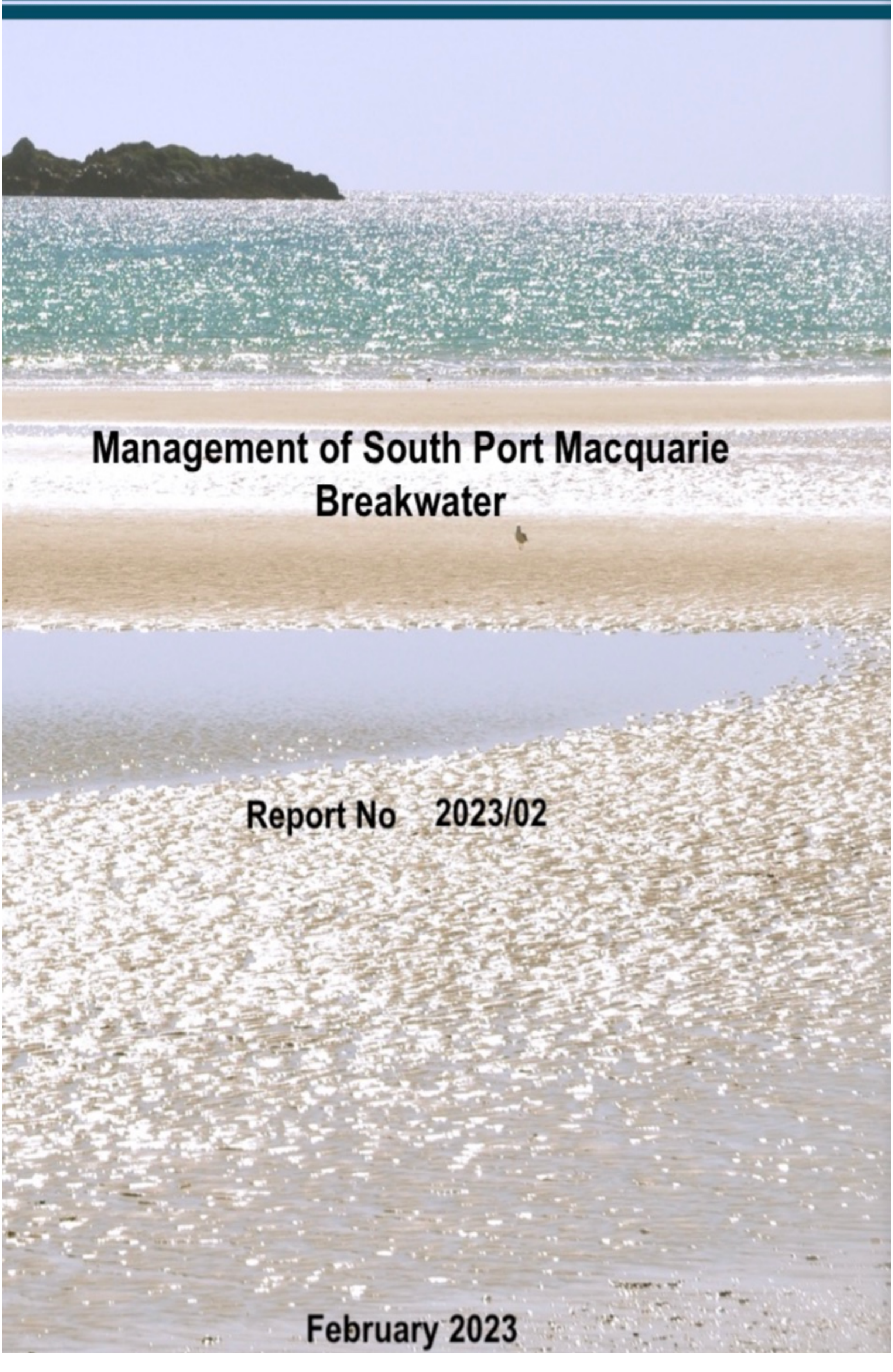
Coastal Zone Management & Planning



Management of South Port Macquarie Breakwater

Report No 2023/02

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EXECUTIVE SUMMARY

Based on inspections undertaken on the 8th and 9th February 2023, and available information from various reports it would seem that, as was previously determined by Manly Hydraulics Laboratory (MHL) and outlined in the inspection report by Royal Haskoning DHV (Haskoning), the South Port Macquarie breakwater is in reasonable condition, but in need of some targeted repairs. A guideline to the areas requiring repair, as determined by this current inspection is covered in detail in Appendix B. The repairs are for two different reasons. The majority are to reduce the risk of injury to persons who wish to depart the crest path and clamber on the rocky slope and the remainder are to address issues of structural stability of the breakwater itself. Hence these are targeted repairs, of a similar nature to those carried out in the past, and not involving a major reconstruction of the structure. It is both interesting and important to note the similarities between the comments and locations where issues were raised between the 1994 MHL report and this current report. While some of the areas identified in 1994 have been repaired many have not and yet the breakwater/training wall is clearly in a similar state 30 years later. The only rational conclusion is that the breakwater has ‘weathered” the last 30 years with surprisingly little degradation, particularly for an adaptive “dynamic” type of structure. However, the suggested repairs (Appendix B) are still recommended so as to enable the breakwater to be readily manage into the future and in particular to minimize the risk of injury for those who choose to clamber on the rocky slope of the trunk.

Traditionally rubble rock breakwaters have been built using a heuristic approach to design. This produced structures that were/are flexible and adaptive. In the post Second World War era a design methodology was developed overseas which resulted in more formal structures built to specific design principals. These newer structures comprise a core material, filter layers to retain that core and then multiple layers of secondary and primary armour, with the primary armour being either rocks or concrete armour units. The traditional form of breakwater was based on using material available from nearby quarries and simply tipping it to form the breakwater structure and placing the larger quarry rock on the outside as armour. This produced a more flexible, adaptive, and self-adjusting “dynamic” structure. compared to the more rigid formal contemporary design approach. It was less expensive to initially construct but more expensive to maintain. However, with the increasing future climate uncertainty it is starting to be recognised that once the design life of the more formal structure is reached, it may be far more difficult and expensive to adapt than the traditional form. Public Works NSW continued to construct breakwaters using the more traditional approach up to and including the late 1970s at which time breakwater management was passed on to Crown Lands. Public Works did so because they recognised the advantages of a more adaptive format. A resurgence in the more traditional approach of adaptive, self-adjusting structures (now referred to as “dynamic breakwaters”) is occurring as they are far easier to adapt to a changing design environment. Further, while traditional designs display a progressive failure mode the more contemporary designs, such as those being considered for an upgrade at Port Macquarie, can catastrophically fail if the design wave conditions are exceeded.

The southern breakwater at port Macquarie is a traditional structure first built over 100 years ago. The passage of time, and its exposure to multiple floods and major ocean storm events has provided plenty of opportunities to adapt to its environment. Nevertheless, by its very nature it can be expected to continue to require some repair from time to time, but this is less with time.

An important issue which seems to have been overlooked is the Northern breakwater's role in sheltering the southern structure. Until the northern wall was built the southern wall was exposed to direct wave attack over much of its length. Post the 1970s the northern wall has provided a significant degree of sheltering and hence the condition of the north wall has a fundamental bearing on that of the southern wall. The Northern wall has also tended to focus tidal and flood flows along the south wall thereby accelerating its adaption to toe scour.

In the early 1990s the MHL was engaged to undertake a major appraisal of all the rock rubble breakwaters in New South Wales. MHL subsequently published a five-volume series of reports covering more than 60 traditionally built breakwaters at over 30 sites. This series of reports is very detailed and contains background information which provides an understanding of the behaviour of this type of breakwater. The condition of the Southern wall at Port Macquarie is appraised in detail in ten pages and summarised in two diagrams. Unfortunately, it would seem Haskoning was not aware of these reports as if they were they would have had answers to a number of their concerns. In particular, these types of breakwaters did not have a "core" type structure and so the protection of the core as proposed by Haskoning is not required. Secondly the "S" shape of the slope as shown in Haskoning's cross sections is a feature of a self-adapting structure. The slope of the structure in the vicinity above and below waterline adjusts to reflect the higher energy dissipation of wave attack that occurs in this region. Thirdly these types of structures experience toe scour from the time of construction and onwards during their "life" which reduces with time as the slope settles into the naturally scoured seabed. That is why the slope takes up the shape described by Haskoning. The excess rock material below water line is a result of past adjustments of the slope and provides additional material should toe scour again occur. Overall, it is important to understand the significant difference in behaviour between a dynamic and a more fixed philosophy to the provision of breakwaters, including that seeking to "upgrade" a dynamic structure to a rigid structure has potentially far more challenges than seem to have been envisaged.

In considering the stability of the Southern wall it is convenient to divide it into 4 zones, each reflecting the forces at play. The first is the western zone, starting at the tide gauge (for convenience and taken as "zero chainage" (Ch) for measurement). The structure in this zone is actually a training wall rather than a breakwater. Armour size in this zone is dependent on flood and tidal flow velocities. This section of wall is in reasonably good condition. The second zone is from Ch 160 east to approximately Ch 500, near the skate bowl. This is a transition zone where, moving east, the wave action progressively dominates the design criteria. However, the waves are mainly of the "surging" type rather than the more damaging plunging type. This zone, although sheltered to a degree by the Northern wall, is in the worst state of repair, a matter picked up in the MHL 1994 report, but only partially attended to. The lack of repair in this region is evidenced by the state of the slope (as detailed in Appendix B and covered in some detail in MHL 1994). The third zone is from Ch 500 to the "head" This is the zone of the trunk that is exposed to plunging wave action. It is in reasonable condition and there is evidence of repair having taken place in the recent past. The repair in this section addresses many of the issues raised in MHL 1994. The fourth zone is the head and immediate surrounds. This is the region subjected to the most aggressive wave attack. The evidence is that it has been recently repaired however more could be undertaken It is concluded that Haskoning's recommendation that some further repair may be worthwhile is therefore considered reasonable.

In summary, differing armour sizes and repair techniques are appropriate for each of the 4 zones. It should be noted that in this report the required sizing of rocks was not attempted as it was felt that the available wave data was inadequate. The inshore wave information seems to be based on desk top modelling using a SWAN model. While this is a state-of-the-art numerical modelling technique, experience has shown it can underestimate wave heights in complex situations such as at Port Macquarie and should be calibrated using a physical model. There is also a need to include in the modelling different seabed/riverbed scour conditions as these will impact on the size of waves that can reach the structure.

It is prudent to recognise that the Southern breakwater does require some attention. There are two different philosophies that can be followed. The simplest, and one with the least social, environmental and economic impact at present is to continue managing the breakwater as a self-adapting dynamic structure with the repair approach that has been a feature of this and other similar NSW breakwaters over more than a century. This involves identification of, and targeting, the currently vulnerable areas on the breakwater and repairing them by placing new armour stones sized to be similar to, or slightly larger than the surrounding stable stones in the adjacent area. Further, undertaking some "trimming" of the slope to remediate areas, as best as possible, where currently "hung-up" rocks exist. This approach does however require on-going monitoring and potentially further repair over time if subsequent severe storms cause the structure to progressively deteriorate.

The second option, which potentially has greater adverse social, environmental, and economic impacts involves "retrofitting" a contemporary design over the existing structure. Counterintuitively such a design approach can produce a structure more vulnerable to catastrophic failure unless the design criteria ensure the worst conditions cannot lead to the loss of the primary armour protecting the slope, especially around water level. This type of failure progresses very rapidly because, unlike the dynamic type of concept where there is a mixture of rock sizes under the primary layer, in the contemporary design the material underlying the primary armour is designed to be one tenth of the weight of the primary armour and therefore very erodible when exposed. Further, in undertaking a contemporary type design it is essential that the difference between "expected life" and "design life" are understood. To progress such a design, it is first essential that the acceptable "encounter probability" is established in order to determine the "design life criteria" that must be chosen to achieve the desired outcome. This normally results in the actual design being based on a far more stringent criterion than say a 1% (1 in 100) or 2% (1 in 50) event, both of which produce results that can result in a relatively high probability of failure during the anticipated life of the structure. Importantly while the issue has been implicit in many designs that have been based on "worst case" scenarios (such as the Port Botany revetment) the need to take an “encounter probability” approach has only been relatively recently recognised as essential. It does have a significant impact on the costs of structures such as breakwaters.

The simple dynamic, self-adapting approach has many advantages in the case of Port Macquarie Southern breakwater but should the decision to be taken for an upgrade, by overlaying the existing structure with a contemporary design, then it would be prudent to realise considerably more work is required to determine the armour sizes needed to ensure the structure has a reasonable likelihood of achieving its “expected life”.

It is noted that none of the reports appear to have considered the NSW Coastal Management act nor the associated Resilience and Hazard SEPP. The breakwater/training wall is a structure clearly within the defined coastal zone in terms of the structure, the memorial rocks and the trees. While it is understood that the Port Macquarie Hastings Council currently has a certified Coastal Zone Management Plan and is working on a Coastal Management Program it is understood that the upgrade of the breakwater/training wall is not included in the certified plan. Transport for NSW, as a State Government agency, falls under the auspice of the Act and SEPP which means that they do not require Development Approval if they undertake “repair” but do so for any other work, such as an upgrade, as the Land and Environment Court has determined elsewhere.

Targeted repairs can minimise the adverse impacts on other cultural aspects of the breakwater’s overall environment such as the memorial cap rocks, the existing trees and the crest pathway.

TERMINOLOGY

Throughout the various documents some puzzlement could be introduced by the terminology used. That is, words such as “breakwater”, “breakwall” “Jetty”, “training wall”, “wave trap”, “head”, “trunk”, “refraction” and “diffraction” may cause confusion. The following is intended as a basic explanation of the use of these words in the various documents.

In the commonly used Australian idiom the word “breakwater” refers a structure at a river or port entrance that extends out into the ocean and whose purpose includes, stabilise the entrance location and geometry, maintaining a navigation channel, assisting in increasing depths over entrance bars, promoting tidal exchange between the river and the ocean and maintaining the entrance open to allow the passage of flood waters. The word “breakwall” is synonymous with “breakwater”. It is a derivation not in common coastal engineering usage but appears in some of the documents involved in this project. The word “jetty” would normally, in the Australian idiom, refer to a wharf, however overseas it is used in such a manner as to be synonymous with the word “breakwater” especially in the United States of America.

A “training wall” is a wall within a river that is intended to direct the flow path, and often is also used to manage riverbank erosion. Commonly “training walls” lead up to “breakwaters” and sometimes it is a matter of conjecture as to where “training walls” finish, and the “breakwater” starts. This is apparent at Port Macquarie where the southern “wall” serves a dual role as both a “breakwater” and a “training wall”. Hence the Southern wall is variously referred to as a “breakwater” (the principal role of the Eastern end) or a “training wall” (the principal role of its western end). The term “breakwater/training wall” encompasses its overall hydraulic functions. This is not to confuse the southern wall with the so called “Eastern training wall” which is the terminology used to describe the wall inside the entrance along the bank on the northern side of the river leading up to the wave trap. The easternmost end of this wall could technically be referred to as a “breakwater”, a role it performed prior to the construction of the “North breakwater” and its ongoing role as the southern end of the “wave trap”.

The "wave trap" can be thought of as being the small beach between the eastern end of the "Eastern training wall" at Pelican Point and the "Northern breakwater". As ocean waves enter the entrance between the main breakwaters they spread as they reach the geometric expansion area before arriving at the internal port area. Some of the wave energy is dissipated on the small beach on the northern side. This feature is referred to a "wave trap" because of its role in reducing the wave energy travelling further upstream.

The word "head" in the Port Macquarie breakwater context refers to the eastern extremity of the breakwater(s) while the word "trunk" refers to the remainder of the breakwater.

"Refraction" of waves is the process by which the direction of a wave travelling in shallow water at an angle to the contours is changed. Whereas "diffraction" of water waves is the phenomenon whereby energy is transferred laterally along a wave crest, particularly after passing an obstruction like, for example a headland. So, ocean waves approaching Port Macquarie's breakwaters from the East to South sector experience both "refraction" and "diffraction" and hence have their "attack energy" reduced. Waves approaching from the North to East sector only experience "refraction".

"Hung up" rocks occur where the other rocks on the slope no longer provide full support

CONTENTS

INTRODUCTION

Coastal Zone Management and Planning was engaged by Damian King, on behalf of the "Save our Breakwall" campaign, to provide an independent review of the current state of the Southern breakwater at Port Macquarie and the recent reports regarding condition, repair and upgrading.

OWNERSHIP AND RESPONSIBILITIES

Setting aside any potential Native title issues, the Port Macquarie breakwaters and training walls are located on Crown Land. This "land" also incorporates the riverbed and the seabed out to the notional 3 nautical mile limit of State waters, (the 3 nautical mile, one league, figure is actually an historic artifact established in 1702 when Cornelius Bynkershoek recommended that maritime domination be restricted to the range a canon could effectively protect) – an interesting concept in a time of ICBMs and nothing to do with the current day Commonwealth territorial waters limits. It is only mentioned in this report because modelling of inshore wave climates sometimes commences with the offshore wave climate near the three-mile limit.

Responsibility for management of the breakwaters and training walls rests with the Maritime Infrastructure Delivery Office (MIDO) section of Transport for NSW (TfNSW) being the State's agency responsible for management of the maritime structures on Crown Land. There is however potential for confusion, and to some extent responsibilities are "blurred" by the Port Macquarie Hastings Council's involvement in what, regardless of the legal ownership, it would seem may be generally perceived to be community open space managed by Council. It is understood that Council does have care control and management over Crown Land's Lot612 DP 754434 and Lot 7025 DP 1060950 both of which back Town Beach, with the former being an area immediately adjacent to the eastern end of the breakwater. In addition, and to add to the confusion, the tourist park to the south of most of the breakwater/training wall is operated by the NRMA but is located on Crown Land (Lots 1 and 2 DP 1233519).

The likely confusion is not simply a matter of the somewhat convoluted responsibilities as land managers, but Council's adoption of the Port Macquarie Hastings Council's *Town Centre Master Plan 2014* and *Breakwall Concept Plan 2016*. These documents reportedly include aspirations as to the "enhancement" of the area in the vicinity of the Southern breakwater/training wall thereby giving rise to an apparent "ownership"/responsibility for the area by Council. MIDO's "Submission Report" (November 2022) clearly indicates MIDO's desire to work with Council to deliver at least some of the envisaged outcomes of the Council's adopted "Concept Plan".

It is noted from experience that it is not unusual for councils to develop concept plans that include Crown Land in order to maximise the enjoyment and amenity for the community and tourists. In fact, it could be argued that such action is a progressive initiative. However, experience also dictates that often councils do not have the expertise, nor even recognise the need to seek specialist advice, when proposing ideas for "enhancements" of areas within the coastal zone where there can be a number of complex issues that may make achieving the desired outcome challenging.

SETTING

The entrance to the Hastings River is tucked into the northern lee of the expansive headland stretching from Tacking Point to the south to Flagstaff Head in the north immediately adjacent to the entrance. This headland tends to shelter the entrance from direct ocean wave attack from the east to south sector, however wave diffraction and refraction processes means waves of reduced energy, from that sector, can impact the entrance as they bend around the headland, particularly in storm conditions. There is however no protection from direct wave attack from the north to east sector, other than the entrance bar and shoals which can, from time to time be scoured out, thereby allowing a greater degree of attack that might otherwise be the case. Unfortunately, this is a factor that is often overlooked.

Prior to the construction of breakwaters, the historical evidence indicates the entrance bar was a very unstable large, shallow dynamic sand shoal that constantly varied in depth and location. During river floods a channel would be scoured across the bar but the channel would quickly infill once the flood action ceased. Because of the overall geometry of the river near the entrance, the ebb tide would tend to follow the southern shore whereas the flood tide favoured the northern side, partially due to the development of a “gutter” in the surf zone along the beach to the north. This flood tide flow down the gutter and associated sand transport would progressively build a flood tide shoal (most likely the original “Pelican Point” feature) which would progressively build up across the entrance further encouraging the southern bank ebb tide tendency. When again a river flood occurred, the flood tide shoal would be washed out, only to start rebuilding as soon as the flood water subsided. As indicated, in between floods the historical tendency was for the channel across the bar to meander and shoal, hence the breakwaters have played an important role in focussing the tidal flows into and out of the entrance thereby lowering the bar and creating a more stable, and navigable, channel than existed before the breakwaters were constructed.

The Manly Hydraulics Laboratory’s (MHL) Waverider data from their Coffs and Crowdy Heads buoys clearly shows the dominant storm wave directions, and wave energy in this area of the NSW coast, is from the east to south sectors and is centred on the SE to SSE direction. However, from time-to-time large storm waves can approach from the east, or even slightly north of east, as shown in the MHL assessment report of 2021, 2804-D.

BACKGROUND - A BRIEF SUMMARY OF THE HISTORY OF THE HASTINGS RIVER ENTRANCE.

In order to best understand the current condition and to place the future management of the Southern Port Macquarie breakwater/training wall into context, it is useful to consider a brief historical summary of its construction and long-term management. While there is some contradictory information from differing sources the general timeline and content is similar, and sufficient for the purpose of this report. The sources for this historical summary include Coltheart (1997), MHL (1994), Druery and Nielsen (1978) and miscellaneous information the author has collected over 50 years of coastal engineering working on NSW breakwaters, including perusing the historic “Work-as-Executed” drawings and notations.

The entrance to the Hastings was first reported in European times by Captain John Oxley in 1819 who described the “forbidding nature” of the entrance bar.

The coastal topography of New South Wales features wide rivers, creeks, lagoons, lakes extensive wetlands and rugged mountain ranges, all of which made the construction of the roads and railways necessary to connect the fledgling European settlements of the early 1800s difficult. So, coastal shipping and hence navigation of entrance bars such as at the Hastings, along with many other NSW river entrances became an essential part of import and export for the early settlement. Historical records show numerous vessels were lost and/or damaged on the wide shallow bar of the Hastings.

It was not until 1909 that a continuous route, was available between Hexham and Tweed Heads albeit that this route was similar to but not that of the current coastal highway. Travel times on this route were extended by the lack of bridges and hence the need to ferry vehicles across many rivers. Interestingly, for example it wasn’t until 1945 that a bridge across the Hawkesbury was completed.

Construction of the North Coast Railway took place between 1911 and 1923. This took some pressure off the need to complete the coastal roadway connection. As a result coastal shipping remained the major avenue of coastal transport for not only most of the 1800s but right through and into the 1900. It continued to have an important transport role today in servicing Lord Howe Island with Birdon’s “Island Trader” providing a regular service from Port Macquarie. In addition, Birdon also has a ship building yard on the Hastings.
Historical records indicate that in 1884 a young engineer in the Department of Public Works NSW produced a paper presented to the Royal Society of NSW which included a proposed design for the Hastings entrance breakwaters. A year later, in 1885, the British engineer Sir John Coode, who had been engaged by the NSW Government to report on solutions to improve navigability of several entrances, provided a design that included both a northern and southern breakwater at Port Macquarie.

In 1897 a contract was let for the construction of the southern breakwater with 255 metres being built in the first year. However, when expenditure reached the limit of its approval the contract was terminated in December 1899. A new contract was subsequently awarded in early 1901 to construct the southern wall to a length of 823 metres. Work was again suspended at the end of October 1901, due to a shortage of funds by which time the wall was 791 metres long. By 1902, due to settlement of the head, records indicate the seaward end of the break water was reduced in length by 3 metres. A proposal for completion of the work was subsequently rejected by the Parliamentary Committee on Public Works.

By 1905 the sandbank that had built up behind the wall extended from the shore to ‘‘within a few feet” of the end of the wall (early version of what is now Town Beach). Also in October 1905, following storm wave activity the low-lying sand “spit” to the north of the entrance was broken through resulting in the development of a broad, hazardous, entrance bar that was both torturous and dangerous to navigate.

By 1923 further breaches of the sand spit were occurring and, in October 1932, a new channel attempted to form about 1,200 metres north of the southern breakwater. This led to a decision to construct a training wall along the northern bank of the river (sometimes confusingly referred to as the “eastern training wall”) in an attempt to cut off further breakthroughs. Work began in October 1933 and by 1938 the training wall had reached 1,311 metres at which point worked stopped as it was considered that it was sufficient to prevent further breakthroughs. Reports indicate that at the time some repairs were also carried out to the Southern breakwater, including placement of some concrete blocks and some “concreting” of the breakwater though no details of what this “concreting” entailed are readily available, which is unfortunate when seeking to analyse the current condition of the breakwater, and in particular its “core”.

In 1939 it was reported that the “concrete core” of the southern breakwater was completed. But again, there are no readily available details to enable the extent or location of this work to be assessed. Hence there is understandable confusion in regard to what this meant given there are no other reports of a concrete core. It is however the reports of construction work in the 1930s that has been incorrectly interpreted to mean the wall was constructed around 1935 whereas the report is simply that the concrete work was completed in the 1930s but the actual rubble rock structure was completed around 1902.

Further work took place on the training wall between 1939 and 1943 including an extension of 235 metres. This “eastern” training wall was constructed as an unemployment relief project between 1940 and 1941. It was finally completed to an overall length of 1,636 metres in 1943. At that time the extension protruded past the beach shoreline effectively becoming a “stub” northern breakwater at Pelican Point. The current northern breakwater was commenced in 1976 approximately 160 metres to the north of the training wall resulting in this training wall extension forming part of the “wave trap” between it and the northern breakwater.

Repairs on the Southern breakwater were reported between 1948 and 1952 when again works stop for lack of funds.

From 1957 to 1958 the “eastern” training wall was repaired over a length of 533 metres. Again, confusion of the “eastern” wall with the Southern wall has led to some misinterpretation as to the repair regime. Reports do show that between 1961 and 1962 unspecified repairs were undertaken on the Southern breakwater.

In January 1968 there was a large flood which carved an easterly orientated channel through the entrance bar and produced seabed scouring between the Northern training wall and the Southern breakwater. The scouring was reported to have produced a channel depth of over 10 metres adjacent to the Southern wall. The flood reportedly deposited a considerable quantity of sand well offshore of the headland to form an extensive shallow region which subsequently, as it moved back onshore built up the swash bar on the northern side of the entrance.

Between January 1968 and August 1972 survey information indicated that there was a period of significant onshore movement of the seabed contours associated with the general flattening of the offshore gradient and a building of the entrance bar making navigation difficult. As previously mentioned, this was most likely the inshore movement of the sand washed offshore by the flood. At about this time it was reported that a floodtide gutter had formed along the northern beach surfzone resulting in a large amount of sand being moved into the entrance resulting in considerable choking of the navigation channel.

1973 there was an extension of the State Government's policy regarding the upgrading of fishing ports to include the provision of funds for facilities to meet the growing needs of tourism and pleasure craft. This enabled the cost of construction for the long overdue Northern wall to be justified and in March 1976 parliament approved a bill sanctioning expenditure of $1.3 million to enable the northern breakwater to be built with the objective of creating of a more stable bar formation and hence a safer and more reliable entrance.

Construction of the 490m long northern breakwater commenced in October 1976 and was completed by March 1978. Subsequently a report in 1978 recommended that the rock armour on the northern face of the Northern breakwater be amended and further works be undertaken to re-armour this region to accommodate the increased wave climate resulting from the absence of the sand shoals previously associated with the bar formation in this area.

A detailed assessment of the conditions of the Southern and Northern breakwaters was undertaken in 1993 and presented as part of MHL report 646B published in 1994. This report comprehensively detailed the conditions of both breakwaters and repair work that might be necessary to maintain the breakwaters in fit-for-purpose condition. It also provided a template for assessing the condition of breakwaters. The “report” was in 5 volumes covering 61 similar breakwaters at 33 sites in NSW.

In 2014 it was reported that major maintenance work was carried out on the Southern breakwater head. At that time it was identified that the trunk also required some repair work but funds for this were not available.

The importance of major floods such as in 1929, 1930, 1963, 1968 and 2020 is that they scoured the channel along the southern breakwater to a considerable depth and so provided ample opportunity for the toe of the structure to settle into a stable configuration, “feeding” itself from material that slumped down the slope of the trunk.

BREAKWATERS/TRAINING WALLS GENERALLY

Breakwaters and training walls can take many forms. At Port Macquarie they fall into the classification of “rubble rock” structures. With very few exceptions breakwaters and training walls on the NSW coast have traditionally been heuristically designed. A nearby quarry was opened, blasting undertaken based on an attempt to generate the largest rock the available machinery can handle, and the material transported to the site where it was simply tipped, creating an unstably steep slope which adjusted itself under wave action. Although focus is on generating large rock, the actual quarry operation usually produced a far greater quantity of smaller material (called “quarry run”) which was then used to establish the basic shape of the breakwater. That is, there was no “core”, “filter layers” or armour “underlay”, waves and currents were left to establish the final configuration.

Usually, in order to be able to transport material to the construction zone, a pathway or roadway was constructed along the crest as construction proceeded. This roadway sometimes was stabilised by concrete and sometimes incorporated a railway track. The largest rock was set aside as “primary armour” for the outer face of the trunk and for the breakwater head. Over time this design/construction approach led to differential settlement requiring the breakwater to be “topped up” by the addition of more armour stone, or concrete armour units if large enough stone was not available.

The earliest breakwater of this type in NSW was initially constructed over 200 years ago at Newcastle between Stony point and Nobbys Head and has been maintained ever since as part of the Newcastle entrance protection works. An interesting example where the quarry was unable to generate large enough stone was at Laggers Point at Trial Bay where the attempt to construct a breakwater had to be abandoned because the wave climate was such that construction efforts were regularly frustrated. This was before the advent of concrete armour units.

By the 1930s some more sophisticated design approaches were starting to be developed in Europe, but it was not until 1954 that a robust approach became internationally available in a document produced by the US Army Coastal Engineering Research Centre. This document was titled “Shore Protection Planning and Design – Technical report No 4”.

Over the years the document and methodology has been refined and reissued. In more recent years more sophisticated approaches to breakwater design have been developed in Europe, but often give similar results to that of the design approach of the early 1950s, known universally as the “Hudson equation”. Basically, this contemporary design philosophy led to a breakwater which had a core protected by filter layers between progressively larger stone layers with typically a double layer of “secondary armour” protected by an outer, double layer, of primary armour. The secondary and primary armour layers were designed so as to be “stable” under wave attack. “Stable” actually meant very little damage, usually less than 5% under design wave conditions. This level of damage was because it is very difficult to ensure all armour rocks are interlocked in such a way they will not move. An important issue when considering the safety of people scrambling around of the “face” of a breakwater. If rocks of adequate size are not available, then alternatively concrete armour units can be substituted. Over the years a wide range of concrete armour unit shapes have been developed, each with its strengths and weaknesses. The drawback to this type of structure is that if the design conditions are exceeded the outer layer can fail exposing the significantly smaller secondary layer thereby making the structure more vulnerable to catastrophic collapse. Whereas the more traditional form of breakwater, because of the mix of rock sizes, tends to experience progressive failure and can be readily repaired.

The more sophisticated design techniques also enabled the outer slope to be flattened to accommodate smaller rock sizes if that were all that was available. An extreme version of this more modern approach was what is referred to as a “Dynamic” breakwater which ends up being very similar to the behaviour of the older traditional breakwaters In that it is “self-adjusting” by allowing the breakwater to adjust its shape to adopt to a shape and behaviour something like a rocky beach. This behaviour and the linkage between the traditional and modern approaches is an important matter to understand when working with the historical breakwater design/construction techniques that were used for most of the NSW breakwaters, including those at Port Macquarie.

The contemporary design techniques not only focussed on the make-up of the trunk and head but also included crest design to manage overtopping and toe design to accommodate seabed scour. Typically, the toe design featured what is sometimes referred to as a “Dutch toe” which is an excess of rock at the toe that can “fold down” into any scour hole that develops.

Nearly all breakwaters in NSW were constructed by the Public Works Department. Most were initially built by the 1950s prior to the emergence of modern design techniques and so were simply tipped quarry stone. As indicated this means they had no specific core material and no particular armour provision other than an attempt to use the largest rocks available on the outside of the trunk and on the head. Given that the technique usually produced an unstable slope of at best 1:1 (1.5:1, that is one vertical to 1.5 horizontal, being accepted as the steepest “stable” slope for rock armour) it has meant that the structures have tended to differentially settle and shed outer rock, particularly during storms, thereby requiring “repair”. However, this repair work has tended to decrease with time as the structures self-adjusted to conform to the wave climate they are exposed to.

The South Port Macquarie breakwater, being initially constructed over 120 years ago, has the characteristics of a heuristically designed self-adjusting “dynamic” structure that has required repair on several occasions, particularly after large storms. In its historical context the design and performance of the breakwater to date is understandable if the relevant information is accessed. Interestingly during the 1970s when sophisticated design techniques were freely available and had been already in use in NSW, in Botany Bay with the airport extension and the Banksmeadow revetment protecting Port Botany, Public Works chose to use their traditional approach of rubble mound construction when building the Northern breakwater at Port Macquarie. This decision was based on an understanding of how such breakwaters behaved including their flexibility and adaptability. However, given this decision the important sheltering role of the Northern wall for the Southern breakwater, means it requires monitoring and the attention required by a heuristic type design.

In the early 1990s MHL was engaged to undertake a detailed appraisal of the condition of 61 breakwaters at 33 sites on the NSW coast. These were all breakwaters built by traditional means. The report was presented in 5 volumes which provided an excellent detailed insight into how this type of breakwater behaves over time. It included the then current condition (detailed both in writing and diagrams) and detailed the repairs required at the time. That is, the report(s) provided a template for how breakwater assets should be appraised and the condition of the breakwaters at the time, which for the Southern Port Macquarie breakwater suggests things have not changed markedly over the past 30 years. Unfortunately, Haskoning doesn’t seem to be aware of the 1990s MHL reports.

THE RELATIONSHIP BETWEEN THE SOUTHERN AND NORTHERN BREAKWATERS

Some 70 years elapsed between the initial construction of the Southern and then the Northern breakwater (setting aside the role of the Eastern Training wall acting as a short northern breakwater for a period of time). That is, the southern breakwater had around 70 years of direct wave attack from the north to the east sector and from diffracted and refracted waves from the east to south sectors thereby allowing it to adjust to the wave climate that impacted on it prior to the northern breakwater construction.

However, the construction of the northern breakwater in the 1970s fundamentally altered the situation by providing protection against direct wave attack on a length of the Southern breakwater/training wall from waves from the northern sector and by truncating the storm wave attack from the east to south sector. The stability and performance of the Southern breakwater has therefore been enhanced by the building of the Northern breakwater. Hence the earlier history of performance and repairs has to be interpreted accordingly as it is not representative for the current situation, with the exception of the unprotected head and the still exposed adjacent trunk area.

That is, the current and future stability and safety of a length of the Southern breakwater is, to an important degree, dependent on the condition of the Northern breakwater. Should the Northern breakwater fail, either in part or overall, it will have implications for the stability of the Southern breakwater. So, the Northern breakwater has an important dual role of providing protection to much of the southern breakwater as well as helping direct the tidal flows across the entrance bar, thereby increasing the depth over the bar and improving navigation. In addition, it also acts as a barrier to prevent development of the gutter which allowed the previous flood tide ingress of sand from the surfzone to the north.

It therefore is of some concern that the emphasis has been on the Southern wall upgrade project alone rather than recognising the importance of the Northern breakwater on the stability and safety of the Southern wall. This is understandable because access to the South wall is relatively easy and hence there is a great deal of community and tourist use of the structure and its surrounds whereas the Northern wall is less readily accessible and therefore its role is not as well understood nor appreciated.

FAILURE MODES FOR ROCK BREAKWATERS

There are five possible failure modes for rubble mound breakwaters such as the Southern Port Macquarie breakwater:

1. Toe failure,
2. Slope failure on the trunk from wave attack and/or tidal and flood flows.
3. Crest failure
4. Overtopping failure
5. Head failure
6. Core failure

1. TOE FAILURE

Toe failure is normally associated with the scour of the river/seabed at the base of the structure and the subsequent subsidence into the scour hole. Toe failure can destabilise the rest of the slope leading to overall slumping and general settling. However, the Southern breakwater/training wall is over 120 years old and has been subjected to many wave and major flood events, and for some 70 years without a Northern breakwater has been exposed, for most of its length, to more severe conditions than exist today. It could therefore be reasonably expected that any toe scour and slumping settlement has already taken place and so toe failure would have a very low expectation of occurring.

It is noted however that slumping of the main slope of the trunk has, and will, result in additional material cascading down the slope from time to time to form a slump deposit in the toe region and that deposit may have a relatively steep “face” which may be unstable. But any further failure of this material does not constitute toe failure that will impact on the rest of the slope which continues to rely on the already scoured , and buried, toe-material’s support. But it can give the perception of potential toe failure to those who don’t understand how this type of breakwater behaves.

2. SLOPE FAILURE

For rubble mound structures built by simply tipping rock this is the most common form of failure. Experience indicates that it starts near mean sea level as this is the region where maximum wave energy is expended, particularly for plunging waves. As the region at mean sea level is attacked rocks, especially the smaller ones, can be removed resulting in a local steepening of the slope further up the face with the potential for slumping of the slope or rocks becoming “hung up”, which can be of concern regarding the stability of the slope but more so for persons who chose to climb on the slope.

Slope stability relies on both the size/weight of the available armour material and the angle of the slope. It can also be dependent on the thickness of the armour with two layers of outer armour being the normal recommendation. Usually “tipping” slopes start of at 1:1 however the maximum steepness for rock armour is 1 vertical to 1.5 horizontal (modern breakwaters are often constructed to slopes in the range of 1 vertical to either 1.5 or 2 horizontal). So tipped slopes are inherently unstable, and often have only one layer of outer armour. Given the Port Macquarie Southern breakwater was a tipped formed structure it is not surprising that the face of the trunk has, over time, required repair as the slope has adjusted to a more stable configuration, and the structure settled.

An unfortunate implication of this self-adjusting behaviour is that the upper slope, including the capping crest rocks (the “Memorial stones”) are vulnerable to being displaced down the slope over time.

3. CREST FAILURE

Waves overtopping the crest of a breakwater can, if repeatedly occurring, remove material including rocks on the crest and scour the crest itself which results in the progressive reduction of crest elevation and/or development of “slots” across the breakwater. Left unchecked this can result in the crest being lowered to water level, or below as has been the case with some NSW breakwaters.

At South Port Macquarie breakwater, a concrete pathway extending for the length of the crest which combined with the cap rocks ("Memorial stones") acts to minimise the potential for a crest failure to develop. The concrete pathway has stabilised an effective crest level of between 4 and 5 metres (including the capping rocks) near the head decreasing to the west where it is of the order of 3 to 4 metres before decreasing to 2 to 3 metres at the western extremity.

Towards the head there is evidence that the second last "panel" of the concrete pathway has been replaced very recently. The last panel at the head is badly cracked and is breaking up suggesting both settlement and potentially overtopping damage. The vulnerability to significant overtopping damage may have been recently reduced as a result of "new" large rocks being added to the head thereby effectively increasing local elevation of the head.

Interestingly however, examination of the cracked slab at the head indicated there was an earlier concrete pathway under the present one, and that pathway had also suffered damage. The matter this raises is as to how old, and how thick is the current concrete pathway that extends for the length of the trunk. That is, its current observable good condition, which suggests settlement is possibly not an issue, may be covering up an earlier pathway that has shown historical damage. The importance of this is that if the current surface is a relatively thin concrete layer, the use of heavy machinery on the crest to repair/upgrade the breakwater could result in far more damage to the structure than envisaged by assuming the current pathway is of a substantial thickness.

So, it would be prudent to further investigate the concrete crest if heavy machinery is to be used on the crest to ensure a breaking up of the path under load (as is also the opinion of Haskoning) will not lead to deeper seated problems.

4. OVERTOPPING FAILURE

Overtopping failure, as compared to crest failure, occurs on the lee side of a breakwater as the overtopping water "escapes" down the shore side slope.

Although from time to time there is reportedly, and some on-site evidence, of overtopping, it is mainly confined to the head and the nearby trunk. In this eastern head region, the relatively low elevation of the crest, as compared to wave runup from ocean wave attack, has reportedly caused significant overtopping during major events. However here the lee side of the breakwater is also a rock slope (northern corner of Town Beach) and so the overtopping causes little damage.

The degree of overtopping and hence overtopping stress any breakwater is experiencing can often be best judged by the amount of damage on its lee side, in the case of the Southern breakwater/training wall, the southern side. Apart from in the immediate vicinity of the head there is no evidence of the type of sustained damage serious overtopping produces along the rest of the trunk. This is not to say that during major storm conditions there might be spray overtopping from time to time.

5. HEAD FAILURE

Head damage is the most common form of failure most NSW rock rubble breakwaters suffer. The head is the most exposed section of a breakwater and because of its curved shape, makes it far easier for waves to dislocate armour stripping it from the head, especially from about mean sea level, and above, and depositing it on the seabed in the immediate vicinity. This results in a flattening of the slope of the head and formation of an underwater rampart around the head, particularly on the southern side in the case of the Southern breakwater as evidenced by the scattered rocks on the Town beach northern corner.

At both Port Macquarie breakwater heads this damage and stripping of material has been on-going since they were first constructed. In order to overcome this form of failure, which can result in progressive retreat of the head, larger armour stones are required or, more commonly the substitution of sufficiently large concrete armour units that are shaped to achieve better interlocking than stones.

6. CORE FAILURE

Core failure occurs when the finer core material is leached out, generally by wave penetration of the structure. When core failure takes place breakwaters collapse internally into themselves. This destabilises the overall formation and can lead to catastrophic collapse. Characteristically it can be detected by evidence of significant structural settlement. At Port Macquarie Southern breakwater/training wall, as previously mentioned, there is a concrete pathway along the length of the crest. With the exception of the section of pathway approaching the head the greater length of this pathway currently shows no signs of distress, such as the cracking which could be expected to occur if active settlement was taking place at present. At the head, and the section of pathway approaching the head, significant cracking is apparent reflecting the instability of this region.

The South Port Macquarie breakwater and training wall was, as discussed constructed using the historical technique of simply tipping quarry run and then placing the larger sized stone on the outer face. This means that the material forming the “pseudo core” was subjected to wave swash as it was being placed and, given the high porosity of this type of structure the open voids also allowed on-going winnowing of any small material. Given there was never a formal core to the breakwater and the exposure of this open structure to wave action for over 100 years, core failure is not a credible failure mechanism. It is however noted that over time some rocks within the breakwater can be expected to be breaking down by weathering and fretting, so small amounts of material are progressively being lost from the overall mass. This could be misinterpreted as being loss of a core material.

One matter which has not been able to be investigated is the reported historical addition of concrete into the breakwater structure. To date no records are available regarding this matter and it is not apparent whether the information relates to concrete being injected into the “core” region of the formation or whether the concreting was an artifact of the construction of the concrete pathway(s) on the crest.

ARE ROCK BREAKWATERS DANGEROUS?

Breakwaters are designed to resist storms. Their design and condition is related to what is termed the “design storm”. Few if any breakwaters are designed to be stable in an ultimate storm, they are generally designed around a criterion such as a 1%, or a 2% event (commonly misnamed a 1 in 100 or 1 in 50m year event respectively). The importance of the terminology is that a 1 in 100 criteria means a storm that has a 1% probability of being equalled or exceeded each year, or a cumulative probability of around 45% of being equalled or exceeded during a life of say 60 years. Which means it is almost 50% likely that a storm equalled to or worse than the design storm will attack the breakwater during its design life. So, most breakwaters are vulnerable to damage and overtopping at some point in time.

Breakwaters of the rubble mound type such as those at Port Macquarie experience progressive, not catastrophic failure. Individual rocks or small groups of rocks can become displaced from time to time due to wave action and in doing so may destabilise some rocks up slope from them. These up-slope “hung-up” rocks may then be displaced by people clambering around on the slope and this can result in injury.

Historically failures of rubble mound breakwaters have occurred during storm wave attack when people should be nowhere near the structure. These failures are again of a progressive form which is why rubble mound breakwaters often have a history of repair following storm damage, as is the demonstrated case at Southern Port Macquarie. There are some rubble rock breakwaters on the NSW coast where post storm repairs have been neglected. These breakwaters have suffered on-going progressive failure over many years, some to the point where their crests have been reduced to near sea level or below. Compared to these breakwaters the South Port Macquarie breakwater is in “good” condition.

It is important to understand that, even rock armoured breakwaters that have been built to modern design and construction standards/methods with armour stone sized to be “stable” suffer some armour failure. As previously mentioned, it is very difficult, if not impossible to ensure all the armour stone is well interlocked to form a totally integrated surface. That is why the modern design criteria specify “zero damage” but accepts and warns that up to 5% failure of the armour can occur.

Apart from armour failure, all armoured breakwaters are inherently dangerous to those who climb on them. During even modest storms wave “sets” can unexpectedly inundate the slope or overtop the crest catching those on the breakwater unaware. This is a particularly dangerous situation near the head of the breakwater where overtopping is more common because of the exposure, the gaps between armour units tend to be greater as the heads are more easily damaged and units displaced, and retreat options are more limited.

The unevenness of the rocky slope of the trunk and the slipperiness of wet rocks, and of those covered by marine growth, makes negotiation of this area hazardous. Often those wishing to fish clamber down the slope to be closer to the water, however they need to be aware of the risks including the difficulties of beating a hasty retreat if conditions suddenly deteriorate. The unwary may also injure themselves by falling and/or slipping their feet or legs into the gaps between the rocks. Further, where rocks are “hung up” there is the added danger that they can be dislodged by persons climbing on them.

Therefore, appropriate signage is required to bring to the attention of those visiting breakwaters the dangers from factors such as overtopping waves for those walking on pathways along the crest, but in particular the dangers associated with climbing on the rocky slope. Such signage is an essential risk management consideration for those responsible for the structure. The aggressive environment means that such signage needs regular attention.

Experience dictates it is essential that the agency responsible for risk management of the breakwater is clearly established and acknowledged. In general, although usually “a lost cause” people should be discouraged from clambering on the sloping surface of the trunk, and especially the head, regardless of how well constructed and maintained the rocky slope.

An interesting further danger is the types of usage of the crest. For example, the concrete pathway encourages a variety of usages that, given it is relatively narrow, may be incompatible and/or limiting. In particular, while the cap rocks along the crest serve multiple purposes of both limiting overtopping and providing a physical boundary to the path, they have also become a surface on which “memorials” have been painted. This has resulted in visitors to the breakwater often stopping along the crest in order to read the “Memorials” thereby potentially disrupting the flow of other users. In addition, some uses of the relatively narrow pathway involving activities other than walking could result in injuries should the users collide with others on the path or with the cap rocks.

FISHING/VIEWING PLATFORM(S)

It is a readily observed fact that people enjoy walking out on breakwaters, wharves, and rock shelves. There is a natural attraction to the ocean, not only for swimming but simply to be able to observe firsthand its timeless motion. People also like to scramble down the rocky slopes of breakwaters to be able to fish.

It is understood that there has been some thought to enhance the recreational opportunities and the general amenity of the Southern Port Macquarie breakwater/training wall by the addition of fishing/viewing platform(s) on the northern face of the breakwater. Apart from aesthetic considerations, which is a subjective choice, there are some other factors that need to be considered.

The breakwater is a flexible, adaptive, structure that experiences differential settlement from time to time. This means the addition of a “fixed” platform and its supportive structure brings with it significant foundation challenges. These not only involve the need to securely “found” the structure through the adaptive breakwater formation but also the need to assess and accommodate local impact on the stability of the breakwater in its immediate vicinity. This is the result of the introduction of a “hard” element which may influence the ability of the breakwater to flexibly adapt to storm events.

The deck level and the type of decking, along with the structure and foundations need to be designed to handle the significant uplift pressures that can occur due to waves hitting the bottom surface of the deck. While this is achievable it does mean the costs may be significantly greater than may be envisaged for a similar structure in a non-wave environment such as on a river frontage.

Perhaps the issue often overlooked with this type of concept is that of public safety. While fencing can be designed to minimise safety concerns under “normal” conditions, the problem arises during storm conditions. Experience indicates that some people are attracted to closely observe conditions during storm events. Unfortunately some people venture into hazardous positions without realising it, only to be injured or killed when a larger wave set produces unexpected conditions. Many rock fishermen are lost every year for this very reason and each year people are injured at rock pools where they mistakenly think it is “safe” to “chain surf” hanging onto the chain fence as the waves wash over them.

While a platform could be designed and signposted to hopefully minimise the potential for injury or death it may still be necessary for the authority responsible for risk managing the structure to physically close off the use of the platform from time to time, a sometimes-onerous risk management task, particularly for storms that peak at night and, experience is that the attraction to dangerous situations is not necessarily a daylight pursuit.

In regard to Council’s responsibilities, it is noted that TfNSW “acknowledges that while the building of fishing platforms is part of the Council’s Breakwall Concept Plan 2016, they were not included in TfNSW’s proposal as the building of such platforms is not within the project’s budget or scope”. Further, TfNSW states that “additional amenities such as fishing platforms are for Council to consider. It is Council’s responsibility to install and maintain such amenities with approval from TfNSW as the manager of maritime assets in the area”. So TfNSW sees Council as the risk manager for the platforms which, given TfNSW is the risk manager for the breakwater, could lead to potential conflicts given the interaction between the two structures.

In summary, unless there are pressing reasons it would be prudent to not consider the concept of a fishing/viewing platform on the northern face of the breakwater/training wall. The possible exception being a location towards the western end, away from ocean wave action. However, the comments in regard to the issues centred on providing adequate foundations for such a structure and in ensuring those foundations do not have a deleterious impact on the local stability of the breakwater/training wall still stand.

BREAKWATER REPAIR, MAINTANENCE AND UPGRADING

As previously mentioned historically the NSW breakwaters were built, maintained, and repaired by Public Works NSW. Apart from any capital funds assigned for new works State Treasury made an annual recurrent revenue allocation for maintenance and repair. This was necessary because of the philosophy behind the historical construction methods involving low initial cost but a need for on-going maintenance. There were however seldomly sufficient funds to keep all breakwaters in good condition. Priority was given to the breakwaters protecting entrances where commercial, fishing or pleasure boating were considered to provide sufficient benefit to justify the costs. Where breakwaters were of a low priority, they were often allowed to progressively degenerate.

When responsibility for NSW's "minor ports" (including all breakwaters) was transferred from Public Works to Crown Lands (notionally 1989, but the actual timing was somewhat convoluted) the annual recurrent funding did not follow. At the time, this was touted by Treasury as a budgetary "savings" generated by the reallocation. So, for a period of time little repair and maintenance took place. Eventually MIDO, as the maritime assets manager for the TfNSW's responsibilities for Crown Land management were able to convince Treasury of the need to provide funds for the repair of a number of breakwaters. However, for budgetary reasons Treasury appears to prefer the allocation of funds for defined projects that have a completion date, not on-going maintenance type operations. Hence the "driver" to upgrade assets to the point where they do not require on-going maintenance developed. So, it is understandable that the desire to evoke a major upgrade at Port Macquarie Southern breakwater/training wall to the point where it supposedly will not need maintenance/repair for many decades has most likely originated.

RECENT ASSESSMENTS OF BREAKWATER/TRAINING WALL CONDITIONS.

There are three reports relevant to the condition of the South Port Macquarie Breakwater/training wall. None refer back to the MHL 1994 detailed Asset Appraisal report which provided both a template and a baseline for potential comparison. An assessment as to the current condition of the breakwater, compared to its historical condition over 26 years ago (see Appendix A), was not undertaken. In addition, there is a "submissions Report" prepared by Transport for NSW. It is not the intention of this current report to review these reports in detail however it is felt relevant to highlight their principal findings as they could be seen to be in conflict if not placed in context and understood.

The reports are:

1. MHL report MHL2804-D "Macquarie River Training Walls Post March Flood Assessment"
2. Royal Haskoning DHV Ref PA2696-RHD-ZZ-XX-RP-Z-001 "Port Macquarie Breakwater Basis of Design" October 2021(a Draft report noted to being a "live" document for internal use but downloadable from the TfNSW project website)
3. Royal Haskoning DHV Ref PA2696 "Port Macquarie Upgrade Design Report" Final September 2022
4. Transport for NSW "Port Macquarie Southern Breakwall Upgrade" Submissions Report November 2022

1. THE MHL REPORT

In 2021 Manly Hydraulics Laboratory (MHL) was engaged by the Maritime Infrastructure Delivery Office (MIDO) section of Transport for NSW to undertake an assessment of the post March 2021 flood damage to the breakwaters and training walls at a number of sites including Port Macquarie. On 20th April MHL provided a "letter report" (MHL2804D) which summarised their findings regarding repairs that they considered were required for both the South and North breakwaters/training walls. In making its assessment MHL had available its 1993 assessment (MHL 1994) and so was in a position to recognise that little had changed over the intervening 30 years. This no doubt influenced its 5 out of 6 rating for the trunk. However the focus on the letter report was that of before and after the recent flood event.

The following is an extract from the summary of the report (please note that the MHL comments on the North wall is included in this current report because of the considered interdependence of the South wall condition on the existence of the sheltering from the North wall):

North Wall

"The 2014 and 2016 (post June) storm surveys indicate that the head is prone to continuous damage particularly from Easterly and North Easterly storms.

Due to the Easterly direction of the March 2021 storm overtopping has caused damaged to the ocean side of the pathway.

The offshore storm associated with the March flood recorded offshore significant wave heights less than 5m and does not appear to have caused appreciable damage despite the associated high-water levels recorded in close proximity to the training wall.

The two storms with offshore wave heights over 6.5m and numerous more over 5m have been recorded (**MHL 2804-B**) since the 2016 inspection appear to have caused continued damage at the head. Hence, a comprehensive repair strategy of the head is recommended.

The post flood photos do not indicate any appreciable damage to the crest and trunk on the river side.

The condition of the wave trap training wall has not deteriorated since the survey in 2012.

Using a Kd *(coefficient of damage)* of 2.5, slopes of Cot α (slope),2 (head) and 1.5 (trunk) it is envisaged that 50 armour stones 4-6 tonne armour for the north facing trunk and crest *(are required)* to reduce overtopping on to the pathway and 100 armour stones of 6 to 8 tonne armour at the head using basalt of s.g 2.7 would suffice to repair the North Macquarie wall. This additional armour on the head should prevent the continuous damage to the pathway in close proximity to the beacon and head.

The 3D digital imagery indicated that 25 m^3 of 15kg-25kg rock is required to fill the large hole resulting from wave overtopping."

South Wall

"Comparison of the 2016 survey indicates a wall in relatively good condition with the ocean side trunk in a relatively undamaged state.

Comparing the 2016, 2020 (preflood) and 2021(post flood) drone photos indicate a damaged head which requires repair if further damage is to be prevented.

The March 2021 flood itself may not have caused appreciable additional damage however it was noted that the head requires repair

Using a Kd of 2.5, slopes of Cot α (slope),2 (head) 50 armour stones of 4 to 6 tonne armour at the head using basalt of s.g 2.7 would be sufficient to repair the South Macquarie training wall.

Repair of 15m of cracked foot path/crest closest to the head would be recommended"

MHL then provided a summary table of their opinion on the condition of the two walls, the scoring was out of 6:

Table S1-Summary of Macquarie Traing wall Condition*

Breakwater/Training Wall	Crest/Pathway	Head	Trunk
Macquarie North	5	2	4
Macquarie South	5	3	5

The score of 5 indicated the opinion was that the structure was not damaged by the June 2016 storm but now shows signs of minor deterioration.

The score of 4 indicated the opinion was that the structure was damaged by the June 2016 storm and now displays signs of minor deterioration.

The score of 3 indicated the opinion was that the structure displays signs of deterioration since last inspection in 2016.

The score of 2 indicated the opinion was that the structure is functional but requires immediate attention to prevent future damage.

Hence MHL's opinion was that at the time of inspection in 2021 the main trunk of the South wall was only showing signs of minor deterioration however the head needed attention. Whereas the north wall showed signs of damage and the North wall head was in poor condition and required "immediate attention" which is critical if the North wall is to continue to provide a level of protection to the South wall. Interestingly MHL provided an estimate of the number and size of rocks needed to repair the head and the trunk of the north wall but only an estimate of the size and numbers required for the head of the south wall.

MHL provided some wave data obtained from the Coffs and Crowdy Head Waverider buoys during the March 2021 storm event which MHL estimated to be less than a 1 year ARI event. Both of these records, reproduced below from the MHL report, indicate that during the storm the offshore significant wave heights in the region were of the order 4.5 to 4.9 m and reached maximum wave heights of 8.5 to 8.9m with approach directions of approximately 80 to 100 degrees (MHL refers to the event as having an "easterly direction". That is, from a direction that could have diffracted and refracted the waves onto the Port Macquarie breakwaters. MHL notes this storm "**may have resulted in the ensuing damage to the North Macquarie trunk (ocean side)".** They also noted the damage to both the north and south breakwater heads.

These results and comments give rise to concerns as to the wave climate obtained from the numerical modelling subsequently undertaken by Haskoning when calculating the size of armour needed for repair or upgrade of the Southern wall. It is noted MHL suggests that, although it was not part of their engagement, the sizing of armour should be based on both numerical and physical modelling. Experience dictates that numerical modelling alone is not sufficient to obtain a confident design wave condition for sizing armour in complex situations like Port Macquarie where there can be refraction, diffraction, shoaling and major changes to the entrance bar configuration, especially under storm conditions.

In addition there is the important issue of where the wave attack is in the form of plunging breakers which have a high energy impact on a breakwater or as surging breakers which have a significantly lower impact. Under storm conditions the head and approximately the first third of the breakwater could be expected to experience plunging breakers whereas the rest is more likely to experience surging breakers. Given that the required armour size is a function of both the wave height but also the type of breaker this is a matter that it would be prudent to re-examine.

Table 2-1 Coffs Harbour Offshore wavea Indicating Time Periods of Hs>4m during the Flood Period (18/3/21 to 30/3/21)

Date/Time	Hsig (m)	Hmax (m)	Tz (s)	Tsig (m)	T P1 (s)	WDIR (deg TN)
21/03/2021 3:00	4.23	7.18	8.02	9.68	9.77	94
21/03/2021 4:00	4.09	6.28	7.99	9.77	10.27	99
21/03/2021 6:00	4.315	6.73	8.41	9.89	11.45	102
21/03/2021 7:00	4.34	7.09	7.84	9.72	10.83	105
21/03/2021 8:00	4.28	6.47	7.95	9.98	10.83	96
21/03/2021 9:00	4.74	8.16	8.41	10.22	10.83	109
21/03/2021 10:00	4.46	7.39	7.99	9.93	11.45	101
21/03/2021 11:00	4.70	7.88	8.3	10.15	11.45	96
21/03/2021 12:00	4.42	7.1	7.88	9.86	11.45	98
21/03/2021 13:00	4.89	7.95	8.26	10.01	11.45	99
21/03/2021 14:00	4.77	7.46	8.32	10.33	11.45	103
21/03/2021 15:00	4.66	7.79	7.9	9.95	11.45	88
21/03/2021 16:00	4.18	8.91	7.87	9.94	10.83	87
21/03/2021 17:00	4.32	7.22	7.96	9.72	11.45	103
21/03/2021 18:00	4.09	7.3	7.62	9.59	10.83	99
21/03/2021 20:00	4.23	7.41	7.94	9.81	11.45	105
21/03/2021 22:00	4.03	6.84	7.82	9.55	10.83	92

Table 2-2 Offshore Data off Crowdy Head Harbour Indicating Time Periods of Hs>4m during Flood Period (18/3/21 to 30/3/21)

Date/Time	Hsig (m)	Hmax (m)	Tz (s)	Tsig (m)	T P1 (s)	WDIR (deg TN)
18/03/2021 23:00	4.14	6.75	7.61	8.99	9.77	101
19/03/2021 9:00	4.07	7.52	7.18	8.79	9.32	103
19/03/2021 10:00	4.07	6.73	7.62	8.72	9.77	99
19/03/2021 11:00	4.14	7.65	7.41	8.76	9.77	99
19/03/2021 12:00	4.00	7.89	7.61	8.84	9.77	117
19/03/2021 14:00	4.24	6.64	7.5	8.91	10.27	101
19/03/2021 15:00	4.15	8.04	7.29	8.85	9.32	105
19/03/2021 16:00	4.09	7.26	7.28	8.73	9.32	127
19/03/2021 18:00	4.13	7.75	7.24	8.8	9.32	112
19/03/2021 19:00	4.16	8.51	7.37	8.77	9.32	119
19/03/2021 20:00	4.39	8.08	7.37	8.68	9.77	101
19/03/2021 21:00	4.42	7.93	7.17	8.78	9.77	102
19/03/2021 22:00	4.43	8.26	7.21	8.71	9.77	84
20/03/2021 0:00	4.79	5.58	424.38	27.44	36.57	96

2. ROYAL HASKONING DHV BASIS OF DESIGN REPORT

The Haskoning "Basis of Design" Report is dated November 2022 on the TfNSW webpage under the listing of "Reports". However, clicking on the link brings up a report dated 6th October 2021 and marked "Draft", for "Internal use only". The second Haskoning report: "Port Macquarie Upgrade Design Report, Final September 2022" lists in its contents "Appendix A – Basis of Design" yet this Appendix has no content on the webpage. Hence it seems reasonable to assume the "Draft" document was re-issued as a final in November 2022 some 2 months after the final design report. Which seems strange as it implies the basis for design was established after the final design was completed. A possible explanation is that the October 2021 "Draft" report was simply reissued as a November 2022 document. In the absence of content for "Appendix A" in the "Final Report" it seems reasonable to assume this was the case. Hence the October 2021 report is what has been analysed.

The report indicates that Haskoning's understanding of "MIDO's objectives and functional requirements with consideration to budgetary constraints are:

"• Remediate training wall to address:

- o toe scour; and,
- o movement/displacement of rock armour.

- Upgrade training wall with consideration of climate change impacts (including sea level rise).
- Improve recreational land use including:

  o widen the footpath along the crest of the breakwater and training wall structures;
  o provide access ways from the road to the footpath; and,
  o consider numerous ancillary structures such as fishing platforms, lighting and shelters as proposed in the Breakwater Master Plan prepared by Port Macquarie-Hastings City Council."

Importantly the documented results of the site inspection of the Southern Breakwater/training wall undertaken by Haskoning in August 2021 produced a similar assessment to that by MHL in April 2021. While MHL had examined the condition of both the North and the South walls, with recommendations for both, Haskoning determination of the design parameters for the Southern wall made no comment on the state of the Northern breakwater, nor its possible influence on the stability of the Southern breakwater/training wall. Although MHL commented on the North wall neither MHL nor Haskoning mentioned its role in sheltering the Southern wall and hence its importance in determining a repair or upgrade to the Southern wall.

Unfortunately, the MHL report did not provide details of their findings but rather just a summary. On the other hand, Haskoning provided more detailed information to support its findings that the "trunk of breakwater west of the most recent repairs to the head of the breakwater is in generally fair condition" and "the profile at the end (easterly facing) of the head was considered to be over-steepened and some damage at the water line was apparent". That is, importantly, in principle, both the MHL and the Haskoning inspection came to similar overall conclusions.

Haskoning provided the following "Initial key general observations from the detailed site inspection":

- "the southern side of the breakwater and transition to back beach revetment at Town Beach was in good condition with no significant issues observed or remediation measures required;
- recent repairs to the breakwater head appear to have been effective in the upper (above water line) profile either side of the breakwater and were in good condition. Note: Below waterline will be assessed using analysis of hydrosurvey data. However, the profile at the end (easterly facing) of the head was considered to be over-steepened and some damage at the water line was apparent. Accordingly, there appeared to be some flattening of the profile below the water line in response to the incident wave climate at the focus point at the end of the breakwater.
- Crest pavement damage at the head of the breakwater indicating ongoing movement of the head;
- Crest at the head of the breakwater is considered too narrow for lookout structure without significant widening of the breakwater crest. Ongoing movement of the head is not conducive to foundations for a rigid structure. An area was identified further

west, adjacent to crest pathway where head widens significantly, which would be more suitable for a lookout (beachside and across the entrance) location;

- Trunk of breakwater west of the most recent repairs to the head of the breakwater is in generally fair condition. Localised areas of oversteepening were identified. Armour stone generally had a wide grading of sizes which creates voids and overhangs.
- Crest “containment” units along the trunk of the breakwater/training wall are generally large and embedded in the concrete pour along the crest pavement slab and form a rigid line. Many locations were observed where these units were overhanging due to the rest of the armour slope, which is a flexible and subsiding structure. Evidence of ongoing deep-seated subsidence of the riverside armour slope. Further analysis of underwater hydrosurvey profile is required.
- Slope instability generally appeared to be due to insufficient filtration design and subsequent scour around the mean water level. The hydraulic stability of the armour was generally considered sufficient for the wave and current climate (estuarine dominated processes).
- Ongoing slope instability indicates that the foundation of rigid ancillary structures on the breakwater slope will not be feasible, without significant works
- Isolated areas at the upstream bend of the training wall trunk were observed, where catchment flooding flows appeared to have caused significant damage to the profile with loss of armour volume and oversteepening due to hydraulic instability. This is
- potentially due to focusing of the catchment flood flow at this location.”

The Haskoning report also provided the following:

“The proposed works include the breakwater head and the inner river revetment and crest path with a distance of approximately 700m in length” presumably the reference to the path is to do with widening it as elsewhere it is stated the “potential upgrade is also to consider the feasibility of; widening the footpath along the crest of the breakwater and training wall structures, providing access ways, and considering numerous ancillary structures such as fishing platforms, lighting and shelters as proposed in the Breakwater Master Plan prepared by Port Macquarie-Hastings City Council.” Thereby tying the “upgrade” directly to the Council’s recreational aspirations rather than MIDO’s specific maritime asset management considerations.

In addition, Haskoning notes “the back beach revetment along Town Beach adjoining the breakwater head is not considered within the scope of the proposed works”. It is not clear from this as to whether the section of breakwater to the immediate south of the head, leading to Town Beach is, or is not included. The dispersal of rock in this part of the breakwater demonstrates the history of on-going damage and vulnerability of the head, so is an important aspect of any repair or upgrade project.

Haskoning having seemingly established that: “The hydraulic stability of the armour was generally considered sufficient for the wave and current climate” they also state: “The training wall requires remediation and potential upgrading to address issues of toe scour, movement/displacement of rock armour and consideration of climate change impacts.” This statement seems inconsistent with the previous and also suggests a lack of knowledge of toe behaviour (which Haskoning later states) and little understanding of climate change impacts during their selected design life.

In regard to the design life of the proposed upgrade Haskoning states that the design criteria are for the breakwater to be “stable” and that the “proposed nominal planning period for the rock works is 40 years, which is the design life of rock armour specified in AS2758.6. If required, the planning period could be extended by specifying more stringent rock (or concrete) properties for the armour units, increasing the size of the rock (or concrete) armour to allow for degradation or including additional maintenance requirements”. However there seems to be some confusion at what is meant by “design life” as it seems the term is being used to refer to “expected life” which would require an encounter probability analysis and a selected risk appetite determination.

Although Haskoning does comment on the fact that given the years of exposure to floods and storm waves (more than70 of which were without the benefit of the sheltering provided by the Northern breakwater) it must be considered highly likely that the entrance channel and the sea/riverbed in the vicinity of the breakwater/training wall would have scoured to its likely maximum depth at least once since its construction. However other comments in their reports seem to overlook this and the fact that as such, this would have resulted in burial and reshaping of the toe over time.

Specifically in regard to toe scour Haskoning opines: “The design scour depth near the head and on the southern side of the training wall would be somewhat shallower due to the progradation of longshore sediment transport and lower currents. The existing bed is at approximately -3.75m AHD. The estimated design scour depth has been adopted as approximately -5.75m AHD.” Yet, as previously noted, the historical measured scour depth following the major 1968 flood (prior to the construction of the North breakwater) was -10 metres.

It would therefore seem the Haskoning suggestion of placing rock armour on top of the existing rock toe would do little to ensure that the “structure is founded at the design scour depth” as clearly the breakwater has survived substantially more scour than the design figure adopted by Haskoning. Haskoning goes on to say: “Should this not be practical, then rock armour should be placed to form a ‘falling/Dutch’ toe or founded at the design scour level”. It would seem this is not applicable unless it is proposed to undertake a major reconstruction of the breakwater/training wall which would necessitate the construction of a new toe. A matter not considered is that a “Dutch” toe could actually provide a navigation hazard until it “falls” as it will protrude underwater into the fairway and, albeit the fairway is wide the underwater protrusion may require marking with a series of buoys.

Based on the available profile information and the visual observation undertaken for this current report, the existing toe appears to already have an excess of rock on it, most likely as a result of past rockslides from the main slope over time. This process would likely have commenced during, and immediately after, the construction phase over 120 years ago. This is consistent with the typical behaviour of these types of tipped rock rubble breakwaters as experienced at numerous locations on the NSW coast. However, this excess of rock tends to produce what could be mistaken as an unstable toe for the overall structure whereas any real or apparent instability is that of the material that has been displaced to cover the underlying toe.

One purpose of the "basis of Design" report was the determination of the design wave conditions to be applied to determining the sizes of rock required for armour. Haskoning presents the directional wave roses from the Crowdy Head Waverider buoy and an extreme wave analysis. The Crowdy Head wave directional statistics suggest the dominant storm direction to be SSE with a potential 1 hour, 1% wave height of 8.5metres. However, the headland between Tacking Point and Flagstaff could be expected to significantly modify waves from that direction so Haskoning apparently only chose to analyse waves from:

- "East northeast (ENE);
- East (E); and,
- East southeast (ESE)."

A SWAN model was used to determine "wave heights near the toe of the training wall and breakwaters". A Swan model is a state-of-the-art desk top numerical model. However, experience dictates it can underestimate wave conditions when applied to complex situations such as the 3-dimensional geometry configuration of the Port Macquarie entrance and environs. Including an unstable offshore bar that can be scoured out during storms.

Haskoning notes "that there is some uncertainty around the nearshore wave climate, which would influence overtopping volumes. Detailed wave modelling may be effective in providing more confidence in nearshore wave climate". They probably should have mentioned not only overtopping but also armour size however it is presumed that was intended.

Given the complexities of the geometry and the importance of correctly identifying armour size and overtopping Haskoning's concerns seems reasonable. Experience dictates that both numerical and physical models should be used in situations such as those at Port Macquarie in order to best determine the design wave conditions and therefore the required armour size and potential overtopping issues along the length of the breakwater. Such modelling could take into account the sheltering effect of the northern breakwater, the effect of different entrance bar configurations and also determine where, along the trunk the more violent plunging breakers on the slope transition to the gentler surging breakers and finally dissipate completely. This would allow an informed determination of the grading of armour units required to achieve "stability" at the various locations along the trunk.

In calculating the design wave heights, it is also necessary to determine the design water levels. Haskoning notes:

"The design still water level, including SLR would be adopted as:

- 1 year ARI event – ~1.35, AHD;
- 50 year ARI event – 1.83m AHD; and

- 100 year ARI event – 1.85m AHD ".

It is important to note that a 100 year ARI can be expressed as the 1% probability of occurring each year which means that over say a 60 year life there is approximately a 45% probability it is likely to be equalled or exceeded. In addition, wave setup has to be included but is a relatively minor influence. Importantly however the height of waves that can attack the structure is not only dependent on water levels but also water depths and so the degree of sea and riverbed scour associated with determining the design conditions is also critical.

The Swan model results (reproduced from Haskoning's report) are as follows:

ARI (year)	Wave parameters	Location									
		A	B	C	D	E	F	G	H	I	J
1	Hm0	4.5	3.2	3.1	2.0	2.5	2.0	1.2	1.0	0.8	0.7
	Tp	11.8	11.9	11.9	11.9	11.9	11.9	11.9	11.9	11.9	11.8
	Tm-1,0	10.6	9.5	9.2	9.1	9.2	10.6	8.7	8.5	7.8	7.6
50	Hm0	6.1	3.5	3.4	2.1	2.8	2.2	1.3	1.1	0.9	0.7
	Tp	12.9	12.9	12.9	12.9	12.9	12.9	12.9	12.9	12.9	12.9
	Tm-1,0	11.0	10.3	10.1	9.9	10.0	9.8	9.4	9.3	8.6	8.3
100	Hm0	6.3	3.5	3.4	2.1	2.8	2.2	1.3	1.2	0.9	0.7
	Tp	13.6	13.7	13.7	13.7	13.7	13.7	13.7	13.7	13.6	13.6
	Tm-1,0	11.3	10.7	10.4	10.2	10.3	10.1	9.8	9.6	9.0	8.7

Positions A and B are offshore, position C (wave height for 1% of 3.4m) is at the head of the Northern breakwater, D (wave height for 1% of 2.1m) is at the head of the southern breakwater and E to J are equally spaced along the breakwater/training wall with wave height progressively decreases to 0.7metres at the western end of the training at J. Interestingly even the offshore location at "A", which is clear of the headland only shows a 100 year ARI (1% probability of occurrence each year) of 6.3 metres which is substantially less than the extreme value analysis figure of 8.5 metres and the Waverider measurements.

There is however a further important matter that being whether the 50 and 100 ARI are valid design criteria. There is an important difference between the reasonably expected life and the design life. If the anticipated or expected life is say 40 to 60 years then a far more stringent "design life criteria" are required to achieve that expectation (see Gordon et al 2019). Further the waves that can reach the breakwater are dependent on the seabed levels at the time so scour of the entrance bar is an important consideration. The depth limit at which waves break is determined, to a significant degree, of the condition of the bar and surrounding seabed at the time.

The damage the head of the Southern breakwater has demonstrably suffered in the past and the distribution of the wave heights along the trunk makes the SWAN model results "barley credible". These relatively small design wave heights at, and near, the head and the inability of the model to differentiate between plunging and surging breakers suggest that Haskoning's own reservations regarding the SWAN model results are justified.

A further complication in using numerical models like SWAN is that in entrance configurations such as at Port Macquarie, flood tides cause wave penetration to be greater with the waves tending to exhibit a spilling type of break whereas penetration on an ebb tide is less with increased wave steepness and plunging type break characteristics. Given the importance of getting the wave conditions for the assessment and potential design of the armour correct, it would be justified to check the results using a physical model. Armour size is proportional to the wave height cubed so even a small underestimate of wave height can have significant implications for the size of armour necessary for “stability”.

Water flow (currents) along the breakwater/training wall is also a design parameter for determining the required rock size for the armour, particularly in the western region of the wall where wave influences are minimal. That is, the upstream area which is effectively beyond the reach of the ocean waves because of the sheltering of the North breakwater. The local wind waves and boat wakes are likely to have little impact on amour stability given the size of the armour already present in this region.

Haskoning’s analysis of existing flood studies led to them adopting of the following flood flow velocities, which, based on experience seem reasonable however whether the 50 or 100 year ARI is the appropriate design criteria requires further consideration of encounter probability as will be discussed further in following sections of this report. The Haskoning’s adopted current velocities for flood flows (which are in keeping with experience elsewhere) are:

- 3.5m/s for a 50 year ARI event
- 4.3m/s for a 100 year ARI event

In addition, Haskoning adopted the MHL determination that for tidal flows the Spring ebb tide and flood tide currents are 1.65m/s approximately 0.55km upstream from the ocean end of the southern breakwater, so the flood flow currents provide the design criteria.

Haskoning provides the following table for armour sizes required to be stable under flow conditions of 4.3m/s:

Design Parameters			50 year ARI	100 year ARI	50 year ARI	100 year ARI	
			Straight Reach		Transition/Outer Bend		
Flood Velocity				4.3 m/s		4.3 m/s	
		M50	60	210	500	1700	No reduction for damage
		Dn50	290	430	570	870	
	Escarameia and May	M50	330	1140	1980	6800	No reduction for damage
		Dn50	500	760	910	1380	

It should be noted that the Escarameia and May references, results that were based on highly turbulent flow downstream of structures such as weirs, are questionably applicable to the

Port Macquarie situation. However, the table provides an indication of the design rock sizes that could apply to the armour in the western zone of the structure, the training wall section.

3. ROYAL HASKONING DHV UPGRADE DESIGN REPORT (Designated "Final")

A significant portion of this report is concerned with locating suitable quarries for stone supply. While this is an important aspect of the overall project it is not necessary to be analysed as part of this current report. As Haskoning points out, if adequate rock is not available then artificial concrete armour can be substituted. Further, comment in the Haskoning report on other issues such as landscaping, and tree removal and replanting is not undertaken as this current report is focussed on the proposed repair or upgrading of the breakwater/training wall. Constructability is another issue not dealt with further in this report as Haskoning has provided some traditional alternatives. However, it is pointed out that there is other more innovative light weight, highly manoeuvrable, machinery.

It is important to recognise that there appears to be two conflicting approaches to the management of the breakwater given that, while the emphasis for management of the trunk focusses on an "upgrade" philosophy, the management of the head takes a more conventional "repair" approach. There is no apparent explanation as to why these two different approaches have been adopted for different sections of the breakwater. Further, the upgrade of the trunk seems to assume the protection of the southern wall by the northern wall will continue with no comment on what may be required to ensure the northern wall remains fit-for-purpose.

There also seems to be an underlying issue of understanding between the historical design philosophy under which the breakwater/training wall was constructed and maintained as a "living structure" and more modern design approach of a stable structure requiring minimal maintenance, **provided that its design wave conditions are not exceeded**. Hence the "upgrade" of the trunk seems to be aimed at converting the historical based maintenance management approach to a more contemporary capitally funded structure. Both options having their strengths and weaknesses.

There is a valid question in regard to the robustness of the determination of the wave climate along the breakwater. As has been pointed out experience dictates there are reservations regarding the use of "desktop" numerical modelling alone for both wave design considerations and the ability to recognise the different regimes of plunging breakers verses surging breakers on the structure. Over and above this is the apparent focus on 1% and 2 % events without examining the outcomes of such design criteria. While it may seem these are rare events, in practice, over the expected life of the structure there is a concerning probability of them being equalled or exceeded and given the relatively vulnerable nature of the proposed "new" design it is critical that there not be confusion between the anticipated life and the design life. The design life has to be considerably longer if the life the structure is reasonably expected to achieve say 40 to 60 years (see Gordon et al, 2019).

Under the heading “Opportunities” Haskoning states:

1. “The nearby beaches and entrance bar are approximately in equilibrium and sand supply from the rivers and from beaches to the south are expected to ensure that the beaches remain in equilibrium in the foreseeable future. It is therefore unlikely that any works to the training wall would alter coastal processes or sediment transport rates.
2. It is assumed that the entrance channel and beach near the southern training wall would have scoured to the design scour depth at least once since construction of the training wall. This would have resulted in reshaping of the toe of the training wall. Placing rock armour on top of the existing rock would ensure that the structure is founded at the design scour depth. Furthermore, in response to sea level rise, the entrance to the Hastings River is expected to shoal at a similar rate to sea level rise to remain in equilibrium. The scour level is therefore expected to reduce (i.e. become shallower) over time.
3. The design wave height at the structure has been refined and reduced by undertaking numerical wave modelling using a detailed SWAN model that considers freshwater and ebb tidal flows, and resolves complex nearshore bathymetry. The model assists in design optimisation including spatially armour requirements and overtopping risk.
4. There is available space landward of the breakwater head, to locate a viewing platform for views of Town Beach and across the entrance. This does not require crest widening or conflict with the crest accessway.
5. The head of the breakwater is in reasonable condition having been recently repaired. At this point in time, minimal works are considered to be required near the head of the breakwater. “

In regard to point 1 there are conflicting views on the sand movements in this region of the NSW coast, however that the proposed works are unlikely to alter coastal processes in regard to sediment movements, is considered reasonable.

Point 2 seems confused. The historical evidence indicated the scour has been down to -10 metres and hence the toe would have scoured, and the toe rocks moved to a greater depth than the proposed “design depth”. So, the argument that the toe needs further work to ensure the structure is founded at the design depth is difficult to understand. The sea level rise argument that the scour depth will be less over time actually means that the existing toe is likely to be more than adequate as the scour will be to a lesser RL in similar events. So, the issues to do with the toe seem inconsistent and poorly thought through or presented.

Point 3 indicates that the design is still relying on the SWAN model rather than recognising its limitations in the complex situation prevailing at Port Macquarie. The issue being that armour size is a function of wave height cubed and hence it is critical the wave conditions the structure is exposed to are confidently established, not just for events such as the 1% storm. This also applies to wave overtopping considerations. The particular issue at the entrance is that the offshore bar can be scoured during major storm events and hence the incident wave height may be greater than otherwise established. So, again a hybrid modelling approach which included physical modelling which could test different seabed configurations would be justified in finalising the design.

Point 4 provides an alternative for a viewing platform which doesn't involve construction on the breakwater/training wall, which seems a sensible way forward if a viewing platform is desired. Note that the concept of a fishing platform is not mentioned.

Point 5 seems counterintuitive and confused given the historical damage to the head and the likelihood that the currently repaired head may face similar problems in the future. That is, whereas the upgrade of the breakwater/training wall proposals look to producing a contemporary result designed to require little or no maintenance for many decades, the comment in point 5 seems contrary to the comments in the following section "Breakwater Head" which indicate the head requires some repair, in the traditional form. Interestingly there doesn't seem to be an engineering analysis as to what is required for the head to be considered stable for the proposed design life philosophy being applied to the rest of the structure. In regard to the head Haskoning states:

- "Typically, recent (2015) repairs to the breakwater head appear to have been effective in the profiles either side of the breakwater and were still in good condition.
- However, specific areas of damage were identified that included the movement and displacement of **individual amour units** from the primary armour layer such that voids in the layer have opened and/or oversteepening of the layer has occurred producing areas of instability in the structure. individual armour units are proposed to be placed where voids have opened and oversteepening has occurred due to armour rock movements since the previous remediation works
- Failure to remediate this damage will leave the breakwater head vulnerable to progressive damage over time and potential failure; and
- The placement of the individual armour units at the head is required to restore the breakwater profile and armour thickness, to the previously installed extents which satisfy the overall design intent of the structure. "

The issue of the breakwater head becomes more contradictory under "Further Design Development" where Haskoning states:

"It is recommended to undertake minimal works to the breakwater head in this campaign given the recent (2014) maintenance work which is considered still in good condition. The proposed work will include top up of individual armour units where localised damage has been identified. It is noted that it is unlikely that the recent maintenance works in 2014 included suitable filtration design and/or geotextile filter fabric installation and the head area will remain susceptible to the loss of fines, and subsequent settlement over time. This would remain a project risk. However, given the age of the structure and the high energy nature of the head location it is considered that loss of fines would have stabilised to a degree in equilibrium with the hydraulic conditions. Accordingly, further loss will be a relatively slow process that may be manageable through standard ongoing maintenance regimes".

So, it would seem that unlike the trunk the head is to be managed by the traditional methods of repair rather than the upgrade to the more contemporary philosophy being applied to the trunk. Further, it would seem that unlike statements in other areas of the report there is a recognition that the age of the structure is such that "it is considered that loss of fines would have stabilised to a degree in equilibrium with the hydraulic conditions. Accordingly, further loss will be a relatively slow process that may be manageable through standard ongoing maintenance regimes". It would appear different people may have written different sections of this report.

Under the heading "Constraints" Haskoning states that:

1. "The southern training wall is identified as a heritage item. Additional consultation is recommended.
2. There are spatial constraints along the training wall, between the training wall and the caravan park, which limits the space available to widen the footpath.
3. Without undertaking significant repair works, ongoing geotechnical instability is expected, which would limit the feasibility of founding ancillary structures on the breakwater/training wall. Furthermore, rock structures are intended to be 'flexible' and regrade in response to a storm event. Installing rigid structures, such as a fishing platform, near the training wall is not recommended.
4. There is a risk of overtopping of the breakwater and training wall, particularly near the head of the training wall. However, due to the depth limited nature of the site, dangerous conditions that result in overtopping of the breakwater are usually self-evident and rogue waves are not expected.
5. There is limited access to the site to undertake construction. The crest of the training wall is relatively narrow, which limits space available for construction plant and equipment. The site is accessible through the NRMA Macquarie Breakwall Holiday Park and the reserve at the eastern end of the site. Access for trucks at either location would be limited. "

Point 1 requires further expansion as to what is considered the heritage aspect. What is the role of the memorial rocks? And what aspects of stability need to be considered in respect to them? Or is the heritage issue more to do with the type of traditional rubble rock construction employed in the building of the breakwater? If this is considered significant then it could rule out any of the upgrade proposals as the breakwater/training wall was clearly conceived to be a flexible structure that would require "repair" from time to time.

Presumably point 2 relates to the potential removal of trees.

Point 3 yet again seems to be dealing with the viewing/fishing platform concept but at the same time makes the key point that the breakwater is a flexible structure that would provide poor foundations for a fixed structure. But this would also seem to accept that the proposed upgrades would not achieve a stable structure.

Point 5 recognises the issues concerning limitations on construction if a conventional approach were adopted.

In regard to the trunk Haskoning puts forward 3 options ranging from a "top up of the rock armour with suitable sized material" although the suggested cross section seems to involve more than a simple topping up of areas which require "repair". Importantly, because of the extent of armour bolstering Haskoning states:

"To allow Option A to be completed, it would be necessary to widen the crest of the existing structure to provide a suitable platform for an excavator to place new rock armour. It is likely that the footpath would be damage during construction and would require complete replacement". There is no comment on the potential destabilising impact on the structure. Throughout the report it seems to be assumed the "mound" faced by the breakwater is a stable homogeneous structure whereas, given the construction method involved when the breakwater was first built it is more likely the "mound" backing the exposed rock slope of the trunk is to some degree quarry material.

While Option A would improve hydrodynamic stability of the armour layer by adding suitable sized armour to the outer layer, it would not address the underlying cause of instability due to inadequate filtration and subsequent loss of fines."

So, even the proposed option A envisages the need to widen the crest and accept that the existing crest is likely to suffer significant damage. Further the second comment about an underling issue of loss of fines appears to reflect a possible lack of understanding of the make-up and performance of this type of breakwater, particularly after more than 100 years of exposure to waves and currents, not to mention the comment above by Haskoning regarding the loss of fines most likely no longer be an issue. Again, the report seems to have been written by more than one person, with more than one understanding of how these structures behave.

Option B is advanced as a "partial reconstruction" but could be seen as an attempt to retrofit a contemporary design to an existing traditional design. It can therefore be expected to result in considerable disturbance and the formation of a a discontinuity within the structure.

Option C is in reality a major rebuild to convert the breakwater from its traditional form and behaviour to a contemporary design which includes filter cloth. Experience dictates this means a major regrading of the underlying material to achieve "a robust and compliant structure that would achieve a 40 year design life with ongoing maintenance and repairs". The problem being that the current surface armour has a great variation in size and shape hence major deconstruction would be necessary to create a workably even surface on which to start the reconstruction.

All of this presumes the wave conditions (plunging and surging) and the wave heights have been accurately determined. To this end it is noted the calculated mass for rock armour on the western zone of the structure appears to be 6.3 T. Interestingly the MHL 1994 report calculated the armour size required for the same region of the breakwater as being 30T.The MHL figure was based on an incident wave height of 6.7m which is possibly very conservative however it did take into account offshore bar scouring and hence the potential for larger waves to attack the structure. Given Haskoning shows a "design scour level" of -7.75 metres it would seem waves larger than they have chosen for their design could reach the structure. Again, a reassessment of the design wave conditions using hybrid modelling would seem prudent and in keeping with recommended international practice.

Finally, Haskoning recommends a hybrid design that incorporates option B and C involving "reusing armour material (where appropriately sized) from the upper portion of the existing breakwater for placement in the lower portion of the breakwater, prior to a full reconstruction of the profile in the upper portion." But again, despite recognising the toe has probably stabilised seems to assume the historical scour and toe collapse into the scour has been insufficient so includes a falling or "Dutch" toe constructed above existing bed levels, regardless of the historical information that indicates the bed has naturally scoured to -10m and the toe has had plenty of opportunities to consolidate at greater depths as a result of wave and flood events over more than 100 years. So, it would seem the proposed "Dutch" toe is required in the design, not to support the existing structure but rather to support the proposed modifications to the lower section of the main slope. That is, there seems to be an implied concern that the recommended repairs/upgrading will create an unstable slope that will need new support in the region of the toe.

Haskoning makes the following comment which seems to demonstrate an unfamiliarity with this type of breakwater, its historical behaviour and the previous 1994 MHL report:

"The filtration design of the existing structure that is over 100 years old is unknown and cannot feasible be deduced. Optimised design described in **Section 6.4.2** requires acceptance of previous filtration design at the breakwater head and to a lesser extent in the lower profile of the breakwater trunk area. It is considered, due to the age of the structure, that loss of fines would have stabilised to a degree in equilibrium with the hydraulic conditions and further loss will be a relatively slow process. "

The recommendation that the provision of a viewing and/or fishing platform on the face of the trunk be not taken further is a well-supported proposition given it would potentially involve placing a fixed structure on a flexible foundation. However, it seems inconsistent that after having recommended against it Haskoning provides a concept design drawing for a structure cantilevered off the crest which, although engineeringly possible could invite members of the community into a potentially vulnerable situation given the exposed location and relatively low level of the platform (shown in the Haskoning diagram as being lower than the breakwater crest).

4. TRANSPORT FOR NSW SUBMISSION REPORT

The TfNSW submissions report is aimed at presenting and discussing the submissions received from the public consultation and the provided TfNSW responses. MIDO states:

"The high number of submissions received demonstrates that the Port Macquarie Southern Breakwall, a state maritime infrastructure asset, has multiple community uses and has become an iconic and loved treasure for the community. In addition to its intended maritime and navigational purpose, it provides many benefits including tourism, economic, health and community identity."

It also indicated that there is a "$5M funding allocation for the Port Macquarie Southern Breakwater Upgrade Project under the Maritime Stimulus package. While the submissions and responses are well detailed the most useful information in regard to the reasons for the Upgrade Project are contained in the executive summary:

"Transport for NSW's (Transport's) is proposing to upgrade the southern breakwall in Port Macquarie to ensure its long-term structural stability as a critical maritime asset.

Transport's primary objective, as manager of the breakwall, is to undertake the necessary corrective maintenance of the structure to:

- "repair damage sustained over years of exposure to the water flows in the Hastings River entrance
- upgrade the structure to contemporary design standards and industry best practice
- extend the life of the asset, ensuring it continues to serve its primary purpose.

  The secondary objective of the proposal has been to consider and incorporate where possible, the additional features of the foreshore walk along the southern breakwall as proposed and adopted by Port Macquarie Hastings Council's *Town Centre Master Plan 2014* and *Breakwall Concept Plan 2016.* The proposal for this project is as follows:
- rebuilding 600 metres of the breakwall along the river section
- completing maintenance of the breakwall head with no change to its footprint
- installing a new five-metre-wide footpath along most of the structure's length. This excludes the last 30 metres at the breakwall head, so the current footprint and width will stay the same

- installing two new stair access points and a new access ramp at the seaward end of the NRMA Port Macquarie Breakwall Holiday Park (NRMA Holiday Park)
- installing safety fencing along the southern side of path
- installing a kerb along the northern side of the path
- installing new lighting on the shared path
- installing six seating areas
- landscaping the area"

These requirements were presented by Haskoning in a somewhat different form as:

"MIDO's objectives and functional requirements with consideration to budgetary constraints are:

Remediate training wall to address:

- toe scour; and,
- movement/displacement of rock armour.

Upgrade training wall with consideration of climate change impacts (including sea level rise).

Improve recreational land use including:

- widen the footpath along the crest of the breakwater and training wall structures;
- provide access ways from the road to the footpath; and,
- consider numerous ancillary structures such as fishing platforms, lighting and shelters as proposed in the Breakwater Master Plan prepared by Port Macquarie-Hastings City Council."

Hence there seems to be a somewhat different interpretation of the MIDO objectives.

The MIDO response to issues regarding the widening of the pathway along the crest is a response to community concerns without necessarily recognising the impact on the Memorial rocks. However it defers, provided by Haskoning as being required, for the facilitation of the movement of construction equipment along the crest as required to upgrade the breakwater/training wall. This however seems to be recognised elsewhere in the MIDO report, under the trees issue, where MIDO acknowledges that the widening of the pathway will result in the loss of 29 trees because:

"The construction method for the major rebuild section of the breakwall will affect the tree protection zone (roots) and will impact the health of the trees and their long-term survivability." MIDO does state that replanting is to take place to offset these losses. However, it is likely to take many years before even "advanced" plantings re-establish the current environment.

But specifically, in regard to the pathway widening MIDO states:

“The Port Macquarie Southern Breakwall is the second most trafficked breakwall path in New South Wales. Population and tourism growth predicted for the area may see a significant increase in the numbers of path users.

Widening the concrete path will improve safety for all users, as it alleviates the existing high probability of conflict between various path users such as walkers, runners, cyclists, dog walkers, roller bladers, skaters, and those with prams or wheelchairs.

- In line with Austroads *Guide to Road Design* which recommends a minimum width of five metres for a path that is used for shared recreational purposes.
- Widening the path will improve conditions for people with mobility issues, making the path more accessible to more users.
- Increasing the width of the footpath is in line with Council’s vision for the area as outlined in their *Town Centre Master Plan 2014* and the *Breakwall Concept Plan 2016*
- Provides a seamless connection with the recently refurbished Town Green precinct, which also features a 5-metre-wide footpath.”

Which does not mention the upgrade construction purposes issue. This raises the important and relevant question as to whether the “upgrade” of the breakwater itself can be undertaken without the need to widen the path. Experience dictates the answer is basically any upgrade can take place without the need for widening the path if that work is undertaken from the water by equipment mounted on barges. This would also minimise the disruption to the on-going community use of the path along the crest and to the occupants of the tourist park. However, experience also dictates this is usually a more expensive exercise, in part because it is more weather dependent and challenging in regard to the movement of the rocks to site.

It would seem the emphasis regarding toe scour emanated from Haskoning as did the comment on climate change. It is not apparent how they concluded climate change impacts (including sea level rise) would be significant in the relatively short “design” life of 40 to 60 years as mentioned in the documents. Based on current scenarios for sea level rise associated with climate change, the likely impacts over the relatively short “design life” could be expected to be minor as compared to recent findings of the effects of coastally trapped and infragravity waves, neither of which are mentioned, nor taken into account in the designs. However, the most difficult issue regarding climate change is the potential for intensification of storm events and their possible increase in frequency. While there is some speculation regarding the climate change impacts associated with storms there are not yet even well supported scenarios. Hence with the lack of information, the only reasonable response is to undertake some sensitivity testing of design parameters. This will simply provide an indication of how critical, or not, the assumptions are to the final design outcome. Such an analysis is not apparent in the available information.

Clearly the brief to Haskoning had a significant emphasis on recreational land use, not simply the condition and structural management of the breakwater/training wall.

Finally, MIDO clearly does not recognise/understand the important sheltering the Northern breakwater provides for the Southern wall and so has not included in the Upgrade Project an assessment and necessary actions regarding management of the vulnerable North wall.

COMMENTS

The on-site inspections of the Southern breakwater/training wall by MHL and Haskoning were undertaken a little over 4 months apart. Both inspections came to basically the same conclusion that the trunk of the structure was generally in reasonable condition, but the head is/was vulnerable. It is interesting to compare, and contrast, the recent assessments with those of MHL in 1993 (MHL 1994) some 30 years ago. In doing so it is apparent the breakwater/training wall has withstood the test of time with surprisingly little degradation.

There are two basic differences between the two assessment reports. The first is that while MIDO engaged MHL to advise on the condition and hence **repairs** that may be necessary following recent storms and floods, it engaged Haskoning to recommend remediation and **upgrading** of the structure along with **"improved recreational land use"** in line with Council's "Breakwater Master Plan". So this introduced additional matters, such as crest widening, removal of trees and removal/alterations to the "memorial stones" that have become a cultural feature of the crest of the breakwater/training wall structure.

The second difference is that while MHL assessed the conditions of both the Southern and Northern breakwaters, Haskoning has only focused on the Southern wall, as would seem to have been their brief from MIDO. The importance of this difference being that the stability and future requirements for maintenance/upgrading of the Southern wall are in part dependent on the sheltering from ocean waves the Northern wall provides to the Southern wall. Importantly MHL pointed out that the Northern wall, and in particular the head, required some immediate attention.

It is useful to consider the Southern breakwater/training wall as having four different zones in accordance with the hydrodynamic forces to which these zones are exposed: Western, Central, Eastern and Head. Different design and stability criteria need to be applied to each zone. The Western zone is the region where the forces the wall must cope with are dominated by tidal and flood water velocities. The Central zone is a transition region between the Western and Eastern zone as the wall becomes more exposed to wave action. At its eastern extremity it experiences similar forces as the Eastern Zone while at its Western end the forces are mainly tidal, and flood induced. Moving Westward from the Eastern zone the wave attack not only decreases but also transforms from high energy plunging breakers to lower energy surging breakers, but increasingly at an angle to the rock slope. The Eastern zone is where the principal hydrodynamic forces are the result of wave attack and in particular plunging wave breaking on the structure. Finally, there is the head, which is subjected to attack from plunging breakers that, because of the rounded shape of the head and the generally less well interlocking of the armour, attempt to strip the rocks off. This is the region of the most severe wave attack and hence usually the zone most prone to damage.

The focus on 1% (also misleadingly referred to as 1 in 100) and 2% (similarly misleadingly referred to as 1 in 50) as design conditions for an upgraded breakwater/training actually means there is a reasonable probability the structure will fail during say a 40 or 60-year life. That is, if, and when the design criteria are exceeded, there could be an unacceptable outcome. In recent times it has been recognised that in order to ensure a structure like a breakwater or revetment has a reasonable probability of achieving its “expected” life of, say 60 years, a far more stringent design criterion is required, as discussed by Gordon et al (2019). That is, there is an important difference between the “design life conditions” used for calculations and the potential resulting “expected life”. The matter of concern being that the proposed contemporary design relies on the layers of outer armour for stability and if the design conditions are exceeded and the outer armour is stripped away at say around water level, which is the usual area most vulnerable to failure, then the much lighter underlay suffers rapid degradation and rapid collapse of the structure can ensue.

Overall, the structure is also exposed to both boat waves and to locally generated wind waves, especially from the west during winter. However, both of these additional wave influences are likely to be small as compared to the other forces at play and therefore have an insignificant influence on the design of the structure.

The western and central zones could most likely be repaired without the need to widen the crest pathway, if repairs were targeted to particular areas and were undertaken from a water-based construction spudded barge, or using light weight articulated equipment like Spider walking excavators fitted with a grab, on the crest and again targeting the specific areas where additional rocks are required. This would mean that the existing trees could be, in the main, retained. However, given the state of the crest path in the eastern zone and the need for the provision of heavier head armour consideration should be given to the repair/upgrade of this section of the breakwater utilising conventional heavy machinery.

The confusion which appears to be inherent in the various documents in regard to the state of the breakwater, the objectives and priorities of recommended actions have resulted in suggestions that some arguments for an **upgrade** rather than a **repair** project seem to have been retrofitted.

A key issue that appears to have been overlooked, and is not even mentioned, is the Coastal Management Act and the associated Coastal SEPP (which is now incorporated into the Resilience and Hazard SEPP). As the breakwater/training wall is within the defined coastal zone compliance with the Act should have been a matter of consideration. In particular:

Coastal protection works by public authority

Development for the purpose of coastal protection works may be carried out on land to which this Policy applies by or on behalf of a public authority:

(a) without development consent—if the coastal protection works are:

1. (i) identified in the relevant certified coastal management program, or
2. (ii) beach nourishment, or
3. (iii) the placing of sandbags for a period of not more than 90 days, or

4. (iv) routine maintenance works or repairs to any existing coastal protection works, or

(b) with development consent—in any other case.

That is, routine maintenance works or repairs are allowed under the Act/SEPP without consent, however based on court findings at Byron Bay, an **upgrade** would not be covered unless there is a certified Coastal Management Program which specifically includes the upgrade project. Advice from Council is that while they have a certified Coastal Zone Management Plan it does not include this current proposed project. Hence a Development Application would be required for anything other than routine maintenance.

CONCLUSIONS

MIDO points out its responsibilities for maintaining maritime assets to ensure their long-term structural stability, particularly in regard to navigation of the entrance. It would seem however that there are a couple of significant underlying influences behind MIDO's proposed upgrade of the South breakwater and its immediate surrounds. The first is the Port Macquarie Hastings Council's "*Town Centre Master Plan 2014* and *Breakwall Concept Plan 2016"* (which is currently not readily available). Undoubtedly this plan went through a normal process of public exhibition at the time, however unfortunately, as is the experience of many councils, the quantity of matters requiring public consultation suffer from the growing weariness of the community to participate in the process. The MIDO's Submissions Report summarises a more recent attempt at community engagement however again, from the numbers of respondents listed in the report as against the numbers present at recent protests, it would suggest community participation may have been less than desirable.

The second underlying driver for an upgrade rather than a maintenance/repair solution is State Treasury's preference for projects over programs. That is, there is a State Government reluctance to provide on-going funds for repairs to breakwaters rather than a single allocation for projects that upgrade assets so, during their foreseeable "life" they do not require further expenditure. However, this is normally examined by a cost benefit analysis (CBA) to determine whether a project or program approach is the most efficacious. At this point in time such an analysis was not readily available.

Both MHL and Haskoning surmise that the trunk of the breakwater/training wall is, based on their inspections, in reasonable condition however both point to the on-going vulnerability of the head of the Southern Breakwater, although it seems it is in a "reasonable" state due to recent repairs. MHL indicates the head of the Northern breakwater requires attention as do several locations along the trunk. It would seem that the principal difference between the MHL and the Haskoning reports is that MHL was engaged to examine the condition of the breakwater and recommend any necessary repairs whereas the focus of the Haskoning engagement was on upgrading the breakwater so that future maintenance requirements would theoretically be minimised.

The design wave conditions for determination of the size of rocks needed for repair and/or upgrade is critical for the longevity of the structure and its ability to achieve an "expected life". Given it is a cubic relationship between wave height and rock weight even a small difference in the selected design wave criterion can have very significant impacts on the outcome. There appears to be a number of matters that it would be prudent to reassess. These include a more robust modelling (numerical and physical) of the wave climate the breakwater is/will be exposed to including at what point the wave attack changes from plunging to surging breakers, and an evaluation as to what wave criteria should be chosen to best achieve the desired expected life by taking into account the concept of "encounter probability". That is, determining an acceptable encounter probability to determine the ARI required for the achievement of the desired project life.

A number of the ancillary works proposed in the "upgrade" are linked to increasing the crest width to accommodate aspirational recreational usage. Unfortunately, it would seem widening the crest has implications for the fate of many of the "memorial" stones and 29 trees to the south of the wall. There is not a coastal engineering basis to argue that crest widening is necessary for the stability of the breakwater/training wall trunk or head. The link to the stability of the breakwater/training wall is that a widened crest might make it easier for heavy machinery to traverse the crest and hence undertake any necessary repair/upgrade of the trunk slope. However, the necessary partial use of the current concrete path, which is believed to be unreinforced, by heavy machinery, is likely to result in its integrity being compromised.

Haskoning has proposed alternatives to the use of heavy machinery on the crest and experience dictates there are further options as well. So, the argument for an increase in crest width should be seen as one to fulfill recreational aspirations, not necessarily breakwater/raining wall stability requirements nor essential for targeted repairs to the trunk. This current report does not seek to comment in any detail on the aspirational recreational and landscape matters, as they are not coastal engineering matters, other than to recognise their role as a "driver" in the decision-making process.

The key "sheltering" benefits of the Northern breakwater for the stability of the Southern breakwater and hence the need to ensure the Northern breakwater is maintained in a fit-for-purpose condition appears to have not been appreciated and hence has been overlooked.

Consideration be given to developing a management strategy for the breakwater/training wall that recognises there are effectively four different zones along the length of the breakwater/training wall. This would enable a different approach to be adopted that reflected the different hydrodynamic conditions prevailing in each zone and hence the armour sizes required to address the differing stability requirements of each zone.

RECOMMENDATIONS

In line with the findings of the MHL inspection it is recommended that a detailed assessment of the Northern breakwater is urgently undertaken. In particular, the condition of its head with the aim of identifying any repair/upgrade that might be necessary to maintain the wall in a fit-for-purpose state. The aim being to ensure that it can continue in its role as providing a degree of "shelter" to the Southern breakwater.

For the purpose of refining the management regime the Southern breakwater be considered as having 4 zones and undertake both numerical and physical modelling for each zone to accurately determine both the wave height and breaker type to be applied so as to develop strategies to best manage each zone.

It should also be recognised that to determine the design wave conditions to be resisted in order to provide a "reasonable" probability of the structure reaching its expected life requires an encounter probability type analysis to ensure that the level of damage over the proposed life is acceptable. A traditional use of criteria such as 1 in 100 or 1 in 50 alone is no longer considered reasonable practice for structures such as the Southern Port Macquarie breakwater.

Further investigation would be prudent of the potential for using non-conventional construction equipment instead of heavy conventional excavators on the crest and the river face of the trunk, like a "Spider walking excavator" equipped with a grab instead of a bucket for working on the crest and face of the breakwater/training wall trunk in carrying out repairs.

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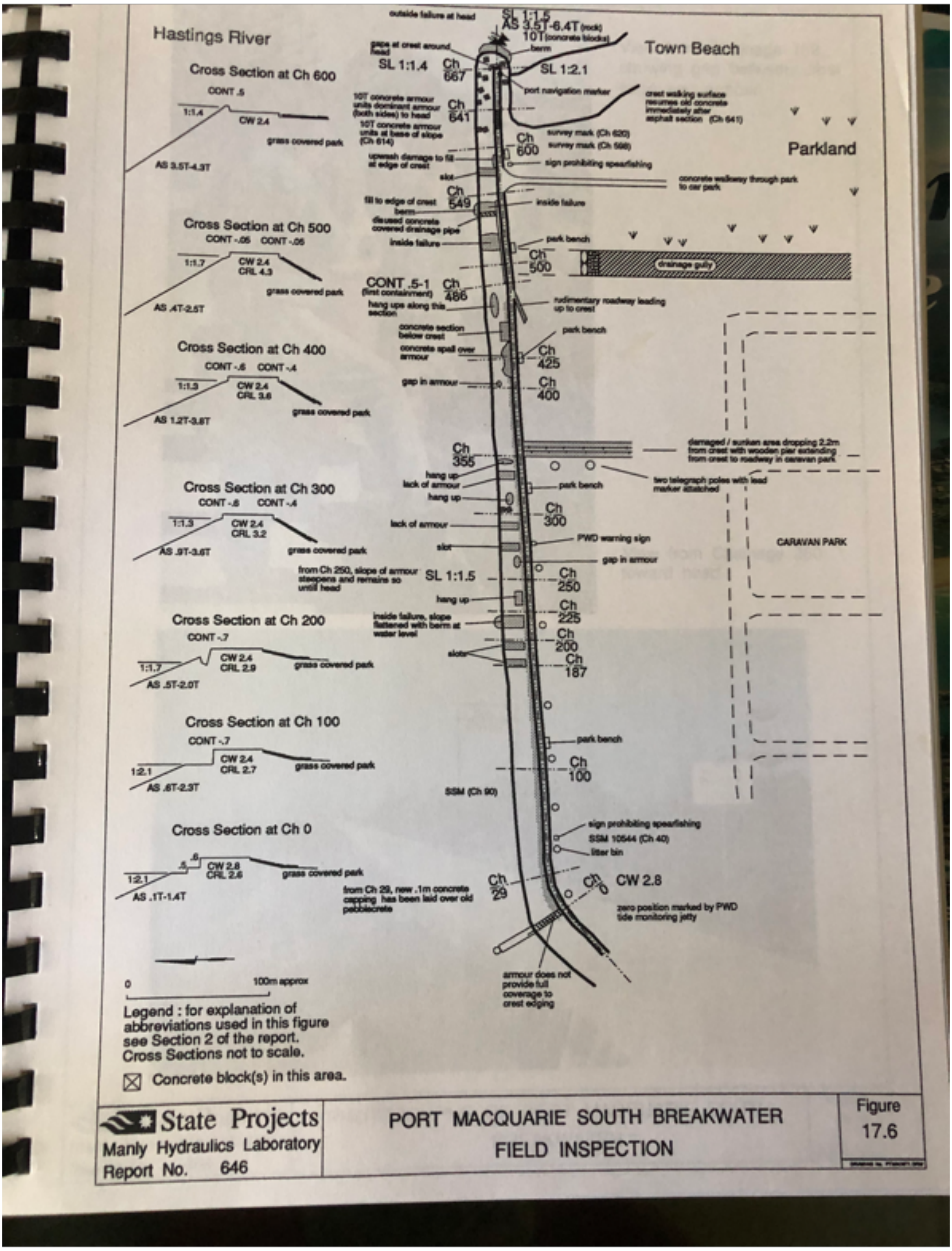
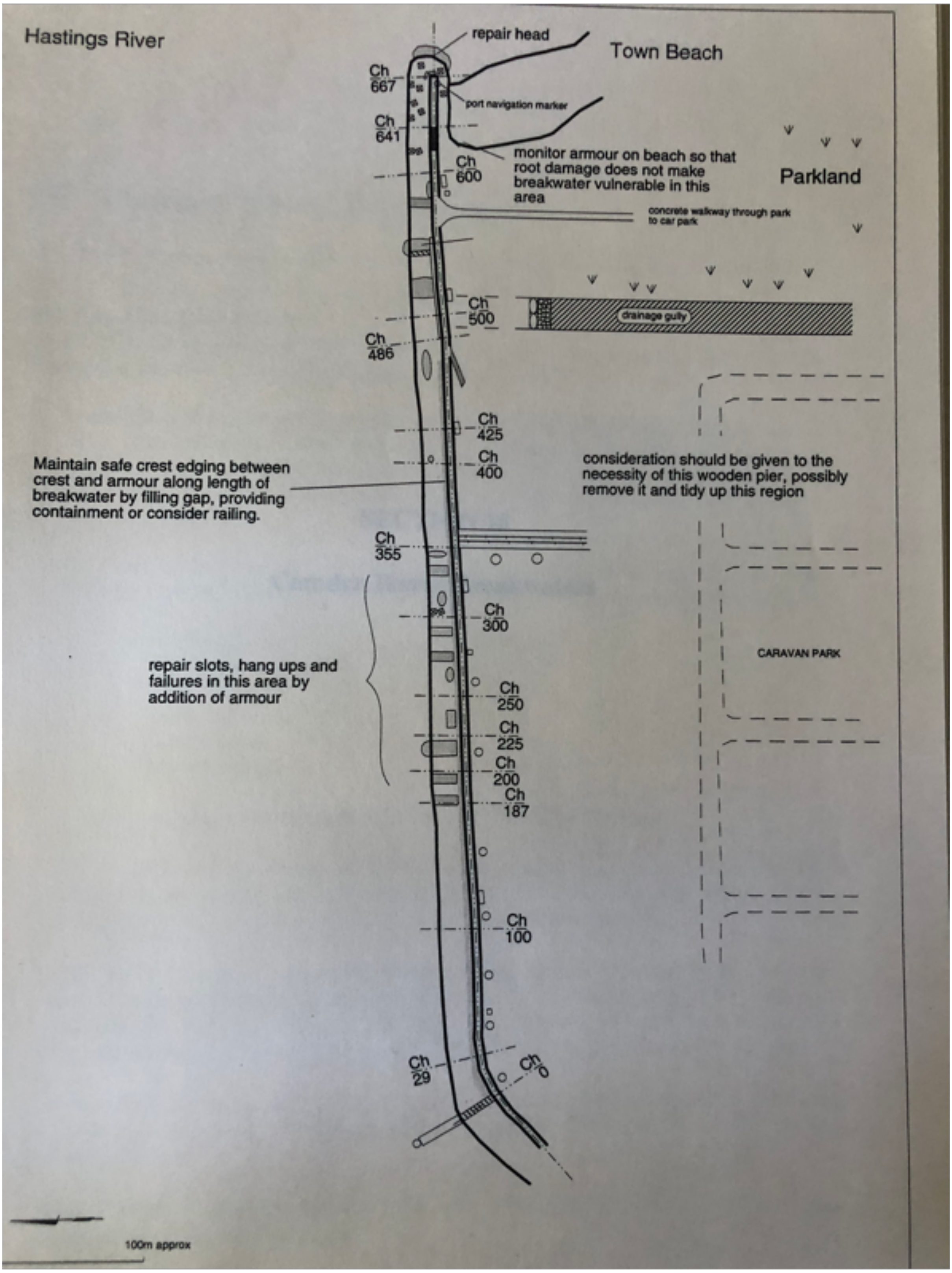
APPENDIX A - MHL 1993 Asset Appraisal and repair recommendations for Southern Wall
(this summary is supported by 10 pages of detail)





It is important to note that in the MHL appraisal report the difference between the areas identified in the field inspection as against the relatively fewer areas considered to need attention particularly between chainages 187 and 365 and from 500 to the head.

APPENDIX B – Field inspection 8th and 9th February 2023

Initial formal inspection and notes: made on the afternoon of the 8th starting at 1200hrs – Tide Low @ 0440 hrs 0.26m. Tide high 1105hrs @ 1.36m
Repeat Inspection and notes: made on morning of 9th starting at 0930hrs – Tide Low @ 0518hrs. Tide high 1139hrs @1.29m

General notes:

Small tidal range both days and although an ebb tide for full duration of inspection on 8th, water surprisingly clear. On 9th mainly flood tide dominated with exceptionally clear water.

Distances (Chainages) all measured with a calibrated measuring wheel. Zero chainage was taken at the same point as the MHL 1993 zero on the breakwater where the tide gauge is located (near western end of training wall). Slopes were measured using an inclinometer.

Notes take in a visual assessment of rocks sizes based on experience and from observation of the sizes of the obviously stable rocks in the near vicinity, as there was concern that the wave height information available along the length of the breakwater/training wall is not adequate and therefore precluded rock size calculations.

The inspection was undertaken in three passes along the length of the breakwater/training wall. The first pass was for familiarisation, the second was one of measurement and professional assessment, and the third was a repeat of the second on the following day to check repeatability of the observations. The results represent reasonable estimates aimed at providing a supportable indication of appraisal of the breakwater condition and an indication of the repairs that may be required or are considered desirable. The detail provided is aimed at focussing future monitoring and/or a more detailed assessment.

Where it is noted that a small number of rocks are missing and/or required, it is generally aimed at even out the slope or support the material further up the slope so as to reduce the safety hazard for people choosing to clamber around the slope. That is, they do not necessarily imply that work is required in that area to ensure the breakwater/training wall needs action to prevent serious structural failure.

Every reasonable care was taken to detail observed issues. Where it is felt, from this initial assessment, the stability of the slope may be compromised in the near future an asterisk (*) is provided to direct attention to the chainage. A double asterisk is shown where it is felt early attention should first be focussed. There are some 113 comments however only 30 have an asterisk indicating early targeted attention desirable and a further 13 with double asterisk indicating priority of attention. The majority of the remaining comments are aimed at potentially reducing risk for members of the community who chose to clamber on the rocky slope.

Chainage		**Comment**
0		At base of tide gauge (as per MHL 1993). Slope in good condition, 1 in 1.5, moderate to small rocks.
10		1 rock missing mid slope, 1 rock missing near waterline
31		2 rocks missing at waterline and mid slope, material in the water. There has clearly been a past slip in this area. But only needs another rock to the immediate east at waterline level
57		Slope a bit flatter. It looks as though a couple of rocks near the base of the slope have been dislodged. Need for 2 reasonable sized rocks
61		Slope generally 1 in 1.5 , need for 2 rocks
63		2 rocks required mid slope
68-70	*	Slumped area, mid slope has failed, and 3 or 4 rocks are required. The cap rocks have clearly started to move out because of that lack of support
73	*	Slump in the lower part of the slope. The upper part of the slope is hung up and there is a need for 3 to 4 rocks in the lower part of the slope down to below water line
77	*	The cap rock and upper third of the slope is hung up, the bottom part of the slope has slipped. There are a couple of large rocks out in the water, they were probably part of the bottom slope
81		2 or 3 rocks required just above water line
84		2 or 3 required from water line to mid slope
87	*	Large rock hung up just above water line. Rocks mid slope would probably follow it in if it were to slump
89		2 or 3 rocks missing mid slope
96	*	Slope quite steep. A number of rocks required from water line moving back up to the lower part of the slope
101	*	Two large rocks appears to have fallen out of the mid slope region and are now at the waterline so a there is a gape mid slope
109		1 or 2 rocks needed around water level and slightly above
113		Need for rock around water level

117	*	Mid slope area has slumped at some time in the past. In the order of 4 rocks required (needs to be monitored)
120-125		Very large rocks on the cap appear stable at the moment but the slope is quite steep
126		Small slump near the base of slope. Need for 3 or 4 rocks in that area
129	*	There is a collapse from lower part of mid slope. There is a need for 4 to 5 rocks in that area to support upper slope where there are quite large rocks
135		Area where mid slope rocks are smaller than the rest, they appear reasonably stable however cap rocks have just started to move down slightly so there is a need for additional support at top of slope
139-145		The slope is quite steep and it looks as if rocks have slumped out of the mid slope area and into the water
150	*	Several large cap rocks with rocks missing from underneath them so that they are slightly hung up. Approximately 6 rocks needed to stabilize this area
155		Looks like rocks have moved out from the bottom of the slope so whilst the main part of the slope is fine the bottom rocks are hung up
159	*	There has been some rocks slide out from the mid slope. They are now near water line and so would need 2 or 3 rocks to support the larger rocks at the top of the slope
165		There are a couple of rocks hung up near the bottom of the slope and there is obviously a rock missing from the base of the slope
166-175	**	Extensive area where it looks as if there has been settlement. There are rocks in the water and the mid slope area, and most of the slope, is in a relatively poor condition and looks as if it needs additional rocks
182		Very large crest rock slightly overhanging the rest of the slope and while the rest of the slope looks fine the cap rock could slide sometime in the future
185	*	Reasonable collapse stretching most of the way up to the crest rocks and it would require 6-10 rocks in that area to stabilize that
200	*	A slump area is obvious in the center of the slope, and to the immediate west and east of 200. Slope is approximately 1 in 1.5 and looks relatively stable. However, there’s quite an amount of smaller material exposed and so there a need for 4 or 5 rocks

209	*	A number of rocks missing from mid slope from waterline up to the cap rock so there'd be a need for half a dozen rocks in this vicinity
215		Quite a few smaller rocks exposed on the slope however not a lot of sign that that area is currently slumping or moving. Just east of 215 there's another area of mid slope where there are obviously larger rocks missing exposing mainly smaller rocks. Probably requires 3 or 4 rocks to stabilize the slope
221		Again material moved out of the mid slope and would require larger rocks, 3 or 4 to stabilize that area
227-231	*	Slump has occurred resulting in the slope now being quite steep, around 1 in 1. There are quite a few larger rocks but they are hung up so it would need repair works starting at the base and working up slope up
235		Mid slope some smaller rocks have been exposed. Need for1 or 2 additional larger rocks in this area
239		Similar situation. To the east of this location the slope is approximately 1 in 1.5 and it's mainly large rocks
247	**	There are a number of rocks missing in this general vicinity probably 3 or 4 metres either side around the water line. Mid slope there are large rocks that are hung up and there is certainly a need for larger rocks from the waterline back up, probably between half a dozen to 10
251	*	Mid slope collapse has exposed an area of smaller material requiring probably 5 or 6 rocks
256-263		Slope is close to 1 in 1 and although the rocks are of a reasonable size a number of them look like they are hung up
271-276	*	The slope in this region is quite steep. There has obviously been a slump. Rocks can be seen in the water, below the waterline. There are a couple of large rocks, particularly one at the eastern end, that is hung up
279		Number of rocks missing at water level all part of slump while rocks in main slope look reasonable, they are hung up as there is little supporting them
285		Obvious loss of rocks around water level with rocks observable underwater. This has triggered failures further up the slope. Still quite large rocks in upper third of slope but they are now slightly hung up.
288	*	Couple of large rocks mid slope really hung up. Looks as if the slope has started to slump all the way from the cap down

290		Rocks hung up near water line some mid slope slumping and crest cap rocks appear to be slightly displaced as shown by cracks in concrete at the edge of the path
294		Past collapse with a lot of rocks in the water but possibly repaired area as the slope looks to be generally in reasonable condition
300	*	Obviously, some slumping with smaller rocks now occupying mid slope. Cap rock hung up so potentially unstable
304	**	Obvious slump into water. Rocks under the surface. Rocks missing from lower slope so upper slope hung up – possibly up to 10 rocks required to stabilize the area
307		Large rock hung up down near waterline to the immediate east while the mid slope looks reasonable the rocks appear hung up near water line
310		Evidence that Cap rock is settling down slope. A large concrete block has been placed mid slope sometime in the past possibly to prevent this settlement from continuing
313	*	Past collapse has exposing smaller material mid slope. Cap rocks hung up so some 5 to 6 rocks needed in this area
315		Clearly been a slump, can see material under the water line. Lower one third of slope looks like it needs 2 or 3 rocks
318	*	Clearly been a slump, quite an amount of rock in the water below waterline. Some mid slope rocks missing. 3 or 4 necessary to support cap rocks
322	**	Significant mid slope slump. Large rocks in water, just below waterline size of rocks on the slope seem reasonable. Slope close to 1 in 1 so a number of rocks hung up, as are the rocks to the immediate east
327	**	Mid and upper slope rocks are all hung up with quite a few rocks missing near waterline, possibly 10 or so rocks needed to stabilize this section
331		Slope near 1 in 1 some rocks in the water that seem to be supporting the slope however the cap rocks are large and hence placing pressure on the mid slope area
334		Slope approximately 1 in 1. Rocks of a relatively large size but there seems to be a couple of rocks missing around waterline, and just above Rocks in the water are possibly from a past slump.

336		Large rocks except about half way up the slope there is a gap so 2 or 3 rocks needed to fill gap
341	**	Rocks missing at base of slope and a bit of a gap to east. To west rocks in the water but here there is a moderate gap. With slope close to 1 in 1, 6 to 10 rocks needed in the general vicinity
345		Slope in reasonable condition at approximately 1 in 1.5 but quite a few smaller rocks in this area suggesting a possible minor repair at some past time
347-352	*	Slope hung up at approximately 1 in 1. Rocks in the water below water level indicating past slumping into water. Work required to reestablish the slope in this area
358		Rocks missing around water level, some rocks in the water suggesting some slumping in this area however rocks in the mid slope of a reasonable size
364		Some slumping however reasonable coverage but the need for 2 rocks mid slope and to the immediate east a similar situation with the need for 2 to 3 rocks mid slope
371-374	*	Quite large rocks but somewhat hung up with steep slope, some rocks in the water and evidence cap rocks have moved recently
377	**	Obviously, a significant slump in the past quite large rocks in the water with mid slope mainly smaller rocks that look very exposed, possibly the need for 10 or more rocks to stabilize the slope
383-385		Generally larger rocks and slope looks reasonably stable even though slightly steeper than 1 in 1.5 and there are some rocks in the water so possibly a repair area
386		Small local collapse with rocks in the water. Rocks in bottom half hung up with the need for 2 or 3 large rocks near water line with a further 1 or 2 up towards the crest
390		Small collapse only 1 to 2 rocks wide but goes from waterline to just under the cap rocks so need for about 3 rocks in this location
393		Some evidence slope has slightly slumped with the cap rocks showing signs of having moved. Although rocks are generally of a reasonable size there is evidence of a larger past slump as rocks can be seen in the water
397		Mid slope there is a very large rock but slope looks reasonable either side
403		Slope reasonable but rock size small, looks like a possible past repair area, really needs a couple of larger rocks to help stabilize area

406	*	Obvious mid slope collapse large rock sitting above water line with rocks in the water. Cap rock and those around ok but half way down slope there are a couple of large rocks hung up and needing support
412		Evidence of cap rocks moving slightly, about 3 large rocks missing from lower third of slope
417		Slope looks interlocked but at approximately 1 in 1. Not many rocks at waterline holding up the slope. Some rocks in the water and up to 10m to the east so seems to suggest this is an area that has slumped and then been repaired in the past
421		Obviously past slump, rocks moderate size but possibly not big enough. Slope to the immediate east 1 in 1 with a number of past slumps in the water below water level
427	*	Very steep slope, past slump obvious with rocks below water level. Slope fairly intact but rocks undersize so possibly a past repair area, certainly around 10 to 12 rocks needed to stabilize area
432		Large rocks from water line up and currently stable but some rocks on the slope with graffiti on them suggesting that at some time in the past the slope had slumped but had been repaired
435		Slope approximately 1 in 1.5 but rocks missing around waterline and just above. Rocks in the water suggesting a past slump, although slope seems reasonable however some undersized rocks mid slope suggesting a past attempt at repair
437-444	**	Upper third of the slope has collapsed with cap rocks hung up and mid slope rocks appear under sized. Need for at least a dozen rocks. To immediate east large rocks and slope approx. 1 in 1.5. There are rocks in the water supporting the slope
444		Same mid slope problem with undersized rocks but slope looks reasonably stable. Some evidence cap rocks starting to move. Also evidence of some concrete cast in breakwater or a concrete block (beam?) partially buried extending over a length of 4 to 5m
448		In past slope has possibly slumped rocks in water slope almost 1 in 2 and rocks of a reasonable size. So maybe a past repair. Cap rocks coming away from path making them. Cap rocks also look vulnerable immediately east
454	**	Evidence of past slump into water. Rock underwater. Mid slope up to cap, rocks are rather small so possibly another 10 to 12 rocks needed
456	*	Large cap rock hung up no support down for top one third of slope. To immediate east slope adjusted itself to 1 in 2 with generally large rock armour

460		Rocks missing at water line and can be seen underwater hence a past slump area, lower part of slope seems to currently have reasonable sized rocks and so looks like a possible repair area
463		Mid slope slumping with rocks in the water indicating a past slump. Some movement of cap rocks towards the upper slope/mid slope area. Rocks in the mid slope look undersized
469	*	Slope very steep approx..1 in 1. Some large rocks in the water around waterline with rocks missing from main part of the slope and cap rocks hung up and to the immediate east
470-478	*	Significant collapse in the past. Slope approximately 1 in 1. Heavy cap rocks hung up and only held up by concrete that has been placed there in the past, so seem ready to collapse. The lower one third has obviously collapsed into the water in the past as there's quite extensive amounts of rock in and under the water. There are medium sized rocks in the mid slope and upper slope but the cap rocks look as if they've started to move out. This is where (approx.. 472) a rock has a large sharks mouth painted on. The slope is very undercut, and so possibly need for at least a dozen large rocks to rebuild the slope in this area seems to be the most dangerous area sighted on the breakwater during the inspection so far
478		Couple of large rocks in mid slope rocks have slipped both below and above. The cap rock is hung up however to the immediate east the situation appears better
482	*	Mid slope has collapsed with rocks in the water, rocks in mid slope look undersize and hung up. Cap rocks also hung up with some signs they are slipping
485-490	**	The slope in general looks unstable in the mid slope area there are a number of rocks that look undersize and appear as if they've been moving. At the toe there has been collapse in the past, there are rocks missing. There are also rocks missing in the upper slope which means the cap rocks are starting to move. This is where the furthest movement was observed for the cap rocks.
490	*	Reasonably sized rocks in upper one third part of slope but rocks missing further down slope with rocks in the water indicating past slumping. Upper part of the slope now hung up requiring 4 or 5 rocks
491		Northern boundary of the tourist park
492	**	Significant slump creating what looks like a channel, a 2 to 3m wide slot running from water line up to near the cap. Rocks in slot quite small so need for 10 to 15 larger rocks to fill the slot

498	**	Fairly serious collapse extending right down to the water line, steeper than 1 in 1, probably a slot that needs about 5 or 6 rocks to support the cap rocks
501		Some slippage of larger rocks into water, generally only moderate size rocks on slope so looks like a repair area needing some larger rocks.
505		Rocks hung up from here to at least 10m east at waterline. Mid slope looks relatively flat but rocks only mid sized, again a possible repair area
517		Slope looks to be in good condition with no obvious slump material in water but could be a past repair area
530		Slope looks reasonable but quite an amount of rock in the water so possibly a past slump and repair area but could do with an additional 2 to 3 rocks
537		Small bush growing on top of breakwater suggesting little overtopping and reasonable stability at this location.
540-east		Slope 1 in 2 and fairly even and stable,
543		Approx. where the first of the palm trees is just east of the skate bowl. Bush growing in the crest. Crest has been bolstered with a second row of cap rocks from here on in to the head. The double row of rocks provides a much wider crest. The slope looks relatively stable
560		Small slump in bottom one third of slope
571		Mid slope slightly cut out with rocks slightly smaller so possibly needs 2 rocks
587		Large rock near waterline with a couple of others nearby but currently slope looks reasonably stable
604		Small gap mid slope - needs say2 rocks
604-608		Mid slope looks stable but possibly recently repaired. Cap rocks appear to have moved in the past (separating from crest path)
615		Large rocks extend from this chainage to the head. Looks like a major repair area up to and including the head
622		Slope a bit steeper down to water line and some rocks under water indicating a slump in the past but in good condition now, suggesting a repair has taken place

628-630		Although there are mainly large rocks the slope looks like it has slumped with a relatively steep slope and potentially a couple of rocks missing. Cap rocks separating from crest path
634-641		New capping slab slope looks relatively stable at present at 1 in 1.5 but cap rocks look like they may have moved sometime in the past
641 to head	*	Concrete path badly cracked and showing signs of breakwater settlement and wave overtopping
647	**	Signs of cap rocks moving away from the crest and settlement, reflecting exposure of site. Path showing more cracking and breaking up, again suggesting settlement.
659		Navigation port hand marker and light. Large rocks on the head look like a recent repair with some concrete blocks embedded under the rocks