

THE FIRST CORE-LOC BREAKWATER

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Abstract

The first project in which Core-Locs were used as the main breakwater armour unit, is a small craft harbour on the south east coast of South Africa named Port St Francis. The Core-Loc was selected based on economic advantages over other units. Stability of 15 t units was confirmed in a full three-dimensional model study. Construction of the harbour commenced in early 1996 and was successfully completed towards the end of 1997. Practical construction issues are discussed in detail. Core-Loc performance under construction stage storms was excellent and virtually no damage has taken place under design storm conditions subsequent to the completion of the structure.

Background

The Core-Loc was developed at the Waterways Experiment Station (WES) Coastal and Hydraulics Laboratory (CHL) in 1992 and patented in 1995 (Melby and Turk 1995). During the initial stages of breakwater design for Port St Francis in 1995, the advantages of the Core-Loc over the commonly used dolos unit became apparent.

Figure 1 shows an aerial view of Port St Francis where Core-Locs were used

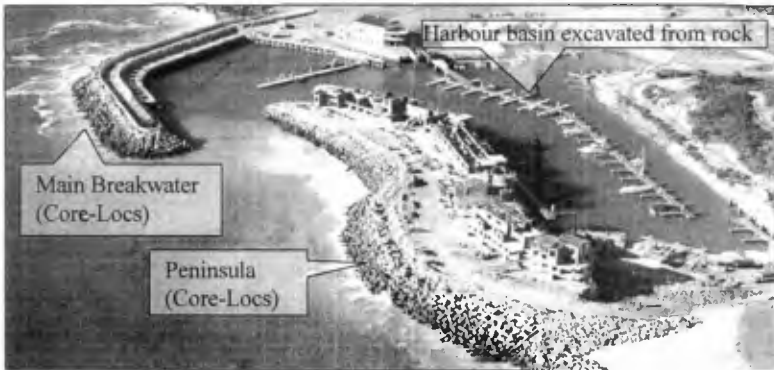


Figure 1. Port St Francis small craft harbour and marina development.

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on the main breakwater and peninsula. Previous attempts to develop a small craft harbour at St Francis Bay were unsuccessful. One of the main reasons for the viability of the present project is minimization of harbour costs. Since the breakwaters make up a large component of these costs, the savings achieved by using Core-Locs as the primary armour, were substantial.

A further significant saving was achieved by optimizing the layout with regard to rock excavation and fill. The harbour basin was positioned to produce sufficient rock excavation for use as breakwater core and armour rock, as well as rock fill required for reclamation of the peninsula as shown in Figure 1.

Core-Loc Design

The initial Core-Loc design was based on Melby and Turk (1995). Maximum design depths of 8 m can be expected for the main breakwater, which means that the design is depth limited. With a foreshore slope of 1:50 and peak wave periods of up to 16 s, the maximum wave height at the structure is estimated to be approximately 7.2 m. Using a Hudson K_d factor of 16, a unit mass of 15 t was selected as a preliminary design.

A three dimensional model study of the proposed design was conducted at a scale of 1:60 in the CSIR hydraulics laboratory in Stellenbosch. The Core-Loc proved to be very stable and even on the breakwater head zero percent damage was achieved. Together with a representative of CERC, Mr J A Melby, who attended part of the model study, the design was finalized as shown in Figure 2.

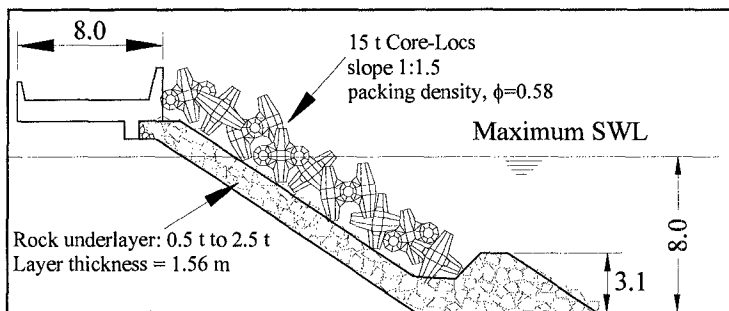


Figure 2. Design cross section for Port St Francis

The rock toe shown in the figure was included to allow for settlement in the case of scour. Probing results indicated that scour would be limited to a depth of approximately 1.5 m by a rock layer underlying the sand on which the breakwater and peninsula was to be constructed.

The packing density and placing grid for the Core-Locs were determined by experimentation since there were no set guidelines established at the time of the study.

A final packing density of $\phi=0.58$ was used, where ϕ is defined as the number of units per area D_n^2 , with:

$$D_n = V^{1/3}$$

V = Volume of one armour unit

The equivalent solid concrete layer thickness, t , for a given armour size and packing density is given as:

$$t = \phi D_n \quad \dots\dots\dots (1)$$

The total volume of concrete, V_{total} , required to cover a given area of slope, A , is:

$$V_{\text{total}} = t A \quad \dots\dots\dots (2)$$

The packing density ϕ , can therefore also be expressed as:

$$\phi = t/D_n$$

The number of Core-Locs per area is given by:

$$N = \phi / D_n^2 \quad \dots\dots\dots (3)$$

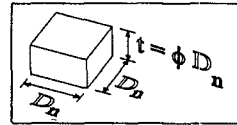
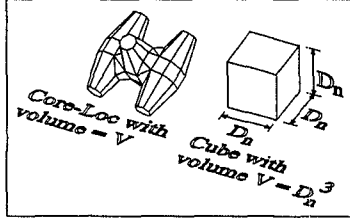
For a 15 t Core-Loc with a concrete density of 2.4 t/m^3 the volume is 6.25 m^3 , and $D_n = 1.842 \text{ m}$. The number of Core-Locs per area is:

$$\begin{aligned} N &= 0.58 / 1.842^2 \\ &= 0.1709 \text{ units/m}^2 \end{aligned}$$

The total slope area that had to be protected was $4\,680 \text{ m}^2$, which required 800 Core-Locs for both the breakwater and peninsula, with a total volume of concrete of $5\,000 \text{ m}^3$. The equivalent solid concrete layer thickness, t was:

$$5\,000 / 4\,680 = 1.068 \text{ m}$$

The only displacements of Core-Locs in the model took place in shallow areas under angular wave attack. This was especially the case at the root of the peninsula where severe plunging breakers caused some displacements of toe units in



cases where they were not interlocked with units behind them. Dolosse with a unit mass of 15 t were also tested and showed the same instability along the toe as that found with the Core-Locs. Since this problem only existed in shallow areas, it would be possible to construct these areas carefully with toe units being visible at low tides. A number of repeat tests were conducted to investigate the probability of toe displacements. These tests confirmed good stability if toe units are placed with reasonable care.

Comparison of Core-Locs with dolosse

Although the focus of the model study was to confirm and refine the Core-Loc design, some tests were conducted with dolosse that allow a comparison of the two units. In general the stability of the two units, both with a mass of 15 t, was good and appeared to be similar (a more rigorous testing program would be required to allow a good estimate of the relative stability of the two units). Dolosse are traditionally placed at a packing density of $\phi=1$ in South Africa which implies a 72 percent increase in both the volume of concrete and the number of units to be placed relative to the Core-Locs (packing density of $\phi=0.58$). CERC (1984) indicates a packing density of $\phi=0.83$ for dolosse, which would bring the 72 percent increase in concrete down to 43 percent, but this is still a very large difference. The minimum packing density that can be used for dolosse is probably $\phi=0.8$, which still implies a 38 percent saving if Core-Locs are used.

Due to the larger packing density of a dolos structure, it can withstand more damage than lower packing densities, without exposing the rock under layers. Another aspect that makes it difficult to directly compare the two units, is that a dolos is more likely to break than a Core-Loc when being displaced by wave action. Since a comparison of units with the same unit mass contains a certain degree of subjectivity, the comparison was also done for equal volumes of concrete. In Table 1, dolos options that have the same total volume of concrete are compared to the 800 Core-Locs with a total volume of concrete of 5 000 m³. Options with equivalent volumes of concrete have the same equivalent solid concrete layer thickness (Equation 1):

$$\phi D_n = 1.068 \text{ m}$$

Using this relationship, dolos options with packing densities of 1.0, 0.83 and 0.8 are compared with the 15 t Core-Locs in Table 1.

Table 1. Comparison of dolos options with Core-Locs that require a total volume of concrete of 5 000 m³ (12 000 t concrete at a density of 2.4 t/m³).

Packing Density, ϕ	Cube Dimension (m)	Unit Mass (t)	Total Number of Units
0.58	1.842	15	800
0.80	1.336	5.7	2 100
0.83	1.287	5.1	2 344
1.00	1.068	2.9	4 100

This table indicates the sensitivity of unit mass to packing density if the same total volume of concrete is specified. A graphical illustration of this comparison is shown in Figure 3.

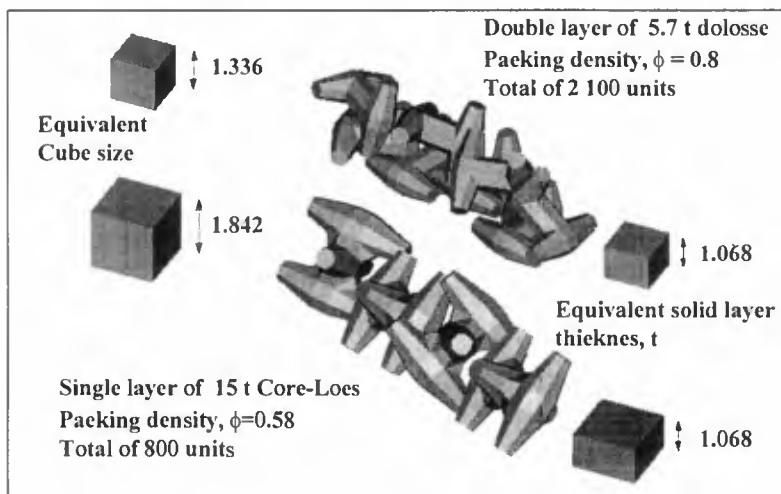


Figure 3. Comparison of dolosse with Core-Locs for equal concrete volumes

The best dolos option is obviously a packing density of 0.8, however the unit mass of 5.7 t is more than 2.5 times lighter than the 15 t Core-Loc and would undoubtedly sustain significantly more damage than the Core-Locs under design wave conditions. This would result in increased maintenance that once again makes the Core-Loc option less expensive. Also, the number of dolosse to manufacture and place is more than 2.5 times the number of Core-Locs. In terms of handling and time for construction, this would tend to make the dolos option more expensive than the Core-Locs.

Construction

The developers of Port St Francis decided against an open tender. A contracting company was formed and a negotiated contract was accepted after a number of modifications to further optimize costs and layout. The contractor had no in-house experience of breakwater construction. Since it was also the first time that Core-Locs would be used, a test section was built in the dry to check the tolerances that would be achieved with the type of construction proposed by the contractor.

The rock toe and under layer were dumped from a skip. Tolerances achieved with this method were checked by measuring rock profiles. It also provided an opportunity to check the toe placement proposed for shallow areas of the breakwater. Figure 4 shows an example of this type of placement and Figure 5



Figure 4. Toe placement used for shallow areas



Figure 5. Completed test section

shows the completed test section in which toe units were placed according to this orientation. In general the rock profiles and Core-Loc placement achieved on the

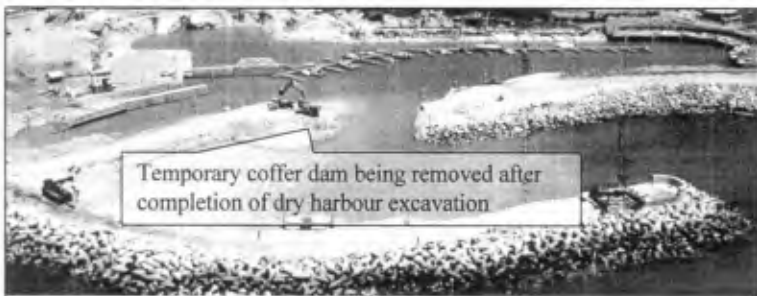


Figure 6. Temporary coffer dam built to allow dry excavation and construction of slipway, mooring piles and quay walls

test section was found to be acceptable. This, together with a specification based on CIRIA (1991), would form the basis for approving actual breakwater sections.

Excavation of the harbour basin commenced in January 1996. It was possible to excavate this area in the dry by constructing a temporary coffer dam as shown in Figure 6. Figure 7 shows the construction site before the harbour basin was excavated. The rock appeared to be good quality sandstone and it was estimated that this quarry would yield approximately 15 percent armour rock based on results obtained from a trial blast and excavation.



Figure 7. Harbour basin before excavation with location of test hole

Figure 8 shows the excavated harbour basin (quarry) which only yielded approximately 6 percent armour rock. This required the opening of a new quarry some 11 km away from the site, causing extensive delays to the breakwater construction.

In addition to the low yield in armour rock, the basin excavation also produced fines and clay that fell outside specifications for breakwater core material and fill material for the peninsula. In spite of the specification, some of this material was used for breakwater construction, leading to the rejection of these sections of breakwater. A compromise was reached where areas containing fines were excavated and sluiced before being replaced.



Figure 8. Harbour basin after excavation. Armour rock yielded ~6%

Storms during Construction

At least 3 severe storms hit the breakwater during construction while a number of smaller storm events caused delays due to the breakwater being inaccessible for the crane. The first major storm hit without any warning. Luckily the waves only started picking up around mid-morning, allowing some time for emergency rock dumping on the breakwater crest. Figure 9 shows the last truck leaving the breakwater just in time. Surprisingly, the Core-Locs sustained almost

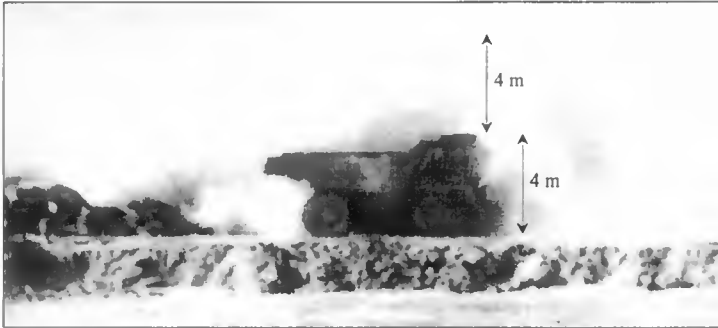


Figure 9. Emergency rock dumping during the first major construction storm. The Core-Locs sustained virtually no damage.



Figure 10. Largest storm that hit the breakwater during construction

no damage during this storm and only the unprotected head of the breakwater was eroded by some 15 to 20 m.

The breakwater was constructed all the way to the head at the level corresponding to the foundation level of the crown wall. Crown wall sections were then cast from the head towards the root of the breakwater (Figure 6 shows the first section of crown wall that was cast on the head). The crown wall construction had just reached the bend in the breakwater when the biggest storm during construction hit the breakwater. Figure 10 shows a wave photographed during this storm with a crane on the completed section of crown wall. Apart from some broken windows the crane was not seriously damaged but the breakwater experienced significant



Figure 11. Construction stage damage mainly due to erosion of breakwater crest. Some Core-Locs rolled further than 10 m without any breakage.

damage along sections where the crown wall had not yet been constructed.

Over the section where the crown wall had been completed, almost no damage took place. However, the section from the bend in the breakwater to the root sustained significant damage. The damage was caused mainly by erosion of the breakwater crest to a level that caused top row Core-Locs to roll toward the lee of the breakwater. This is illustrated in Figure 11 where some Core-Locs rolled over a distance of more than 10 m without breaking.

Engineering Evaluation

Construction of the first section of breakwater in relatively shallow water, proceeded without many problems. However, the crane and ball survey method that was used for surveying core and underlayer rock, had obvious limitations that gave rise to differences between the Contractor and Engineer. The survey was done using the breakwater crane shown in Figure 12. The height of the crane coupled with wave action made horizontal



Figure 12. Crane and ball survey equipment

positioning of the ball very difficult so that surveys often took very long to complete. This was especially frustrating when rising wave conditions made it urgent to protect breakwater sections where under layers had been placed. Before placement of Core-Locs could proceed, under layers had to be surveyed, plotted and approved by the engineer.

When the breakwater reached the bend it appeared that some problem was experienced with the positioning system since a number of Core-Locs were placed too far seaward, resulting in units that were isolated from the rest of the slope. This presented a problem since the isolated units could not interlock and were obviously rolled during some of the storms, as revealed by diving inspections during periods when underwater visibility was sufficient to evaluate the toe of the structure.

In some instances underwater inspections also revealed gaps between Core-Locs that were outside the placing specification. To resolve a differences between the Contractor and Engineer regarding these issues, an accurate survey of the structure was undertaken. A survey system was specifically made up for this purpose that could measure accurately and quickly. It consisted of a measuring staff that slid vertically within a pipe fixed to a crane. The pipe was fixed vertically so that the horizontal position of the staff was always directly below the pipe. The entire breakwater was surveyed in one day, with both the Engineer's representative and the Contractor's site agent agreeing on each point that was measured.

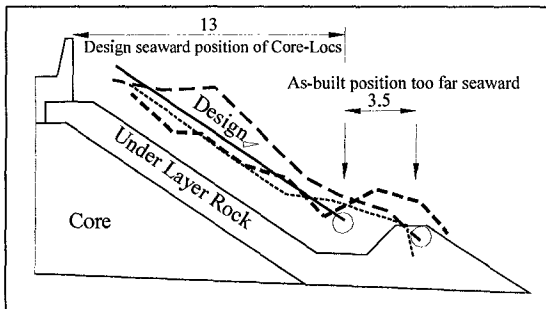


Figure 12. Design versus as built Core-Loc profiles

The survey results showed that some sections of the breakwater did not conform to the contract specifications. Figure 12 shows 3 profiles spaced at 7 m intervals approximately 50 m from the breakwater head. In some instances the toe Core-Loc was more than 3 m too far seaward. Areas were identified where remedial work was required. This mainly entailed removal of toe units that were positioned

away (seaward) of the slope. Approximately 20 units were removed underwater and re-positioned on other areas of the breakwater.

In one area (approximately 15 m length of breakwater) Core-Locs had to be removed to enable rectification of underlayer profiles (placing additional underlayer rock) before replacing the Core-Locs. Removing units from the slope was easy, even underwater, and the remedial work could be completed in less than one week.

Figure 13 shows some typical breaks that occurred during storm conditions on uncompleted sections of the breakwater and on units that were placed incorrectly (mostly seaward of the toe). In most cases the percentage loss in mass is as low as 5 percent. It was therefore decided not to reject Core-Locs with such minor breaks, especially where a displacement causing the break resulted in the unit moving into a more stable position.

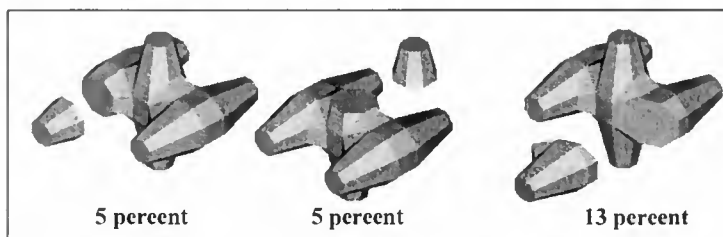


Figure 13. Percentage loss in mass for different types of breakages

Placement of Core-Locs on the peninsula was done after the majority of Core-Locs had been placed on the main breakwater. The experience gained on the breakwater lead to much improved work on the peninsula. Also, no major storm hit the peninsula before the structure had largely been completed. The final Core-Loc protection on the peninsula was of a high standard and was accepted without qualification in terms of the contract specification.

Figure 14 shows parts of the Core-Loc structures that were accepted in terms of the contract specifications, with one section of the breakwater being accepted subject to reduced payment. The acceptance of this part of the breakwater was qualified since it was not built to the original contract specification. However, the relatively minor deviations from the design profile was evaluated as possibly resulting in increased maintenance rather than causing major damage during the design storm event. An amount relating to the estimated increase in maintenance was used as the basis for reducing payment on this section of the breakwater.

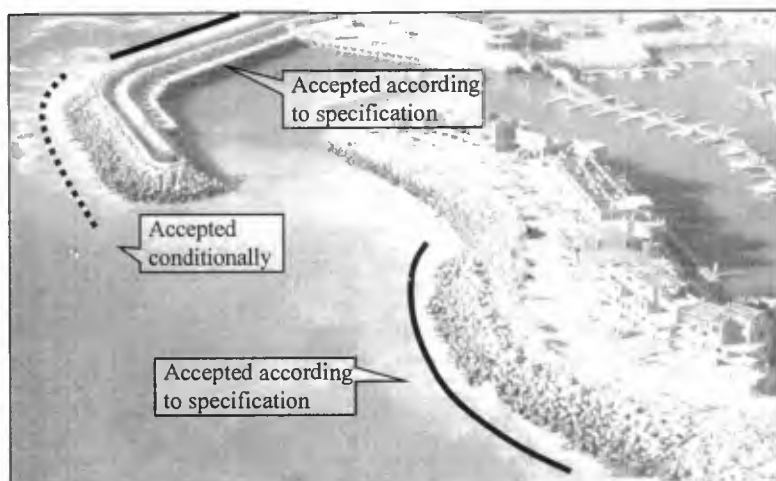


Figure 14. Contractual acceptance of Core-Loc structures

Conclusions

The first Core-Loc breakwater has been successfully completed. It played a significant role in the viability of the Port St Francis development by saving an estimated 30 percent on the cost of the breakwater armouring, compared to the traditional dolos protection used on most South African breakwaters.

Three storms, that approached the breaking wave design condition, were experienced during the approximately 2 year construction period. Core-Loc stability on the uncompleted structure was excellent under these conditions. During one storm, the breakwater crest and core was eroded by overtopping waves, causing some of the top row Core-Locs to roll on to the eroded area. Some units rolled up to 10 m and more without any breakage, clearly illustrating superior structural characteristics.

Post construction performance has been good so far and very little maintenance is expected in future. The structure will be monitored on a yearly basis by the CSIR as part of their research work on prototype breakwater performance.

Acknowledgement

The author wishes to thank the U. S. Army Coastal Engineering Research Center, and specifically the designers of the Core-Loc, Mr J A Melby and Mr G Turk, for their co-operation and assistance with the project.

References

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