

Lessons Re-Learnt from the Failure of Marine Structures

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Introduction

Codes of practice in the UK and overseas assist the designer in scheme planning and detailed design phases of a project, which lead to the development of the Tender and Contract documents and to construction. While it is normal to follow these codes, it is also often prudent to carry out additional investigations, say of the soils, as well as modelling.

Despite all the advice and guidance and the increasingly sophisticated methods of analyses available in recent years, failure of structures still occur. While these failures can be the result of the structure being exposed to extreme events, greater than the structure was designed to withstand (say due to global warming), failure is mainly because of deficiencies in their design or their construction. Such failures are, however, rarely due to a single 'error', but can be the combination of, say, an inappropriate aspect of the design, lack of quality control during construction or of construction methods or of taking the present levels of knowledge beyond its limits.

An example of the latter was the \$178 million Sines breakwater in Portugal which failed catastrophically on 26 February 1978. It was built in deep water exposed to very large Atlantic waves (design H_s was 11m), used an armour unit almost twice as heavy as any used before, whose stability factor (K_D) was subsequently reduced and where the scale of the structure, its exposure and methods and made construction difficult. Reconstruction costs are believed to be about 30% of the original cost.

Happily not all failures have the same scale of damage or are as costly to repair as Sines, but repair costs can represent a significant percentage of the original cost (over 100% in one known case). They still occur and not only affect the insurance covers of all concerned but, in making the structure no longer fit for purpose, can also lead to costly delays while the repairs are being considered, designed and carried out and finally deciding who should pay for the repairs. Once a structure is damaged, especially during construction, the damage can progress quite rapidly, even though the subsequent storms are not as severe. This is especially so for the more sophisticated armour units, which rely on their interlocking properties and where the debris from broken units can cause further damage.

Rubble mound structures are not alone in being subject to failure and the paper examines historic and recent failures of a number of marine and coastal structures that have been damaged, normally soon after completion, by smaller storm events than the ones they were designed to withstand. The failed structures investigated range from breakwaters (rubble mound and vertical) through to revetments embankments and scour blankets. Many lessons can be learnt from their likely modes of failure, but many are in fact a re-learning process. This is despite the fact that in many cases the designs have been physically model tested. It

does not always follow that if proved safe in the model (whether it be a physical or mathematical hydraulic model, or geotechnical model), then any failure must be down to poor construction. Model testing of whatever type can still be a very useful tool as it allows the design to be modified prior to construction. Failure modes seen in the field can also be investigated by modelling.

History of some breakwater failures (rubblemound and vertical)

Sines breakwater in Portugal, the largest breakwater of its kind in the world, was armoured with some 21,000 Dolosse, and failed catastrophically on 26 February, 1978. This failure occurred as construction neared completion and, by the time storm had abated, two thirds of the Dolosse units making up the armour layer of the breakwater were apparently lost (Herzog, 1982). Five simplified failure possibilities were evaluated (ASCE, 1982), namely that:

- The design criteria were exceeded.
- The breakwater construction was faulty.
- That materials used for construction were sub-standard.
- The procedures during the design were incomplete or incorrect for this specific set of severe environmental conditions.
- The design was inadequate.

The analysis of the damage to the breakwater showed that, in general, the units above water level were in fact less damaged than those below. This led the general conclusion that the failure had occurred due to a general slumping of the Dolosse down the slope. What is not known, or can ever be known, is whether the slumping was due to the fracture of these large 42t un-reinforced units, almost twice as large as those ever used before. This fracturing could have been the result of the rocking of the units, leading to collision and damage or the poor placing of units and loss of underlayer rock between them, or damage during their initial placement or the movement of already broken units present after earlier storms, or the destruction of the 16t to 18t rock at the toe of the Dolosse slope. All could result in widespread slumping and damage.

Perhaps the most important lesson coming from this failure is that it is not sufficient to apply principles known for smaller structures in shallower water, to larger structures in deeper water, exposed to more severe conditions. It is also apparent that little thought was given as to how it would be built and the risks involved.

Subsequent studies have also suggested that failure occurred because of erosion of the seabed of toe and slumping of the scour rock into it, the predictions for the depth of this erosion hole being up to 10m (Sylvester, 1989). What was also clear from this and other failures is that the grouping of waves, which was not tested in the 'design' hydraulic model, was subsequently found to be an important aspect and, in deep water, the maximum wave height during a storm can produce a devastating effect.

The destruction of Sines breakwater also led to a major studies of the strength of the Dolosse units and whether or not they should be reinforced (see for example Burcharth et al, 2000), which also includes Tetrapods. Model units were also built for testing in flumes to see the stresses that were built up during the passage of the wave across the armoured layer, at different elevations within the layer (Scott et al, 1990). These tests showed that failure could occur, despite the fact that under normal test conditions they were stable. While 2% of broken

Dolosse were found before the storm (ASCE, 1982), Herzog (1982) claimed 10% of the Dolosse were found to be broken before the February 1978 storm. The ASCE (1982) indicate that, of the Dolosse used in the temporary protection during the winters of 1975/76 and 1976/77, some 14% of the recovered units were broken.

In comparing failures of Dolosse breakwaters, Burcharth and Liu (1992) found that at Crescent City (USA), Richards Bay (SA) and Sines the reported displacement and breakage was 26.8%, 4% and total collapse respectively, with the size of units being 38t, 20t and 42t. The predicted displacement was similar for Sines and Crescent City (3.6%), but less for Richards Bay (0.6%), with the predicted breakage being the same for Crescent City and Sines ($\geq 10\%$), and 5.7% for Richards Bay.

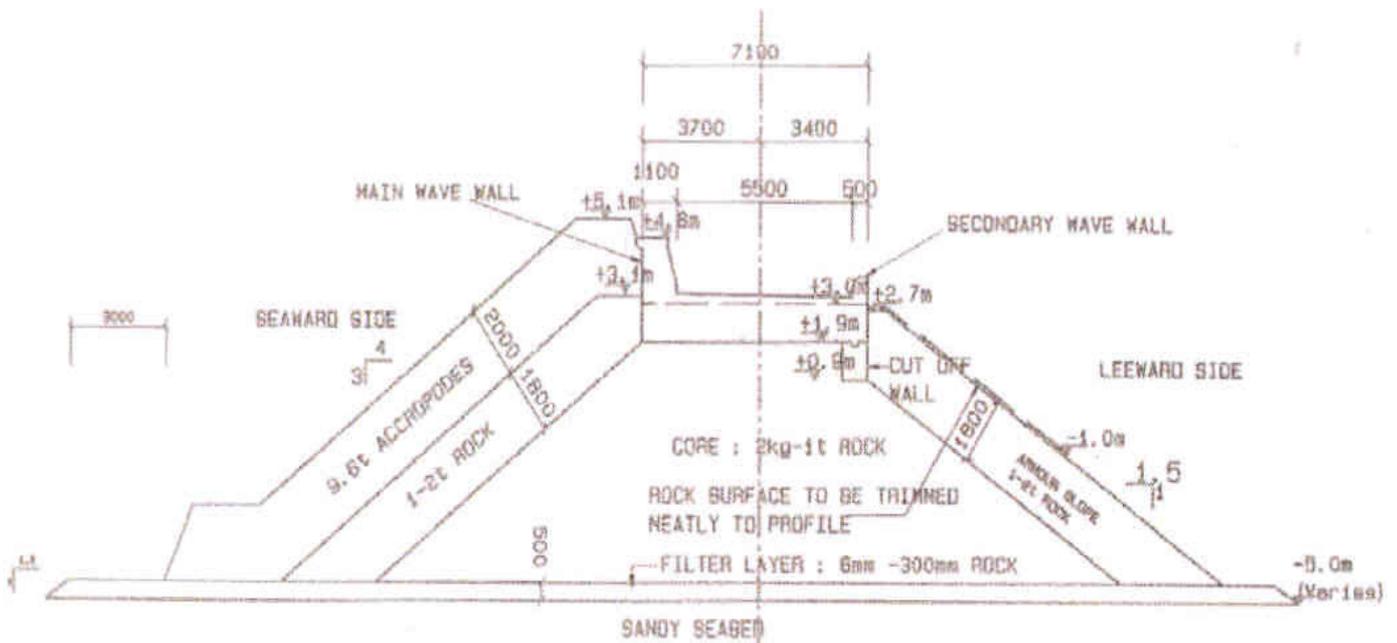
In Kahului, Hawaii, the breakwater which was built with heavy armour stone had to be rebuilt using Tetrapods after it was badly damaged. When the Tetrapods were destroyed by storms it was then rebuilt with Tribar armour units and, when these were washed away the present structure was protected by Dolosse (Harlow, 1980). Significant damage to Tetrapods have also been seen in a number of structures around the world, including Tripoli in Libya. It should be noted that the design wave was underestimated at Tripoli as little regard had apparently been paid to wave breaking and shoaling transformations. The units too also suffer from stress related damage, with legs being broken off and the broken pieces then acting as battering rams causing further damage within the armour layer. Stresses in these units were examined by d'Angremond et al (1994).

The two breakwaters of the Diablo Canyon Nuclear Power Plant that were built in 1970-71 were armoured with Tribars. In January 1981, the west breakwater was severely damaged by a storm where the wind waves were increased by over 50% by swell waves to give a maximum H_s about 6.3m (Lillivang et al, 1984). Not only were large numbers of Tribars destroyed, but several sections of the concrete capping, weighing some 300t, slid into the sea. The recommendation was that properly scaled 3D-physical modelling is essential, unless the Owner and Engineer understand and are prepared to take the risk.

Damage to breakwaters is not a new phenomenon and, in 1881, a new breakwater built in Port Erin in the Isle of Man was destroyed. The breakwater was protected by 18t concrete Brick Shaped units which, looking at their weight and placing pattern, should have been capable of withstanding a design storm wave H_s of about 5m. What would appear to have been the case for this breakwater is that, while the armour units should have been large enough, the core rock from a local quarry consisted of Manx slate, the largest size of which would have been about 2 tons. Looking at the breakwater today, which has survived about 120 years of storms, is that it has simply settled, with the core rock being washed out and possibly settling into the sandy bed, leaving behind a relatively stable semi-submerged breakwater, exposed at about mean tide level. This breakwater, like many others subsequently, was also damaged during construction and it is likely that this damage contributed to the final failure.

Vertical and composite breakwaters are also subject to damage and Oumeraci (1994), identified 17 failures of vertical breakwaters and a further 5 failures of vertical breakwaters which had been armoured. He concluded that almost none of failures reported had occurred without prior warning given by the experience seen during less violent storm events. He also believed that many of the failures occur because the breakwaters represented attempts to design and build to a scale not seen before. Franco and Passoni (1994) believed the failure of

Naples' caisson breakwater in 1987 was mainly due to exceptional spectral wave characteristics.



**Figure 1 Design Cross-section of the Club Mykonas Breakwater
(after Bartels et al, 2003)**

Some rubble mound structures are designed to “fail”, such as the Berm breakwaters. These structures have been built in areas where the size of rock available was limited and the cost of protecting breakwaters with concrete armour units was prohibitive. While they can never be considered maintenance free, there are many structures which are relatively stable and efficient in destroying storm wave energy. However, failures do occur as was the case with St Paul breakwater, Alaska. This breakwater, completed in October 1984, was severely damaged during two storms in November and December the same year, which reduced its affective length by about 60%. While the structure had been model tested, it is apparent that the materials used in construction were not the same grading or size of those used in the model (Sorensen and Jensen, 1990). The breakwater was subsequently redesigned and required a 14t armour stone on its trunk with 18t armour on its head, with the slopes being 1 in 2.5 and 1 in 3 respectively.

Other reasons for failure have been morphological changes, for example Hirtshals Harbour in Denmark where the extension of the main breakwater in 1970-71 led to a starvation of sediment to the eastern breakwater, which failed in 1973 (Sorensen and Jensen, 1986). Steep rocky foreshores can also be problem and during a storm in 1979, the entire seaward armour layer of Dolosse sustained severe damage at Azzawiya Refinery in Libya. Steep slopes seawards of a shelf on which a structure may be built can influence the wave characteristics at the structure. Hydraulic model tests carried out by Allsop et al (2002) using a 1 in 2 offshore slope, indicated that violent breaking occurs where the height of the offshore wave exceeds the shelf depth and that violent plunging occurred relatively close to the edge of the shelf. Neither the Goda nor Weggel methods of calculation were able to predict the continuation of breaking on the shelf.

The presence of a rock bed can present other problems, for example at Llanca in Spain with Accropode units where, while some were anchored to the rock bed, others were not and moved during a major storm. This movement caused a failure to the outer section of the breakwater. In Port Arzew El Djedid, the main breakwater of Tetrapods suffered severe damage in December 1999 (Abdelbaki and Jensen, 1983). A diver survey showed that just below water level up to 80% of the Tetrapods in both layers were broken in certain sections. Model testing showed that this was mainly due to settlement and compaction within the structure, with settlement values in the order of 2m, within a range of 1 to 4m. In this case the Tetrapods weighed 48t.

Over the years much attention has been paid to the hydraulics stability of the armour layer and crest, with limited attention to geotechnical stability, even though it could be a major contributor to failure. Consequently, the Zeebrugge breakwaters design studies looked at the shallow slip surfaces, which develop in the breakwater core and the deep slip surfaces reaching the soil layers beneath the structure. These studies indicate that within a wave period, the minimum factor of safety occurs immediately after the maximum run up ie when the seepage forces are at their maximum. This is in contradiction to the wide spread view that the wave trough in front of the breakwater slope is the most unsafe condition, which is only true for shallow slip surfaces.

While rubble mounds are inherently safe structures in terms of seismicity, in Patras in Greece failure coincided with moderate seismic activity (Memos and Protonotarios, 1992). The reason for the failure appears to have been the ground conditions, with the breakwater being built on relevantly soft soils and following the earthquake, the crest of the breakwater had settled by over 5m. The earthquake in this case had a magnitude of 3.5 to 4.5 on the Richter scale and so was not particularly large.

Even without the impact of earthquakes, soil types and soil strengths are important, not only in controlling the flows behind a revetment (Bezwjen, 1991), but they can be liquefied by storm waves. Wave induced soil response have been examined by Lin and Jeng (2004), together with liquefaction and shear failure depths, together with the added stresses due to the mass of the breakwater.

While physical and mathematical model testing using random waves has greatly improved their reliability as design aids, their use should also include analyses of the irregularity of the waves and the storm duration. This is because the effects of wave grouping and spectral shape of irregular ocean waves, storm duration and wave/wave interaction are important parameters (Franco and Passoni, 1994). This must be considered with resonance phenomena (Ryu and Kim, 1994).

Recent Examples

The following examples come from recently published papers as well as from the author's own experience of investigating failures. In the latter case, as many of the investigations are ongoing, it has not been possible to name the structure and the causes of the failures are the author's interpretation of the information he has been given and the results of his investigations into the failures. The lessons that may be re-learned following these failures are included after their description.

The recently constructed 457m long offshore breakwater which suffered damage during construction, especially its north head, was Ventura Harbour, California. It was armoured

with two layers of 13t rock, but its head was repaired with a single layer of 18t Core-loc units. The construction/repair damage was partly because, as an offshore breakwater, it was built using floating plant even during winter. Not surprisingly, it suffered damage during its construction (Mesa, 2004), as for example during construction wave heights were consistently in the range of 1.8m to 2.4m. While the design packing density coefficient (number/area) of the Core-loc armour layer, which varies with slope angle, was 0.58, the as-built density on the 1 in 2 slope was lower i.e. about 0.52, reflecting the difficulties when building. On about 31 January 2000, it was severely damaged by a storm with an H_s of 3.7m, especially at the breakwater head. The damage here involved a complete displacement of the armour layer, underlayer and core. The zone of damage stretched from the seaward side to the rear side of the centre line of the breakwater close to the most vulnerable zone for the roundheads.

Lessons: Proper consideration should have been given to the whole of the design and construction process i.e. including hydraulic model testing and site supervision. If not, then even breakwaters, protected by the most up to date units can still fail.

Some 140 breakwaters armoured with Accropode units have been built worldwide and they too have been damaged. This is despite the fact that they are considered to be robust units, more so than say the Dolosse or Tetrapod. In the case of the Club Mykonas breakwaters, South Africa (Bartels et al, 2003), it was perhaps the robustness of the units that limited the damage and allowed the 100m long failed area to be repaired without a wholesale rebuild.

This breakwater was constructed in 1988 and was protected by 9.6t Accropodes (see Figure 1). However, by 1991 significant settlements to the armour layer was apparent (up to 3.5m), which included a movement away from the wave wall behind them (up to 1.5m). Surveys revealed that a 3.5m deep erosion hole had developed in 5m of water near to the landward end of the breakwater. The cross-sections for the breakwater shows the 9.5t Accropodes on a 1 in 1.33 slope (3 to 4) with the toe of the units resting on a 0.5m thick filter layer of 6mm to 300mm rock on the sand bed.



Photograph 1 The Jalali Dolosse revetment after the storm

The breakwater was subjected to major storms both during construction and prior to failure and, in post-repair period, the wave conditions have been more severe. It was concluded by Bartels et al (2003), that the steepness of the Accropode slope and its associated turbulence contributed to the erosion of the sandy seabed. However, the good interlock of the units prevents a catastrophic failure. The provision of a 6m wide, 2t to 4t rock toe in 1998 has proved effective in preventing further settlement. What is unclear, however, is whether or not the placing of these 9.6t units on a 0.5m thick layer of 6mm to 300mm rock, contributed to the failure, given that the breakwater was exposed to quite significant wave action during and after construction.

Lessons: Steep placed rock slopes reflect wave energy, especially if their voids ratio is low. The reflected energy will produce turbulence and possible bed erosion. The toe or filter rock must be properly sized and dimensioned, reflecting the soils beneath and their exposure during and after construction.

A further example of the instability of a breakwater protected by 11.9t and 16.7t Accropode units occurred in November 1999, shortly after completion. In this case the rubble mound breakwater, almost 300m long and protected with large armour rock had been destroyed in 1997, mainly near its head. Subsequently it had to be upgraded with Accropode units, the work taking some 7 months, with some damage seen early during construction.

The design drawings show the filter layer consisting of 0.01 to 0.25t rock on a sandy seabed, extending beyond a toe of 2t to 3t rock and 2t to 4t, supporting the 11.9t and 16.7t Accropode slopes, respectively. The width of the filter varied from 6m to 8m from the toe of the slopes for the 11.9t and 16.7t units. The design was not model tested.

The November storm, while significant, was not particularly severe, having a return period of between 1 in 10 and 1 in 25 years (the breakwater was designed to withstand a 1 in 100 year storm). The damage reported one month after the failure showed that there were 14 broken units and, in addition, 3 had been pulled off the slope and were lying on the seabed, with most breakages being to the 11.7t units. The report also indicates that, in the main area of damage, the toe had slumped by as much as 2m, which led to a 1m slump of the top of the Accropode slope.

The most likely cause for the failure appears to have been the loss of the filter layer at the toe, but whether or not this triggered the failure of the rock at the toe of the Accropode slope or vice versa, is unclear. The rock in the filter layer appears to have been too small (0.01 to 0.25t) and the layer too thin (0.5m) and the toe rock may have been too small.

Lessons: The toe and filter rock must be properly sized and dimensioned, reflecting their exposure during and after construction.

Not far away the failure of another Accropode breakwater appears to have been due to the fact that the client would not pay for the toe units to be anchored onto the rock seabed. Where he did pay, the breakwater was stable, where he did not, it failed.

Lessons: The toe of any rubble mound structure on a rock bed must be adequately anchored (say in a toe trench) and the armour properly interlocked.

Dolosse units have always been considered to be relatively fragile and it is important to choose the appropriate waist ratio and placing density. The use of steel reinforcement is not recommended for any armour unit. In the case of a Dolosse revetment at Jalali in Oman, the structure was almost totally destroyed in August 1983. The storm waves were generated by a cyclone which produced an H_s of some 3.8m offshore, with the peak of the storm being coincident with high water. The storm, however, was only unusual in that it was the first recorded cyclone to have occurred during the month of August. The significant wave height on the revetment was estimated to be about 3.5m and, after run up, could have reached some 1 to 2m above the height of the revetment. Indeed, significant flooding occurred with rocks and broken Dolosse being deposited on the adjacent helipads, with services and cables being washed away.

The damage to the revetment, as can be seen on Photograph 1, was severe and, at the end of the storm, no part of the revetment consisting of two layers of 6t Dolosse units was left undamaged. Again, the storm does not appear to have particularly severe, with a predicted return period of 1 in 25 years, with the structure apparently designed to withstand a 1 in 100 storm.

The reason for the failure was not specifically because of the fragility of the Dolosse units, but the fact that the low crest level of the revetment allowed considerable overtopping, dislodging the Dolosse at the top of the slope, which were not retained in any way. The fact that the Dolosse were sitting on a toe of filter rock weighing on average 1.3kg, undoubtedly also contributed to the failure. For the reconstruction, heavy blocks were placed at the top of the slope and the revetment protected with Shed units, which sat on a concrete toe beam where rock was exposed and a piled concrete beam where the bed was sand. The latter was further protected with a rock armour apron. The heavy concrete blocks placed at the top of the slope were provided with large vents; some 200mm in diameter at 2m centres to resist the uplift forces (see the discussion by Read, 1985).

Lessons: Damage due to overtopping of any rubble mound structure can be severe and can lead to damage to the front face. Reinforcing concrete armour units is not recommended and it is essential to properly design the toe of any structure.

The failure of a revetment on newly reclaimed land in an estuary took place at high water. The form of the failure was 'S' shaped, with material being eroded at the high water level, exposing the geotextile beneath, being deposited as a low berm on the slope below. The reason for the failure was that, even though the designer had carried out extensive studies of potentially significant wave energy entering the estuary and impact of the banks etc, he had neglected to establish the wave heights generated within the estuary at high water. These latter waves were more severe than the much larger deep water waves by the time they had been diffracted and refracted into the site. The chosen stone size was therefore too small.

Lessons: Consideration must be given to the generation and impact of waves from all wind wave directions in enclosed areas which must take into account variations in water levels and any complex geometry and bathymetry, which can produce focussing.

The failure of scour protection around an 8m diameter offshore outfall diffuser rising some 3m above the bed in a water depth of about 8m below ACD, can also be spectacular, given the right combination of tidal currents and waves (see Figure 2). This particular structure was protected by a 'ring' of scour rock placed as a cone around the structure, with a total placed width of 8m, equivalent to the diameter of the structure ie an inner 3.25m wide ring of 300kg to 1000kg rock, a middle 1.75m ring of 60kg to 300kg rock and an outer 3m ring of filter rock, with each layer resting on the other and the filter layer on the sand bed. It was in theory designed to resist the 1 in 100 year design combination of waves and currents, but was virtually destroyed by a storm which it is believed occurred in November/December 2000. The surface of the apron was lowered by 2m and a pile of rock some 2m high was left downstream of the circular scour hole, which was about twice the diameter of the outfall (see Figure 2).

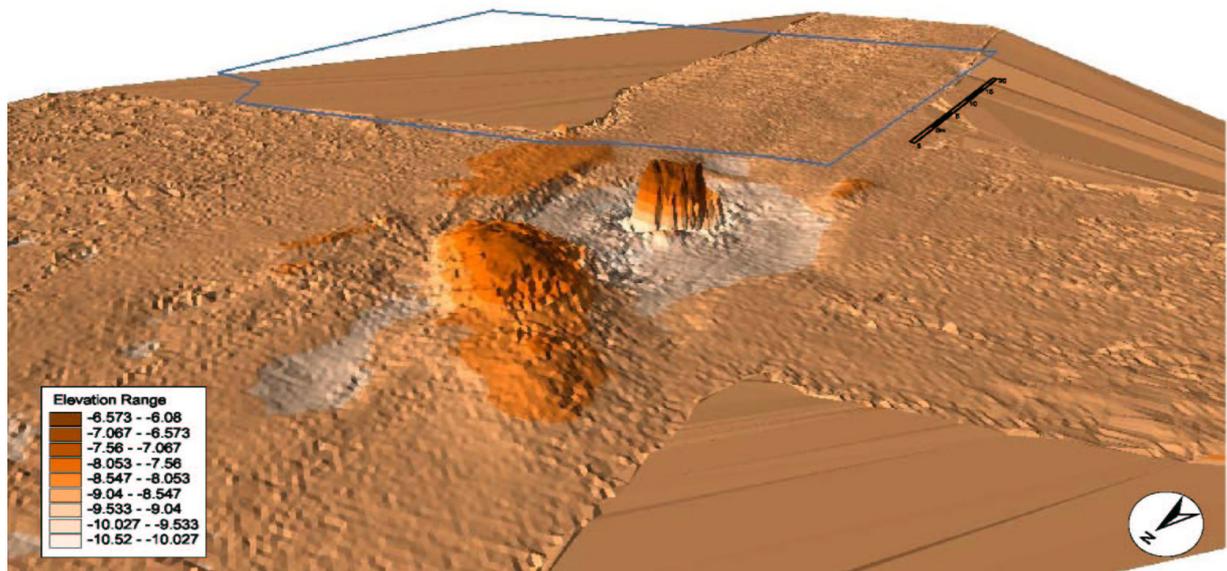


Figure 2 Scour and accretion around the outfall

The scour hole created was elliptical with the erosion being greater on the seaward side than the landward side. It should be noted that there were four ports on the seaward side discharging water at 3m/s at an angle of 20° to the horizontal. There was also accretion on the seaward side but, away from the structure, bed levels changed little.

The analysis of the design indicated that the amplification factor for the currents could be as much as 10, which would have been capable of moving the main armour rock. The area of erosion seen was very much as predicted and, despite the increase in the size of the filter rock during construction, it is unlikely that it would have remained stable. It would appear from subsequent model tests that it was the loss of scour rock both from the bed and through the underlayer rock that triggered the failure, along with the passage of sand through the filter rock. The increased turbulence from the outfall jets also appears to have had an impact as would any problems and difficulties during construction.

Model testing of the storm conditions thought to have lead to the failure also produced damage, but not on the scale seen in nature. The revised design, which was more traditional

with the armour rock extending for about 1.5 times diameter of the outfall, was also tested on the model. Geotextile was also provided.

Lessons: Consideration must be given in the design to likely difficulties there will be in constructing the design and, if the exposed filter rock can be considered ‘sacrificial’, what will happen to the structure when it is eroded. Physical modelling of complex structures should be carried out.

An example of the use of flume and 3D models to assist the design was for Koeberg intake basin in South Africa in 1975. These tests showed that the proposed rock toe for the Dolosse slope was inadequate and it was replaced by a toe of Dolosse units (Loewy et al, 1976). Partly because the design wave of 6m was predicted to occur annually at the head of the breakwaters and the main breakwater had an impermeable core whose impact was apparent in the physical model testing, a K_D of 12 was chosen, which was half that recommended at the time. Annual surveys have shown the breakwater to be in good condition some 30 years after construction in a very harsh wave climate.

Lessons: 2D and 3D physical modelling was able to illustrate the reflective properties of the impermeable core and show the frailty of the original toe design.

The offshore breakwater for the revamped Beirut International Airport was model tested post-contract, as a contractual obligation of the successful contractor. The breakwater, which was in theory designed to overtop, failed dramatically due to overtopping during the testing of the 1 in 100 year design condition, although there was much subsequent discussion as to the adequacy of the original model testing. The redesign took almost a year to complete and contractual positions of the Parties took a further seven years before it was established in Arbitration. Legal cases can be expensive and time consuming (Maddrell and Gowan, 2001).

Lessons: Conduct physical modelling before letting the contract and, if overtopping is a design criterion, adequately armour the backface.

The sensitivity of soil conditions does not simply relate to areas of seismic activity and Dinardo (1991) described the failure of the Rhu marina in 1983/84, during its construction, the last failure being a major collapse in December 1983. Subsequent investigations showed uniform soft silty clays, some 20 to 25m thick beneath the structure, which was sited in water depths varying from 8m to 14m. For its construction, the rock was dropped directly through the water column onto the filter fabric weighed and strengthened with rebar and Dinardo attributed the falling velocities of the rock and the weight of the structure to the failure of the soils whose shear strengths ranged from 2 to 9kN/m². He found that the spread of the basal rock was twice that of the design width of the original structure, which created 1 in 5 slopes.

Lessons: Un-consolidate fine bed materials will consolidate and move laterally under loading and filters, when continuous, will only prevent the material moving vertically.

A recurring feature of failed structures appears to be the fact that they were damaged during their construction. In the case of the outer lay-by sea defences at Shoreham (Maddrell and Vaughan, 1991), a spur groyne was created early during the Contract to protect the root of the existing breakwater and the Contractor elected to have his access road in this area. Construction commenced in May 1989 and was due to be completed one year later. By September 1989, the Contractor had only just started building the Seabee revetment and it

was damaged twice that month, by which time action had been taken to protect the open face of the Seabee slope with armour rock and by pinning the units together. The 28/29 October storm destroyed approximately 50% of the completed revetment face and a decision was made to suspend the work until the next spring. When it did restart, strong points, comprising rock filled gabions were introduced at 75m centres to protect the core and underlayer and the Seabee units were anchored back into the strong points. The 500m long revetment was finally completed in July 1990 and, prior to the placing of the gravel for the recreational beach, the units did suffer some surface damage from a combination of wave action and the gravel it contained.

Lessons: Be aware of the likely wave climate changes and incorporate strong points, especially for armour units whose effectiveness relies almost entirely on the units around it. Consider the abrasion due to shingle in the design life of the structure.

Damage resulting from the construction of marine structures can also be quite severe, say by causing significant downdrift erosion. However, quite often the design does not fulfil all its functions. An example of this is a marina and entrance, which was designed to be low maintenance. However, no real studies were carried out regarding the alongshore and on-offshore transport, despite the fact that the dredged depth in the unprotected section of the entrance channel was in excess of 2m. Surveys prior to construction also showed there had been significant accretion in the littoral zone. This led to a cheaper breakwater structure (shallower bed depth), but no analysis of this change on channel or marina accretion rates appears to have been made at any time. The channel has never been dredged to its design depth and Client has subsequently had to carry out regular maintenance dredging and has also had to obtain the necessary permits for a new offshore dump area in which to dispose of the dredged spoil.

Lessons: The calculation of maintenance dredging must involve all types of material and their modes of transport must be assessed. A change in layout or bathymetry requires the recalculation of the likely accretion rates.

The sensitivities of soil to wave action and liquefaction are becoming more recognised. In the case of a long sea sewage outfall, consisting of HPDE pipes with concrete collars, a section of the pipe, which should have been buried, was found to be above seabed level close to the outfall. It was assumed that the reason for the failure was because the pipe “floated”, mainly due to ground liquefaction during storms, but this may have been enhanced by trapped air. Consequently, the soils along the whole route had to be examined and, in the end, a rock blanket had to be placed above the pipe throughout virtually its whole length.

A similar ‘floating’ of a power cable occurred in the littoral zone at Folkestone. In this case it is likely that the reflected energy in the trench cut into rock liquefied the cable and the loose backfill around it.

Lessons: Pressure changes due to wave action can cause the liquefaction of soils. This requires a detailed knowledge of the wave climate and the property of the soils.

Lessons to be re-learnt

The following are some, but not all, of the points we still seem to miss or get wrong during the design and construction processes. First and foremost there are many excellent design

codes covering aspects such as the studies required, rock requirements, the manufacture of materials e.g. concrete etc, which do not appear to have been followed in many failed structures. This paper does not have time to look at the 'miss-use' of the various codes and the following are observations drawn from the failures discussed in this paper.

Studies and Design

- i) Soils. Carry out an adequate soils investigation of the whole site, which allows the engineering properties of the soils to be established. This should include the potential for liquefaction, varying grain sizes and layering.
- ii) Morphology. Studies should include the past and likely future geomorphological changes and the likely impact of the structure on that morphology.
- iii) Marine Conditions. Depending on the type of structure and the degree of exposure, it may be essential to measure all marine conditions on site e.g. waves, rather than remotely using wind records. Joint probabilities should be used and all possible conditions must be considered.
- iv) Modelling. Mathematical models will establish changes to the design storm waves; say due to changing water depths, sheltering etc. Flume and 3D physical models may be essential to confirm stability and can be used to refine a design, thus saving money. These models can not establish the relationship between the structure and their foundation soils. While the models will be built to specification, your structure may not and this aspect must be considered.
- v) Materials. While it is usually cost effective to use local materials, their availability, say in relation to weight and shape and long term durability must be established. Good strong, dense concrete is always essential especially for armour units. Steel reinforcement should only be used if essential and not in areas where movement can open up cracks and long term durability is required.
- vi) Structure. Do not simply scale up structures or simply modify an existing design without considering all the implications. A design is only as good as the information that goes into it, as is the case for models.
- vii) F.O.S. Allow for adequate factors of safety and check against likely costs, as a safe design is not necessarily an expensive one. An overall F.O.S of 1.2 will be acceptable, providing that it has been looked at in terms of all the variables e.g. soils, wave heights, testing carried out etc, etc.
- viii) Consider the likely ease or difficulties involved in the construction and factor them into the design.

Contract

- ix) Materials. Ensure that the design materials are properly described and are appropriate.
- x) Ascribe risks to those in the best position to deal with them.
- xi) Consider very carefully any alternative designs.
- xii) Give the Tenderers all the design information e.g. the analyses of waves, all soils data etc.
- xiii) Make the requirements clear, at all times

Construction

- xiv) Consider very carefully how the structure will be built and the conditions under which it will be built.

- xv) If damage occurs, assess the reasons for the damage, ensure it is properly repaired and thoroughly check the repair.
- xvi) Consider how best to protect the structure during the construction process.
- xvii) Ensure that the work is adequately supervised at all times, preferably by the designer.
- xviii) Take action immediately if placing densities are not being achieved.

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