CHAPTER 136

Model Study of Reservoir Riprap Stability

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Abstract

A series of large scale (1:15) experiments have been undertaken to assist the Societé d'Énergie de la Baie James (SEBJ) in the development of new design guidelines for riprap. The stability of revetments resting on 1:1.8 and 1:2.25 (V:H) slopes is considered. In an attempt to quantify the relative roles of stone size, gradation and armour layer thickness, these tests were undertaken with stone gradations varying from (M_{max}/M_{min} ranging from 1 to 10) and with armour layers both $2.2D_{n50}$ and $2.7D_{n50}$ thick.

Introduction

The La Grande Hydroelectric complex includes over 100 km of earth dykes. Over the past 15 years a small percentage of these dykes have suffered wave damage and have needed repair. (Caron et al, 1993, Levay et al, 1993 and 1994). The Societé d'Énergie de la Baie James (SEBJ) was mandated to review the designs of these dykes and contracted the Canadian Hydraulics Centre (CHC) to undertake a physical model testing program. In the first phase of this study, repair schemes were developed for several dykes which have experienced damage (Mansard et al, 1994) The second phase of this study focused on improving riprap design techniques. This paper reviews some of the findings of this second phase.

In undertaking the original construction and the repair works, SEBJ has garnered extensive experience with riprap design and construction. Their experience suggests that riprap specifications based on a minimum acceptable stone size (M_{min}), along with controls on stone gradation can provide an efficient and practical method for design, construction and quality control (Tournier et al., 1996).

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Experimental configuration

The cross-sections studied in these experiments were typical designs for earth dykes in reservoirs exposed to wave action (see Figure 1). These are adaptations of the earth dyke designs used in the La Grande Hydroelectric Complex. A Froude-scaled model of these cross-sections (scale 1:15) was built in a 2m wide flume built within the Multidirectional Wave Basin (MWB) at the Canadian Hydraulics Centre. The basin configuration is shown in Figure 2. This layout allowed waves reflected by the structure to diffract and dissipate as they propagate towards the wave generator. This minimises the presence of re-reflected waves at the test section.
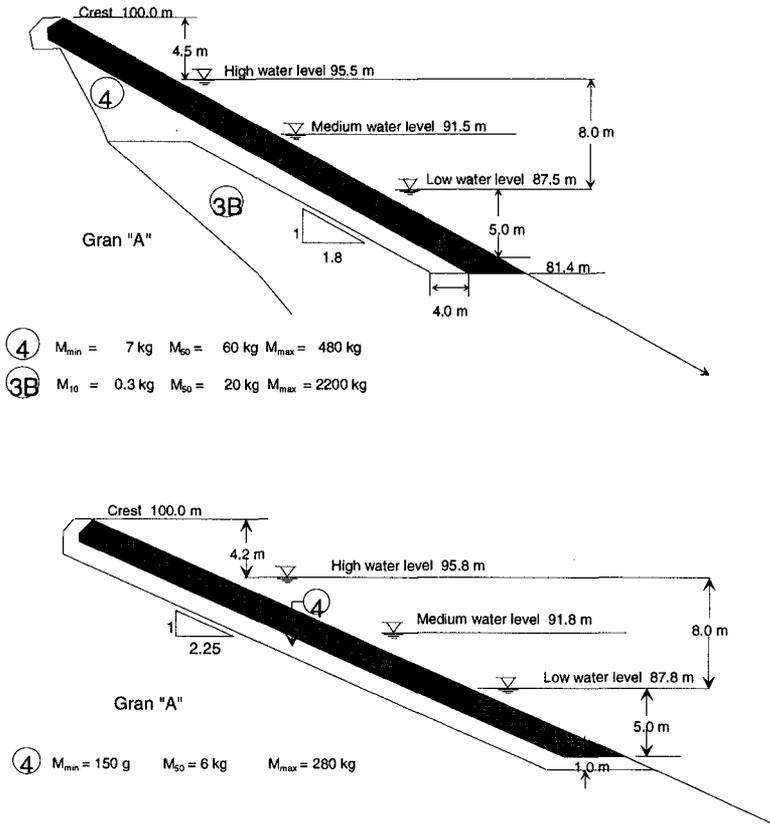

Figure 1 Cross-sections used in test program

Armour layer gradations for the model were obtained from crushed limestone from a local quarry. This rock was first mechanically sorted using the CHC's high capacity rock sizing facility. The tightly controlled gradations required for the armour layer were than obtained by individually weighing the stones from the mechanical sorting. Approximately 1 tonne of armour stone was required for each test configuration.

Test Conditions

Table 1 summarises the tests undertaken. The two slopes were both nominally designed to withstand a design storm of $H_s=2.5$m. This resulted in median stone masses (M_{50})for the two slopes of 2600 and 2100kg for the 1:1.8 and 1:2.25 slopes, respectively. This typical design is listed at Tests 1 and 4 in Table 1. Tests were conducted with extremely uniform stone distributions where the median stone mass was set equal to the value of the minimum stone mass of the initial tests (see Tests 2, 3, 5 and 6 in Table 1). These uniform gradations of minimal stone size were tested with layers both $2.2D_{n50}$ and $2.7D_{n50}$ thick. Additional tests were also undertaken with two broader gradations $M_{max}/M_{min}=5$ (Test 7), and $M_{max}/M_{min}=10$ (Test 8) along with a final test (Test 9) with a uniform gradation ($M_{max}/M_{min}\approx1$) in which the median stone mass was the same as in the original design. Tests on the typical designs were conducted at three water levels; high, medium and low, in order to establish any potential influence of overtopping and toe instability on the overall stability of the dyke.

Table 1 Test Program

Test	Slope	M_{50} [kg]	D_{n50} [m]	Armour layer thickness, t_a [m]	Filter layer thickness [m]	Gradation M_{max}/M_{min}	Elevation of SWL [m]
1	1:1.8	2600	0.99	2.2 (2.2)D_{n50}	1.9 (1.92D_{n50})	2.5	95.5, 91.5 and 86.5m
2	1:1.8	1500	0.82	2.2 (2.7)D_{n50}	1.9 (2.32D_{n50})	1	91.5
3	1:1.8	1500	0.82	1.8 (2.2)D_{n50}	1.9 (2.32D_{n50})	1	91.5
4	1:2.25	2100	0.92	2.0 (2.2)D_{n50}	1.5 (1.63D_{n50})	2.5	95.8, 91.8 and 87.8 m
5	1:2.25	1200	0.76	2.0 (2.7)D_{n50}	1.5 (1.97D_{n50})	1	91.8
6	1:2.25	1200	0.76	1.6 (2.2)D_{n50}	1.5 (1.97D_{n50})	1	91.8
7	1:1.8	2600	0.99	2.2 (2.2)D_{n50}	1.9 (1.92D_{n50})	5	91.5
8	1:1.8	2600	0.99	2.2 (2.2)D_{n50}	1.9 (1.92D_{n50})	10	91.5
9	1:1.8	2600	0.99	2.2 (2.2)D_{n50}	1.9 (1.92D_{n50})	1	91.5

Note: Tests were performed at a scale of 1:15, the structures had a crest elevation of 100.0m, and the basin floor was at elevation 59m. This table reflects target values for gradations and armour layer thickness.

Figure 1 shows the three water levels tested. Water depths and freeboard (the vertical distance between the still water level and the crest of the structure) were varied in the tests to cover the full range of likely operating conditions for the reservoirs. Table 1 lists the water depth for each test. The majority of the tests reported here were conducted at the medium water level, that is with a water depth of 32.5m and a freeboard of 8.5m. This represents a non-overtopping condition for the design wave height of $H_s=2.5$m, and the structure's behaviour approaches that

of a uniform, infinite slope. The milder 1:2.25 sloped structure required a slightly lower stone mass for stability and required less freeboard to minimise overtopping (i.e. 4.2m instead of 4.5m).

Wave conditions used in test program

All sea states used in this study were irregular waves based on the JONSWAP spectrum (γ=3.3). Waves were synthesised using the method of random phases as described in Funke et al (1988). The time series of each sea state was chosen to be rather long in order to minimise any statistical variability of wave parameters associated with short records. The length of time series corresponded therefore to two hours of storm duration at prototype scale, containing approximately 1100 to 2600 waves depending on wave period (see values of N in Table 2). This choice of long records also ensured that the wave heights fit a Rayleigh distribution. The ratio of H_{max}/H_s was between 1.8 and 2.0 for all sea states. Table 2 summarises the wave conditions used in this study.

Wave Calibration

Generally, waves are calibrated in the basin (with an efficient absorber in place) prior to building the breakwater. During the testing program (with the breakwater in place) reflection analysis is performed on the measured wave records to separate the incident and reflected significant wave heights. Since the design of the 2 m section within the 30 m wide basin ensured a good diffraction zone for minimising the re-reflected components (see Figure 2), good agreement was obtained between the incident wave heights measured during calibration and those measured during the testing program. (This comparison can be found in Mansard et al, 1994). This good agreement, in conjunction with the CHC technique for dynamic wave machine calibration, allowed the use of the reflection analysis results (with the structure in place) for establishing incident wave conditions during the tests. Furthermore, during this testing program, CHC had developed and validated a new algorithm that can provide not only the significant wave heights but also the statistics of the incident wave train (Mansard, 1994).

Experimental Procedure

Testing involved the exposure of the structure to a sequence of wave conditions of increasing severity. Testing started at a 'no-damage' wave height and built in 0.5 m wave height increments to a wave height sufficient to cause major damage to the revetment (see Table 2). Each sea state that induced damage to the breakwater was run for approximately 5000 waves (corresponding to 8 hours of storm duration). For this test program, sea states with H_s<2.0 m did not cause any significant damage to the structure.

The 5000 wave duration was chosen to represent relatively long-term exposure at each storm level. This is supported by the work of Thompson and Shuttler

(1975) and by van der Meer (1988) who both show damage evolution varying logarithmically with time and reaching roughly 80% of the maximum possible damage levels within 5000 waves.

Table 2 Wave conditions used in testing.

H_{m0} [m]	T_p [s]	Number of waves per cycle, N	Number of cycles tested	Total number of waves, N_{tot}
1.0	4.0	2600	1	2600
1.5	5.0	1750	1	1750
1.75	5.4	1613	1	1613
2.0	6.0	1487	4	5948
2.25	6.4	1377	4	5508
2.5	6.75	1328	4	5312
2.75	7.25	1239	4	4956
3.0	7.5	1162	4	4648
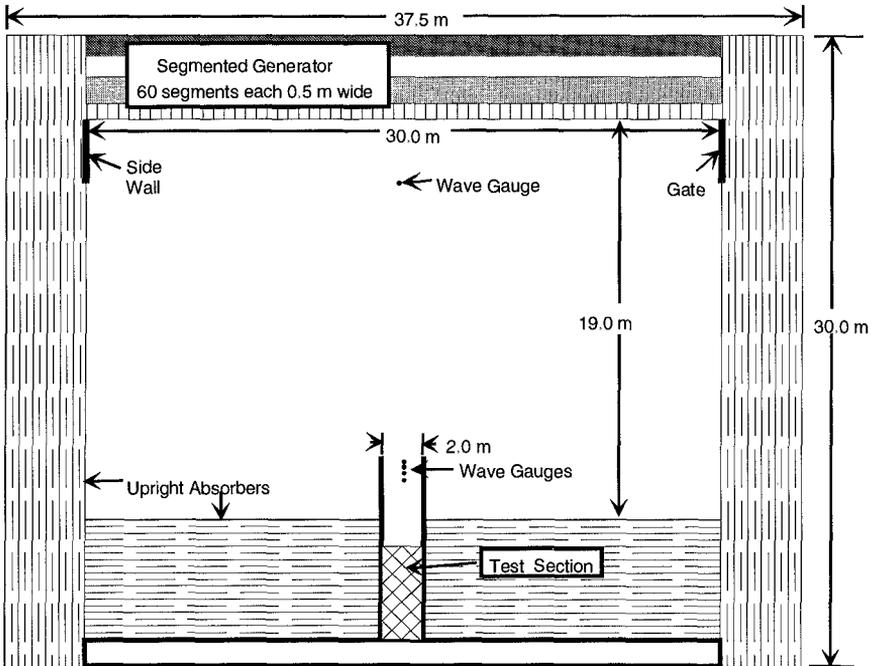
Figure 2 Test layout in Multidirectional Wave Basin

Measurement of damage

During construction of the underlayer and the armour layer the NRC electro-mechanical profiler was used to ensure model tolerances were met. This profiling system is described in Davies et al, 1994. This instrument uses a contact wheel on a pivot arm which is mounted to a traversing carriage. High-precision potentiometers

are used to measure the horizontal and vertical location of the contact wheel on the revetment slope. The slope is located using a counter-balanced mechanical contact wheel. This technique provides profiles of the structure accurate to within ± 2 mm and can be used either above or below the water line. This avoids the need for either draining or flooding the basin as is common with other techniques such as acoustic or electrical resistivity methods.

Profiles are analysed to determine the eroded cross-sectional area, A, by comparing the damaged profile at the end of each storm sequence to the original profile. The damage index, S, is then calculated as:

$$S = A/D_{n50}^2 \text{ where } D_{n50} = \sqrt[3]{M_{50}/\rho_s} \qquad \qquad \textbf{Equation 1}$$

where ρ_s is the density of the armourstone [kg/m^3]. Damage was also measured by counting the number of stones displaced. For low damage levels these two methods provide comparable results (see Davies et al, 1994), while for high damage levels, the counting of stones become impractical.

Another damage index used in this study is the concept of depth of cover, d_c, presented in Davies et al (1994). This index provides a measure of the thickness of protection that remains on the breakwater section after every storm sequence rather than the area eroded from it. It is calculated as the minimum slope-normal armour layer thickness within the zone of wave action. This is based on the measured armour layer profile and the profile of the filter layer obtained during construction. Viewing damage in terms of the depth of cover remaining on the structure can facilitate the interpretation of the results, particularly if the breakwater sections is not a classical 2-layered section.

Test results for typical design revetment

Figure 3 shows typical damage progression for the 'typical revetment', with M_{max}/M_{min}=2.5 results are presented in terms of the damage level, S as a function of the cumulative number of waves to which the structure was exposed. Note that only the data at high and medium water levels are shown here since a different test protocol was followed for the low water level test. The low water level test was carried out only at the design wave height mainly to verify that there was no toe instability when the level of protection extends to 2 times the design significant wave height below the low water level.

Figure 3 shows that the damage progression curves for the 1:1.8 sloped structure are similar at the high and medium water levels. During the H_s=3.0m test at the high water level, reduced damage levels were observed. This is possibly attributable to the high degree of overtopping observed during this test.
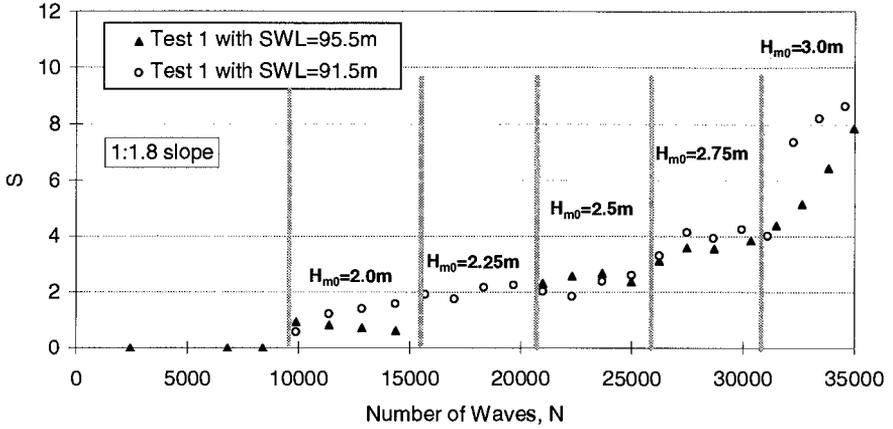
Figure 3 Damage progression for 1:1.8 slope - typical revetment (Test 1).
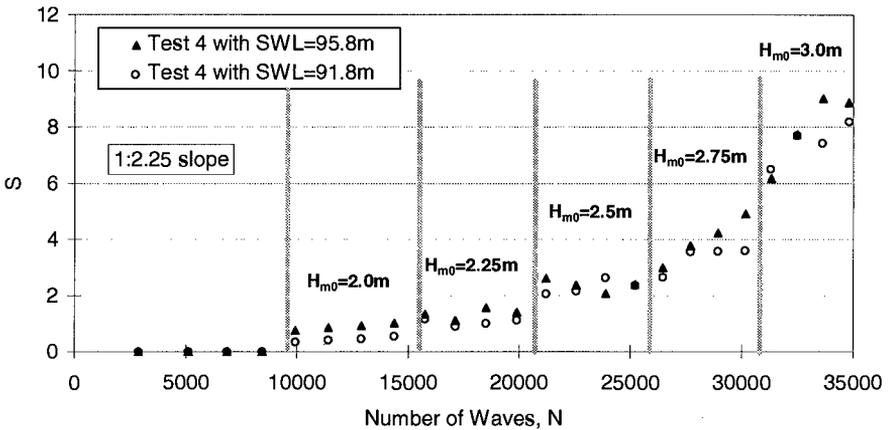

Figure 4 Damage progression for 1:2.25 slope - typical revetment (Test 4).

Similar damage evolution curves were observed for the mild sloped structure. Figure 4 shows the good agreement between the high and medium water level results for a 1:2.25 slope.

The Shore Protection Manual (SPM 1977, 1984) uses Hudson's formula to evaluate riprap stability as a function of stone size, incident wave height, and structure slope. Using the stability number, $H_o = H_s/(\Delta D_{n50})$, Hudson's relationship can be expressed as:

$$S = \varphi \left(H_o^3 \middle/ \cot\theta \right)$$

The coefficient, K_d is the threshold for the onset of damage, commonly taken to be at S=2. K_d is the value of $H_o^3/\cot\theta$ at S=2. Figure 5 presents the S values, obtained after 5000 waves at each sea state, plotted as a function of $H_o^3/\cot\theta$. Through inclusion of the structure slope, θ, in the abscissa, both the steep and mild sloped structures can be viewed on the same plot. Results from the low water level test are also included in this figure. It can be seen from this figure that for a damage equivalent to S = 2, the K_d value ranges between 1.2 to 1.7. Note that this value of K_d is based on the significant wave height (H_s) of the sea states and not on the value of $H_{1/10}$ proposed in the Shore Protection Manual of 1984. Davies et al (1996) compared these experimental results with predictions using the formulae of van der Meer (1988). A good match between measured and predicted values exists for values of van der Meer's permeability factor P around 0.14. Here, the van der Meer formula was applied with N=5000 waves. For a more exact comparison, the cumulative effects of antecedent storm conditions should be included.

Overall results

The results discussed so far correspond only to the first test series where graded revetment with M_{max}/M_{min} = 2.5 was used. A similar comparison of the entire dataset in one figure is difficult because of differences in the stone masses, stone gradation and layer thicknesses listed in Table 1. Furthermore, the damage index S discussed above is the eroded area normalised by the square of the nominal diameter of the stone which is also different in the subsequent tests. It is possible, however, to review the ensemble of the results using simpler parameters such as eroded area, A, and significant wave height, H_s.
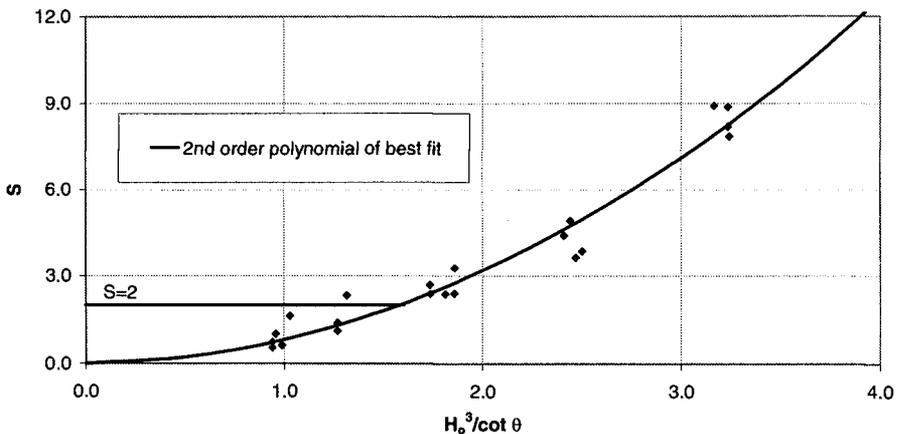

Figure 5 Damage S vs $H_o^3/\cot\theta$ - results for both Test 1 (1:1.8 slope) and for Test 4 (1:2.25 slope).

Figure 6 shows the values of A vs H_s for all the structures listed in Table 1. Note that for the graded revetment the averages of the values obtained during high and low water levels are presented here. This figure shows that the eroded area remains nearly the same for all configurations -- there is little difference in the values of the eroded area of the structures $2.2D_{n50}$ thick, whether they are made up of uniform or graded revetment. There is however a trend for the eroded area to be smaller when the layer thickness increases from 2.2 to $2.7D_{n50}$. The following sections provide different interpretations of the dataset which more clearly illustrate the effects of these various parameters.
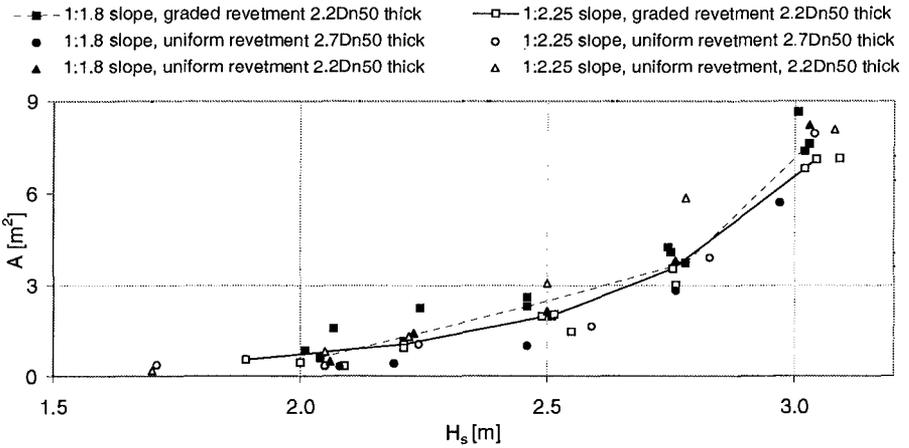
Figure 6 Eroded area, A vs significatn wave height, H_s.

Effect of uniform gradation

Figure 7 presents the results of the graded and uniform revetments $2.2D_{n50}$ thick in terms of S versus $H_o^3/\cot\theta$ for the 1:1.8 slope. Here, we have not included test results performed with the reduced median mass (i.e. Tests 2 and 3). Tests 2 and 3 were performed using the same filter layer geometry as the rest of the tests, consequently, the relative permeability of these tests is higher (i.e. the ratio of filter layer thickness to D_{n50} is larger for these tests). This figure shows a trend for uniform revetments to provide higher stability. The influence of gradation is small, particularly at low damage levels. Values of M_{max}/M_{min} between 10 and 2.5 all show quite similar stability. For extremely narrow gradations (M_{max}/M_{min} approaching 1), stability is seen to increase.

Figure 8 shows the influence of gradation in terms of the value of $H_o^3/\cot\theta$ required to cause a given damage level (S=2 and S=8). This figure shows that the extremely uniform gradation is more stable than the broader gradations. Similar analysis was undertaken by van der Meer (1988) – in considering two gradations

(equivalent to M_{max}/M_{min} of 11 and 2) he concluded that gradation had "no or minor influence on the stability and that, within this range, the armour layer can be described simply by the nominal diameter, D_{n50}". The present work is in general agreement with these findings, however, these results show that for extremely narrow gradations ($M_{max}/M_{min}<2.5$) the gradation can significantly improve stability.
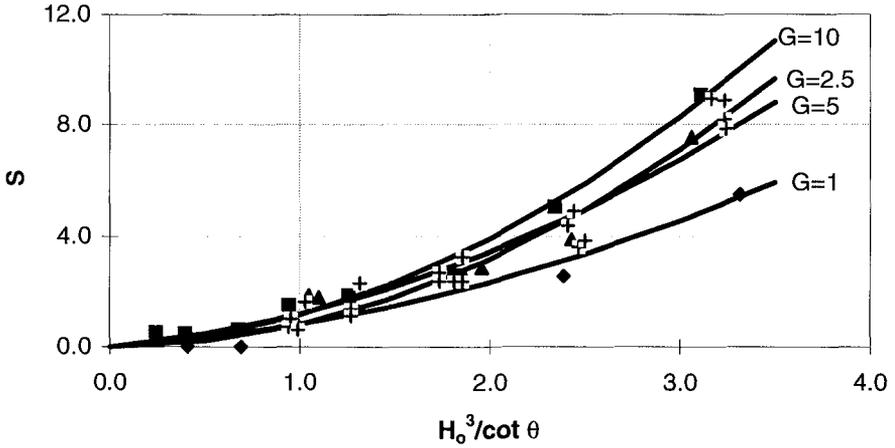

Figure 7 Influence of gradation, 1:1.8 slope (Tests 1, 7, 8 and 9)
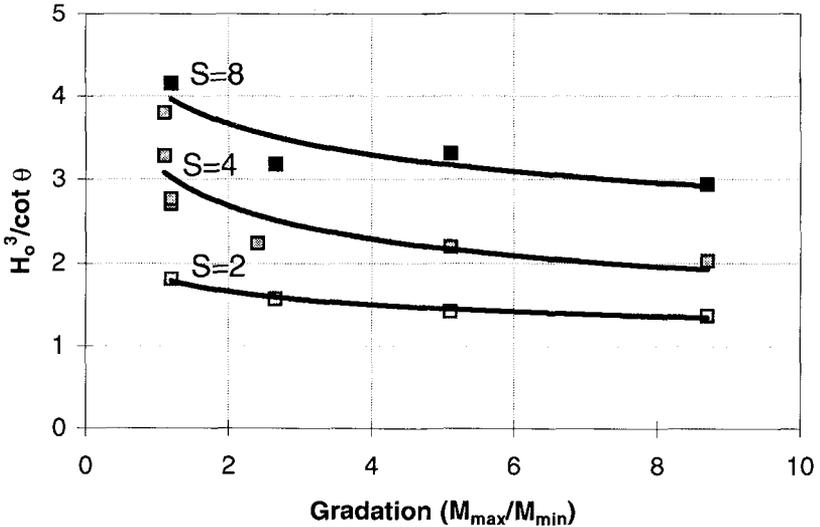
Figure 8 $H_o^3/\cot\theta$ to cause a given damage level as a function of gradation (Tests 1, 7, 8 and 9).
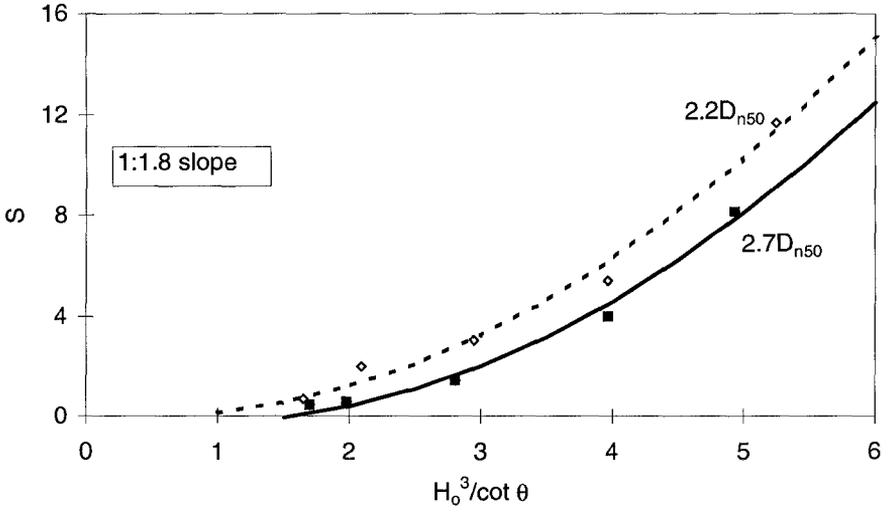
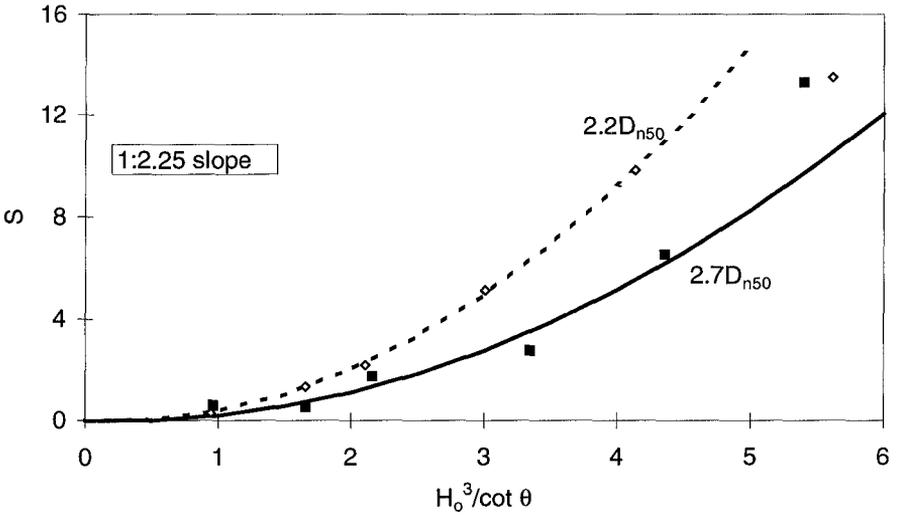**Figure 9 Effect of layer thickness for 1:1.8 slope (Tests 2 and 3).**

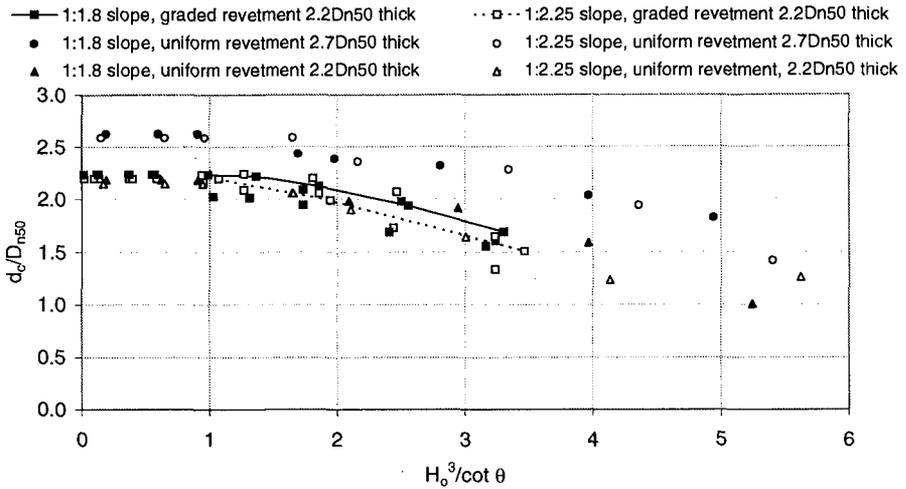**Figure 10 Effect of layer thickness for 1:2.25 slope (Tests 5 and 6).**

Figure 11 Response in terms of depth of cover, d_c

Effect of layer thickness

Figures 9 and 10 show the effects of layer thickness for the steep and mild slopes, respectively. All other test conditions were held constant for these tests so that layer thickness is the only variable. It can be seen that the thicker armour layer ($2.7D_{n50}$) provides better stability. This is as would be expected through the influence of permeability – as layer thickness increases, the ability of the slope to 'absorb' wave-induced flows increases.

The influence of layer thickness can also be viewed in terms of the depth of protection, d_c. Figure 11 presents the depth of protection normalised with respect to the nominal median diameter, d_c/D_{n50} as a function of $H_o^3/\cot \theta$. As one would expect, the tests with greater layer thickness show a smaller loss in the depth of protection compared to its counterparts with two layers. Furthermore, the remaining protection on the structure is also higher – that is, not only are the damage levels lower (i.e. the amount of material removed is reduced) but, by virtue of the three-layer thickness, the structure can tolerate a higher damage level before the underlayers are endangered.

Conclusions

This study has led to the creation of a large-scale dataset on revetment stability. The parameters S and $H_o^3/\cot \theta$ have been seen to accurately describe the revetment stability. The results for the typical revetment design lead to values of K_d ranging between 1.2 and 1.7 (for a damage level of S=2). In establishing these K_d values the median mass of the revetment and the significant wave heights of the sea states were used.

Uniform stone distributions have been shown to provide better stability than graded revetment. However, the influence of gradation is small for ratios of M_{max}/M_{min} between 2.5 and 10. For uniform stone distributions, increased layer thickness reduces actual damage levels and increases tolerable damage levels. This response is described well by the remaining depth of cover, d_c.

Application of these test results into practical design techniques for dam revetment have been undertaken by the Societé d'energie de la Baie James and are presented by Tournier et al (1996).

Acknowledgements

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