

ICELANDIC-TYPE BERM BREAKWATER FOR THE HAMBANTOTA ARTIFICIAL ISLAND REVETMENT, APPLICATION OF GEOMETRICAL DESIGN RULES

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The paper discusses the development of a berm breakwater revetment design for protection of an artificial island on the southern shore of Sri Lanka. The island will primarily be constructed with excavated materials from the Phase 2 Hambantota Port construction activities, which include considerable volumes of slightly weathered rock. A value Engineering Study considered three different design options for the protective revetment: two rock options, conventional rubble mound and berm breakwaters, and an option using Chinapode concrete armour units. There were considerable cost-savings in choosing an Icelandic-type berm breakwater. The geometrical design rules of Icelandic-type berm breakwaters, developed by Van der Meer and Sigurdarson (2014), were used to develop different cross-sections based on different armourstone classes. Based on a quarry yield prediction and hydraulic model studies, the design was optimized using the available rock as efficiently as possible. Refinement of the design during the hydraulic model testing resulted in increased stability of the berm breakwater. Due to lack of experience in the production of large armourstone, the yield of the 5-10 tonnes class was rather low during the first months of construction. A number of measures were recommended resulting in more than tripling the yield.

Keywords: berm breakwater; Icelandic-type; armourstone; quarry yield; design rules

INTRODUCTION

The Hambantota Port Development project is planned to develop a major industrial and service port on the southeast coast of Sri Lanka. It is situated along the shipping route between the Strait of Malacca and the Suez Canal. The construction of Phase 1, which included two breakwaters, navigational channel, turning basin and several quays, started in January 2008 and finished in November 2010. Phase 2 includes excavation of a larger basin, deep water quays and an artificial island.

The paper discusses the development of a berm breakwater revetment design for protection of the artificial island, see Figure 1. The island will primarily be constructed with excavated materials from the Phase 2 Hambantota Port construction activities; the availability of this material is also the main reason to construct the Island. Protection of the island with interlocking concrete units was included in the initial design phase. As large volumes of rock would become available from the excavation, efforts have been made in the Value Engineering Study to come up with an optimized design using this rock as efficiently as possible. The maximum rock size for design is limited by the available equipment for rock handling and was set to 10 tonnes. After preliminary designs indicated that a wide berm revetment would be feasible, a more comprehensive study was performed for further design.

CHEC Sri Lanka (China Harbour Engineering Company) is the EPC Contractor of the Project, which is constructed by FHEC (Fourth Harbour Engineering Company). The client is Sri Lanka Ports Authority (SLPA). CDR International B.V. was engaged for specialist consultancy services including the Value Engineering Study, in which the berm breakwater concept was proposed, and a comprehensive design study including Metocean design conditions. Van der Meer Consulting and IceBreak Consulting were engaged for berm breakwater design and armourstone quarry assessment.

The geometrical design rules of Icelandic-type berm breakwaters, developed by Van der Meer and Sigurdarson (2014), were used to develop different cross-sections based on different armourstone classes. These are a part of a cooperative work on berm breakwaters with several papers presented in the past, Sigurdarson and Van der Meer (2013).

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Figure 1. The Hambantota Port Development Project on south coast of Sri Lanka, artistic view of the final phase. The artificial island is in the lower left side of the figure, left of the harbour entrance.

AVAILABILITY OF MATERIAL AND QUARRY YIELD ASSESSMENT

The harbour basins of the Hambantota port will be excavated into the land in dry conditions with land based equipment. Therefore large quantities of soil and rock become available and will be used for the construction of an artificial island west of the current western breakwater, in total about 20 million m³ of different characteristics with the following estimated quantities:

- 10 million m³ of sandy soil, including some heavily weathered rock and medium weathered rock;
- 5 million m³ of medium weathered rock and heavily weathered rock;
- 5 million m³ of slightly weathered rock.

It is assumed that only the slightly weathered rock may be used as the primary armour protection, about 5 million m³ in total.

The design phase included an assessment of the possible yield of large armourstone from the slightly weathered rock in the harbour basin excavation area. The assessment was based on rather limited information of the actual rock masses to be quarried. There existed drilled cores from the area but only five of those were from the actual rock masses to be quarried. Therefore it was necessary to rely on more indirect information that included armourstone quarried during Phase 1 of the project, visit to an open quarry and rock outcrop, both close by on opposite sides of the quarry.

Apparently the analysis of the 5 drilled cores of the existing geological investigations from the harbour excavation area was not sufficient to give a representative sample of the 5 million m³ of slightly weathered rock. But if these boreholes were taken as representative of the whole slightly weathered rock mass, analysis revealed that only one fourth of the total volume was likely to yield large armourstone.

A site visit revealed that a stockpile of armourstone laid aside during the excavation of Phase 1 of the Hambantota Port project mainly consisted of 1 to 5 tonnes stones, with very few stones between 5 and 10 tonnes. As the purpose of the excavation during Phase 1 was not to produce larger armourstone than 5 tonnes and as it was difficult for the available equipment to load, transport and handle stones larger than 5 tonnes, it was not likely to find many larger stones on the stockpile. Therefore the non-existence of stones larger than 5 tonnes should not be taken as an indication that these could not be produced from the rock masses.

Also a lee side revetment constructed during Phase 1 consisted of armourstone which sides were mostly fresh cut from the blasting operation, although some were from natural joints and fractures. The fresh cut sides indicated that with different blasting design it would have been possible to produce larger armourstone than were needed for Phase 1.

Based on the information above and examination of rock faces in a quarry close to the harbour basin, the 19# Mountain Quarry, about 2.5 km northeast from the harbour basin excavation site, as well as on the fracture pattern of a rock outcrop about 5.5 km west of the basin, it was deemed highly likely that it would be possible to extract armourstone into a stone class of 5 to 10 tonnes and even very likely

in less quantities into a class of 10 to 20 tonnes. This was assuming that the extraction methods would be designed to suit armourstone production.

Figure 2 presents two quarry yield predictions A and B, upper and lower predictions, for the slightly weathered rock mass of the harbour excavation area. The figure also includes achieved yield of armourstone from a harbour project on Sri Lanka's east coast. According to predictions A and B the quarry may possibly yield 10% to 19% over 5 tonnes and 5% to 10% over 10 tonnes. If stones larger than 10 tonnes are not utilised for the project then the 5-10 tonnes class will yield 5% to 9%. But the yield may be increased by careful secondary splitting of oversized stones that cannot be handled with available equipment.

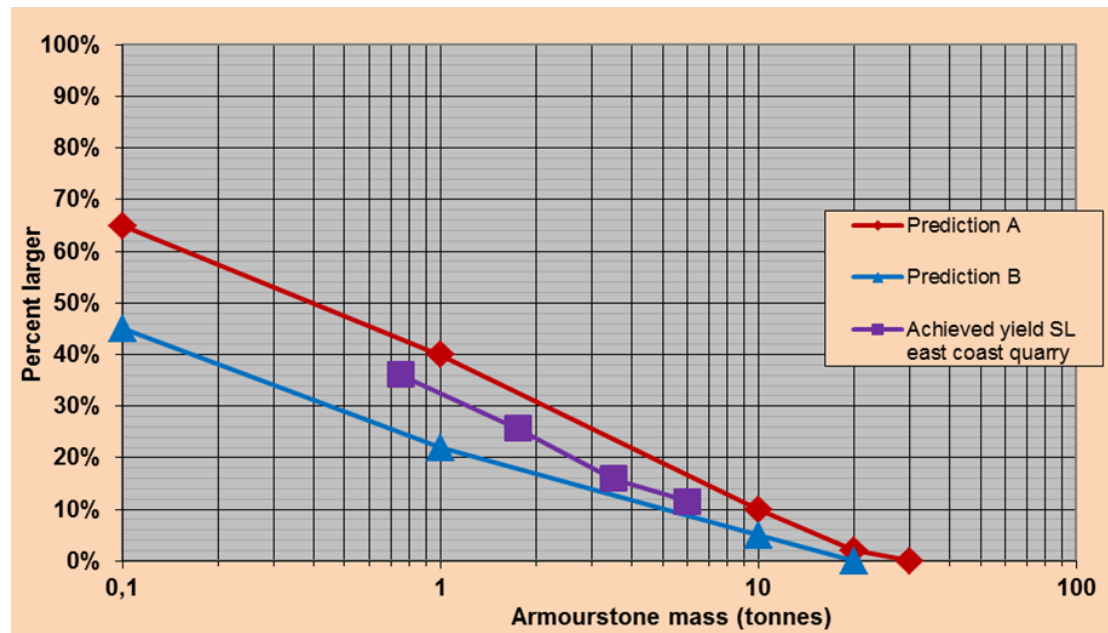


Figure 2. Quarry yield prediction for the slightly weathered rock in the harbour basin excavation area. Predictions A and B, upper and lower yield predictions respectively. Also shown is achieved yield from a harbour project on Sri Lanka's east coast.

At the time of the Berm Breakwater design phase the contractor had already planned and purchased the equipment he intended to use for the project, as previously the intention was to construct the Breakwater Revetment using interlocking concrete units. This equipment, excavators and trucks, were geared to handle armourstone weighing up to about 10 to 12 tonnes. Although it was deemed possible to increase the maximum armourstone size up to 15 tonnes with alterations of the already purchased excavators it was decided to design the breakwater with a maximum stone size of 10 tonnes.

DESIGN WAVE CONDITIONS

In a Metocean study, performed by CDR, operational and extreme wave conditions have been derived for swell conditions, (local) monsoon wave conditions and Cyclones. Nearshore design wave conditions have been established for a combination of swells and monsoon waves and for Cyclone waves separately. Due to the location of Hambantota Port, swell waves are governing in the mixed-sea state and therefore the swell wave period is used; corresponding wave heights are based on the combined energy of swells and monsoon waves.

Cyclone conditions are very rare in South Sri-Lanka, they have however occurred and it is possible that they will occur more often in the future. As relevant data is very scarce, cyclone data for a large area have been included in the Cyclone Wave Analysis, with the assumption that these cyclones may hit Hambantota directly. As this assumption is very conservative, cyclone data from a large area is used and the probability of a Cyclone in South Sri-Lanka is low, the 25-yr Cyclone wave as derived in the analysis has been used as a 100-yr design Cyclone wave. The derived 100-yr Cyclone wave, which in fact is the worst-case-scenario for this area, has been used as overload condition. With this practical approach the Cyclones have been taken in consideration, without over-designing the structure.

Results from the nearshore design wave study have been used to establish extreme wave conditions along the perimeter of the artificial island, see Table 1. For all wave heights a range of peak wave periods are given.

| | H_s | T_p (50%) | T_p (5%) | T_p (95%) |
|--|-------|-------------|------------|-------------|
| 1/yr Swell | 4.7m | 17s | 15s | 21.5s |
| 1/100yr Swell | 5.6m | 18s | 15.7s | 22.2s |
| 1/25yr Cyclone Wave (1/100yr design) | 6.9m | 12.5s | 11.5s | 15s |
| 1/100yr Cyclone Wave (Overload design) | 7.4m | 12.5s | 11.5s | 15s |

DESIGN OPTIONS

The initial intension of CHEC was to use Chinapode interlocking concrete armour units as armour layer for the Artificial Island Breakwater Revetment. The cost of these concrete units is high and as large volumes of rock would become available from the excavation, several design alternatives were proposed in the Value Engineering Study.

After a review of the local wave conditions and assessment of the nearshore design wave conditions, three different design options were considered:

- Conventional rubble mound breakwater – rock armoured;
- Berm breakwater – rock armoured;
- Conventional breakwater – armoured with 18 tonnes Chinapode concrete armour units.

For the wave conditions at the site a conventional rubble mound breakwater with a practical seaside slope of 1:2 – 1:3 is not viable. With the maximum size of armourstone of 10 to 12 tonnes that can be handled in this project, a conventional rubble mound breakwater is only feasible with a slope of 1:5. This slope is very gentle and difficult, time consuming and costly to construct using land based equipment.

A preliminary design of a berm breakwater consisted of a rather wide berm mostly of 1-5 tonnes rock with larger rock, 5-10 tonnes, on top of the berm and on the front slope. Compared to the option above, a conventional rubble mound breakwater, the volume of the berm was less or about 84%. But more importantly the volume of the largest stone class was only about 58%. In total the berm breakwater option would be less voluminous than the conventional rubble mound option, it would need less volumes of the largest stone class and less costly construction methods could be used.

The required weight of concrete armour units on a slope of 3:4 was determined based on the cyclonic wave height as 18 tonnes Chinapode. This unit would be able to withstand overload of 20% in wave height without significant damage. Although being about 20% less voluminous compared to the berm breakwater, this option came out as being considerably more expensive due to the cost of the concrete elements.

The reason why the contractor with his already purchased equipment would have been able to handle 18 tonnes Chinapode, but not armourstone heavier than 10 to 12 tonnes, was that the Chinapode armour units would be lifted up and placed by a crane while the rock would be handled with an excavator. A 280 tonnes crane was planned for handling the Chinapode while the rock would mostly be placed with 40 to 50 tonnes excavators. This adds to the economy of the berm breakwater as both as the placing rate of excavators is larger as well as the hourly cost is much lower, Sigurdarson et al. (2008).

As fairly large rock material is available for free from the excavation of the harbour basin and the cost-savings were considerable, it was decided to design and construct an Icelandic-type berm breakwater. This was not an easy decision as the design and construction of a berm breakwater was new to the contractor as well as for the region.

GEOMETRICAL DESIGN OF THE ICELANDIC-TYPE BERM BREAKWATER

The geometrical design rules for berm breakwaters presented in Van der Meer and Sigurdarson (2014) are based on cooperation between the two authors over a long period, both in the scientific as well as in the practical field with a number of papers presented.

Classification of berm breakwaters distinguishes between mass armoured berm breakwaters, MA, and Icelandic-type berm breakwaters, IC, Sigurdarson and Van der Meer (2012). The mass armoured

has a homogeneous berm of mainly one rather wide graded stone class while the Icelandic-type is constructed from several stone classes, usually more narrowly graded.

The classification takes into account the structural behaviour of the berm breakwater, the degree of reshaping. Mass armoured types are mainly partly and fully reshaping (PR and FR), where the Icelandic types are mainly hardly or partly reshaping (HR or PR).

As part of the Value Engineering Study three different options for Class I rock were considered for an Icelandic-type berm breakwater, namely option A with 5-10 tonnes, option B with 8-15 tonnes and option C with 10-20 tonnes. The evaluation of using rock larger than 10 tonnes was done ahead of the knowledge that the contractor would not be able to handle armourstone larger than 10 to 12 tonnes.

With a stability number, $H_s/\Delta D_{n50}$, larger than 2.5 for option A, it is classified as fully reshaping structure, FR, while options B and C fall into the partly reshaping regime, PR. All geometrical parameters for the cross section were then determined for each option, by the methods presented in Van der Meer and Sigurdarson (2014), see Table 1. Both swell waves and cyclonic waves were considered for this study while only the results for swell waves are reported in this paper.

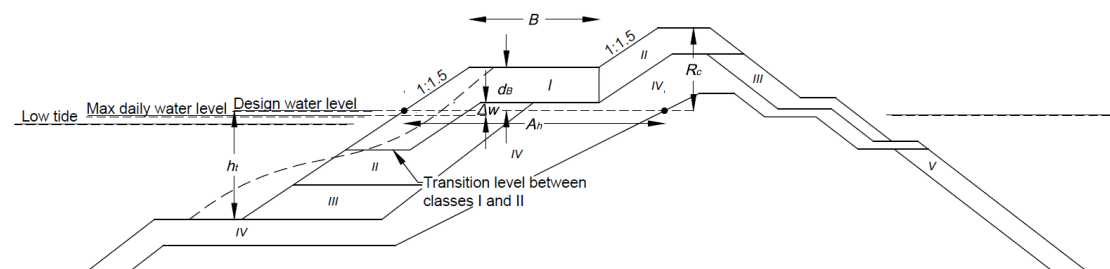


Figure 3. Principal cross section of an Icelandic-type berm breakwater.

The first parameter to determine is the berm width B , see Figure 3. The recession is calculated for both the design wave height as well as for the overload. The wanted resiliency of the berm, that is the remaining part of the original berm width after reshaping, is chosen differently on the basis of the stability number. A higher stability number, meaning lower stability, goes with lower resiliency as the berm reshapes to a large degree. Then there is a geometrical criterion for the berm width, it shall not be less than the recession plus $1 \cdot D_{n50}$ of Class I rock with a minimum of $3 \cdot D_{n50}$.

There are two criteria to look at when determining the next parameter, the berm level d_B . Firstly the berm level shall not be lower than $0.6 \cdot H_{SD}$, in order to achieve higher stability according to the scoring system presented by Van der Meer and Sigurdarson (2014). Secondly constructional issues play a role. A construction level is chosen at a certain distance, Δw , above the mean high water spring tide or maximum daily water level. This is the working level for the excavator. Two layers of Class I armourstone are placed on top of this level. The choice of Δw depends on the wave climate during the construction time. To be able to work into the monsoon periods, Δw is chosen rather high in this case or 2.5 m.

The required horizontal armour width, A_h , is defined as the horizontal distance at design water level from the seaward slope of the armour to the transition between rock armour and the core. The armour width is important for dissipation of wave energy and depends on design wave height and permeability, $A_h/H_{SD} = 2 \cdot H_s/\Delta D_{n50}$.

The next parameter is the transition level between classes I and II on the lower front slope. Van der Meer and Sigurdarson (2014) propose to use $0.4 \cdot H_s$ as a limit, where water levels with different wave heights should be considered.

The crest level is determined based on overtopping criteria both for the design wave condition as well as for the overload condition where the overtopping is calculated with the formula of Van der Meer and Sigurdarson (2014). The minimum crest width is taken as the significant design wave height.

As the structure will be built on sand a filter layer between the sand and rock layers is necessary to prevent the rock sinking into the sand. This layer is taken as a 1.5 m thick layer of quarry run and will extend the armour as apron with a width of about $1 \cdot H_s$.

| Table 1. Geometrical design considering swell waves for three different options for stone classes. | | | | |
|--|------------------|----------|----------|----------|
| Design conditions | | Option A | Option B | Option C |
| Design wave height | H_{SD} (m) | 5.6 | 5.6 | 5.6 |
| Peak period | T_p (s) | 18.0 | 18.0 | 18.0 |
| Overload wave height | H_{SOL} (m) | 6.2 | 6.2 | 6.2 |
| Design water level, DWL | (m CD) | 1.0 | 1.0 | 1.0 |
| Allowable overtopping q for H_{SD} | ($l/s/m$) | 10 | 10 | 10 |
| Allowable overtopping q for H_{SOL} | ($l/s/m$) | 30 | 30 | 30 |
| Selected stone classes | | | | |
| Class I | (t) | 5-10 | 8-15 | 10-20 |
| Stability number, $H_s/\Delta D_{n50}$ | | 2.6 | 2.2 | 2.1 |
| Type of berm breaker | | FR | PR | PR |
| Length parameters | | | | |
| Recession for H_{SD} | Rec (m) | 5.2 | 2.8 | 1.9 |
| Recession for H_{SOL} | Rec (m) | 6.4 | 3.5 | 2.4 |
| Wanted resiliency | | 40% | 30% | 20% |
| Min berm width from resiliency | B_{resil} (m) | 12.9 | 9.2 | 9.3 |
| Min berm width from geometry | B_{geom} (m) | 6.5 | 4.7 | 5.1 |
| Min berm level from waves | d_{Bw} (m CD) | 4.4 | 4.4 | 4.4 |
| Min berm level from constructional issue | d_{Bc} (m CD) | 5.8 | 6.3 | 6.5 |
| Min horizontal armour width | A_h (m) | 29.1 | 25.1 | 23.1 |
| Min transition level between Classes I and II | h_{I-II} (m) | 2.2 | 2.2 | 2.2 |
| Crest level and crest width | | | | |
| Overtopping influence factor | Y_{BB} | 0.60 | 0.51 | 0.54 |
| Required crest level for H_{SD} | R_c+DWL (m CD) | 10.8 | 10.0 | 10.0 |
| Required crest level for H_{SOL} | R_c+DWL (m CD) | 10.4 | 9.5 | 9.5 |
| Required crest width | (m) | 5.6 | 5.6 | 5.6 |

Figure 4 shows the optimized preliminary cross section of the Icelandic-type berm breakwater based on Option A with 5-10 tonnes armourstone. The required berm width for the cyclonic design wave is higher than for the swell wave, therefore a berm width of 18m has been applied.

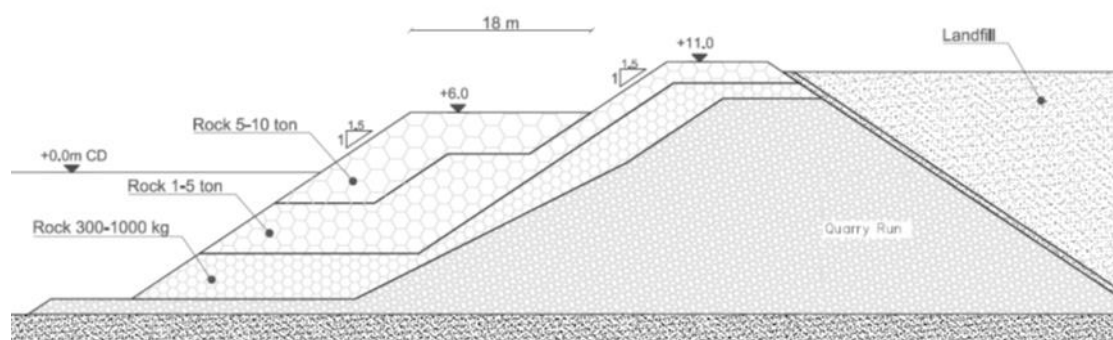


Figure 4. Preliminary design of the Icelandic-type Berm Breakwater after the desk study.

HYDRAULIC MODEL TESTING AND FINAL DESIGN

Model testing was carried out in Tianjin Port Engineering Institute (TPEI) in China. The wave flume was 98 m long and 4 m wide with a 1 m wide test section, tested in a scale 1:38, Figure 5. During the tests, the design of the Icelandic-type berm breakwater was validated and improved. This included the crest free board, berm width, the slope and the size grading of the armourstone classes.



Figure 5. The testing facility in Tianjin Port Engineering Institute, flume and tested cross section.

In total 8 different berm breakwater cross sections of the Icelandic-type were tested and one test of mass armoured type. The depth at the toe of the structure varied from -8 m CD to -14 m CD. The Icelandic-type sections were mostly tested with 5-10 tonnes as Class I while one was tested with 7-12 tonnes as Class I. The mass armoured berm breakwater was tested with 1-10 tonnes class. The berm width of the structure at -14 m CD water depth varied from the initial 18 m to down to 10 m.

The test programme included up to 10 different wave conditions with several 1y, 10y, 100y and overload conditions for both sea waves, swell waves and cyclonic waves. The duration of the sea and swell wave conditions was 6 hours prototype, while the cyclonic conditions lasted 4 hours.

Yang (2014) analysed the results of the model tests for Hambantota and compared them with selected results from hydraulic model tests of similar Icelandic-type berm breakwaters, a dataset described in Sigurdarson and Van der Meer (2012). The measurements of recession or profiles in the Hambantota tests were not performed to a high standard, which gave quite some scatter for interpretation of the results.

Test results showed a much higher stability than expected according to the recession formula presented by Sigurdarson and Van der Meer (2013), Figure 6. During the first test this was caused by fresh broken rock with sharp edges and recently painted with a sticky paint. This was corrected before test two, the rock was a little rounded and the paint was removed.

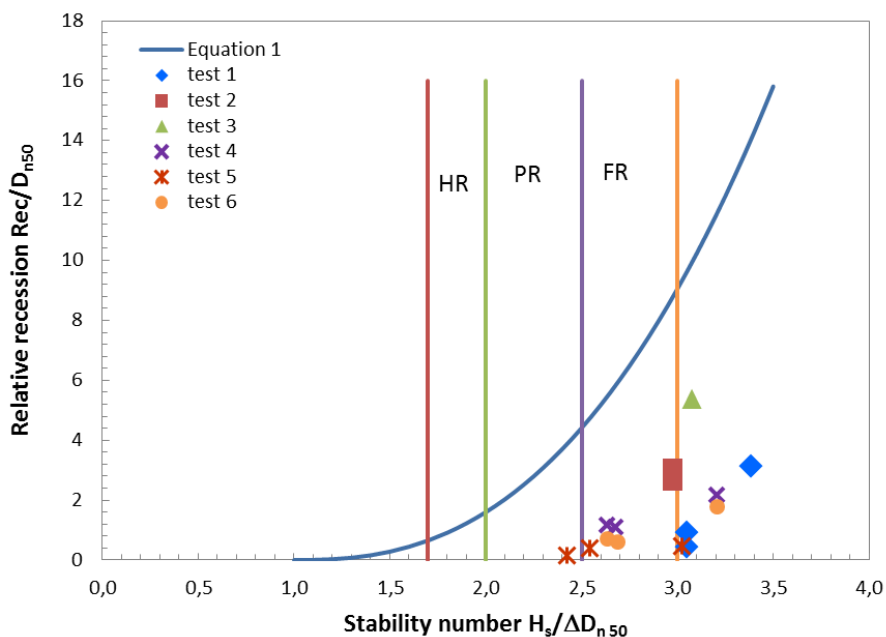


Figure 6. The result of the model testing, dimensionless recession Rec/D_{n50} , versus stability number $H_s/\Delta D_{n50}$, compared with the Sigurdarson and Van der Meer (2013) prediction formula and classification of berm breakwaters into the groups, Hardly Reshaping, Partly Reshaping and Fully Reshaping.

The analysis of the test results was done by shifting the recession formula along the horizontal axis of the figure to fit each test. This showed that the best fit through test 2 needed a shifting of $0.75 H_s/\Delta D_{n50}$ towards higher stability. In test 3 the berm width was decreased to 10 m from the original 18 m which receded the fit by $0.3 H_s/\Delta D_{n50}$, till $0.45 H_s/\Delta D_{n50}$ from the formula, Figure 7. Interpreted on the vertical axis of the figure, the recession in test 3 is $2*D_{n50}$ more than in test 2.

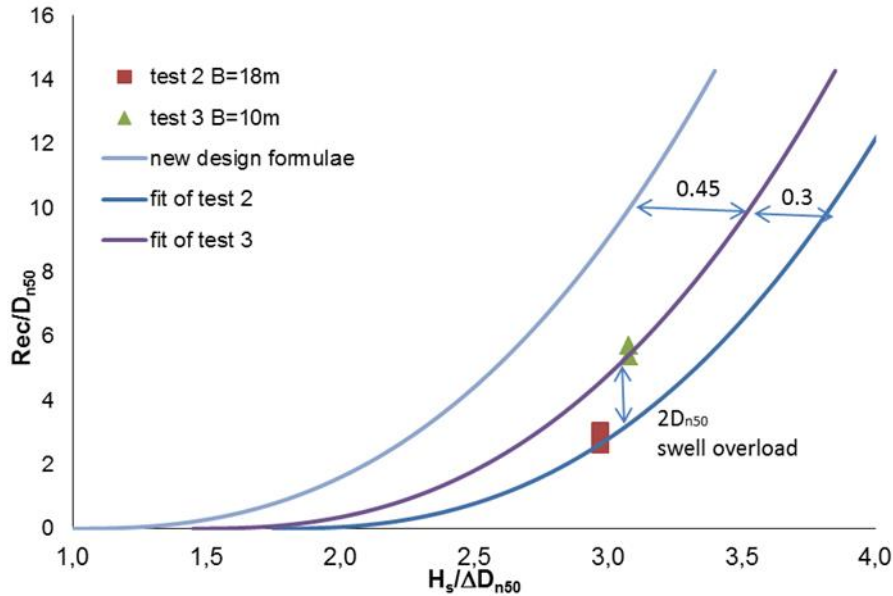


Figure 7. Influence of berm width of Tests 2 and 3, dimensionless recession Rec/D_{n50} , versus stability number $H_s/\Delta D_{n50}$, compared with the Sigurdarson and Van der Meer (2013) prediction formula.

Comparison of Class I as 7-12 tonnes instead of 5-10 tonnes in tests 4 and 5 showed that the rock diameter, which is already considered in both the x and y-axis on the graph, is a good parameter to describe the behaviour of the structure.

Comparison between mass armoured and the Icelandic-type berm breakwater, tests 10 and 2, showed that the recession followed a similar curve, test 10 shifted by $0.25 H_s/\Delta D_{n50}$ to lower stability. This again shows that the rock diameter is good in describing the recession, while the lower stability of the mass armoured structure could be due to less positive effect of the high and long berm and/or the wide grading.

To investigate how the down slope and toe depth influences the Hambantota design, Project 4, Sigurdarson and Van der Meer (2012), was chosen to compare with the Hambantota cross section. The comparison was carried both for Icelandic type and mass armoured type cross sections which are section 2 of Project 4 with test 2 of Hambantota case and section 3 of Project 4 with test 10 of Hambantota case respectively.

Both comparisons reveal that the gentler down slope of the Hambantota design has the effect of reducing the recession. The shallow toe berm depth also has a positive effect for test 2. For partly reshaping berm breakwaters, the gentle slope and shallow toe berm depth could cause 1-3 rock diameters less recession than expected. For fully reshaping berm breakwater, the toe doesn't have a significant influence on recession, but the gentle slope may give 3-5 rock diameters less than the design curve at most. Moreover, the wide berm is also important, influencing the recession of the Hambantota design.

The conclusion of this analysis is that a combination of a relatively shallow berm toe, a gentle down-slope and a wide and high berm results in less recession than the design formulae of Sigurdarson and Van der Meer (2013).

Sigurdarson and Van der Meer (2011) stated that the wave period doesn't have influence on the recession of berm breakwaters when the stability number $H_s/\Delta D_{n50} < 3.0$. To investigate if this is true, Sihan (2014) analysed data sets of Icelandic-type berm breakwaters including the Hambantota case by plotting the dimensionless recession Rec/D_{n50} against several parameters including the wave period. The result was that the scatter increased compare to the wave height stability parameter. Therefore the

result is that the wave period doesn't have an influence on recession in the Hambantota tests and the stability parameter $H_s/\Delta D_{n50}$ is the best parameter to describe the recession.

The refinement of the preliminary design done during the hydraulic model testing included:

- Decreased berm width from 18 m to 15 m, still with good resiliency with the expected recession not exceeding 40% of the berm width.
- Additional toe berm of Class III 0.3-1.0 tonne rock at -8 m CD level, with toe-depth ratio h_t/H_s of 1.53 accounting for double positive score according to the design rules.
- Decreased crest height from +11 m CD to +10 m CD as the measured overtopping rates were very low.

The additional berm at -8 m catches the stones falling down during the reshaping process, stones that otherwise would end on the lower berm and thereby accelerates formation of a more stable S-shape profile. Figure 8 shows the final design of the Icelandic-type berm breakwater for the Hambantota case.

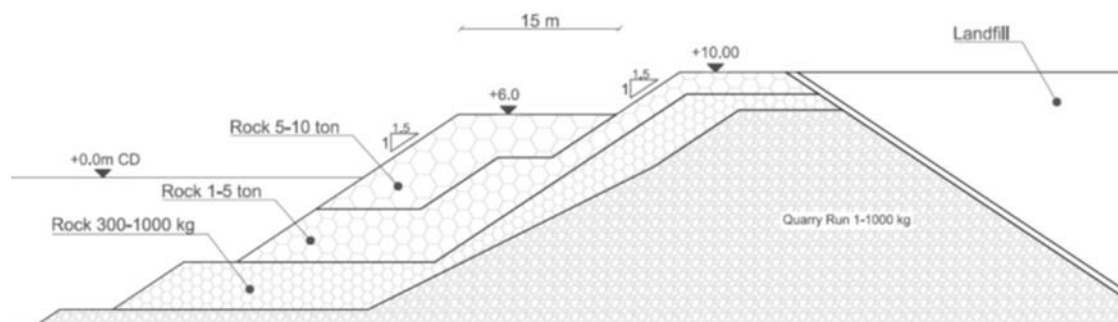


Figure 8. Final design of the Icelandic-type Berm Breakwater optimized during physical model testing.

IMPROVED YIELD OF 5 TO 10 TONNES ARMOURSTONE

Quarrying for large armourstone is a discipline that not many contractors master. Large rocks will not be available after blasting, 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.

In Hambantota the original plan was to get the majority of the 5-10 tonnes armourstone class from the excavation of the harbour basin as the quarry yield assessment showed that this would be possible, Figure 9. In the case the assumptions about the slightly weathered rock in the excavation area would not withstand, the rest of the 5-10 tonnes rock would come from the 19# Mountain quarry.



Figure 9. Extraction of the slightly weathered rock in the harbour basin area down to a final level of -17 m.

Due to lack of experience in the production of large armourstone, the yield of the 5-10 tonnes class from the excavation of the harbour basin area was rather low during the first months of construction, only between 2 and 3%. This called for a site visit of IceBreak Consulting Engineers to improve the rock extraction methods, Figure 10.

IceBreak recommended a number of measures for improving the yield of 5-10 tonnes rock, where the most important were:

- One row blasting instead of multi rows;
- Decreased specific charge;
- Changing the distribution of charge within the blasting hole;
- Optimise the utilisation of the bench;
- Emphasize on secondary breakage of oversized rock.

Three weeks after the site visit the yield of 5-10 tonnes rock from the harbour basin had more than tripled, from 2.8% to 9.7%. Later it was reported that the yield varied between 6%-15% depending on the rock characteristics.

If it is assumed that about half of the stones from the splitting of the oversized rock, heavier than 10 tonnes, have contributed to the 5-10 tonnes class, then the total yield above 5 tonnes is about 13% to 15%. This places the achieved yield from the harbour basin about midway between quarry yield predictions A and B in Figure 2. This means that it is possible to quarry all of the 5-10 tonnes stones from the harbour basin area and unnecessary quarrying in 19# Mountain can be avoided.



Figure 10. The FHEC team examining the blasting pile after test blasting in the harbour basin area.

CONSTRUCTION

The construction of the artificial island as well as the breakwater or revetment started in June 2013 and is planned to finish in November 2015. During the monsoon season from May to September the work on the breakwater revetment is halted.

The main construction equipment includes:

- 79 backhoe excavators – 20 to 85 tonnes
- 413 dump trucks
- 1 Crane – 280 tonnes
- 11 Bulldozers

During the first non-cyclone season the average weekly production and transport rate was about 140,000 m³ of rock and core material to the breakwater revetment and about 105,000 m³ of soil excavation to the backfilling, in total nearly 250,000 m³ per week. At the end of May 2014 the construction of the breakwater revetment had reached a length of 850 m of a total of about 1600 m. Of these about 500 m were fully protected with Class I rock while the rest only partly and needed a temporary protection during the monsoon period, Figures 11 to 14. In total about 82,000 m³ of Class I rock, 5-10 tonnes, had been placed on the breakwater revetment and about 11,000 m³ were stored in stockpile.

No deformation or reshaping is reported after the first monsoon period, neither of the fully nor of the partly constructed berm.



Figure 11. Partly constructed berm.



Figure 12. Fully constructed berm consisting of 5-10 tonnes rock.

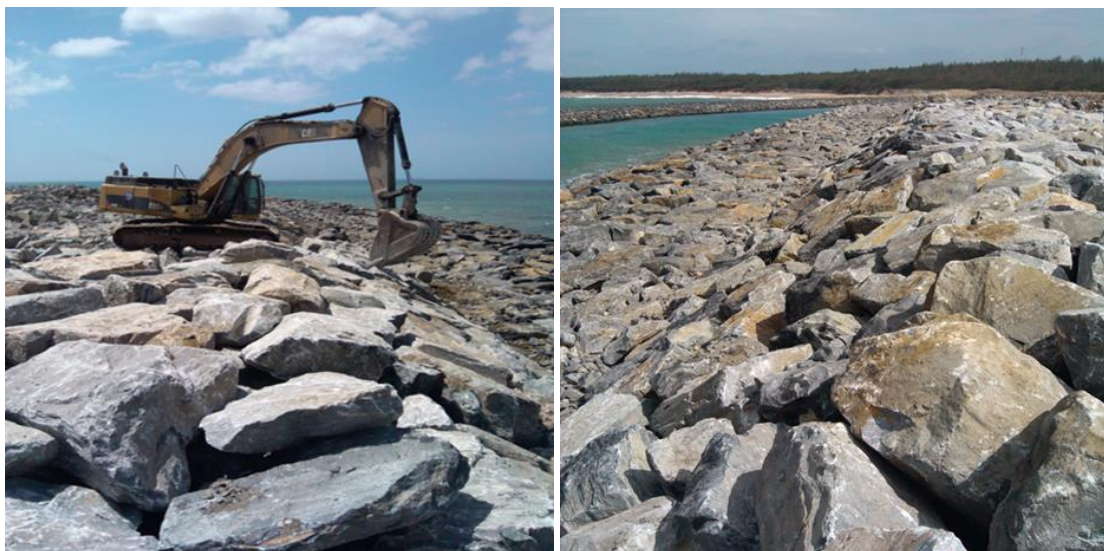


Figure 13. Placement of 1-5 tonnes rock on the breakwater crest at elevation +10 m.

CONCLUDING REMARKS

Based on a Value Engineering Study an Icelandic-type berm breakwater was chosen for protection of the Hambantota artificial island ahead of a conventional rubble mound structure and a revetment protected with Chinapod concrete armour units with considerable cost savings.

Of about 20 million m³ of soil and rock to be excavated from the harbour basin of the Hambantota port there are about 5 million m³ of slightly weathered rock. The design phase included an assessment of the possible yield of large armourstone from the slightly weathered rock. Based on rather limited information a quarry yield prediction was presented, predicting that it would be possible to quarry about 8 to 12% into an armourstone class of 5-10 tonnes.

The geometrical design rules of Icelandic-type berm breakwaters, developed by Van der Meer and Sigurdarson (2014), were used to develop different cross-sections based on different armourstone classes. Based on available equipment for rock handling and transport the maximum rock size for the design is limited to 10 tonnes. A design based on optimization of quarry yield can save significant costs in rock projects.

The preliminary design was optimized in hydraulic model studies with several refinements of the design adding to the stability. The results of the hydraulic model study showed higher stability than expected from the design formulae of Sigurdarson and Van der Meer (2013). Analysis showed that this was due to several design improvements. By applying the geometrical design rules it is possible to achieve higher stability and less reshaping.

Most contractors lack the experience in quarrying for large armourstone and during the first months of excavation the yield of the 5-10 tonnes class was considerably lower than predicted. During a site visit of the design team several measures were recommended changing the blasting technique and quarry procedures. This resulted in more than tripling the yield of 5-10 tonnes and reaching the predicted quarry yield. Correct quarrying for the required rock sizes is very important for the viability of the project.

At the time of writing more than half of the island and the protective revetment has been finished and has survived the first monsoon period.

The large porosity in this Icelandic-type berm design increased the stability significantly; however correct construction of the porous berm avoiding contamination of finer material is very important for the final stability of the structure.

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Figure 14. Fully constructed berm during the monsoon period.

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