8th INTERNATIONAL CONFERENCE ON COASTAL AND PORT ENGINEERING IN DEVELOPING COUNTRIES

COPEDEC 2012, IIT Madras, Chennai, INDIA.
20-24 Feb. 2012

ARMOURSTONE FOR THE ICELANDIC-TYPE BERM BREAKWATER

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Abstract: This paper discusses the design concept of Icelandic-type berm breakwater and the key differences in the use of armourstone compared with mass armoured breakwater designs. Through reference to two breakwater case studies, a comparison has been provided on the damage performance of each design when exposed to wave conditions up to and exceeding the design event.

In contrast to mass armoured breakwaters, the Icelandic-type berm breakwater is designed to be statically stable with only limited reshaping. As it only allows limited stone movement on the reshaped profile, it overcomes the problems of degradation and sorting of the armourstone and therefore maintains its stability and overtopping performance throughout its design life.

This is an important outcome for those responsible for the ongoing maintenance and upkeep of breakwater structures, in particular reducing annual repair costs and extending the period over which any major reconstruction will be required.

***Keywords**: berm breakwater; mass armoured breakwater; armourstone; quarry yield; breakwater maintenance.*

INTRODUCTION

The purpose of this paper is to explain the design concept of the Icelandic-type berm breakwater and identify its advantages in regards to damage performance when compared to mass armoured breakwater design. This has included a comparison of the damage performance of each design concept through reference to two breakwater case studies where wave conditions greater than the design event have been experienced since construction.

The Mass Armoured Breakwater

Australia played an important role in the early development of berm breakwaters with many innovative structures being built in the 1970's and early 1980's.

The design of the Hay Point tug harbour in Queensland utilised the experience from the above mentioned structures, Bremner et al. (1987). Interpretation of preliminary quarry investigations and trial blasts in a nearby quarry assumed a maximum available rock size of 2 to 3 tonnes. However further investigations showed that it was possible to quarry armourstone of 3 to 7 tonnes in large quantities. The development of design utilising these armourstone lead to a definition of the mass armoured breakwater that is designed and built in an initially unstable form, but with sufficient material provided to allow natural forces to modify its shape to a stable profile. Among the advantages of the mass armoured breakwater is the use of natural rock in its available sizes.

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The Icelandic-type Berm Breakwater

The Icelandic-type berm breakwater concept has been in development over the past 28 years and nearly 40 structures have been constructed worldwide for a wide range of wave climates, water depths, tidal and subsoil conditions. Since the year 2000, several projects have made use of extra large armourstone, with the largest armourstone class weighing more than 15 to 20 tonnes.

The Icelandic-type berm breakwater is built up of several narrowly graded armour classes with the larger classes placed at the most exposed locations within the breakwater cross section. These narrowly graded armour classes have a higher porosity than wider graded armour classes and therefore higher permeability, which increases the stability of the structure and decreases both the overtopping and reflection from the structure.

One of the aims in designing an Icelandic-type berm breakwater is to maximise the possible output of the armourstone quarry, often specifying the largest class armour rock in the range 20 to 35 tonnes. There is already good experience in Iceland and Norway with the quarrying and handling of this range of rock classes and further studies have been undertaken in regards to the quarrying of these stone classes in other countries (e.g. Sigurdarson et al, 2009).

AVAILABILITY OF LARGE ARMOURSTONE

Quarry yield prediction has played an important role in the design phase of harbour breakwater projects in Iceland since the early 1980's. The prediction is based on analysing drilled cores from the potential rock mass. It has proven to be a valuable part of the design process in preparation for successful breakwater projects. Preliminary designs are based on initial size distribution estimates from potential quarries, and the final design is tailored to fit the selected quarry. Quarry selection is a process which aims at providing rocks best suited to the wave conditions of the construction site and at the same time minimising transport costs and environmental disturbance.

Often the owner/designer has to rely on the contractor or quarry operator for information on the maximum quarry yield or the size of the largest stones obtainable from the quarry. These estimates are very often biased by the size of equipment the contractor/quarry operator has available.

Dedicated armourstone production, and in particular large armourstone production, is not common and therefore there are not many contractors with sufficient experience in this field. Current guidelines for blasting for armourstone are also insufficient and only a few contractors have much experience in drilling and blasting for breakwater construction. It is therefore important that the supervision team has the expertise to supervise the quarry management.

The availability of armourstone is a very important aspect in the planning and design of breakwater projects. This is particularly true for the design of the Icelandic-type berm breakwater where the information on the availability of large armourstone is regarded as equally important as the information on the wave loads which the structure will be exposed to.

In many countries it is believed that armourstone quarries only yield up to 6 to 8 tonnes armourstone and rarely 20 tonnes armourstone. But in many cases it is possible to improve the yield of large amourstone with proper measures.

Recent cases of the Icelandic-type berm breakwater illustrate that optimisation of the extraction process has to have a focus on the potential for production of large blocks for armourstone right from the outset of the quarry development.

Large armour stones will not be available from the blasting pile unless it is properly planned and the contractor is executing blasting and other production activities appropriately, typically with the technical assistance of the design/supervision team and others with experience in producing large armourstone.

COMPARISON BETWEEN MASS ARMOURED BREAKWATER AND BERM BREAKWATER

This section compares two different design approaches for berm breakwaters, the mass armoured breakwater and the Icelandic-type berm breakwater by comparing two prototype structures.

The Mackay Small Craft Harbour breakwater was designed as a mass armoured structure for a design wave height of about 4 m. On the other hand, the Sirevåg berm breakwater was designed as an Icelandic-type berm breakwater for a design wave height of about 7 m. Both structures were constructed from armourstone quarries yielding about 30% of the total rock resource above 3 tonnes with a maximum stone size of about 30 tonnes. Both were constructed about a decade ago and both have experienced waves close to or even exceeding their respective design wave conditions. The intention of the section is to show that it is possible to design for higher wave load by better utilising the larger rock fractions available from the quarry.

The Mackay Small Craft Harbour breakwater, Queensland, Australia

From the 1930's the Mackay harbour in Queensland, Australia, was protected by the Southern and Northern breakwaters. In 1998 a mass armoured berm breakwater was constructed to protect the Mackay Small Craft Harbour basin, Figure 1. It is a reshaping berm armoured breakwater and has been maintained over time, following cyclones, by "topping-up" the breakwater profile with additional armour, Colleter et al (2011).

Figure 1. Mackay harbour, the Small Craft Harbour breakwater to the right on the photo protecting the marina, from NQBP (2011).

The breakwater is built with rock from the Mt Basset Quarry that has been used for construction and maintenance of breakwaters in Mackay since the 1930's. In the design phase of the Small Craft Harbour breakwater, a surface and subsurface quarry investigation was undertaken to determine the potential quarry yield in terms of blast block size distribution and to assess rock quality, Johnson et al. (1999). The design of the breakwater required an optimum yield of about 50% armour and 50% core and the adopted block size distribution separated armour and core at 30 kg, with armour heavier than 30 kg and core weighing less than 30 kg. Figure 2 shows the grading curves for the estimated and actual quarry yield

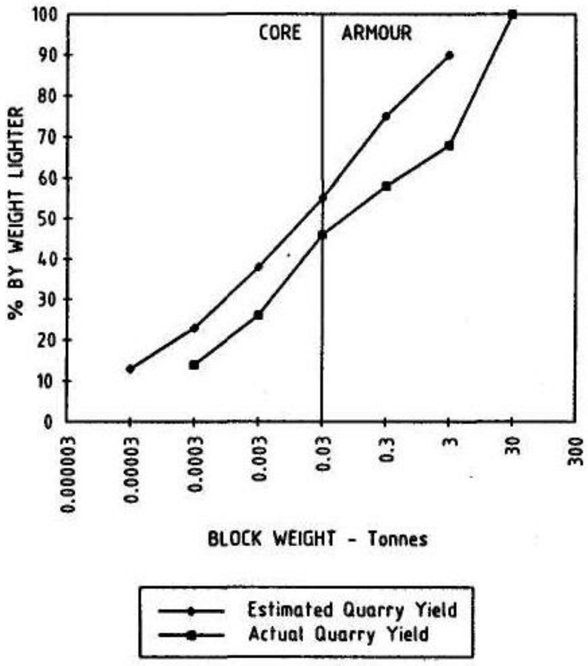

Figure 2. Grading curves for estimated and actual quarry yield for the Mackay Small Craft Harbour breakwater, from Johnson et al. (1999).

As shown in Figure 2, the actual yield turned out to be considerably better than the predicted yield. Prior to construction, it was estimated that less than 10% of the quarry yield would exceed 3 tonnes, and this was also the largest rock size shown on the quarry yield curve. During the quarry development, in fact, the actual quarry yield above 3 tonnes was over 30% with armourstone weighing up to 30 tonnes. In addition, the median weight of the armour is close to 3 tonne, as estimated from the grading curve.

The Mackay breakwater was designed for various combinations of wave height, wave period and water level, with the most severe conditions being a 50-year return period wave event of H_s=4.1 m, T_z= 9.0 s and still water level of +4.8 m LAT.

The adopted design cross section is shown in Figure 3, Johnson et al. (1999). The crest level was set at +9.6 m LAT to match with the existing breakwater. The berm level was set at +6.5 m LAT just above the highest astronomical tide (HAT). The idea was to build the berm at a level that makes the initial profile approximate the final S-shaped equilibrium profile as closely as possible. Based on 2D and 3D model tests the adopted berm width was 14 m. While the tested cross sections rapidly reshaped to form a long, flat, S-shaped profile, it was noted that larger rocks displaced to the toe of the profile whilst the smaller rocks remained on the flat part of the slope. The smaller rocks moved up the slope with each wave run-up and then down with each wave recession gradually forming a dynamic equilibrium.

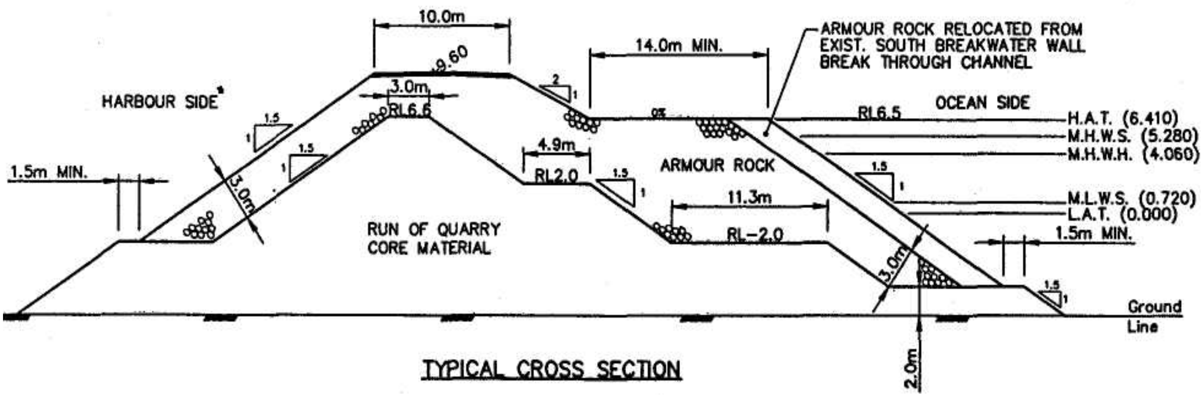

Figure 3. Design cross section of the Mackay Small Craft Harbour breakwater, from Johnson et al. (1999).

The breakwater was constructed with land based construction equipment and most of the material was placed by means of end tipping. This caused problems during construction with segregation as larger rocks were falling to the lower batter levels (below LAT) and smaller/finer material was being deposited on the upper slopes in the tidal range and active wave zone. According to Johnson et al. (1999) this was rectified at least to some extent by a dragline, when a 6 m^3 bucket was used to pull larger rock from the lower batter up the slope and deposit the larger rock at the higher levels.

The breakwaters protecting the Mackay harbour, Northern, Southern and the Small Craft Harbour breakwaters, are mostly reshaping breakwaters and have needed a year to year maintenance. Over the period 1997-2008 the total maintenance effort is estimated to be 65,000 t of additional rock added to the profile corresponding to, on average, about 28tonne per meter length of breakwater.

During the Tropical Cyclone Ului in March 2010, the breakwater was significantly damaged and substantial wave overtopping occurred, Colleter et al. (2011). At the peak of the storm, the significant wave height along the breakwater varied between H_s=4.0 and 4.3 m. The reshaping extended well into the crest section with the bitumen road on top of the breakwater damaged in areas. Rock up to 0.5 and 1.0 tonne was left on the crest and the inner harbour revetment was damaged by overtopping flow. After the storm the median rock mass on the emerged reshaped slope was estimated as 1.5 tonne with significant variations along its length. This corresponds to a stability number at the peak of the storm of $Hs/\Delta D_{n50}$ = 3.0 to 3.2.

The authority responsible for the maintenance of the breakwater, the North Queensland Bulk Ports Corporation (NQBP, 2011) has recently commissioned an independent review of the breakwater following the significant damage experienced during this storm event.

Figure 4. The Mackay Small Craft Harbour breakwater after the Tropical Cyclone Ului, from Colleter et al. (2011).

The Sirevåg berm breakwater, Norway

The Sirevåg breakwater is located in a narrow bay on the west coast of southern Norway and was constructed in the period from January 2000 to July 2001, Figure 5, Sigurdarson et al. (2001) and (2003). The breakwater was designed and constructed as a partly reshaping Icelandic-type berm breakwater for a wave height with a 100-year return period. It should also withstand a wave height with 1000-year return period without total damage.

The Sirevåg breakwater is exposed to heavy waves from the North Sea. The design 100-year recurrence wave height at the location of the outer part of the breakwater was established as H_s = 7.0 m with T_p= 14.2 s.

The breakwater is partly located on rocky bottom and partly on fine quartz sand. The depth of the rocky bottom is very variable from 3 m to 22 m with steep slopes. Under the outermost 150 m a flat sand bottom is present. The breakwater is about 500 m long with a total volume of approximately 640.000 m^3.

The quarry investigations included drilling of 25 cored drill holes and surface scan-lines. Three possible quarries (A, B and C) all from the same rock formation were assessed for the Sirevåg breakwater. A quarry yield prediction was carried out for the three quarries for a 640,000 m^3 breakwater. The armourstone material is anorthosite gabbro rock of good quality with specific gravity (SSD) of 2.7 t/m^3 and water absorption between 0.2 and 0.3. The point load index exceeds 10 MPa. The quarry yield prediction, Figure 6, for a carefully worked quarry was about 30% over 3 tonnes.

Figure 5. The Sirevåg berm breakwater.

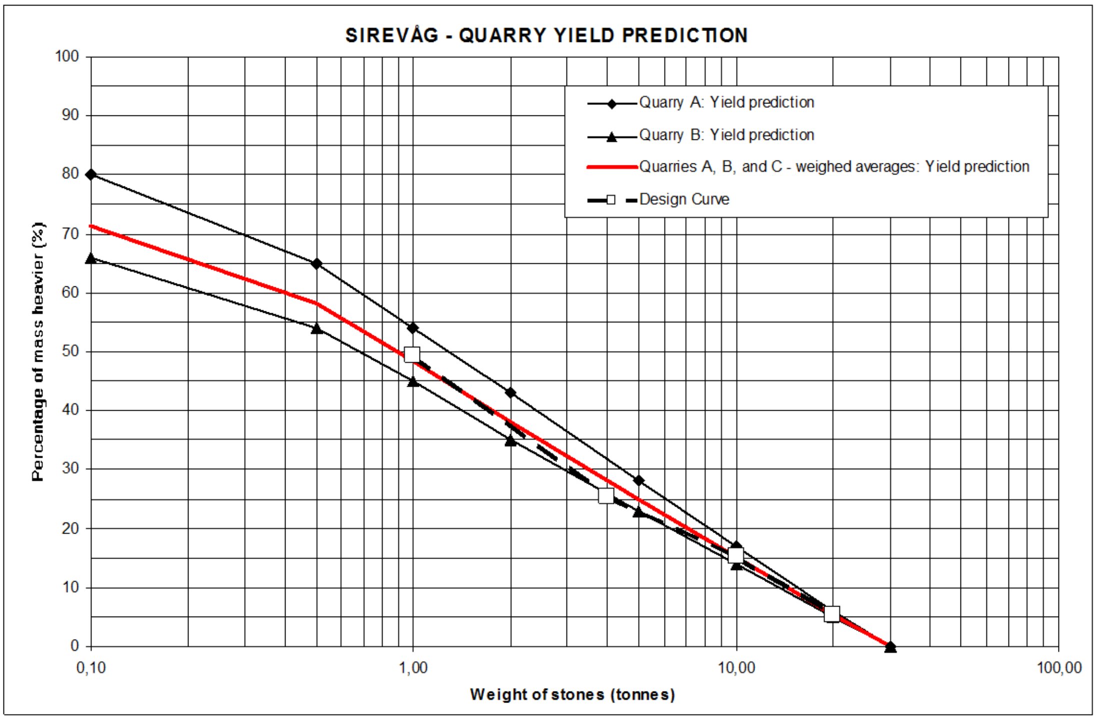

Figure 6. Quarry yield prediction and design curve for the Sirevåg breakwater.

In the preliminary design, three sets of stone classes were considered based on initial size distribution estimates from the potential quarries. In the final design, after thorough quarry investigations, one set was chosen based on the overall utilisation of all quarried material according to a quarry yield prediction and fulfilment of stability criteria for all sections of the breakwater, Table 1.

Table 1. The Sirevåg breakwater, the stone classes, the quarry yield prediction and the stability number for the most exposed section under design conditions.

Stone Classes	W_{min}-W_{max} (tonnes)	W_{50} (tonnes)	d_{max}/d_{min}	Expected quarry yield	$H_s/\Delta D_{n50}$
I	20.0–30.0	23.3	1.14	5.6%	2.0
II	10.0–20.0	13.3	1.26	9.9%	2.4
III	4.0 – 10.0	6.0	1.36	13.7%	3.2
IV	1.0 – 4.0	2.0	1.59	19.3%	4.6

The design wave height varied along the 500 m long breakwater and the design consisted in total of 10 different cross sections. A cross section of the most exposed part of the breakwater is shown in Figure 7. The design fully utilises all quarried stones over 1 tonne and a 100% utilisation of all quarried material was expected and achieved for the project. The Class I stones were specified to be individually and orderly placed with interlocking.

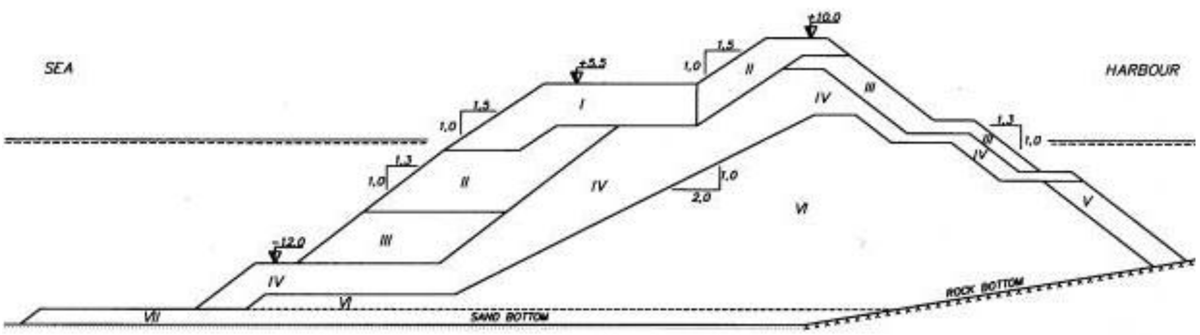

Figure 7. Sirevåg berm breakwater, cross section of the most exposed part of the breakwater.

On January 28, 2002, only six months after the construction finished, the Sirevåg breakwater was exposed to a severe storm. A Waverider buoy, located 450 m off the breakwater head at 20 m water depth, measured wave heights at half hour intervals. The maximum recorded significant wave height was H_s = 9.3 m and the wave height exceeded H_s = 8.0 m for a period of 3 hours. Tørum et al. (2005) estimated the wave height at the breakwater to be in the range of H_s = 7.1 – 8.7 m, which significantly exceeded the 100-year design wave height of H_s = 7.0 m.

Inspection of the breakwater shortly after the January 2002 storm showed that in three areas stones at the still water line were displaced, one at the roundhead and two on the breakwater trunk. At the time of this inspection, the reshaping had yet not progressed upwards to the top of the berm.

Only three years later on 11 to 12 January 2005 the breakwater was again exposed to a major storm called "Inga". At this time there were no wave measurements outside the Sirevåg breakwater. Tørum et al. (2005) estimated the wave height at the breakwater to be in the range H_s = 6.3-7.7m. These estimates were based on wave measurements about 60 km north of Sirevåg, calibrated wave hindcast closer to the breakwater and wave refraction up to the breakwater. Eyewitness accounts from experienced seamen in Sirevåg stated that the waves impacting the breakwater during "Inga" were heavier and more powerful compared to 2002 storm. During "Inga" damages occurred on old breakwaters, seawalls and lighthouses, up and down the coast to Sirevåg, the oldest structures from the early 20th century.

For analysis of these storms, a conservative approach is applied, by assuming the lower estimated values of the two storms. The 2002 storm will be taken as H_s = 7.1 m with T_p = 14 s and the 2005 storm as H_s = 7.0 m with T_p = 16 s. This means that the breakwater has twice been exposed to the 100 years design wave conditions. The two storms the breakwater was exposed the first 4 years in service indicate that the design wave conditions might be underestimated.

During a site visit to the Sirevåg breakwater in September 2008, the breakwater was inspected (see, Figure 8). The outer part of the breakwater and the roundhead suffered recession which was quite more extensive than after the 2002 storm. The recession has been estimated from an aerial photo, Sigurdarson et al. 2009. On the breakwater trunk the maximum recession is of the order 2.1 to 6.2 m. With a mean diameter of Class I armour stones of 2.05 m. this corresponds to a recession of about 1 to 3*D_{n50}. On the breakwater head the maximum recession is about 8.4 m which corresponds to about 4*D_{n50}.

Figure 8. Inspection of the Sirevåg breakwater after being twice exposed to design wave conditions.

Comparison of the performance of the mass armoured and Icelandic-type berm breakwater

Table 2 presents a comparison between the mass armoured Mackay breakwater and the Icelandic-type berm breakwater at Sirevåg.

Table 2. Comparison between the Mackay Small Craft Harbour breakwater and the Sirevåg breakwater, wave load, construction material, experienced storms and maintenance need.

	Mackay Small Craft Harbour breakwater	**Sirevåg breakwater**
Design method	Mass armoured breakwater	Icelandic-type berm breakwater
Construction year	1999	2001
Design wave height	H_s= 4.1 m	H_s= 7.0 m
Quarry yield 3 to 30 tonne	30%	30%
Breakwater type	Mass armoured breakwater, fully reshaping	Icelandic-type berm breakwater, partly reshaping
Cross sectional volume	Similar	Similar
Maintenance before storm	Following cyclones “topping-up” with additional armour, average of all breakwaters 28 tonne per m length	None
Experienced storms close to design event	1 storm (2010): H_s= 4.0 to 4.3 m	2 storms (2002 and 2003): H_s= 7.0 and 7.1 m
Relative wave energy	**1**	**5**
Maintenance immediately after the storm	Initial maintenance of 60 tonne per m length	None

In reference to this table, the following observations are made:

- The breakwaters are exposed to different wave climate but for large armourstone from 3 to 30 tonnes, the construction quarry yield was exactly the same;
- In the design of the Mackay breakwater no attempt was made to sort the larger armourstone and place this in the most exposed areas of the cross section, in contrast to the Sirevåg breakwater where this was done;
- Both breakwaters have been exposed to design wave condition and the Sirevåg breakwater twice. However, in terms of stone size, the Mackay breakwater was only exposed to only one fifth of the wave energy that the Sirevåg breakwater has been exposed to; and
- Significant maintenance of the Mackay breakwater has been required prior to the most recent storm (March 2010) and further considerable repair is now required, whereas the Sirevåg breakwater has not required any maintenance at all since its construction.

It is considered that the main reason for these significant differences is that the mass armoured design does not make the best use the available large armourstone, instead these have a tendency to end up on the sea bed, either when constructed or as a result of the reshaping process. The reshaping process also appears to lead to a reduction in the size of the armourstone through breaking and splitting of stones, which then fills the voids in the mass armour structure, reducing the breakwaters ability to absorb the incoming wave energy and leading to higher overtopping volumes.

The Icelandic-type design, on the other hand, ensures that the largest armourstone is placed where it can have the maximum benefit at dealing with the wave energy and the development of the Icelandic-type filtering, ensuring maximum performance from the structure and a significant reduction in overtopping in comparison with the mass armoured design.

ROCK ARMOUR SPECIFICATIONS

Projects involving the Icelandic-type berm breakwater differ in two main ways from many rubble mound breakwater projects, Sigurdarson et al. (2011). Firstly, they are usually based on volume of different armourstone classes rather than mass which has consequences on the quantification. Secondly, the definition of the armourstone classes is different than advocated by the Rock Manual and EN 13383.

The Icelandic-type berm breakwater has developed through projects with a dedicated armourstone quarry usually operated by the contractor constructing the breakwater. The projects are typically based on volume rather than mass. This eliminates the uncertainty of placing density from the price paid for the project. As a consequence these projects require the use of survey methods to define the rock surface. This is in contrast to the Rock Manual where the main focus is on defining layer thicknesses and bulk mass densities as the projects are usually based on mass.

A common definition of the theoretical surface in North Atlantic: “The rock surface shall be defined as the plane through which armour stones protrude by one third of the surface area”.

In recent Icelandic-type berm breakwater projects some, with armourstone classes in the order of 15 to 35 tonnes, the profile measurement has been performed with a GPS staff measuring on the top of the armour stones.

The Icelandic-type berm breakwater is normally designed with continuous armourstone classes, with the aim of utilising all size grades from predicted quarry yields. Armourstone classes are generally defined with stricter size grading than those presented in the Rock Manual and EN 13383. The key to the use of the Icelandic berm breakwater design is in its ability to match the predicted quarry yields of the potential quarries. Full utilisation of all size grades from 0.5 or 1.0 tonne up to 25, 30 or 35 tonnes, has been achieved in many projects, both small and large. This has been possible through reliable quarry yield prediction.

CONCLUSIONS

This paper has been prepared to explain the main difference between the mass armoured and Icelandic-type berm breakwaters, with the inclusion of two case studies where similar armour yields were achieved from their respective quarries but where the design and use of armourstone has lead to completely different outcomes for the respective structures.

In the case of the Mackay breakwater, the lack of sufficient experience in accurately predicting the likely quarry yield, led to the development of a breakwater which was designed to reshape dynamically during the life of the structure. During construction of the breakwater, it appears that poor control over the placement of the armour materials led to the “loss” of larger armour stones towards the toe of the structure, where they are of little benefit. It appears that some measures were undertaken during construction to address this but these are likely to have been largely ineffectual. This resulted in armourstone in the active portion of the breakwater being significantly underweight, and has subsequently led to considerable damage of the breakwater with an ongoing program of maintenance and, more recently, significant damage to the structure requiring major reconstruction.

For the Sirevåg breakwater, on the other hand, there was a more accurate estimate of the quarry yields which enabled more effective use of the large armourstone in the design profile. As a result, despite twice experiencing storms well above its design wave height, the breakwater has experienced minimal reshaping and has required no maintenance since its initial construction.

This comparison is particularly important when one considers that, despite using rock from quarries which achieved almost exactly the same quarry yield (30% above 3 tonnes), the design wave height for the Sirevåg breakwater is H_s = 7.0 m, whereas for the Mackay breakwater, only H_s = 4.1 m. As wave energy is proportional to the cubed root of the wave height, this represents a five-fold required increase in wave energy for the Sirevåg design, using the same quarry materials.

If the Mackay breakwater had been constructed to the same design and specification as provided for the Sirevåg breakwater and using the quarry yield achieved in practice, it is considered that this breakwater would have been able to achieve and significantly exceed the required design wave height with no maintenance, based on the outcomes achieved at Sirevåg.

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