GANSBAAI FISHING HARBOUR - THE DESIGN AND CONSTRUCTION OF A BREAKWATER ON A HOSTILE COAST

by

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ABSTRACT

This paper describes the design and construction of a fishing harbour on a rocky coastline exposed to the prevailing south westerly swell of the South Atlantic and to severe westerly gales.

Because of economic pressure the first phase of the development was undertaken without adequate knowledge of the wave regime or the topography of the sea-bed in the area and resulted in a virtually unusable harbour.

The second phase of construction was therefore only embarked upon after extensive hydrographic surveys, wave recording and analyses, and probably the most exhaustive series of model tests ever undertaken for such a small project.

These investigations and the good co-operation between research staff, engineers and contractors resulted in the elimination of most of the initial problems and the creation of a functional fishing harbour.

INTRODUCTION

Gansbaai is a small fishing village situated 170 km south east of Cape Town as shown in figure 1, and only a few kilometres from Danger Point, scene of the tragic wreck of the Birkenhead in 1852. When this ship went aground more than 400 soldiers *en route* to the eastern frontier of the Cape Colony perished.

This coastline near the southern tip of the African Continent is exposed to the prevailing south westerly swell of the South Atlantic and the full force of westerly gales.

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A landing place for fishing vessels developed at Gansbaai because a deep channel in line with the direction of the waves provided an approach route relatively free from breaking waves.

Although the fishing village has been in existence since the late 19th century, the first noteworthy harbour works were constructed in 1939-1942 when breakwaters were built from the northern and southern shores to enclose an area of approximately 4 hectares, of which less that 1 hectare was deeper than 3 metres at low water ordinary spring tide. Unfortunately the northern breakwater cuts across the deep entrance channel forcing boats entering the harbour to deviate from the channel towards the less safe shoaling bottom south of the channel (see figure 2).

Rapid expansion of the pelagic fishing industry in the late 1950's created a demand for improved facilities at Gansbaai as at other fishing harbours, particularly on the west coast of South Africa.

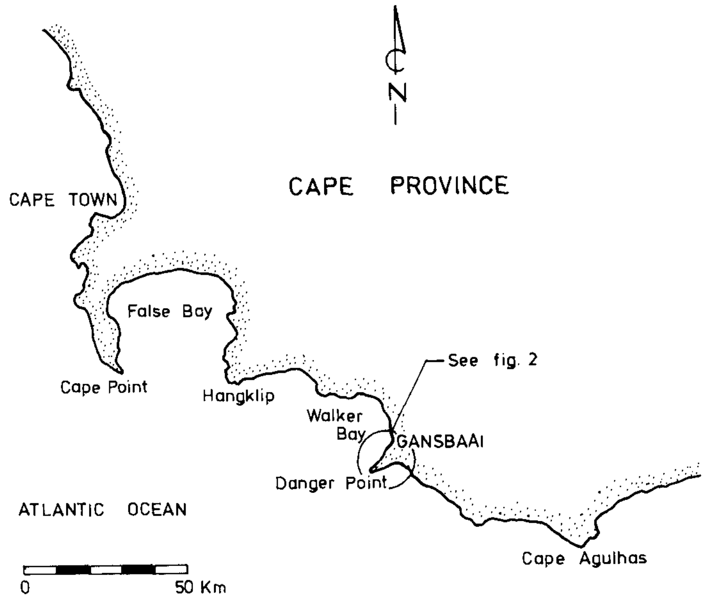


Fig 1. Location Plan

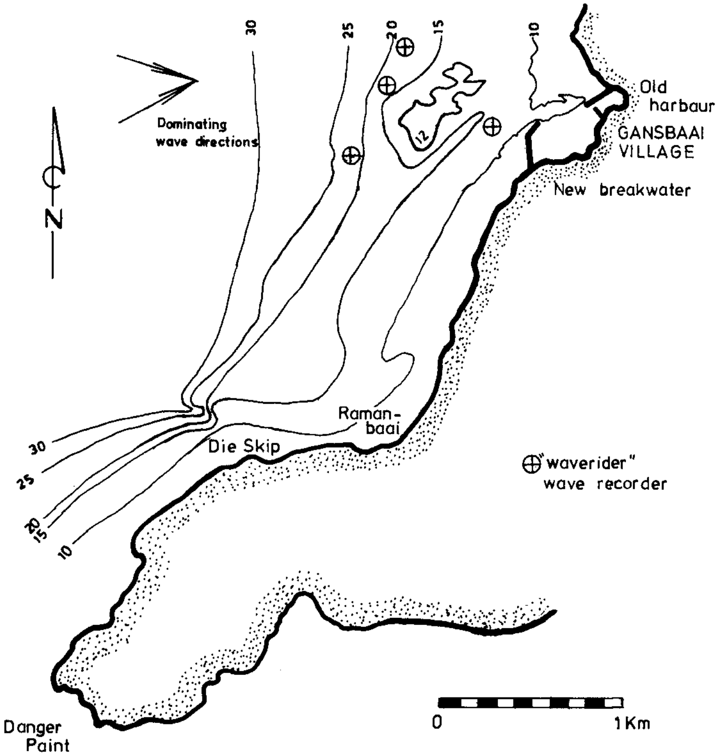


Fig 2. Coastline around Gansbaai

As no obvious solution for the improvement of the existing harbour presented itself, other sites in the vicinity notably Roman Bay and "The Ship" on the north western shore of the Danger Point Peninsula (see figure 2) were investigated. These options were discarded because of the high cost of breakwaters in the deep water at these two sites and the additional cost of a road from the existing village and it was decided to construct a new breakwater south west of the existing harbour at Gansbaai.

FIRST PHASE OF DEVELOPMENT

The new breakwater was to serve a two-fold purpose, viz to protect the entrance to the existing harbour and to provide additional mooring space which could be developed into a fully fledged fishing harbour at a later stage.

The alignment of a breakwater required to effectively protect the entrance to the old harbour, presented difficulties because it would have to extend into or across the deep channel, as shown in figure 3. Such a breakwater would be expensive and could create the same problem as that caused by the existing northern breakwater, i.e. forcing incoming boats into the shoaling water on the opposite site of the channel. Reflection of wave energy by the new breakwater could also create turbulent conditions in the entrance channel.

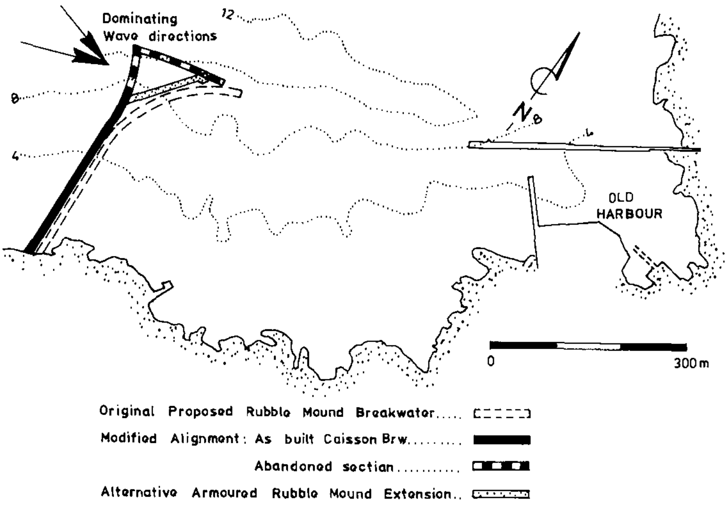


Fig 3. Layout of Breakwaters - Phase I

In 1964 the South African Council for Scientific and Industrial Research (CSIR) was requested to carry out a model study to determine the most suitable alignment for the new breakwater. A fixed bed model was constructed in an existing tank in Pretoria at a scale of 1 in 100.

The study was to be based on hydrographic and other information supplied by the Fisheries Development Corporation of SA Ltd. Unfortunately the urgency of the project precluded the collection of adequate wave data. From the deepsea wave recordings of the fisheries research vessel Africana II in 1962/63 (ref 1) it was concluded that wave heights of 6,1 m from the west and 5,5 m from the south west occurred in the area about 10 times a year for a duration of 6 hours.

Experienced fishermen in the area maintained that deepsea waves exceeding about 6 m in height would break on a series of reefs situated about 1 to 1,5 km west and south west of the harbour (see figure 2). Under storm conditions this break was continuous right across the bay making the harbour inaccessible. It was therefore concluded that even the highest waves, on reforming, would not exceed 6 m in height at or near the proposed breakwater. (Later wave recordings proved this assumption to be incorrect because of the focussing effect of the reefs.)

The hydrographic survey of the area was also inadequate. The original survey was carried out in a small dinghy with position fixing by theodolites on shore. The high swells and the distance from shore led to inaccuracies and an inadequate coverage of the area seaward of the proposed harbour. Consequently the modelling of the sea-bed was not sufficiently accurate and did not extend far enough out to sea to ensure correct reproduction of the waves in the vicinity of the proposed breakwater. It was not realised at the time that the outlying reefs resulted in marked focussing of waves at certain frequencies on to the line of the proposed new breakwater as shown in figure 6.

Before completion of the model study the pressure for the provision of additional harbour facilities became so great that a contract based on an armoured rubble mound breakwater was advertised on the assumption that minor deviations in the alignment of the breakwater indicated by the model study could be accommodated in the course of the work.

The lowest tender for the work was based on an alternative design by an international construction company for a vertical wall consisting of concrete caissons anchored to the sea-bed with post-tensioned cables. Although a vertical wall was not favoured and it was realised that the seaward face might ultimately require some form of energy absorber, it was argued that the cost of such absorber would be offset by the advantage of a quay on the inside face of the wall.

Considerable doubts had also arisen as to the practicability of obtaining suitable large rubble from the highly fractured quartzitic sandstone in the local quarry. The contract was therefore awarded in 1965 on the basis of the alternative design. Details of the original caisson structure are shown in figure 4 together with the modifications which later proved to be necessry.

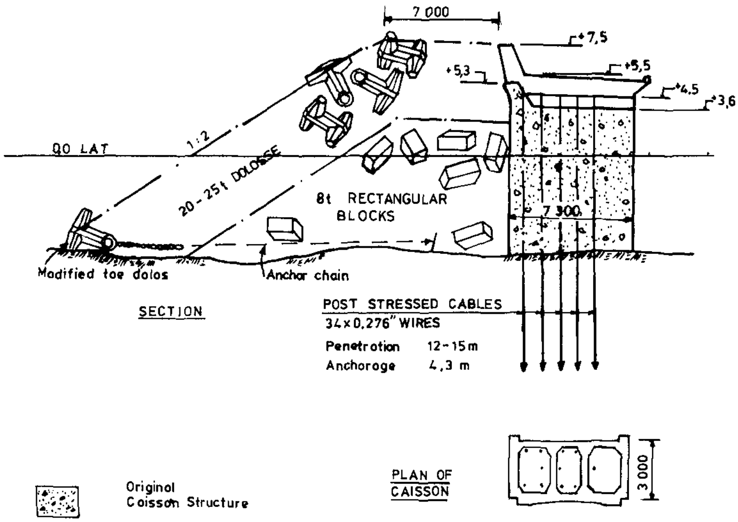


Fig 4. Caisson Breakwater (with armouring added later)

The wall, 6 m wide in shallow water and 7,3 m wide in deeper water, was constructed in open reinforced caissons 3 m long in the direction of the wall and divided into 3 cells with 300 to 400 mm thick walls. The units, approximately 3 m high, were stacked one on top of the other on a bed of broken stone and filled with concrete to within ± 400 mm of the top.

The deck was cast to a level of 3,6 m above low water and 90 mm diameter holes for the installation of the anchor cables drilled to a depth of 12 to 15 m below sea-bed level. The holes were pressure grouted to seal off fissures in the bedrock, interstices in the broken stone bed and construction joints in the concrete structure, and re-drilled. The wires were grouted in over a length of 4 to 5 m at the bottom of the hole, stressed and finally grouted in up to the level of the deck.

During construction of the wall, sea conditions were found to be more severe than anticipated. Placing of caissons was only possible when the wave height was less than about 1,5 metres. Overtopping hampered the work to such a degree that the level of the deck had to be raised from 3,6 m to 4,5 m above low water and the splash wall had to be re-designed at a very early stage of the work.

Fig 5. Wave attack on vertical breakwater

During storms the gantry crane which was specially built for the work and ran on rails concreted into the deck had to be moved back onto land. The crane was washed off the breakwater twice during the course of the work. It also became apparent that it would not be possible to use the wall as a quay and that boats would not even be able to moor near the wall because of the excessive overtopping.

The rock formation on the sea-bed proved to be highly fractured which made sealing of the holes for the stressing cables very difficult. The grouting and redrilling often had to be repeated several times before the hole could be considered watertight. (This method eventually proved to be ineffective in protecting the cables against corrosion. This was proved by ultrasonic tests on the stressed cables in 1977 and confirmed when a portion of the wall was demolished during a subsequent contract.)

When the caisson wall was incorporated in the model the excessive overtopping of the wall and the reflection of waves from the vertical face was clearly demonstrated. It was evident that the reflection of waves from west north west and north west off the last section of the breakwater would result in extremely rough conditions in the access channel to the old harbour. Various remedial measures were considered but eventually a revised alignment was proposed whereby the wall would initially curve westwards and then at chainage 360, bend sharply eastwards so that the first section would reflect waves to the south western shore and the second section would be almost parallel to the direction of the waves (see figure 3).

The adoption of this unusual alignment was unfortunate because it moved the breakwater towards deeper water and a concentration of wave energy near the proposed bend. Refraction diagrams based on more extensive hydrographic surveys carried out later confirmed this focussing effect. An example is shown in figure 6.

As the construction of the wall approached the proposed bend, sea conditions became more severe and the contractor became more and more concerned about the prospect of not being able to retrieve the crane during stormy weather via the sharp bend in the new alignment of the breakwater.

Doubts as to the efficacy of the stressing system increased as the water depth increased and the sea-bed became more highly fractured. Eventually it was decided to abandon the idea of a sharp bend in the breakwater at 360 metres, and to end the vertical wall at 300 metres. The remainder of the breakwater would be constructed as an armoured rubble mound as shown in figure 7 which would reflect less energy into the approach channel.

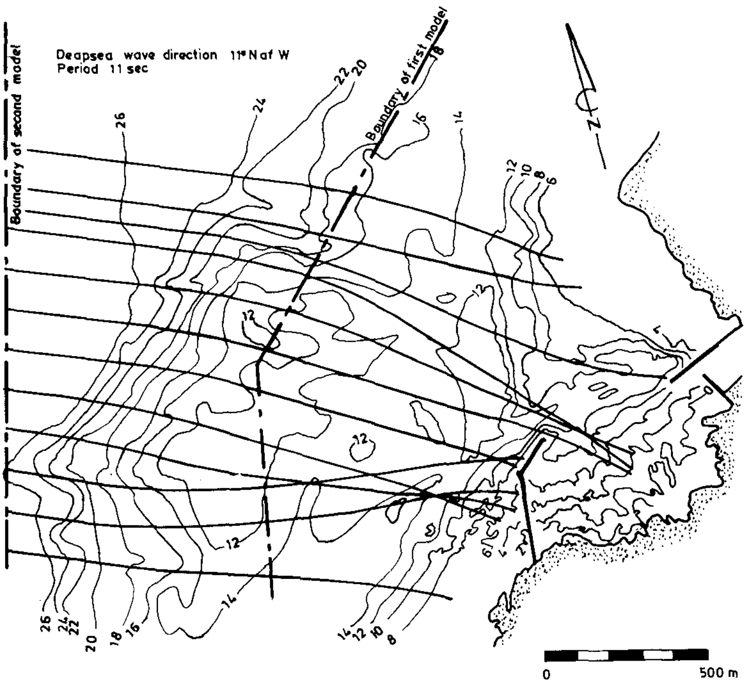
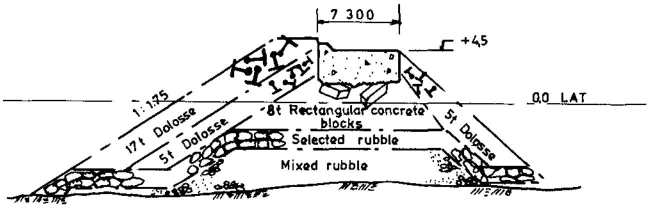
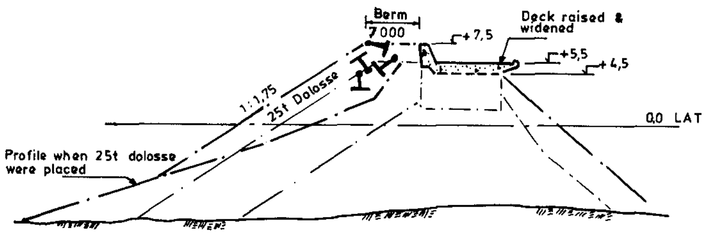


Fig 6. Wave Refraction



ORIGINAL PROFILE



MODIFIED PROFILE

Fig 7. Armoured Rubble Mound Breakwater

Construction of this rubble mound extension of the breakwater proved to be no less difficult than construction of the caisson wall. The fractured quartzitic sandstone available for rubble in the core contained a high percentage of fines of which large quantities were washed away. Due to the shortage of the specified selected rubble a layer of 5 ton dolosse was initially placed over the rubble core. Later 8 ton rectangular blocks were used for this purpose. These units helped to prevent the immediate loss of core material but allowed considerable quantities of fine material to be leached out with the passage of time. This loss of core material combined with the structural weakness of the 5 ton dolosse, resulted in marked settlement of the deck over several sections. One section of the deck had to be demolished and rebuilt in order to fill the large cavity which had developed under the deck.

Wave recordings which had commenced in 1968 at a depth of 15 m on a position inside the reefs as shown in figure 2, had indicated that waves up to 8 metres could be expected at the breakwater. Further model tests had shown that the 17 ton dolosse used for the armouring might not be adequate. The contractor's plant, however, did not allow for the placing of heavier units at the distances required and the design could therefore not be altered without prohibitive additional costs.

The weakest section of the rubble mound breakwater was found to be at the interface with the vertical wall. This was probably due to the focussing of waves on to this section of breakwater and the lack of interlock between the vertical wall and the rubble mound breakwater. The deflection of wave energy off the flat end of the caisson wall could also have contributed to the damage which occurred repeatedly to the breakwater along this section. Several unsuccessful attempts were made to reproduce this damage in a wave flume and in a later 3 dimensional model study. The damage was probably aggravated by fracturing of the 5 ton dolosse between the core and the deck or the overlying 17 ton dolos layer and possibly by low packing density of the dolosse.

The leeward slope of the rubble mound breakwater which was protected with 5 ton and later 12 ton dolosse also suffered considerable damage in the first year after completion because of heavy overtopping and because of the penetration of wave energy through the highly permeable 5 ton dolos layers underlying the deck. Reasonably successful attempts were made to repair the erosion of the inner face by sealing the cavities with concrete pumped into filter cloth bags and providing a steeper leeward slope so that overtopping waves could expend their energy into the enclosed basin.

It was feared that the roundhead at the end of the breakwater would be particularly vulnerable but it suffered relatively little damage possibly because the armour units were chained together above the low water level.

Although at this stage the objectives of the scheme (i.e. to provide safe mooring space for additional and larger fishing vessels and to improve the entrance conditions at the old harbour where the factory was still situated) had not been achieved, the completion of the rubble mound breakwater in 1969 marked the end of the first phase of attempts to improve facilities at Gansbaai. The fishing industry had by this time entered into a period of decline and additional funds for further development were not readily available.

SECOND PHASE OF DEVELOPMENT

Although the urgency for further development of the harbour had abated, a number of questions regarding the durability of the completed portions of the work had to be resolved. The overall stability of the rubble mound breakwater was a matter for concern because of the quality of the rubble and the inadequate mass of the armouring of 17 ton dollosse. (This concern was proved to be justified when a section was destroyed in 1979 shortly before it could be reinforced with 25 ton dollosse.)

An even more serious problem was the uncertainty regarding the stability of the caisson wall. This wall which had originally been designed with a factor of safety against overturning of 1,5, had subsequently been raised from an effective height of 3,6 metres above low water to 4,5 metres above low water and was also being subjected to attack from waves higher than the original design wave height. New calculations had shown that the factor of safety was lower than originally intended. Factors such as the force taken up by the higher splash wall, an occasional higher still water level due to set up and transverse resonance (recorded locally and confirmed by a mathematical analysis by the Danish Hydraulic Institute) were taken into account. The higher SWL also accounted for higher overtopping than expected.

A series of two dimensional tests confirmed that the factor of safety was nearer 1,0 and in some sections of the wall probably less than 1,0. The 2 dimensional tests which were being conducted to investigate the caisson stability were extended to explore various ways of reducing the overtopping of the wall (such as different shapes of splash wall and a lower splash wall on the seaward side with a secondary splash wall on the leeward side).

Serious doubts had also arisen regarding the soundness of the stressing cables which supplied the major proportion of the moment of resistance against overturning. Ultrasonic tests conducted in 1977 (ref FDC report F92-2) revealed that these wires were in fact seriously corroded especially at the interfaces between the caissons and the sea-bed and the caissons and the deck slab. The continued stability of the wall could probably be ascribed to the angle of wave attack which resulted in the forces on the wall only reaching maximum values over very short sections of wall at any given instant and the transfer of these forces to adjacent caissons through the deck and the keys between the caissons.

In order to solve the many problems which had by now been identified, i.e. stability and excessive overtopping of the completed new breakwater, and also provide design guidelines for the completion of the harbour works preferably as one combined larger harbour, a second 3 dimensional model study was commissioned in 1975. Again the FDC would be responsible for the field work while the CSIR would carry out the model study.

An intensive study of the wave regime was carried out between July 1975 and September 1978. To obviate the mistakes which had occurred in the first model a series of three "Waverider" recorders were installed seaward of the reefs and one waverider nearer the breakwater. As the direction of wave attack on the breakwaters had proved to be critical, a wave direction recorder (DOSO) developed by the FDC was installed seaward of the reef close to the central waverider. The effect of the reefs on wave refraction could therefore be accurately determined. The positions of the recorders are shown in figure 2.

More detailed hydrographic surveys were also carried out (including side scan surveys to determine the shape and nature of the rock strata). Accurate refraction diagrams, verified by numerous aerial photographs under the complete spectrum of wave attack, as recorded by the 4 waveriders and DOSO were used to calibrate the wave reproduction in the model. The model was constructed on a scale of 1 in 80 over a much larger area than for the previous model in order to reproduce the wave regime as accurately as possible.

The result of the 2 dimensional tests on stability of the caisson wall were verified in the 3 dimensional model by constructing the caisson wall of 100 mm wide hinged box sections fitted with strain gauges at several elevations. The forces and overturning moments were determined and the results compared well with those of the previous 2 dimensional models.

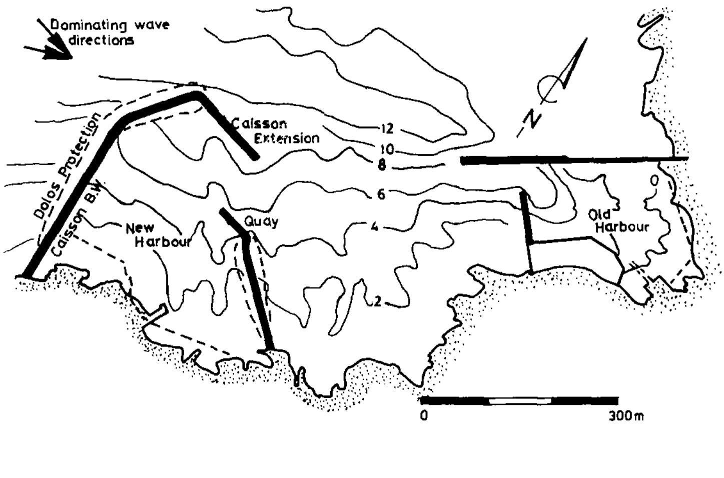


Fig 8. Revised layout of new harbour

Many different alignments, sections of breakwaters and absorbing structures seaward of the caisson wall were tested under all recorded wave directions, heights and periods, as small variations in these parameters affected the focussing of waves along the breakwaters considerably. A solution was found for the safeguarding of the existing new breakwater and an extension of this breakwater to create satisfactory entrance and mooring conditions in the "new" harbour was successfully tested. No really successfull solution was found for the combination of the old and new harbours because of the distance between the ends of the main breakwaters of the two harbours. One layout tested involved the construction of a T-breakwater halfway between the two harbours. The cost of this scheme, however, rendered it unacceptable. Even the most favoured scheme involving the extension of the north breakwater of the old harbour was not justifiable. The main part of the model study was completed in 1978 but additional tests involving mainly construction techniques continued up till 1981.

In 1978 it was decided to proceed with a scheme involving the safeguarding of the existing caisson and rubble mound breakwaters, the extensions of the rubble mound breakwater and the construction of a quay and approach mound from the southern shore. This would enclose a protected water area of approximately 7 hectares of which 3 hectares would have a water depth exceeding 4 metres at low tide.

The measures adopted to protect the existing caisson and rubble mound breakwaters took into account the variation in design wave height along the length of the wall in the design of deck and splash wall height, mass of armour units and width of berm at the top of the armoured layer. The final deck height was 5,5 m above low water and the top of the splash wall varied from 6,5 to 7,5 m.

20 and 25 t dolosse placed to a slope of 1:2 on a core of 8 t corrugated rectangular blocks were used on the outside of the caisson wall as shown in figure 4 and 25 t dolosse placed to a slope of approximately 1:1,75, were used to protect the rubble mound breakwater as shown in figure 7. A horizontal 7 m wide berm at splash wall level (7,5 m) was used for additional absorbtion of wave energy.

Specific attention had to be given to the stability of the toe of the dolos layers because of the relatively smooth sea-bed seawards of the caisson wall coupled with reflected wave energy from the wall. Model tests indicated that the toe line of dolosse would move out of position at relatively low wave heights (1,5 m). A special reinforced "toe dolos" was developed with a lowered centre of gravity to prevent overturning and with two projections on the ends of the lower fluke to prevent rocking (see figure 9). To prevent sliding these dolosse were tied back into the mound with heavy chain and a rudimentary anchor.

Fig 9. Reinforcing of toe dolos

Fig 10. Storm damage to rubble mound breakwater

The contract for the above work was awarded at the beginning of 1979, but in August of the same year, before any remedial work could be carried out, a major storm breached the existing rubble mound breakwater. Other storms followed and eventually almost 60 m of the breakwater was destroyed as shown in figure 10. Model studies were again carried out to determine the most effective way of repairing the breach, and a construction method using 8 t corrugated concrete blocks as a core with a double layer of 25 t dolosse as armouring was eventually successfully used. During the repairs to the breach the junction between the vertical wall and the rubble mound breakwater was reconstructed. The section of the vertical wall extending beyond the rubble mound breakwater was eliminated and a smooth curve introduced. Fortunately no severe storms occurred during these critical stages of the work.

The extension of the main breakwater by means of another vertical wall more or less parallel to the direction of the waves and the construction of a secondary breakwater with caisson quay from the eastern shore were also completed without serious problems.

CONCLUDING REMARKS

The construction of the extension to the Gansbaai fishing harbour has been successfully concluded but only after a disproportionate volume of model testing and other investigations. The project was carried out over a span of almost 20 years which coincided with a period of rapid development in the science of coastal engineering.

The initial mistakes were due to inadequate field data, a lack of appreciation of the effect of distant topographical features of the sea-bed on the wave regime, the adoption of an untried design without model testing and the commencement of construction before completion of the model tests.

The project has highlighted the need for closer co-operation between research personnel, designers and contractors.

ACKNOWLEDGEMENT

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