armour stones in the form of stability number \( N_s \equiv \frac{H_o}{D_{n}} \); in this new approach, the stability is relied on the overall stability of the mound. Secondly, in traditional design, slope (usually steeper than 1:5) and allowed damage are firstly chosen within the certain range of experience (experiments); in this new approach, available materials are first analysed then berm slope is determined based on EBP allowing slope gentler than 1:5. Thirdly, from Grassy’s breakwater experience, the new design approach will be much cheaper in cost as result of efficiently use of material and ordinary equipment.

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References