RAISING THE ALARM:
HOW THINGS GO WRONG

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Way back in 1979 the author was privileged to provide a concept design to replace the 1930s vertical reinforced concrete retaining wall that had collapsed in the storms on 1974. This design was developed during a period of economic constraint. And ventured the use of the newly developed design rules for Seabees and Maccaferrri gabions when subject to wave action. The structure was intended to be built by direct labour of the local authority. While attending ICCE2022 in Sydney, the author visited the seawall at Cronulla which was now of a different form, showing signs of distress and wondered how things turned out that way. The abstract was accepted for aural presentation in Rome, and this paper represents a more generalized view of how things have gone wrong on various projects the author has been involved with since 1964. An attempt perhaps to get the reader to learn from the mistakes of others.

Keywords: mistakes, inspection, understanding, SQEP, communication, decimal point.

INTRODUCTION

This paper was originally concerned with the design, construction and inspection of sloping revetments and seawalls and the problems of independent advisors and testing laboratories with ensuring that critical details described in advice, correspondence, test reports and the like are properly incorporated into the design documents and into the built structure, Inspection and maintenance teams need to understand their structure and inspect and maintain it accordingly. All of this require adequate budgets and powers.

During the development of the presentation, it became apparent that the problems encountered bore more widely across the gamut of civil engineering, and so the scope became an examination of 'How things go Wrong' and means of prevention, chiefly competent inspection and powers of enforcement.

Examples from a spectrum of maritime structures are cited from the author's experience, all affected by tides, and of which all illustrate one or more of the pertinent causes.

Recent experience of attempts to *Raise the Alarm* for a distant society to the damage sustained during and after a stormy year, and the subsequent discovery of critical faults will also be described.

HOW THINGS GO WRONG

Over a sixty year career a variety of problems were encountered, which all amounted to mis-communication of one form or another. In some cases, remediation was achieved, in others the changes led to eventual failure or damage to third parties.

If you employ human beings, you must expect mistakes to occur. So, it is important to have effective procedures to catch them before they hurt. People must be encouraged to identify and own up to their own mistakes as well as those of others. The aviation industry leads us all by example.

In broad terms, mistakes may occur due to any or many of the following:

1. Lack of communication
2. Misinterpretation by evolving design teams,
3. Inadequate management of interfaces
4. Lack of understanding of the processes involved
5. Incompetent or lack of Design Change Control procedures - design changes not re-tested or referred back to originating designer or adviser, including
6. The wider implications of changes
7. Wilful changes by contractors without reference to the design authority or expert advisor,
8. Failure of inspection during to identify or note deviations from the design drawings and specification.
9. Lack of testing during construction
10. Deliberate omission, theft and fraud.

CASE HISTORIES

The following case histories contain one or more examples of these mistakes. The reader is invited to consider which were most relevant

Eden Woodchip Export Berth [1971-2]

In the late 1960s a woodchip plant was constructed on the South side of Twofold Bay. The out-loader berth was designed to comprise a set of rock filled sheet pile rock filled caissons, both as berthing dolphins and the loading tower base, embedded some 6m into the seabed. The contractor was unable to complete any of the circular caissons due to the exposed nature of the site, with 5-6m swells occurring every few

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weeks. The clapotis on the incomplete circles of sheet-piles collapsed three dolphins before the work was abandoned, leaving alone jury rigged temporary outloader to service the plant.

A subsequent redesign was developed by the Japanese co-owner, with the dolphin and out-loader deck supported on 1m dia. tubular steel piles. These piles were required to have an embedment of some 16m below the seabed, some 16m below sea level, with the working decks 8m above sea level.

The 1m piles were exported as deck cargo from Japan, sealed at both ends for delivery at sea in the middle of Twofold Bay, to be stored on the beach for later use at the berth location 400m from shore. marked T and B at opposite ends. No detailed pile drawings were supplied.

The author was appointed as Resident Engineer for the construction of the berthing dolphins and tower base by the Sydney based consultants employed as liaison between the owners (Harris-Daishowa), the Berth Designer (Obayashi-Gumi) and the Contractor (Thiess Bros).

Our first review showed that the original site investigation extended less than 0.3m below the tips of the new piles, so a new investigation was commissioned, which showed a peat lens under the line of the new berth; an old river channel form 12,000 years ago. A proposal to extend the piles to safe end bearing below the peat was accepted by Obayashi-Gumi, but using 0.5m tubular piles rather than heavy section H-piles. (This caused problems in construction in dewatering the piles and concreting he annulus).

The approved method statement was to tow a pile out form shore to the barge mounted piling rig lift the pile clear of the sea and cut off the base seal, to allow for the pin pile to be driven down later. Imagine the surprise when it became apparent that the end of the pile to be driven by the hammer was cross braced, and was marked B for Butt and T for Tip. Luckily, we had been provided with spare piles so the pile was taken back to shore and replaced. A problem overcome, but a day lost.

Other problems came and went and eventually the tower base was handed over for the Tower fabricator to erect the outloader. This contract had been let by the Facility Manager, using a simpler one page purchase order, with all the contract provisions on the reverse.

And we were then appointed to supervise the erection and commissioning of the new loading tower. That's when we discovered that the steel fabricator had not been deck level had been raised and the conveyor link had now allowance for the movement of the new loading deck under wave action. A major series of design changes on the run. The work was commissioned on time on Nov 5th 1972 – Figure 1

Figure 1. Eden Woodchip Berth 1972-2015

Figure 1. After the 2015 storm & 17m waves

Credit to the late Frank Dahl and Giuseppe Laginestra for help sorting out the mess and chelping to commission the facility on the original programme.

Some 45years later the approach jetty and conveyor were washed out (Figure 2), but the tower survived the high waves (17m recorded at the waverider buoy before it was lost). So, the raise in deck elevation was the result of a proper assessment, but just not communicated properly. Nor did anyone, author include, raise the question that if the loading deck level had to be raised, what about the lol level approach jetty: no over-arching peer review of the whole installation.

Wamberal 1978-2020:

A prototype compound structure, comprising a buried gabion scour apron and compound gabion and blockwork revetment was built between rock walls founded on top of the beach (Figure 3). It was a prototype structure constructed by voluntary labour, using design rules were being promulgated for both Seabee and Maccaferri Gabions as coastal revetments.

Intervention by neighbouring residents led to a limit on excavation depth, resulting failure to reach original scour horizon. The structure lasted over 40 years before this weakness was discovered by the sea. The structure failed by sliding due to deep scour and probable slumping of wet sand below. The two lateral gabions wings, designed to cater for erosion on each side of the revetment lasted well. Provision of a mesh link to the crest along the whole width may well have saved the structure from this failure mode .

Figure 3. June 2016: a rare survivor of the storm: performed as expected, but failed by scour & sliding in 2020.

But a 25m wide coastal revetment in the middle of a 3km beach has no long term future. This structure outlived its 25 expected life by 17 years, waiting in hope for implementation of a general scheme that never happened.

Abbot Point 1981:

An armoured mole supporting a major export conveyor. The structure commenced on exposed rock and extended over a sandy bed. Two toe designs were specified, the first to be built over rock, the second to be in an excavated trench. The contractor forgot or ignored this, and bult the rock-bed detail over the sandy bed and it was not picked up by the site inspectors. This was not noticed until an independent inspector, visiting the site to discuss aspects of the armour unit production noticed and reported the discrepancy. This was retested following discovery and a revised perched toe agreed.

Cronulla 1979:

In the late 1930s a vertical reinforced concrete seawall was built to protect the Prince Street dune in North Cronulla, NSW from storm erosion (Figure 4), which erosion was exacerbated in later years

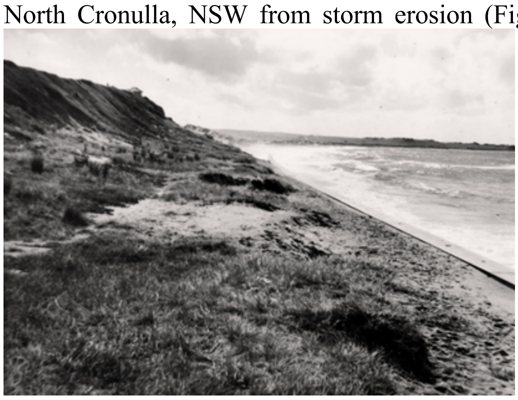

Figure 4. North Cronulla seawall in 1949

Figure 5. Immediately after the storm, 1974

by sand mining in the lee of the dunes near Kurnell.

This loss of sand from the dunes of Bate Bay increased markedly in the postwar years. In 1974 a severe storm demolished this wall, resulting significant erosion of the dune face. A programme of studies was initiated to reduce the loss of sand to the North, and how to prevent further recession of the dune.

By 1979 the rate of recession was deemed critical (Figure 6) and a range of options to stabilise the dune and secure were considered. The proposal was put forward to excavate the dune to renourish the beach and construct a seawall well back from the beach was the chosen solution.

Figure 6. Continuing recession of the dune and burial of the remains. Unfortunately, not surveyed and soon lost to view.

The use of a flexible scour apron was considered to be significantly cheaper than a sheet pile toe, and avoided the problems with the hidden remains of the old wall. The concept was to locate the scour apron below the level of 5% AER risk of scour of exposure (Figure 7) to the seawall design wave climate of 3.6m 12 second waves

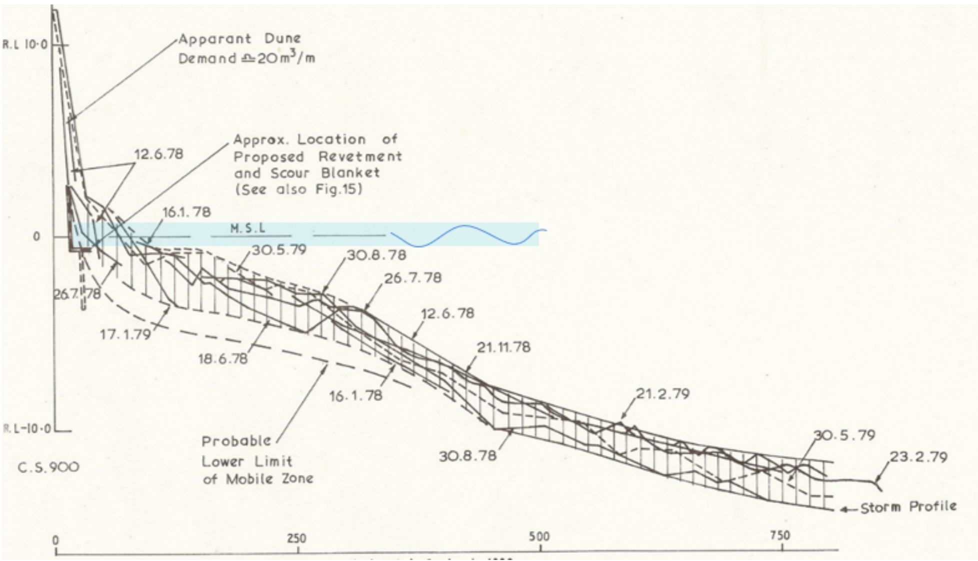

Figure 7. Location of seawall in beach erosion hysteresis

Under cost pressures from the client, the crest level was lowered, and more importantly, the scour apron became a sloping revetment, with transition raised to High Water mark, so becoming a revetment in its own right, and the whole profile was moved seaward, becoming subject to frequent exposure to wave action (Figure 8). The design of this lower gabion revetment was not checked against the design criteria established in 1979.

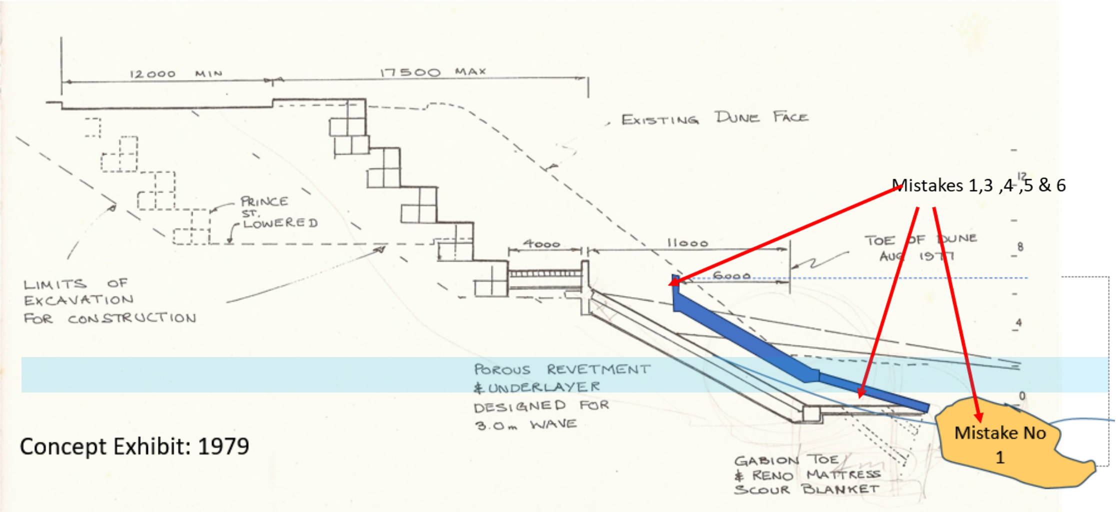

Figure 8. Evolution of Design from Concept to Construction

Figure 9. Early realisation of the concept with gabion scour apron supposed to be al low water mark. –

Figure 10. 1986 – Damage in first months of service

Figure 11 2007 Damage at the downdrift end

The site was not cleared of waste. In its first year of operation, the whole scour apron/lower revetment was subjected to significant wave action, and damaged by uncleared waste being thrown up on to the gabions (Figure 10). The gabion structure survived some 25 years before sustain severe storm damage caused a replacement of the gabion revetment (Figure 11). This compares with the design rules promulgated in the Gabion Report (Brown 1979), which set a limit of 3m waves for gabion revetments, for very infrequent exposure.

Subsequent restoration involved removal of the gabion revetment, and installation of a contiguous bored pile retaining wall, with a capping beam to support the Seabee revetment, all built within a sheet-pile coffer-dam (Figure 12).

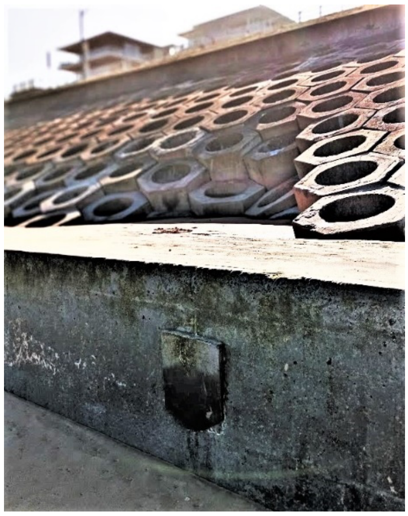

Figure 12. Contiguous pile wall after completion: note the pile gaps **Figure 13. Loss of fill through gaps leads to loss of fill below the filter cloth.**

For such wall to retain the sand fill the piles have to be contiguous, if not secant (Figure 13). Close inspection of this Figure 13 shows a degree of irregularity in the placement of the piles. Competent inspection by the designer prior to construction of the capping beam would have allowed a reappraisal of the effectiveness of the piles as constructed. The presence of 10cm gaps between occasional piles would surely been not been corrected before placing the capping beam. But the designer was not briefed to inspect the works, nor to brief those who did, who seemed to have considered the piles as solely supporting the toe beam.

Shoreham 1989:

A Seabee revetment was designed by a reputable experienced consultant, intending to utilise coarse crushed concrete as the underlayer over native shingle as bulk fill. Problems of contamination delayed production of crushed concrete and alternative rock underlayer was specified, but deliveries were inadequate to meet the program (Figure 14).

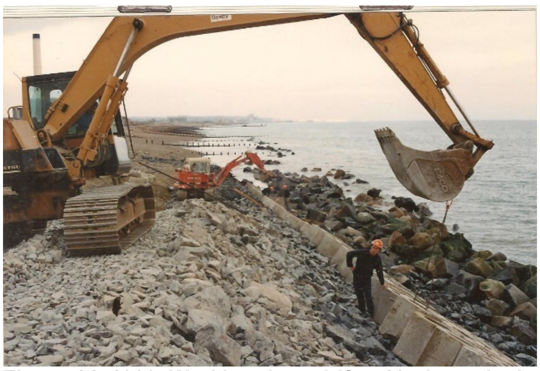

Figure 14. 1989 Working downdrift 5-10min cycle time. Figure 15. 1990 Working updrift, 22second cycle time

The vulnerability of the structure during the construction phase to overtopping by storms, and the washing out of the shingle bulk fill (placed at slope of 1:1.5, far steeper than the natural angle of a shingle beach) was not appreciated by the construction team, with inevitable consequences.

Following a construction sequence review over the following winter, a revised construction programme was developed, which saw the whole revetment completed in less time that had been spent in the previous year's attempt (Figure 15). Critical points of the review were the protection of the shingle fill during the construction phase. The was achieved by ensuring continuity of supply of underlayer materials, working quickly away from one end towards the weather, and establish gabion bastions at intervals to provide fall back retention of the shingle. Due to the new efficient method of working, the 500m main revetment was completed in 13 weeks without any weather incident.

Ardglass 1991:

The existing near-vertical harbour wall was to be faced with a sloping Seabee revetment to reduce wave splash and overtopping during storms (Figure 16) The design was subject to three dimensional flume tests as the structure comprised seawall with a trailing leg, with units placed on a coarse rock underlayer sized to minimise the risk of damage during the construction period, leading to a rough surface.

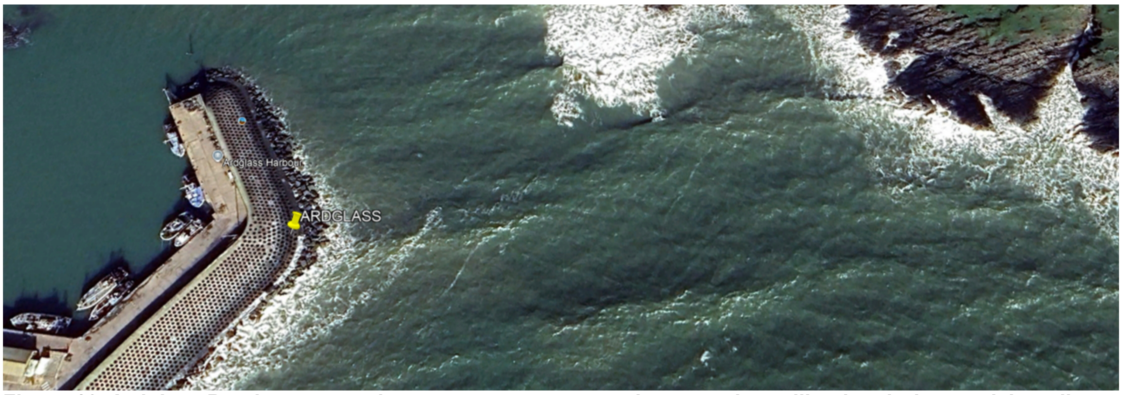

Figure 16. Ardglass Breakwater: mach-stem waves were experience on the trailing leg during model studies

Figure 17. Mach stem wave in the flume..

An open trench was left at the top of the slope, below the crest level of the existing vertical wall. Testing was carried out to 130% of nominated design conditions to investigate potential problems of Mach-stem reflections on the trailing leg (Figure 17).

No overtopping was recorded on the seaward face, extreme runup falling safely into the trench. It is probable that the importance of these details was not emphasized in the laboratory report.

During construction, the contractor chose to dress the rough underlayer to allow installation of a smooth slope. It was also decided to fill the trench entirely with mass concrete dressed up to the crest of the wall. The inevitable result of these changes came sometime later, when the storm run up was not resisted by the unusually smooth slope, and having nowhere to go, went over the crest of the wall. This crest was significantly above the harbour quay, and the overtopping water fell clear of the quay into the open hold of a fishing vessel, sinking it.

The effects that these design changes might have on the performance of the structure was not appreciated on site nor was it referred back through the design team. The problems had been considered and addressed on other projects and suitable advice could have been provided, had it been requested.

Underquay Revetment

A new berth in a shelter location, with a narrow window of exposure to the ocean had been designed as a suspended deck of a rock revetment. After the certificate of practical completion had been issued, a severe storm occurred during the maintenance period, beyond the agreed design basis event. The berth operator swept the berthing area after the storm and reported the presence of one or more large rocks that were present, and were impinging on the safe berthing draft.

A strong protest was received and the Author was despatched to undertake an inspection and review. The construction sequence had been to drive the piles, then advance the fill and construct the armoured revetment followed by the suspended deck, retaining wall and laydown areas. The actual armour layer had been measured by weighbridge dockets – no profile surveys could be found in the records. And no final inspection had been carried out before the end of the Maintenance Period, so the contractor was free of defects responsibility. No perched rocks were revealed by a sidescan survey along the completed berth face. A strong protest was received and the author was despatched to undertake an inspection and review the situation.

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Back in the office it was found that the design conditions agreed with the client at the start of the project were those of a time-limited exposure to a Design Basis Event (DBE) of a ‘1 in 50 year storm’, aka 0.2% AER. Wind records showed that the actual event had exceeded the ‘ 1 in 75 year’ limit and was beyond the agreed design basis. So, what had gone wrong?

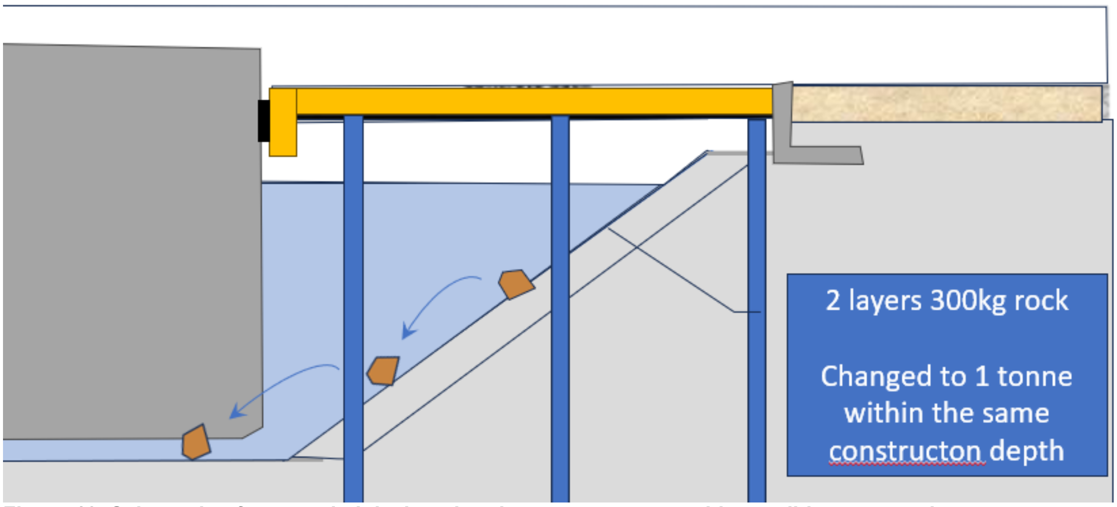

Figure 18. Schematic of suspended deck and underquay revetment with possible errant rock

Further investigation found that the original designer, an experienced maritime structures engineer, with decades of experience of piles, berths, quays and the like, but not of revetment design, had undertaken a fair assessment of the wave conditions at the berth, and had designed the revetment in accordance with the USACE Shore Protection Manual, to provide a double layer of 300kg rock. In his final review, he referred to a Norwegian manual on Harbour Engineering which advised never to use less than 1 tonne rock in underquay revetments. He decided to adopt this advised the (trainee) draftsman to change the revetment to use 1 tonne rock. The draftsman changed the note, but not the drawing. This mistake was never picked up, as the rock volume, or total mass, didn't change, and placement was by measured delivery at the weighbridge. The difference between UK and Norwegian sites is that the sea does not free off the UK coasts].

It was considered that it was quite possible that a single layer of 1 tonne rock would actually have the capacity to resist the DBE. These findings were presented to the senior partner of the consultancy with the recommendation to contact the P.I. insurers and discuss the options. The insurers fully supported the proposal to submit the documentary report to the client, with the offer to test the structure at the country's leading hydraulic laboratory at the consultancy's expense. If the structure failed at or below the DBE, the consultancy would accept full responsibility.

A suitable brief was prepared and accepted by the client, and tests commissioned at the laboratory to extend up to and beyond the 1 in 75 year event.

Some weeks later, the laboratory called to say that they could not place two layers of rock within the profile as drawn. The answer was, good, that is the point of the tests: you are in the same position as the contractor. Please continue with 1 tonne rocks within the profile.

The subsequent test continued up to and beyond the DBE, and the client said enough. But was not satisfied.

It is probable that the laboratory made a much better single layer than the contractor, but there was no denying that the storm event exceeded the agreed design basis. This reviewer was left writing strong recommendations about design approval, design change management, site supervision and inspection. And it did not help that the senior partner, a mechanical engineer, was not SQEP to review work of this kind.

The consultancy experienced a downturn in business with this client.

Blackwall Yard

During the re-development of London Docklands in the late 1980s, the London Docklands Development Corporation (LDDC) realised that there were very fer historic structures left. During work on the redevelopment of the Blackwall Yard site, which contained two drydocks, it was determined that the smaller 1803 dock should be retained for posterity, whilst the larger western dock would be sealed to allow construction of a new high-tech telecoms office block within the dock.

With no surviving drawings, there was little information about the dock, except the alignment of the dock walls, and that it was tidal and partially filled with mud and silt from the Thames. This was deemed a contaminated waste and had to be removed (Figure 19).

The consensus of opinion was that the dock was entirely brick in the form of an inverted arch. The LDDC and the architect involved wanted the original brickwork to be retained in its entirety using soil anchors, and the scheme objective was to rehabilitate the dock as a working Dry Dock, and not a wet-dock or riverside pond. The existing dock gate, a floating caisson, was unserviceable and would need to be replaced. But the probable depth of the dock was estimated from the level of the dock gate sill on the river side.

But first, the site investigation to determine the nature of the structure. The advising consultant designed a series of large diameter (c 500mm) tubular props to be erected above the silt to stabilise the walls and allow adequate working height beneath. These were quite expensive, and the deputy project manager effected a 5% saving by having these replaced with a series of scaffold trusses, which were about 2.4m between chords, but with posts and diagonals going below. This cost time later.

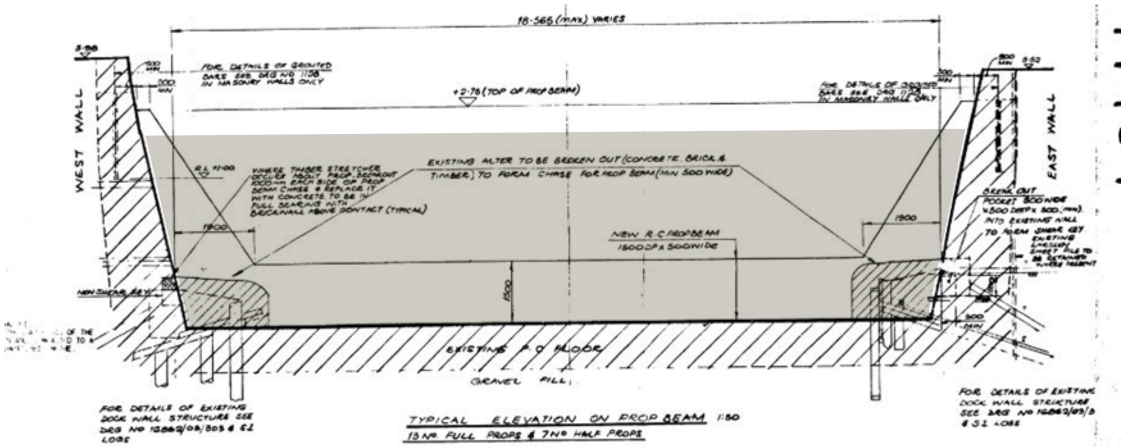

Figure 19. Blackwall Yard 1803 graving dock cross-section, full of silt to the level shown by shading at commencement.

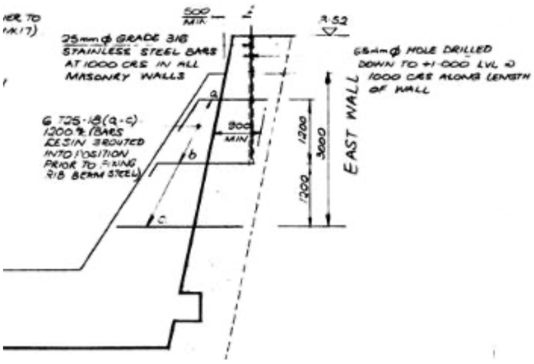

Figure 20. Detail of anchors

Figure 21. Future plans for development

To effect removal of the silt, a temporary seal gate was erected in front of the caisson, bearing on the existing entrance structure. On removal of the silt, it was found that the dock walls were found on a sloping timber deck supported by triple-pile timber bents, with a mass concrete dock floor set below this, between the ancient timber sheet piling bents retaining the soil between the bents. Trial pits were also opened to determine the nature of the floor construction. It was found that in places the timber deck had decayed, and the walls were arching from bent to bent. A new cellular cofferdam was built out into the river to provide room for the construction of a new entrance for the dock,

All work let on the project was let as fixed price lump sums, and no contractor would be found who would undertake such work entirely at their risk. The eventual solution was to form reinforced concrete ribs to resist the soil pressure when the dock was empty, and with a shear key and epoxy anchors to support the wall against further decay and against the possibility of subsequent excavation of the retained soil when the dock was full (Figure 20). The anchors were 25mm stainless steel bars to be installed in 900mm predrilled holes, with 3 pairs per rib. The loads were very conservative and it was deemed that testing would be unnecessary. A great mistake.

The dock was shortened with a mass concrete retaining wall, which the architect required to be clad in matching bricks and to the same coursework as the existing walls. These were to be retained by stainless steel ties against the pressure of tidal water trapped behind the bricks. Between one evening and the next visit at the next day, a large amount of brickwork was installed where on the earlier visit there had been no brick ties. A request was made (as a Resident Engineer it would have been an Instruction) for a panel of bricks to be broken out to check. Instead, a report was received, reported made using a metal detector, showing all the ties to be in place as specified.

With the dock empty a new Caisson dock gate specification was drawn up, with asymmetric plan to meet the aesthetic needs of the project, and a two-hour open window specified during which a vessel could enter or leave the dock. The caisson would be have had to be floated clear before this and have time to be relocated after the vessel was clear and safe. There was no room for a sliding caisson gate. Nor could new roundheads be designed to support the new gate as a beam within the available footprint, but required most support from a shear key in a new piled sill.

The gate was bult at Nijmegen to be brought over under tow. A visit was arranged to occur with the fabricator before work commenced, but on arrival it was found that work was well advance with the hull of the caisson substantially complete. Reviewing the design with the fabricator, it appeared that the design criteria had been misinterpreted by the fabricator and the caisson would only float free of the sill shear key for a few minutes. A caisson had been provided with 400x400 rubbing strakes. It was found that substituting these with 400x300 stell sponson capped with a 100mm thick greenheart strake would provide the necessary additional buoyancy.

This project was run with the main contractor functioning as project manager and responsible for primary inspection. The consultant was retained as designer and technical adviser (TA), and did not have the powers of a resident engineer, whereas the architect had the power to reject any work that would affect

As with the new end wall, the new roundhead structures, which was to function as a two-way seal to both seal and retain water and prevent overturning of the caisson. On inspection of the seal bearing faces, the TA advised that reinforcement was incorrectly placed and did not intersect the shear plane as required. Nevertheless, the deputy PM proceed with the concrete work. This cast work had to be rejected and either broken out and redone, or corrected with the addition of new shear ling by drilling and inserting suitable reinforcements. Due to time pressures project team adopted the latter course, which require accurate installation of shear reinforcement using a special epoxy grout with a limited pot-life of 1 hour. The workmen were briefed accordingly, but walked off the job at their normal lunch break with the work half done, quite contrary to the agreed conditions. This was again rejected by the TA, but the Superintendent carried on installing the new shear links regardless.

While this work on the new entrance was proceeding, when the majority of the ribs had been constructed, a young engineer going down a ladder to the dock floor grabbed one of the protruding anchors of a rib under construction, and as he reported, ‘it bent under my weight!’. He was not that heavy. Immediate inspection found that by wiggling a bar, the whole length could be removed from the wall. The only grout was a bead of epoxy mortar sealing the entrance of the hole – the rest of the bored hole bore no trace of any form of grout or mortar, let alone that specified. Records were requested for the work on the ribs already cast, but no adequate records were forthcoming.

So much for not specifying random testing of the anchors.

Despite strenuous objections from the TA, the contractor then decided to hurry things along by removing the cofferdams, before the caisson could be safely installed. Soon after, as the tide went out, the brickwork on the new end wall fell off, as there had been no brick ties installed in that suspect panel.

So now the dock had again become tidal, subject again to siltation with major rectification now required to be undertaken. The TA objections were now referred beyond the site team, to a panel comprising the building lessee, the developers, director of the main contractor and a partner of the TA consultancy. Luckily for this presentation in a matter of only a few weeks the epoxy mortar of the shear links had begun to fail and leachate was appearing around the embedments. All the complaints raised by the TA were accepted by the panel.

The contractor now had to make the following repairs under tidal working:

- Break out and repair the shear corbels of the roundheads with reinforcement n the correct location
- Implement new provisions as determined by the TA to meet the wall support criteria of the original design
- Restore the brickwork using approved ties on the new end wall.

It was a mistake by the TA designer and the consultants own internal peer review procedures to omit random testing of the rib anchors. Contractually, it also left the consultants in a reputationally vulnerable position to have agreed to the role of advisors and not certifying inspectors, relying on the contractor to act on advice to break out bad work. Good intentions cannot be relied upon. Nevertheless, oversight by personnel understanding what the design intent of each element meant that these errors were identified and corrected. Recent review suggests that these details have not been recorded on the available as built drawings (Figure 21).

Rada Tilly 2000:

In the late 1990s a back beach revetment was proposed for the southern Argentinian (Patagonian) coastal resort of Rada Tilly. The site is some 12km South of Comodoro Rivadavia, almost in the middle of the Golfo de San Jorge. The site is characterised by a flat beach with a semi-diurnal tide range of almost 6m at Spring Tides. The hinterland slopes back at almost the same gradient with a step of barely 3.8m from the toe to the crest of the proposed revetment slope.

The design criteria adopted for the site were for a Storm Surge level of +7.0m at the structure, and 2.4m waves with a period of 12.2seconds.

The proposed revetment design was quite similar to that proposed for Cronulla in 1979 (above) and Wamberal in 1997, with a porous revetment armoured with Seabees of a sufficient surcharge. However, instead of the toe being embedded in the beach to low water level (as was the contemporaneous seawall at Blackpool South Shore) this wall was to be constructed on a flatter slope (1:2.5) with the toe embedded some 0.45m below the beach profile (as surveyed) at Neap Tide level, supported by precast toe-beams and protected with a Flexmat scour mat.

The Flexmat had been developed in Argentina for river bank protection, and comprised a bespoke filter cloth fabricated with upstand loops, over which tapered rectangular blocks are cast and by which the blocks are anchored to the mat. These mats were made up in panels about 2.8m long and of legal width for road transportation.

Laboratory tests were carried out at UNSW Water Research Laboratory at Manly Vale Sydney in 1998 using a moveable bed model and there were problems of instability of the narrow scour mat, such that a geotube was recommended at the seaward end (Figure 22).

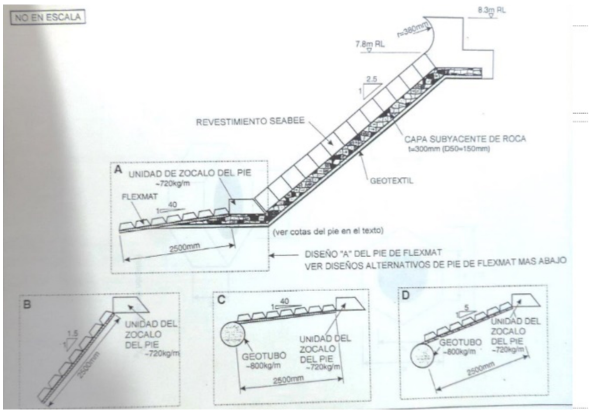

Figure 22. Examples of Scour Apron profiles tested

A video recording of the construction shows a different profile being built, with no geotube at the toe and probably to Profile A in Figure 22 (without the underlayer) rather than the profile recommended by the Laboratory (Figure 23). No documentation has been discovered to explain this discrepancy, neither contract nor as-bult drawings, or any other reports. The shape of either toe beam was such that seaward rotation and sliding is possible if the scour mat fails to retain the underlying sand.

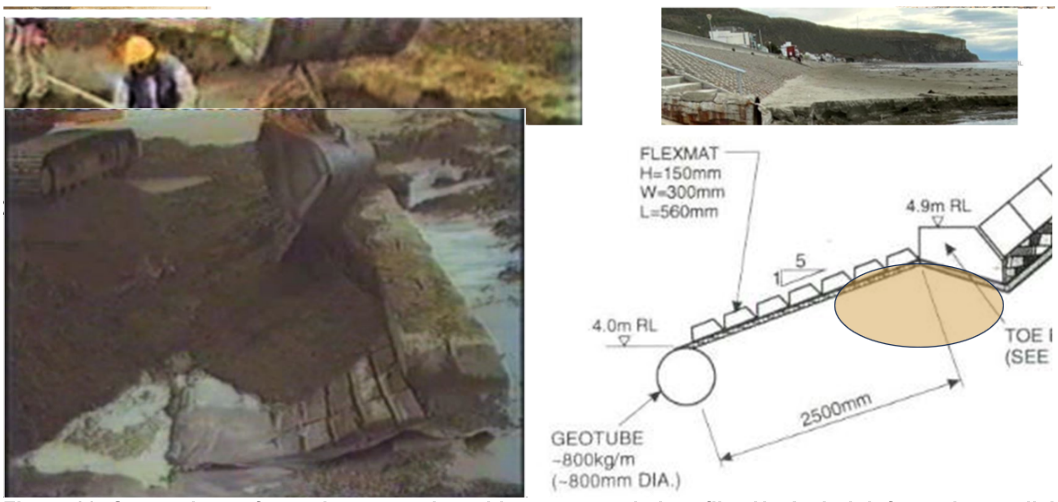

Figure 23. Comparison of actual construction with recommended profile. No As-bult information available

Presumably for reasons of extreme economy, the whole profile is set very high in the flat beach, at about neap high water levels (4.6m). The Seabee were placed with flat face down with a downslope aspect ratio of 0.96, reducing the downslope stability of the bottom row of units of the toe beam were to slide away and diminishing the arching capacity of the array. No records or as-built drawings have been located to determine the actual levels worked to on site.

Seabee production was by a vertical extrusion process, producing one unit every 2mins or less, followed by steam curing for 24 hrs. Units produced had an aspect ratio of near unt, allowing for a low mass and easy handling, but being more susceptible to downslope toppling if not adequately supported at the toe. Maximum downslope stability required that the units be placed with points pointing down the slope. This method of placement also allows rapid placement of units' side by side on the lower row, as described in the Seabee Users Manual (1983) et seq.

The author first became aware of this project in 2010, by virtue of photos on Google Earth, and monitored it on and off. In 2014 he obtained a copy of a video of the construction from the produce of the Seabee units for the project. In 2016 he became involved with work in Micronesia. An abstract on the performance of flexible scour aprons was submitted and accepted for ICCE 2020, but this conference was postponed due to the COVID19 pandemic.

In 2021 while googling Rada Tilly to update the abstract for the 2022 rerun of the conference, something in the background of a video was out of place. Some of the Seabees near the toe were out of alignment and standing high. Examination of a video-still raised serious concern After a month of searching for photos and videos on the internet, it was possible to provide a timeline of probable cause and effect.

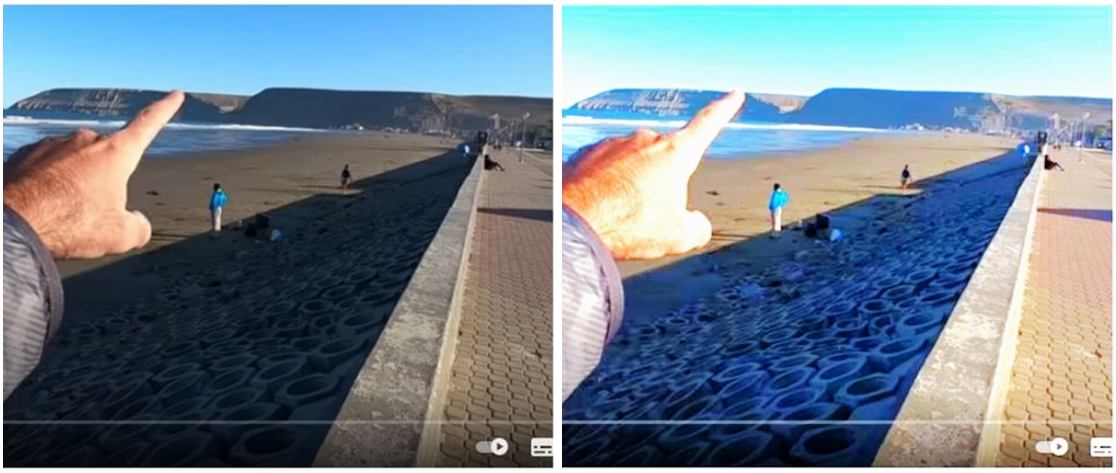

Figure 24. Video still -as seen on screen **Video still, shadows enhanced: units in loose array.**

Since then, a history of events has been compiled entirely by searching social media for photos, videos and other documents, but the only information elicited from relevant authorities was the laboratory test report.

Figure 25: Rada Tilly - one year on, 8th Dec 2018

These inquiries found that in March and April 2017 catastrophic rainfall events occurred in the area. devastating Comodoro Rivadavia and at Rada Tilly the outwash and waves lowered the beach over a significant length, exposing the whole face of the toe beam, exposing and disturbing the Flexmat scour apron.

The beach did not recover during that autumn and winter and subsequent equinoctial storms in September dislodged lengths of the toe beam which slid and rotated seawards. Overtopping similar to that reported in the model tests was observed, so conditions may have approached the design limits for the structure. The addition of a cabled-block mat or a gabion toe and scour apron over the top of the Flexmat could have stabilized the situation in the winter of 2017 at very low cost.

No records of post storm inspections have been found, and the slow degradation of the structure has been allowed to continue.

Figure 26. Rada Tilly as it was 2013 [photo H.F Garrudo]. Was the bay chord stolen to gain ground?

A local action committee of residents is now being formed using social media to seek out the construction records, work out what has actually been built and develop a restoration programme based on this knowledge, with over 30 local residents supporting Malecon Rada Tilly on Facebook.

All this is taking place in a country then blighted by inflation at an annual rate of 140%, so it is perhaps unsurprising that no action has been taken by any of the authorities, municipal, regional or government.

DISCUSSION

The above examples describe projects where a variety of mistakes have occurred, in design, project evolution, construction inspection and maintenance and the lack of either, abuse of trust and possible cases of fraud. They can fill in or add to this matrix.

Avoiding mistakes

The question is, how to avoid them. Early positive experience work as a contractor under the supervision of Kaiser Engineers in Jamaica showed the benefits of calm, rigorous inspection by inspectors with adequate powers to have the work done right.

And later on, after ten years in the nuclear related world of drydocks for submarines, increased respect grew for internal peer reviews, independent technical assessment (ITA) and regulators and inspectors with teeth, alongside a suspicion of Alliances: they rely on trust and a very vulnerable to the behaviour of the untrustworthy.

Clients must resist the temptation to separate the designer from the oversight of the works. The temptation to have staff member with general experience inspect coastal or maritime work must fail the SQRP test.

At the very least the Designers need to brief both Inspectors and Constructors on the critical elements of the work. And they should be invited to inspect the work before critical elements are covered up by later work.

Trust, or Inspection?

Together with Vitruvius and Gorbachev, the only reliable maxim must follow their advice:

Fidete, sed verificate (Vitruvius)
доверыаи, но проверыаи
doveryai, no proveryai (Gorbachev & Reagan)
Trust, but verify

In today's world, perhaps more caution is required?

'Don't Trust, but Supervise and Inspect – or else'

RECOMMENDATIONS

When do you get the chance to identify and correct a mistake? Or detect fraud or theft?

Concept and Design – the cheapest

At initial concept review: it may well cost some man hours, travel, investigations but it is the best time. Have a free radical in the room to ask awkward questions.

During design: especially when design drawings are signed off: Do you have internal peer reviews? Still only time and paper. One or more suitably qualified and experienced people (SQEP) are essential at this stage.

At final review: when signing the contract: be careful to review the videmus – your low bidder may have spotted the error and loaded his rates in the expectation of an additional windfall. Add public/professional embarrassment to paper and time.

During construction – costly,

As well as errors by the contractor, there is still time to recognise a design or concept mistake and make corrective action before the affected element is covered up, after which correction may then require demolition and removal. Costs escalate rapidly if a mistake is covered up by later work before discovery.

After Construction: - can be ruinous

It is essential to have a proper inspection before issuing the certificate of practical completion and again well before the end of the maintenance period and the work is finally accepted.

AFTER A NEAR DESIGN EVENT

After a near or beyond design basis event: did the works behave as expected. Do you understand the consequences of failure. Did you assess and undertake necessary repairs.

Final Cautions

Any the opportunity to be involved in the design and construction of a coastal structure must be carefully considered to ensure that it is properly funded, and has adequate contractual controls in place.

DON'T TRUST: REVIEW, INSPECT, VERIFY, AND MAINTAIN, and make sure you are properly funded. Mistakes are always ready to creep in

CONCLUSIONS

The paper seeks to raise awareness of the potential loss of understanding as projects evolve from initial concept through laboratory testing to design documentation and final construction but a series of case histories.

In the examples cited, three structures have had a significant reduction in life, while others have been rescued because of chance observations by ad hoc inspections by non-team members. And at least one, hurt has been caused to a third party. In many cases, decisions made by the construction team are not referred back to the concept developers.

Formal design change control procedures are recommended, and the review of such changes should be referred back through the entire team, including relevant advisers and suppliers.

It is important to keep proper records, including As-Bult drawings. For reliable reference, these should be updated revisions of the contract drawings, not a separate file of notes that may well get separated as the years pass by.

ACKNOWLEDGEMENTS

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REFERENCES & BIBLIOGRAPHY

Eden Woodchip Berth. *https://foreshoremarine.com.au/portfolios-view/anwe-wood-chip-wharf-eden-nsw/*

Foster, D.N. and Gordon, A.D. (1978). Beach Protection, Cronulla. *Proceedings 4th Australian Conference on Coastal and Ocean Engineering, Institution of Engineers*, Australia, Adelaide, November 1978.

Foster, D.N and Brown C.T. (1978) Protection to Prince Street, North Cronulla by beach nourishment

Uni. of N.S.W., WRL Tech. Report 78/1, January 1978.

Brown C.T. (1979), Gabion Report. WRL Report No 156 *(MEngSc Thesis, Uni. of NSW)*

Brown C.T. & Haradasa D. (1980) Proposals for the long-term protection of Prince Street and the improvement of the amenity of North Cronulla Beach, Sydney: Water Research Laboratory

Brown C.T., Smith T.A.D. & Foster D.N.(1983) *Design and use of rock filled mattresses for coastal protection* Copedec 111 Si Lanka 1982—

Brown C.T. (1984) *Flexible Revetments, International Conference on Flexible Revetments*. pp 1-12. ICE 1984

Hirst, E.H.W. and Foster, D.N. (1987). The Design and Construction of Prince Street Seawall, Cronulla. *8th Australian Conference on Coastal and Ocean Engineering, Institution of Engineers, Australia, Launceston*, December 1987.,

Turner I.L. Couriel E.D. et al (1997) Wamberal Beach terminal Protection structure -*Physical Modelling Study. Report No. UNSW WRL Technical Report No. 97/26 Water Research Laboratory (1998) Informe 98/44 Estructura de Revestimento Para Rada Tilly*

Turner I.L. Cox R.J. et al (1998) Water Research Laboratory (1998) *Informe 98/44 Estructura de Revestimento Para Rada Tilly*

Donini H.J. (2012) Defensa Costeras de Rada Tilly Chapter 2.3 in *Recomendaciones en el diseño, construcción y uso de las obras de abrifo de talud tendido – experiencias en la Provncia del Chubut.*

Brown C.T. (2022) 40 years of flexible scour aprons, *Proceedings of 37th International Conference on Coastal Engineering (in press)*