

REPAIR TO A DOLOS ARMoured BREAKWATER

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1. Introduction

In recent international experience several breakwaters armoured with concrete units have failed shortly after or during construction of the structure. Although investigators have not necessarily agreed on the causes of these failures, it has been generally implied that there is a relationship to breakwater construction in deep water with large units and severe wave conditions (4,9). However, the damaged sections of the dolos armoured breakwater that is the subject of the paper, are in water depths of 6 to 8m with depth limited wave conditions.

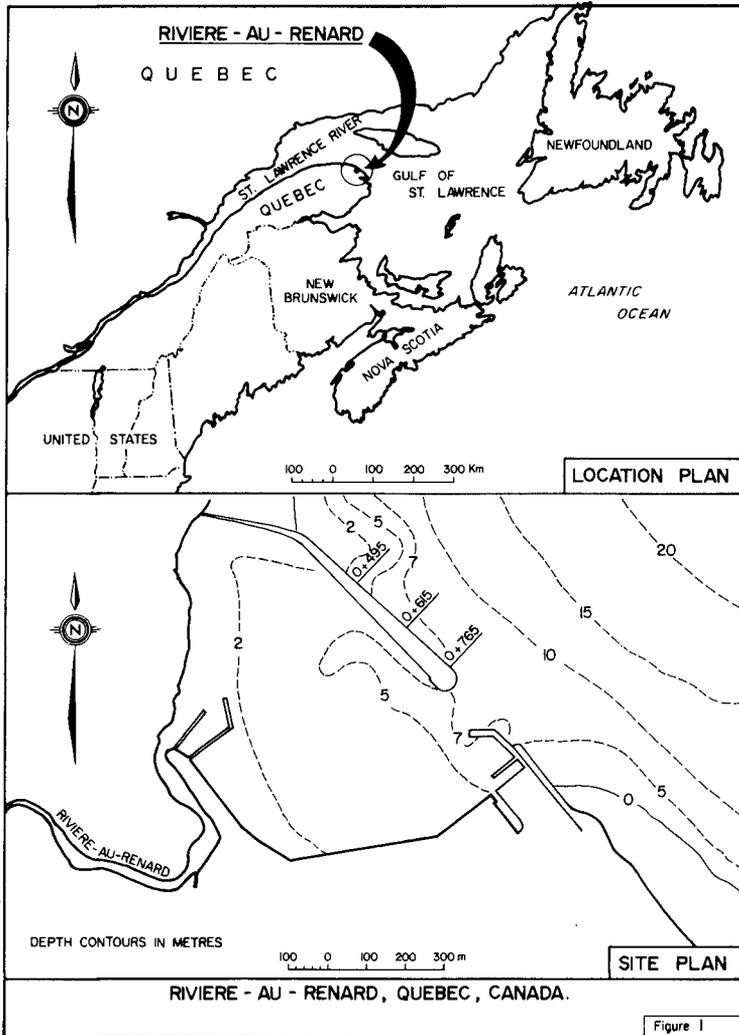
The original design parameters for selecting the dolos weight appear to have been reasonably accurate by current estimates. The recent hydraulic model tests indicate that the breaking wave conditions and overtopping of the breakwater have combined with the breakage of dolosse in the resulting failure.

The recommended repair work which was the primary objective of the study, consisted of armouring the crest of the damaged section with large stone (20t) and leaving the damaged front layer of dolos below low water level. It was judged that the further use of concrete dolosse was not economically justifiable and that there did not exist a satisfactory procedure to design dolos armour within the scope of the study.

2. Site and Damage Descriptions

2.1 Location and Breakwater

Rivière-au-Renard is located on the eastern end of the Gaspé Peninsula on the Gulf of St. Lawrence in Québec, Canada. The main breakwater is about 785m in length and encloses a natural bay to protect a fishing harbour facility. The breakwater construction was completed in 1972 (see Figure 1).



The outer portion of the structure of about 350m in length, was armoured with concrete dolos units of two sizes. Units at the head are 12.7t and extend back about 30m on both front and back slopes in water depths of 7 to 8m. Dolosse in the trunk are 4.5t and cover the crest and front slope only in depths of 4 to 7m. Typical cross-sections are shown in Figure 2. Unit placement was in double layer random in the densities estimated from the S.P.M. (1)

The design crest elevation of the breakwater varied from 5.3 to 8.0m over the concrete armoured section of the structure.

2.2 Construction

Some problems were encountered during construction which may have had a bearing on the subsequent damage.

Initially the contractor had some difficulties obtaining the required strength specification of 31 MPa at 28 days. An unspecified number of units near the time of the job completion were placed after a striking time of about 12 hours.

The contractor was also allowed to cover sections of the crest filter layer with quarry run material to permit equipment access. This finer material was not removed from the filter layer.

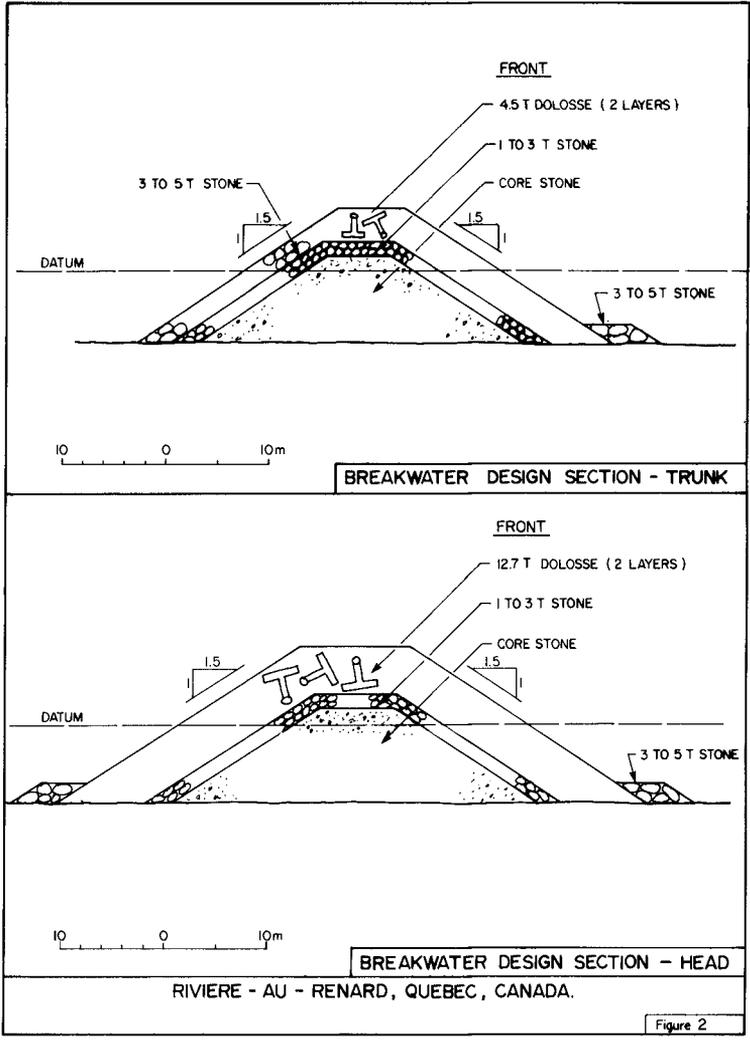
2.3 Damage Surveys and Inspections

Rivière-au-Renard has been cited in previous publications as an example of a successful dolos installation (7). It can no longer be considered as such.

In 1978 it was visually observed that the crest elevations in sections of the main trunk had decreased. Storms in the fall of 1980 eventually opened a major breach at one location and severely damaged another section, both in the 4.5t units.

Field inspections in 1981 revealed the following:

- There was a high percentage of broken dolosse along the full length of the structure including the 12.7t head section. Estimates for broken units on the front slopes above and below mean water level ranged from 10 to 50% not including the areas of the breaches.
- The major breach extended for a length of about 50m and a typical section is shown in Figure 3 (i.e. Section +615). Estimates of broken dolosse in the breach are about 30% on the front below low water, 90% on the crest and front above low water and 65% on the back.
- The secondary breach was in deeper water but extended only 10m in length and was not as extensively damaged. However, dolosse were also piled at the back side of the breakwater (section not shown).



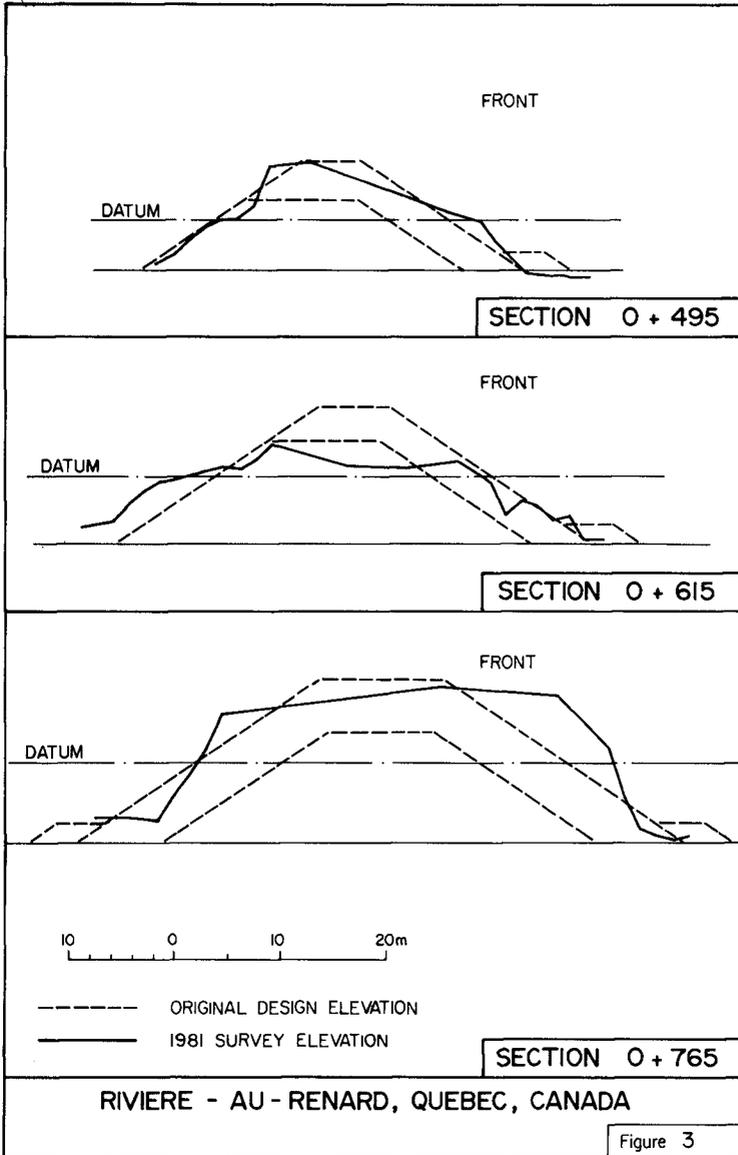
- Overall the slopes were very steep on the landward side of the structure in the area above mean water level. It appeared as if the dolosse in the crest area had been pushed back en masse (see Figure 3 of Section +495), however it is not known whether the original placement met specifications. Seaward side slopes were varied and irregular but showed some tendency to form an 'S'-shape.
- The 12.7t dolosse at the breakwater head showed unusual and steep slopes that could not be accounted for without assuming significant deviations from the design specifications (see Figure 4 of Section +765).
- Over most of the breakwater crest the visible filter layer material appeared to be composed of or filled with much smaller core type stone.
- Most of the rock berm protecting the front toe was either damaged or missing and in areas where the structure resided on sand there was a small trough in place of the berm.

3. Original Design Parameters

Although details of the original design calculations are not available at the time, the Hudson stability formula was used with the K_D , or stability coefficient probably of the order of 15 to 22 for the breakwater head and trunk respectively in breaking waves on a 1 to 2 slope. Assuming a K_D of 22 for the trunk on Rivière-au-Renard, the 4.5t dolos were designed for a wave height $H = 5.1m$; similarly for the 12.7t dolos with K_D of 15 the $H = 6.3m$ (Note: slope was 1 to 1.5 vertical to horizontal).

The design crest elevation of the dolos section ranges from 5.3 to 8.0m above datum and high water large tides are 2.44m. At the time of the design and even to date, there are very limited data available for estimating wave runoff and overtopping and the effects on the breakwater stability (8). However, it would appear the breakwater was intended to be overtopped especially in the initial sections of the 4.5t units. In fact, economic considerations likely dictated minimizing material quantities.

To verify the original design data, wave conditions were hindcast from wind data over an 11 year period by a system of computer programs based on the S.M.B. technique (2). The system produces hourly values of significant wave height and peak period by deep water compass directions. Within this period of data, the largest significant wave heights are of the order of 6.5m with an associated peak period of 12 to 14s. The deep water wave directions for these conditions are almost perpendicular to the breakwater alignment. The wave climate in 1980 reaches similar levels to the previous 10 years, however, the occurrences of the high conditions are greater than the average year.



No detailed refraction analyses were undertaken for this study. The bathymetric contours are reasonably uniform and parallel to the breakwater and the higher waves approach such that there would only be a slight reduction in height due to refraction. The depths and slopes in the approaches are such that larger waves are depth limited and break offshore and in front of the breakwater.

It appears that the original 'design wave' data used were of the correct order of magnitude with the possible exception being the selection of the height for the 4.5 t dolos. It is noted that significant damage has occurred in the larger units as well. The design is complicated by other factors such as overtopping and breaking of waves, however the resultant level of damage indicates that the analysis procedure for stability is not adequate for concrete dolosse.

4. Hydraulic Model Study

4.1 Background

The primary objective of the test program was to recommend remedial works to the breached sections of the breakwater with the 4.5t dolos. Consideration was given to replacing the lost armour material with concrete units of different types or sizes and with large armour stone.

The model test facility available at the time was the 1.8m wide wave flume at the National Research Council Canada in Ottawa. The flume is approximately 1.25m deep by 67m deep and has the capability of generating and measuring irregular waves. Model dolosse were available in two sizes in a sulphur concrete material that gives the units correct specific gravity and dimensions but does not scale concrete strength (i.e. units are not breakable in model loading conditions).

The model scale chosen was 1 to 35 linear such that weight and volume were proportional to the cube of the length scale. Preliminary calculations had indicated that much heavier armour material would be needed but it was also required to model the existing failure as well as possible in the 4.5t units. This presented a conflicting requirement as only one scale could be used within the time allotted. The result was that the smaller lab units were modelled as 12.8t in prototype and these units were in fact used in the 4.5t breached section. This did not represent a gross inaccuracy since the prototype dolosse in the breach were essentially all broken pieces and tests were not intended to totally recreate the whole failure process. The approach slopes in the flume were modelled out to a prototype distance of 800m and depth of about 30m and the modelled section was .9m in width.

Wave conditions generated by each test were for irregular sea states (6). The measurement system in the flume consisted of three wave probes at locations along its length. The wave conditions generated for the tests were selected combinations of deep water significant wave height and peak period, as listed in the following prototype parameters: 2.5m and 6s; 4.0m and 8s; 5.1m and 10s; 6.2m and 12s; 6.8m and 14s.

4.2 Test Series

In general, the wave conditions were run on a test section by starting at low levels and increasing to the maximum with a constant water level (usually at high tide). Each wave condition was run for 15 to 30 minutes in model time (note: time scale approximately 1 to 6 for model to prototype).

4.2.1 Damaged Section +615

Section +615 was chosen as the model section that was typical of the main breach. Two tests were run on this section before remedial works were undertaken. The section was constructed with deliberately broken dolos pieces in the percentages indicated from surveys and diving inspections. The purpose of testing this section was to observe the wave-structure interaction at this stage of damage and to devise repair work accordingly.

The following are observations of the hydraulic tests and describe the progression of the failure at this stage:

- Dolos pieces were rocking and displacing at low wave heights (i.e. 4.2m).
- Overtopping occurred for 4m waves or larger and was quite severe at higher levels.
- Units and pieces below low water on the front became packed after initial movements and remain stable.
- Dolos pieces at the leading edge (i.e. front at crest) were pushed across the top and piled up at the back of the crest.
- Dolos units and pieces were progressively pushed over and rolled down the back slope.
- The crest was eventually stripped level to the core.
- The core became dished out behind the leading edge of the armour allowing more pieces to move across the top.
- Continued wave attack piled a mixture of all the materials behind the back slope.

It was concluded that heavier armour material would be required on the crest to prevent units from being rolled down the back from wave overtopping. Given the excessive movements of modelled dolosse and the evidence of breakage even in the 12.7t prototype units, it was assumed that a small increase in dolos weight would not be an adequate remedial measure. However, a large increase would not be economical especially since there were no available methods to estimate the structural loadings and behaviour of dolosse.

4.2.2 20 t Armour Stone

Three remedial test series were undertaken using 20t stone to fill the breach of Section +615. These included a single layer of armour stone, a double layer and a double layer with additional filter stone added (i.e. each section elevation was progressively increased).

The single layer was built directly over a tested section and run through a complete wave series at mean water level. The section was severely overtopped but remained stable. One or two stones were displaced near the front.

The double layer section was constructed by placing more armour on the previous test section. The section was run at mean water level for the 6m wave conditions. Overtopping was still occurring, but the section remained stable with no armour stone displacements.

The third test was constructed on a rebuilt damaged section and included 1 to 5t filter stone beneath the double layer of armour such that the original design height was achieved. The section remained stable with no armour displacements though it was still overtopping at higher wave conditions.

4.2.3 15 to 11 t Armour Stone

A series of tests using a double layer of smaller armour stone to repair the breach were undertaken in response to initial indications that sufficient 20t stone might not be available. The two test series were constructed over the damaged section and included 1 to 5t filter stone. The heavier armour (15 to 13t) was placed at the front of the crest and the lighter (13 to 11t) to the back of the crest.

The first test section was constructed to the original breakwater height and run at mean then high water level. The armour on the front experienced some initial rocking with no displacement while several stones were displaced down the back at the 6m wave conditions. During the high water level conditions one stone was displaced at the front, two or three more at the back and the overtopping was severe. The section remained stable but sustained significant damage on the back of the crest.

The crest elevation was raised by about 1.5m in the second test section with the increase provided by adding core material in the centre of the breach. At mean water level the section initially resisted overtopping but the 13 to 11t stone began to be rolled down the back with 5m waves. After continued testing at higher wave conditions and high water level, the elevation of the front crest was decreased by 2m and the back by 3m. The severe overtopping eventually stripped the back of the crest down to the core material.

4.2.4 12.8 t Dolos

Two test series were undertaken using 12.8t dolosse in the cross-section configuration of the original trunk design to provide a limited assessment of the stability of the units and the causes of the failure.

The first test used all intact units and appropriate filter layers. These tests began at mean water level and indicated rocking of several dolosse on the front crest near still water level for 4m waves and greater. The rocking was quite severe at higher wave conditions and at high water level there was a major displacement of units down the back. Further testing could not properly simulate the failure at Rivière-au-Renard as the dolosse in the model were not breakable.

The test was repeated with broken pieces introduced in the front of the crest down to low water level (i.e. about 30% broken units). The section went through the stages of: units and pieces rocking at the front then displacing down the front; gaps opening in the front; several units being rolled over the back; units and pieces at front below low water stabilizing; the top of the crest being eaten down to core with material piled over the back. The results appeared to be similar to the damaged section at the site.

4.3 Summary Comments

Under the severest of wave and water level conditions, the 20 t armour stone remained stable. The 15t stone indicated adequate stability with only a minor displacement, however lighter units experienced significant displacement.

The tests with dolosse demonstrated the necessity to account for the breakage of units in hydraulic modelling though the techniques used here cannot readily be proposed for other applications.

5. Conclusions

5.1 Hydraulic Tests

Within the limitations of the test procedures, the model appeared to give a reasonably accurate scenario of the intermediate stages of the failure. Sections in the model were shaped in a similar manner to those in the breach and materials were damaged or displaced in the same areas.

The model did not account fully for the structural behaviour of the dolosse even with pieces artificially injected and it may be subsequently argued that their performance in all the test series is not valid.

The model was not used in a predictive manner to estimate number of storms vs degree of damage and it is not possible to accurately estimate the time frame for the survivability of the existing breakwater. However, it would appear that the remainder of the breakwater is susceptible to a similar mode of failure at any location (probably in 4.5t section first).

5.2 Causes of Failure

It is not clear that any one specific factor has resulted in the extensive damage to the breakwater at Riviere-au-Renard. It may be more broadly attributed to the design procedures recommended for concrete dolos units.

The breakage of the dolosse at the front and crest appears to precipitate an almost total destruction of the units in a particular area. The breakage appears to be related to the potential for rocking or displacement of individual dolos and the unit's inability to resist the resultant loadings (3). The model tests indicate that the breached sections would not have failed in the observed manner if the units had remained intact. However, this is not conclusive in that heavier units were used in the hydraulic testing compared to the prototype dolosse in the breached sections.

The hydraulic testing and prototype surveys (even for the heavier dolos) demonstrated a mode of failure due to overtopping that could not be accounted for in the original design procedure. The severity of the overtopping was capable of displacing units from the back of the crest and down the slope. This phenomenon is related to a number of factors such as the height of the breakwater, the breaking wave conditions at this particular site and the degree of absorptivity of the filter layers beneath the dolos armour.

5.3 Remedial Works

Based on the hydraulic studies and the performance of the existing dolosse, the recommended remedial work consisted of filling the breaches with a double layer of 20t armour stone (17t minimum) and filter material of 3 to 5t. Dolosse were not recommended as there was no satisfactory design process to ensure that an increased size or a reinforced unit could withstand the loadings in the breakwater. The selection of a heavier concrete dolos (i.e. above the 12.4t) would not have proved economical or have provided any guarantee of the unit's structural integrity.

It was assumed from observation of the model tests that the existing dolos armour on the front slope below low water would remain stable. However, it is recommended that regular inspections of this area are undertaken to assess this assumption.

The repair was undertaken in the fall of 1981 and the breakwater was inspected in the summer of 1982. The remedial work had remained stable though a new breach appears to be in the process of opening between the two reararmoured sections. It appears that a long term solution will be required to maintain the remainder of the breakwater.

6. Acknowledgements

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7. References

1. SHORE PROTECTION MANUAL. Second edition. U.S. Army Coastal Engineering Research Centre, Department of the Army Corps of Engineers. 1975.
2. BAIRD, W.F. and GLODOWSKI, C.W. "Estimation of wave energy using a wind wave hindcast model". International Symposium on Wave and Tidal Energy. Canterbury, 1978.
3. BAIRD, W.F., CALDWELL, J.M., EDGE, D.L., MAGOON, O.T., and TREADWELL, D.D. "Report on the damages to the Sines Breakwater, Portugal". Proc. of 17th Coastal Engineering Conference, ASCE. Sydney, Australia, 1980.
4. EDGE, B.L. and MAGOON, O.T. "A review of recent damages to coastal structures". Proc. of Coastal Structures '79, ASCE. Alexandria, 1979.
5. HUDSON, R.Y. "Laboratory investigation of rubble mound breakwaters". Proc. of ASCE Journal of Waterways and Harbors Division, Vol. 85. September 1959.
6. JOHNSON, R.R., MANSARD, E.P.D., PLOEG, J. "Effects of wave grouping on breakwater stability". Proc. of 16th Coastal Engineering Conference, ASCE. Hamburg, 1978.
7. LILLEVANG, O.J. and NICKOLA, W.E. "Experimental Studies in stresses within the breakwater armor piece-dolos". Proc. of 15th Coastal Engineering Conference, ASCE. Honolulu, 1976.
8. MERRIFIELD, E.M. "Dolos - a new breakwater and coastal protection block". The Dock and Harbour Authority, Vol. 50, No. 594. April 1970.
9. WHILLOCK, A.F. "An appraisal of rubble mound breakwaters". The Dock and Harbour Authority, Vol. LXII, No. 732. November 1981.