Wave Transmission at Submerged Rubblemound Breakwaters

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Abstract

Submerged rubblemound breakwaters are becoming more popular as a potential alternative to coastal protection measures where a moderate degree of energy transmission is acceptable. Such situations include areas where vegetative shore protection is existing or proposed or in the event that an existing shore protection structure has become damaged or under designed and a method is needed to reduce the incident wave energy. Although there have been previous investigations on the performance of submerged rubblemound breakwaters, there are only a few design equations available to the design engineer. Those available are based on a limited range of input design variables and as a result are insufficient in some cases.

Physical model studies were performed at the Queen's University Coastal Engineering Research Laboratory (QUCERL) in Kingston, Canada to assess the performance of submerged rubblemound breakwaters under a wide range of design conditions in twodimensional (2-D) and three-dimensional (3-D) settings. The tests include a number of wide crested structures to provide data where previous investigations have not. The results show that the relative submergence, incident wave height and structure crest width are the most important design variables.

A number of potential design equations were evaluated by statistical analysis methods. The proposed design equation fits the 2-D test data well and provides moderate agreement with the 3-D test results. Although physical testing is suggested for all design applications due to the complexity of site specific considerations, the proposed equation does provide a good preliminary design tool for submerged rubblemound breakwaters.

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Submerged rubblemound breakwaters are simply rubblemound structures constructed with a crest elevation below the local water level. Although they are well suited to situations where minimal visual intrusion is desired and where it is desirable to maintain a moderate degree of energy transfer between the shoreline and the offshore region for environmental reasons, their performance is sensitive to water level changes and it is not practical to expect transmission coefficients as low as those achievable with conventional surface piercing structures. However, there are benefits associated with the potentially smaller material requirements for stable submerged structures and the ability to rehabilitate existing structures by simply reducing the incident wave conditions with a submerged breakwater.

Numerous physical and numerical investigations have been performed for various submerged structure configurations and materials. In general, the physical processes at submerged rubblemound breakwaters can be defined for three regions in the vicinity of the structure as indicated in Figure 1. Relevant nomenclature is also indicated.

In Region 1, the incident wave shoals on the rising face of the breakwater, considerable non-linear wave transformations take place as bound waves are developed (Beji and Battjes, 1993) and some wave breaking is initiated. A portion of the incident energy is also reflected from the front breakwater face. Wave breaking continues into Region 2 where significant non-linear interactions occur between the various wave phases. Harmonic generation occurs as energy is transferred from the fundamental wave frequency to higher harmonic frequencies (Driscoll, Dalrymple and Grilli, 1993). Some wave energy is also dissipated on the breakwater crest through friction and air entrainment as well as within the breakwater structure. In Region 3, the free and bound transmitted waves dissociate as they travel into the deeper water. This generally results in a broadening energy spectra as the various wave components travel with their own celerity.

Numerical modeling efforts have met with some success in representing the transformation of weakly non-linear incident waves (Ohyama and Nadaoka, 1993; Driscoll, Dalrymple and Grilli, 1993; Beji and Battjes, 1994; Losada, Silva and Losada, 1996). Although all models are reported to reflect some of the physical modeling data well, none of the approaches can fully model the breaking and non-linear decomposition process on a theoretical basis. Therefore, given the uncertainties associated with the present state-of-the-art in numerical modeling, it may be most appropriate for the design engineer to consider more general design equations based on physical modeling results.



Relatively few design equations have been developed to date for submerged rubblemound breakwaters. Those available have been developed by Seelig (1980) for surface piercing and submerged permeable breakwaters, by Ahrens (1987) for reef-type breakwaters and by Van der Meer (1991) for low crested and submerged rubblemound structures. Seelig's equation was developed with very little submerged breakwater data and Ahren's equation is not directly applicable to conventional submerged rubblemound breakwaters given the reshaping nature of reef breakwaters. Van der Meer's equation was developed from a considerable volume of test data from a number of authors but some variables were not varied to a large degree; a limited variation in crest width was perhaps the most important shortcoming of this research. As a result, none of the existing equations are sufficient for application over a wide range of design conditions.

The objectives of the research presented in this paper are therefore as follows:

- 1. To test a sufficiently wide range of submerged breakwater geometries, ensuring a broad range of crest widths, under a relatively large range of incident wave conditions in a 2-D setting to assess the validity of the existing design equations.
- To extend and modify the existing design equations as necessary to provide some physical basis for the dimensionless parameters utilized.
- To perform 3-D testing for a number of conditions similar to those tested in the 2-D apparatus to assess the validity of the proposed equations for application in more realistic 3-D environments.

The majority of the physical tests were 2-D in nature and were performed in a 1 meter wave flume at QUCERL. These test results provided the data for development of the proposed design equations. Subsequent 3-D testing was carried out in the wave basin at QUCERL using a smaller set of test variables. The results of these tests were used in the evaluation of alternative design equations.

2-D Testing

The testing setup for the 2-D tests is shown in Figure 2. The wave flume is 47.0 m long, 1.2 m deep and 1.0 m wide and is equipped with a flapper type wave generator. A plywood beach was constructed in the flume, upon which the test breakwaters were constructed. The beach permits testing of the submerged breakwaters in relatively large incident waves. The submerged breakwater cross section consisted of a core of relatively course core material ($D_{50c} = 0.017$ m) and two layers of primary armour ($D_{50a} = 0.059$ m). A second armour size ($D_{50a} = 0.037$ m) was used in some tests. The armour size was determined such that the breakwater remained stable during testing. The stone size required for the most severe testing condition was determined using the stability equation of Vidal et al. (1993). The stone size recommended for the "total slope" section was used for the entire breakwater.



Figure 2 : 2-D Testing Configuration

Incident and transmitted waves were measured by two wave probe arrays, located in front and rear of the breakwater respectively. The probes were capacitive type water level gauges, sampling the water surface at 20 Hz. Reflection from the rear wall of the wave flume was minimized using a 1:10 beach of rubberized hair in front of a porous matrix of concrete blocks.

In total, approximately 800 tests were performed with irregular waves. A number of tests were also performed with regular waves in order to confirm the presence of physical phenomenon observed by previous authors. The testing program involved 13 submerged breakwater geometries tested under 5 different water levels with a number of incident

wave characteristics.

All irregular wave spectra tested were Jonswap with $\alpha = 0.0081$ and $\gamma = 3.3$. The signals were generated using the National Research Council of Canada's (NRC), GEDAP wave generation and analysis package. Given the physical characteristics of the flume and the mechanical response of the paddle to the input signal, the generated wave spectral characteristics may vary from the target characteristics by 5% to 10%. A summary of the irregular wave characteristics tested is provided in Table 1.

Wave Set	W01	W02	W03	W04	W05	W06	W07	W08	W09	W10	W11
Hmo (m) *	0.05	0.05	0.05	0.10	0.10	0,10	0.15	0.15	0.15	0.20	0.20
Tp (s)*	1.2	1.5	2.0	1.2	1.5	2.0	1.2	1.5	2.0	1.5	2.0

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* Note: Wave characteristics are target values - those measured in flume may vary to some degree.

3-D Testing

The wave basin at QUCERL is approximately 30 m by 25 m by 1.2 m deep and is equipped with a piston-type wave paddle 10.5 m long. All dimensions for the 3-D tests were scaled at 0.63 times those of the 2-D tests and the submerged breakwater structure was constructed on the floor of the basin. A concrete beach was constructed behind the breakwater and covered with sand to minimize the reflection and provide a qualitative assessment of beach development. A subset of testing parameters was used in the 3-D tests, with 3 breakwater geometries and 3 water depths, all tested at incident angles of 90 ° and 60°. The general testing configuration is shown in Figure 3 below.





The submerged breakwaters used in the 3-D tests were constructed using a relatively fine core material ($D_{50c} = 0.004$ m) and the smallest armour tested in the flume ($D_{50a} = 0.037$ m). The wave signals were generated from Jonswap spectra. Because the breakwater for the 3-D tests was not constructed on an elevated platform, it was not possible to generate waves as large as those used in the 2-D tests. A summary of the irregular wave characteristics used in the 3-D tests is provided in Table 4.

Wave Set	W01	W02	W03	W04	W05	W06	W07	W08	W09	W10	W11
Hmo (m)*	0.032	0.032	0.032	0.032	0.063	0.063	0.063	0.063	0.095	0.095	0.095
Tp (s)*	0.95	1.19	1.59	1.98	0.95	1.19	1.59	1.98	0.95	1.19	1.59

Table 2 : Irregular Wave Characteristics in 3-D Tests

* Note: Wave characteristics are approximate - those measured in basin may vary to some degree.

Water level probes were placed throughout the area behind the submerged breakwater, and one probe was located near the paddle to provide an indication of the incident wave height. Two velocity probes were also moved throughout the area behind the submerged breakwater to provide a qualitative indication of the general velocity patterns. The locations of the various probes are shown in Figure 3 above.

Data Sampling and Analysis

Sampling for each test was performed at 20 Hz over a period of 100 waves. Full reflection analysis was undertaken for the 2-D data to separate the incident and reflected spectra. The reflection analysis was performed using a least squares analysis of 3 probes of the 5 probe array (Mansard and Funke, 1987). The incident wave characteristics at probe arrays 1 and 2 were used to define the transmission coefficient K_t such that:

$$K_t = \frac{H_t}{H_t} \tag{1}$$

where H_i and H_i are the incident H_{m0} values at probe arrays 2 and 1 respectively.

The significant wave height (H_s) at individual probes was used to define the transmission coefficient for the 3-D tests. The incident wave was defined by Probe 5 and the transmission coefficient was computed at various locations behind the submerged breakwater. Although the use of H_s instead of H_{m0i} to define K_i is not consistent with the analysis of the 2-D data, it reduces the effect of energy that has leaked into the testing area or been reflected into the lee of the breakwater, on the characteristic wave height estimate.

The transmission coefficient values were analyzed with respect to the various incident wave and structure characteristics. This analysis involved a simple graphical trend analysis of the data, followed by a comprehensive statistical analysis of alternative design equations relating K_t to the most important design variables. The trend analysis was conducted with

dimensional and dimensionless variables considered to be important to the transmission process. The most important dimensionless variables were ascertained by dimensional analysis of the transmission process.

It is generally accepted that the transmitted wave at a submerged breakwater is a function of a number of variables.

$$H_{t} = f(\rho, g, \mu, D_{50a}, n, H_{t}, L, d_{s}, \theta, B, h_{s}, h)$$
(2)

where ρ , g and μ are density, gravitational acceleration and dynamic fluid viscosity respectively and the other variables are defined in Figure 1. A dimensionless form of this expression can be developed using ρ , g and H_i as basic or repeating variables such that:

$$K_t = \frac{H_t}{H_j} = \varphi \left(\frac{\rho H_{ij} \sqrt{H_i g}}{\mu}, \frac{D_{s_{0_a}}}{H_i}, n, \frac{L}{H_i}, \frac{d_s}{H_j}, \theta, \frac{B}{H_j}, \frac{h_s}{H_i}, \frac{h_j}{H_j} \right)$$
(3)

These basic dimensionless variables were used to develop alternative dimensionless variables which are more relevant to the physical processes affecting transmission at submerged breakwaters. Dimensional and statistical analysis of these relevant variables was undertaken to develop a design equation for transmission at submerged rubblemound breakwaters.

3.0 Results

The observations made during testing were generally consistent with observations noted from previous investigations. The incident wave spectra were broadened with a shift of energy to higher frequencies as the waves passed the breakwater and there was evidence of harmonic generation and subsequent dispersion as individual waves passed over the structure. The most obvious process affecting K_i was wave breaking. Inspection of the general trends defined by the 2-D test data indicate that K_i is most sensitive to the depth Gsubmergence d_s , the incident wave height H_i and the crest width B. To a lesser degree, Kis influenced by the period of the incident wave (T_p) , the breakwater armour dimensions (D_{50a}) and the breakwater slopes (θ) .

Typical trends observed during the 2-D tests are summarized in Figures 4 through 8. It is evident that the transmission increases with increased d_s , increased H_i and increased B. I is also shown that a small increase in K_i is observed with increasing T_{p} , increasing D_{50a} an steeper slopes (increasing Tan θ).



Figure 4 Effect of d, and H, on K,

0.15

A 2

Ktys ds (B=0.6 m. Tp ~ 2.0 s)

1

n 9

0.8

0.7

0.6

205

0.4

0.3

0.2

0.1

۵

۵

0.05

n.1

ds (m)



Figure 6 Effect of B and T, on K,



ds (m)



Figure 7 Effect of t and T, on K,



Figure 8 Effect of D_{50a} and T_n on K,

Three dimensional test data showed similar trends in general but the scatter in the data was much more evident. The transmission coefficients were generally higher in the 3-D tests: this is attributed to a number of factors including diffraction of wave energy into the lee of the breakwater and reflection of wave energy from the testing apparatus. The results show that the relative submergence is the most influential factor under low submergence conditions while crest width is important under higher submergence conditions.

The effects of submergence depth, incident wave height and crest width are shown in Figure 9. This figure is based on the transmission coefficient immediately behind the midpoint of the breakwater with incident waves perpendicular to the structure.



Figure 9 : Typical 3-D Testing Results

The predicted values of the transmission coefficient for the 2-D test variables were generated using van der Meer's Equation for submerged and low crested breakwaters and Ahren's Equation for reef breakwaters, and are shown in Figures 10 and 11.



Figure 10 : Predicted K, - Ahrens' Eqn.

Figure 11 : Predicted K, -Van der Meer's Eqn.

The results show that these equations are not suitable to represent transmission for structures tested in this study, particularly when the crest width is large. The inability of the existing design equations to predict suitable transmission coefficients for the tested conditions indicates that there is a need for an improved design equation.

4.0 Development of an Improved Design Equation

Previous investigations have indicated that d_s/H_{m0l} is the most important dimensionless variable affecting transmission. This observation was supported by these tests. Numerous dimensionless variables considered, including variables discussed in previous authors works. Only those found to be significant in defining the transmission process are discussed here.

Dimensionless variables representing wave breaking, overtopping, frictional losses and internal flow losses were found to be important in defining the transmission process. These variables are discussed below.

i. The wave breaking process is considered to be represented by the dimensionless variable d_s/H_{m0i} (relative submergence). The relationship between K_t and d_s/H_{m0i} for all of the 2-D test data is shown in Figure 12. The effect of relative submergence is very evident when d_s/H_{m0i} is small. This is expected since the majority of waves are breaking and under breaking wave conditions, the unbroken wave (transmitted) has been found to be closely related to the water depth (d_s) . Given the scatter, there are obviously other factors playing an important role in the process as well.



Figure 12 : Effect of Relative Submergence

As the relative submergence increases, its influence is reduced substantially. This is expected as the relative portion of the incident waves which would break is reduced as the submergence depth increases.

ii. The effect of overtopping is expected to be relatively important in defining the transmission coefficient at submerged breakwaters, especially under wave breaking conditions. As the relative submergence approaches zero, transmission at the submerged breakwater was observed to become a function of the potential for overtopping as well as transmission through the breakwater structure, in particular, the large armour crest material.

Typically, overtopping rates are a function of the wave steepness and structure geometry. A dimensionless form of the structure crest width and the local wave height (H_{m0}/B) was found to be representative of the overtopping effect.

- iii. As the wave passes over the breakwater, some energy is lost to frictional dissipation on the surface of the structure. The dimensionless variable representing this process was loosely based on the empirical Darcy-Weisbach expression for head loss. This requires the assumption that the flow velocity can be represented by the velocity of a gravity wave and results in the dimensionless variable $d_s H_{m0}/BD_{50a}$.
- iv. Flow within the breakwater structure will also result in some energy loss as a wave travels over a submerged breakwater. Given the relatively high porosity of the armour layer, the effect of the wavelength on the fluid velocity, the effect of submergence depth on the portion of flow within the armour layer and the effect of the crest width on the overall drag losses, the dimensionless variable selected to represent drag losses was Bd/LD_{50a} .

On the basis of these dimensionless variables, statistical fitting was used to develop a suitable design equation. A number of alternative equations were considered in an effort to develop a design equation which provided:

- a good statistical fit (R²),
- a normal distribution of residuals,
- predictions which are well bounded (ie. $0.0 \le K_r \le 1.0$),
- physically relevant variables with a minimum number of fitted parameters

Through trial and error, a finalized form of the design equation was developed such that these criteria were generally satisfied for the 2-D test data.

$$K_{i} = 1 - \left(e^{\frac{-0.65(\frac{d_{i}}{H_{i}}) - 1.09(\frac{H_{i}}{B})}{L D_{50_{a}}} + 0.047(\frac{B d_{s}}{L D_{50_{a}}}) - 0.067(\frac{d_{s} H_{i}}{B D_{50_{a}}})\right)$$
(4)

The proposed equation fits the 2-D test data well, resulting in an R^2 value of 0.914. Given the uncertainty associated with the fitting of statistical parameters, all of the parameters of the proposed design equation were adjusted to their upper or lower 95% confidence interval values such the change in predicted K_i value was maximized. The results, as shown in Figure 13 show that the prediction is not very sensitive to these changes and as a result, the prediction is relatively robust in its relation to the physical variables affecting the transmission phenomenon.



Figure 13 : Sensitivity to Parameter Estimates

Although the proposed equation does not fit the 3-D data as well, the prediction is still relatively good at low to moderate transmission coefficients (Figure 14).



Figure 14 : Observed vs Predicted K, - 3-D Tests

5.0 Conclusions

Based on an extensive set of 2-D and 3-D tests of wave transmission at submerged breakwaters, a number of conclusions can be drawn.

i. The transmission coefficient at submerged breakwaters is most sensitive to the

relative submergence (d_s/H_i) .

- ii. The relative crest width is another very important factor which has not been adequately accounted for in previous design equations.
- iii. An improved design equation for transmission at submerged breakwaters would be:

$$K_{i} = 1 - \left(e^{-0.65\left(\frac{d_{i}}{H_{i}}\right) - 1.09\left(\frac{H_{i}}{B}\right)} + 0.047\left(\frac{B \, d_{s}}{L \, D_{50}}\right) - 0.067\left(\frac{d_{s} \, H_{i}}{B \, D_{50}}\right)\right)$$
(4)

- iv. The proposed equation represents the comprehensive set of test data well (R²=0.914) and is robust in its relation to the physical variables which significantly affect the transmission process.
- v. The equation is well bounded over the range of test data, which is considered to be representative of typical design conditions. The equation does, however, become unbounded when B becomes very large or very small due to the size of the 3rd and 4th terms. Therefore, it is recommended that caution be used when applying the equation outside of the following variable ranges.

$$0 \le \frac{B d_s}{L D_{50_a}} \le 7.08$$

$$0 \le \frac{d_s H_i}{B D_{50_a}} \le 2.14$$
(5)

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