Rottnest Island Authority

Investigation into Structural Failure at the Rottnest Army Jetty Wharf Structure

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EXECUTIVE SUMMARY

1.1 BRIEF

On the 24th of October 2018, a precast concrete deck Panel on the Army Jetty Wharf Structure at Rottnest Island (RIA) collapsed into the sea injuring members of the Public.

On the 26th of October 2018, RIA requested Wallbridge Gilbert Aztec (WGA) to undertake a visual inspection of the Wharf Structure as part of the investigation steps necessary to respond to the following question put by RIA:

"Undertake an assessment into and determine the likely cause/s of the structural failure that resulted in the collapse of a section/s of the concrete jetty deck of the Army Jetty on 24 October 2018".

This report contains the assessment of the facts relating to the collapse of the concrete slab and the likely causes of collapse.

This report has been issued prior to the completion of a more comprehensive investigation, that would explore the history, previous condition assessments and residual structural capacity of the remaining wharf elements. As such, this report should not be viewed as a complete structural review or investigation into the entire Wharf structure and is thus limited in scope to the pertinent facts relating to the collapse of the concrete deck planks alone.

1.2 CONTEXT

1. The Army Jetty Wharf structure is believed to have been constructed in 1971 and is thus 47 years old.

2. The structure comprises a pedestrian-trafficable deck comprising of precast reinforced concrete panels (Panel) that are approximately 3.0 m long by 1.5 m wide and 0.20 m thick.

3. After the collapse of the first Panel on the 24th October 2018, sometime later on the 24th October 2018, a second concrete panel collapsed, but with no further injuries reported owing to the Wharf being restricted from access.

4. Over the weekend of the 27th October 2018, a third Panel collapsed, again with no further injuries reported.

5. All three Panels that have collapsed are in the most seaward row of Panels of the deck structure.

Figure 1 below shows an aerial view of the first Panel Collapse on the 24th October 2018.
1.3 SITE SURVEY

Luke Campbell and Nick Deussen of WGA visited the site on 26th October 2018 and undertook a visual survey of the Wharf structure only. No testing nor sampling was undertaken.

1.4 KEY REMARKS AND LIKELY CAUSE OF COLLAPSE OF DECK PANEL

WGA provide the following observations and conclusions on the likely causes of the structural failure of the reinforced concrete deck Panel of the 24th October 2018:

1. On the day of the survey, there was distinct evidence of significant, widespread and active corrosion of the primary tensile reinforcement to a number of the reinforced concrete Panels forming the deck (trafficable surface) of the Jetty structure.

2. The corrosion was most extensive and apparent in the most seaward Panels, which are more exposed to the marine conditions. It was one of these panels most seaward Panels that had collapsed on the 24th October 2018.

3. The two other panels that subsequently collapsed on the 24th October 2018 and over the weekend of the 27th October 2018, were also in the most seaward row of panels.

4. The corrosion to the most seaward Panels was of such a degree and extent that some of the primary tensile reinforcement in the existing Panels had separated from the concrete, leading to an immediate elimination in the contribution to the load carrying capacity of the Panel from that reinforcement.

5. The residual primary tensile reinforcement on some of the Panels observed was extensively corroded, leading to a significant reduction in load carrying capacity from that reinforcement.

6. Structural failure of the Panel in question would have most likely arisen from a combination of either, or both of the following mechanisms:
• Tensile failure of the corroded reinforcement — i.e. the primary tensile reinforcing bars had corroded to such an extent that the reduced area that left was incapable of supporting the weight of the concrete panel itself and any additional loads from the people standing on the panel.

• Widespread loss of bond (slip) between the corroded primary tensile reinforcement and the concrete. This loss of bond in a reinforced concrete panel will essentially allow the concrete to behave as if it were unreinforced and lead to a sudden and rapid loss of structural integrity.

7. The above contributory mechanisms would result in a transverse crack forming in the middle of the precast panel, leading to a mechanism forming and an immediate collapse of the Panel, with little or no load (e.g. from people) on the Panels and with little warning.

8. The final confirmation of the absolute cause of failure would require further investigations of:
   • Residual reinforcement thickness and the extent of the loss of bond of the primary reinforcement in the soffit of the Panels.
   • Whether the degraded condition of the steel tubular piles and corroding concrete encased steel beams have contributed to a loss of support to the panels.

However, the investigations above are deemed unnecessary given the evidence from the survey and photographs and the subsequent collapse of two further Panels under their own weight.

1.5 IMMEDIATE ACTIONS REQUIRED

1. While the condition of the most seaward panels were observed as generally sustaining the most severe corrosion to the primary tensile reinforcement, based on the poor condition of these seaward deck panels, it is assessed that the most seaward Panels particularly and, the remaining Panels of the structure, may have a very limited load carrying capacity, to a point where some are in danger of immediate collapse. Thus, as a minimum the Wharf must be fenced off and guarded to prevent any access to the Wharf, i.e. made safe.

2. Further, owing to the risk of collapse of further deck panels, any underwater or under-deck surveys directly beneath the concrete deck Panels, must be prohibited.

3. The immediate next steps should be the demolition of the remaining deck Panels, which would provide a precaution against the consequences of further uncontrolled collapses of the Panels. This would also allow safer inspections of the piles and beams of the Wharf once the panels are removed, subject to assessments prior to surveys commencing.

4. Public and non-public access should be prohibited until after demolition of at least the remaining Panels has occurred whereupon further surveys or the remaining Wharf Structure could be undertaken. If parts of the remaining Wharf structure are to be left in place after the Panels are removed, removal of any access controls to the remaining structure has to be assessed and the residual risks deemed satisfactory by the RIA. Otherwise full demolition of the Wharf structure is considered the safest option.

5. The risk assessment discussed above must consider the levels of access controls permitted until such time for decisions to be taken on the remaining structure and the future of the Wharf have been decided.
6. The safest means of partial or full demolition must be carefully determined to avoid danger to demolition personnel. In addition, the allowable environmental controls in regard to allowing dust or rubble to drop into the sea beneath the Wharf should also be determined with the appropriate Bodies.
2 CLIENT’S BRIEF

2.1 ARMY JETTY FACILITY DESCRIPTION

The Army Jetty Facility (Facility) at Thomson Bay, Rottnest Island, comprises three main elements:

- A 120 m long stone Breakwater.
- A 21 m long Barge Ramp on the northern side of the breakwater at the junction of the Breakwater and the shore, and
- A 24 m long piled concrete Wharf structure.

The Facility was constructed by the Royal Australian Engineers, circa 1971 and is managed by the Rottnest Island Authority (RIA).

This report concerns WGA’s visual inspection of the piled concrete Wharf structure (Wharf), but with a particular focus on the concrete deck Panels.

2.2 CLIENT’S BRIEF

On the 24th of October 2018, a precast concrete slab on the Wharf structure at Rottnest Island (RIA) collapsed into the sea injuring members of the Public. On the 26th of October 2018, RIA requested Wallbridge Gilbert Aztec (WGA) to undertake an inspection of the Wharf and to prepare an answer to the following question:

"Undertake an assessment into and determine the likely cause/s of the structural failure that resulted in the collapse of a section/s of the concrete jetty deck of the Army Jetty on 24 October 2018".

This report contains the assessment of the facts relating to the collapse of the concrete slab on the Wharf structure and the likely causes of collapse.

This report has been issued prior to the completion of a more comprehensive investigation, that would explore the history, previous condition assessments and residual structural capacity of the remaining wharf elements. As such, this report should not be viewed as a complete structural review or investigation into the entire Wharf structure and is thus limited in scope to the pertinent facts relating to the collapse of the concrete deck planks alone.

2.3 CONTEXT

The structure comprises a pedestrian-trafficable deck comprising of precast reinforced concrete panels (Panel) that are approximately 3.0 m long by 1.5 m wide and 0.20 m thick.
After the collapse of the first Panel on the 24th October 2018, sometime later on the 24th October 2018, a second concrete panel collapsed, but with no further injuries reported owing to the Wharf being restricted from access.

Over the weekend of the 27th October 2018, a third Panel collapsed, again with no further injuries reported.

All three Panels that have collapsed are in the most seaward row of Panels of the deck structure.

2.4 ARMY JETTY WHARF STRUCTURE DESCRIPTION

The following data on the Wharf structure has been gathered from a historical construction drawing of the Army Jetty Facility, dated 1971, provided as Appendix A of this report:

1. The date of the drawing suggests that the Wharf is around 47 years old.
2. It was believed that the Wharf was originally intended to service boat, vehicle and pedestrian use.
3. Referring to Figure 2, the Wharf structure comprises:
   - Three rows of Concrete-filled 112 lb BHP steel tubular piles, 23 piles in total.
   - 6-inch diameter steel tubular bracing elements between the two seaward rows of piles.
   - The tubular piles support concrete-encased steel Universal Beams of unknown size.
   - The concrete-encased Uniform Beams support a deck of precast reinforced concrete panels.
   - The reinforced concrete deck panels (Panels) are approximately 3.0 m long by 1.5 m wide and 0.20 m thick.

Figure 2: Section through Wharf, taken from 1971 drawing, See Appendix A for full drawing
4. The concrete specification, reinforcement size and reinforcement arrangement are not indicated on the drawings provided.

5. The Panels are supported along their short (1.5 m) edges sitting on the concrete-encased Uniform Beams. The Panels are located in position by two simple steel dowel bars, which are grouted to the Panels.

6. There is no structural continuity between adjacent Panels and thus the panels are "simply supported" with a design span of 3.0 m.

2.5 PREVIOUS SURVEYS

WGA have been supplied and have reviewed the following historical surveys for the Army Jetty Facility:


3. "Maritime Archaeological Assessment of the Army Jetty, Thompson Bay, Rottnest Island, Western Australia", ResearchGate, November 2012 – this report is not a structural or condition survey, but an archaeological assessment of the Army Jetty Facility.

2.6 SITE SURVEY

Luke Campbell and Nick Deussen of WGA visited the site on 26th October 2018 and undertook a visual survey of the Wharf structure only. No testing nor sampling was undertaken.

A summary of observations arising from the survey, with regards to the precast concrete deck Panels especially, are provided below:

1. The survey and site photos revealed in some cases a complete loss of the protective concrete cover to the soffit reinforcement of the Panels, revealing the primary steel reinforcement beneath, refer Figure 3. The corrosion was more extensive in the most seaward Panels, which would be more exposed to the marine conditions.

2. The revealed primary tensile reinforcement shows extensive corrosion, and hence represents a significant reduction in structural capacity as part of its role in the nature of the design of a reinforced concrete Panel.

3. Some of the panels revealed primary tensile reinforcement was hanging down beneath the Panels (Figure 3), hence there has been a loss of bond to the concrete and thus a complete loss of the strength contribution from that reinforcement to the Panel’s structural capacity.
4. In other locations, there is evidence of spalling of the concrete cover to the soffit of the Panels, revealing corroded reinforcement underneath and “drooping” / delamination of the concrete cover to the Panels (see Figure 4).

Figure 3: Soffit to precast concrete Panel, showing loss of cover and corroded reinforcement. Panel photographed was between the two collapsed panels found on the day of inspection and collapsed approximately 1 week later. One additional seaward panel was in similar condition. There were signs of reinforcement exposure and sagging noted on further panels closer to the rock groyne that could not be inspected closer.

Figure 4: Spalling of concrete cover in the soffits to Panels and “drooping” or delamination of concrete cover. This was the more typical condition of the deck panels, although the extent of the visual assessment of the panels closer to the rock groyne was limited.
5. The Panels appear to have been constructed with only reinforcement in the bottom of the panel, refer Figure 5 and thus the soffit (bottom) reinforcement alone provides the primary reinforcement for the structural integrity and design capacity of the Panels.

![Figure 5: Panels have only a single layer of reinforcement, provided in the bottom of the Panel](image)

6. From the visual survey and photos, it is apparent that the primary tensile reinforcement in the concrete Panels has been undergoing active corrosion for some time. The cause of the corrosion is most likely to be ingress of marine salts from sea spray/splashing onto the soffits of the Panels. The marine salts permeate into the concrete and set up active corrosion cells in the reinforcement. Corrosion behaviour of this type is well known in older marine structures of this type.

7. The cracking and cover delamination that has occurred to the Panel soffits is being caused by bursting pressures arising from active corrosion of the soffit reinforcement beneath.

8. This delamination will allow faster access for marine salt water, increasing the rate of corrosion of the slab reinforcement and allowing debonding of the reinforcement and thus leading to accelerated structural; degradation.

### 2.7 ADDITIONAL OBSERVATIONS FROM THE SURVEY

1. The photos also show longitudinal cracking of the concrete casing to the seaward edge of the supporting steel beam framing, showing that corrosion is actively occurring to the beam inside, refer Figure 6. Thus, the structural capacity of the steel beams to support the existing Panels or replacement Panels, is reduced and will continue to reduce with time, without intervention and refurbishment. It is also likely that this corrosion has commenced to a lesser extent on the remaining beams on the structure.

2. The concrete-filled tubular piles supporting the support beams show active and significant corrosion, (Figure 6). Hence, the residual capacity and design life of these piles cannot be determined by visual means alone and must be subject to further investigation if they are intended for extended structural service in a refurbished Wharf. This comment also applies to the concrete encased steel beams.
3. Referring to Figure 6, the 6-inch steel tubular bracing which should be present here is missing. It is likely that the tubular bracing was severely corroded, and the falling Panel has broken the bracing. The need for replacing the missing tubular bracing would be subject to further structural assessment.

2.8 DISCUSSION ON THE LIKELY CAUSES OF THE COLLAPSE OF THE DECK PANELS

2.8.1 Context of the design of reinforced concrete.

The nature of the structural design of reinforced concrete requires the complementary contribution of steel reinforcement and the concrete, acting together, or “compositely” as the term is known in engineering. This “composite design” provides both design structural capacity to resist loads and provides ductility to the elements, i.e. a gradual cracking and deflecting in an overload situation, providing warning of an overload, allowing people to remove themselves from the situation thus reducing loading.

In the situation of the Army Jetty Wharf structure deck Panels, where the reinforcement has completely or almost completely corroded away and / or debonded from the concrete, the contribution of the primary reinforcement is lost and thus the ability of the panel to withstand its design loads is significantly reduced. Further, the absence of the structural contribution of the reinforcement will make the concrete behave in a brittle manner where the failure is sudden, with little or no warning.

Having only a single layer of reinforcement in the bottom of a Panel of this type is allowed even in current reinforced concrete structure design with simply supported end conditions (Australian Standard AS 3600). The presence of any top reinforcement, if present would not contribute significantly to the design capacity of the Panel but would perhaps aid in the ductility of the Panels on failure.

The steel reinforcement in reinforced concrete is typically protected from the design environment by embedding it within the mass of concrete. This protective layer is known in engineering terms as “cover” to the reinforcement. In the case of the Wharf Structure, this
cover was of the order of 50 mm, which is an expected level of cover for contemporary marine structures, depending on the desired service life.

2.8.2 Discussion of the failure mechanisms

The following statements have been put forward by WGA with regards to the likely causes of the collapse of the precast reinforced concrete deck Panels of the Army Jetty Wharf structure.

1. The bottom reinforcement in the Wharf Panels is the only reinforcement in the precast deck slabs and is the primary mechanism by which the reinforced concrete panel gains its design strength.

2. In the case of these simply-supported Panels, the steel reinforcement that would resist design loads would be expected and was placed in the bottom of the Panel, protected with a generous concrete "cover". This is the case with the Wharf Panels, but there is only a single layer of steel reinforcement, provided in the bottom of the Panels.

3. It was seen that there is extensive corrosion in the primary steel reinforcement in the Panels, with a complete loss in some cases of physical connection (bond) of the reinforcement to the concrete. This is a substantial threat to the composite nature and structural integrity of the design of reinforced concrete Panels of this type.

4. The corroding reinforcement and loss of bond of the reinforcement to the Panel will lead to two structural degradation mechanisms in the reinforced concrete Panel:
   - A loss of structural thickness (or complete loss) of the steel reinforcement, thus leading to a reduction or complete elimination of its contribution to Panel design capacity.
   - A breakdown in the bond between the primary reinforcement and concrete, thus leading to "slip" between the reinforcement and the concrete within which it is embedded.
   - Both of the above mechanisms are threats to the structural integrity and thus design capacity of the composite nature of the reinforced concrete Panels.

5. With the significant reduction of the vital contribution of the primary tension reinforcement discussed above, the precast Panels would have a vastly reduced, or even compromised structural capacity.

6. Considering also the possibility of thermal or shrinkage cracking through the precast panel, the residual capacity would be reduced to the point of having the ability to perhaps only support its own weight, until such a time when it would collapse under its own weight.

7. WGA postulate that structural failure of the Panel would have most likely arisen from a combination of either, or both of the following mechanisms:
   - Tensile failure of the corroded reinforcement – i.e. the primary reinforcing bars had corroded to such an extent that the reduced area that left was incapable of supporting the weight of the concrete panel itself and any additional loads from the people standing on the panel.
   - Widespread loss of bond (slip) between the corroded reinforcement and the concrete. This loss of bond in a composite concrete panel will lead to a sudden and rapid loss of structural integrity.

8. The above contributory mechanisms would result in a transverse crack forming in the middle of the precast panel, leading to a mechanism forming and an immediate collapse of the panel.

9. The final confirmation of the absolute cause of failure would require further investigations of:
• Residual reinforcement thickness and the extent of the loss of bond of the primary reinforcement in the soffit of the Panels.
• Whether the degraded condition of the steel tubular piles and corroding concrete encased steel beams have contributed to a loss of support to the panels.

However, the investigations above are deemed unnecessary given the evidence from the survey and photographs and the subsequent collapse of two further Panels under their own weight.
Following the visual survey, WGA provide the following conclusions on the likely causes of the structural failure of the 24th October 2018:

1. On the day of the survey, there was distinct evidence of significant, widespread and active corrosion of the primary tensile reinforcement to a number of the reinforced concrete Panels forming the deck (trafficable surface) of the Jetty structure.

2. The corrosion was most extensive and apparent in the most seaward Panels, which are more exposed to the marine conditions. It was one of these panels most seaward Panels that had collapsed on the 24th October 2018.

3. The two other panels that subsequently collapsed on the 24th October 2018 and over the weekend of the 27th October 2018, were also in the most seaward row of panels.

4. The corrosion to the most seaward Panels was of such a degree and extent that some of the primary tensile reinforcement in the existing Panels had separated from the concrete, leading to an immediate elimination in the contribution to the load carrying capacity of the Panel from that reinforcement.

5. The residual primary tensile reinforcement on some of the Panels observed was extensively corroded, leading to a significant reduction in load carrying capacity from that reinforcement.

6. Structural failure of the Panel in question would have most likely arisen from a combination of either, or both of the following mechanisms:
   - Tensile failure of the corroded reinforcement – i.e. the primary tensile reinforcing bars had corroded to such an extent that the reduced area that left was incapable of supporting the weight of the concrete panel itself and any additional loads from the people standing on the panel.
   - Widespread loss of bond (slip) between the corroded primary tensile reinforcement and the concrete. This loss of bond in a reinforced concrete panel will essentially allow the concrete to behave as if it were unreinforced and lead to a sudden and rapid loss of structural integrity.

7. The above contributory mechanisms would result in a transverse crack forming in the middle of the precast panel, leading to a mechanism forming and an immediate collapse of the Panel, with little or no load (e.g. from people) on the Panels and with little warning.

8. The final confirmation of the absolute cause of failure would require further investigations of:
   - Residual reinforcement thickness and the extent of the loss of bond of the primary reinforcement in the soffit of the Panels.
   - Whether the degraded condition of the steel tubular piles and corroding concrete encased steel beams have contributed to a loss of support to the panels.
3.1 RECOMMENDATIONS AND ACTIONS REQUIRED

1. While the condition of the most seaward panels were observed as generally sustaining the most severe corrosion to the primary tensile reinforcement, based on the poor condition of these seaward deck panels, it is assessed that the most seaward Panels particularly and, the remaining Panels of the structure, may have a very limited load carrying capacity, to a point where some are in danger of immediate collapse. Thus, as a minimum the Wharf must be fenced off and guarded to prevent any access to the Wharf, i.e. made safe.

2. Further, owing to the risk of collapse of further deck panels, any underwater or under-deck surveys directly beneath the concrete deck Panels, must be prohibited.

3. The immediate next steps should be the demolition of the remaining deck Panels, which would provide a precaution against the consequences of further uncontrolled collapses of the Panels. This would also allow safer inspections of the piles and beams of the Wharf once the panels are removed, subject to assessments prior to surveys commencing.

4. Public and non-public access should be prohibited until after demolition of at least the remaining Panels has occurred whereupon further surveys or the remaining Wharf Structure could be undertaken. If parts of the remaining Wharf structure are to be left in place after the Panels are removed, removal of any access controls to the remaining structure has to be assessed and the residual risks deemed satisfactory by the RIA. Otherwise full demolition of the Wharf structure is considered the safest option.

5. The risk assessment discussed above must consider the levels of access controls permitted until such time for decisions to be taken on the remaining structure and the future of the Wharf have been decided.

6. The safest means of partial or full demolition must be carefully determined to avoid danger to demolition personnel. In addition, the allowable environmental controls in regard to allowing dust or rubble to drop into the sea beneath the Wharf should also be determined with the appropriate Bodies.
APPENDIX A

HISTORICAL WHARF CONSTRUCTION DRAWING
Rottnest Island Authority

Final Inspection, Refurbishment at Army Jetty Wharf

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Appendices

Appendix A IFC CONSTRUCTION DRAWINGS
1 CONTEXT

1.1 BRIEF

On 24th of October 2018, a precast concrete deck panel on the Army Jetty Wharf Structure at Rottnest Island collapsed into the sea injuring members of the Public.

On 26th October 2018, Wallbridge Gilbert Aztec (WGA) undertook a visual inspection of the Wharf Structure as part of the investigation to understand the cause(s) of the panel collapse. The inspection and findings are contained in WGA Report “WGA181787-AA-0001” dated 5th December 2018.

Subsequently, WGA produced a report looking into the demolition and refurbishment options for the Army Jetty Wharf structure and this is documented in WGA report WGA Report “WGA181787-AA-0002” dated 30th November 2018.

Following a presentation and consideration of the options, Rottnest Island Authority (RIA) made the decision to completely demolish the Wharf Structure, and to refurbish the underlying rock armour slope to match the remainder of the Army Jetty breakwater.

WGA have produced IFC drawings documenting the necessary works to provide for the complete demolition of the Wharf Structure and the refurbishment of the underlying rock armoured slope; these drawings are provided in Appendix A.

1.2 PURPOSE OF SITE SURVEY AND REPORT

The purpose of this report is to document a site visit and visual surveys by WGA coinciding with the completion and handover of the refurbishment works by McMahon Services Pty Ltd (McMahon), and identify whether the Works are fundamentally complete in accordance with the IFC drawings. The survey is also aimed to determine whether there are any defective works requiring rectification by McMahon prior to reopening the Army Jetty Wharf to the Public.
2 DESCRIPTION OF DEMOLITION AND REFURBISHMENT WORKS

2.1 DESCRIPTION OF WORKS

The demolition and refurbishment works are provided on the WGA drawings in Appendix A.

The works comprised:

1. The sequential demolition of the 1970's Army Jetty Wharf structure, including removal of deck panels, substructure and cutting of the piles at seabed.

2. Backfilling and placing rock armour to the existing slopes, which were exposed as a consequence of the removal of the old Army Jetty Wharf Structure.

The drawings allowed for either the demolition or retention of the shore abutment, described as Option 1 and Option 2 on the WGA drawings. In the event, the condition of the abutment was assessed by RIA and WGA staff and it was decided to retain the abutment and continue with the refurbishment works as described in Option 2.

The demolition and refurbishment occurred in the period 8th to 17th December 2018.

All supervision of the demolition and refurbishment works was provided by RIA staff.
3 SITE SURVEY

3.1 DATE OF SURVEYS

WGA staff undertook two site inspections during the course of the demolition works. Nick Deussen of WGA and Clint Hull and Nello Siragusa of RIA visited the site on the 12th of December during the demolition works. Subsequently, Ian McRobbie of WGA and Clint Hull of RIA visited the site on 18th December 2018 and undertook a visual survey of the completed works at the Army Jetty only. Two personnel from McMahon were present. No testing nor sampling was undertaken.

3.2 RECORD OF VISUAL SURVEY AND COMMENTS

3.2.1 Interim Inspection of Abutment / Option Selection

During the inspection held on 12th December 2018, enough of the Wharf Structure had been demolished and removed from site as to uncover the abutment at the rear of the Wharf.

The abutment is an L-shaped concrete abutment that had been poured with in-situ around the piles at the rear of the wharf. The abutment spans approximately 6m between each pile and was underpinned in places by the rock revetment. Refer to Appendix A for Two options that have been prepared for treating this abutment after the inspection.

1. Option 1 considered the demolition of the abutment. This was considered the default option considering the unknown condition of this structure.

2. Option 2 was to retain the abutment structure, underpinning it where necessary and burying it within the refurbished rock revetment. There was consideration that this abutment was currently acting as a retaining wall for the breakwater core fill, and that removal of this would reduce the stability of the breakwater.

The site inspection revealed that the abutment was externally in fair condition, with no signs of visible spalling on the soffit with only minor cracking evident. In most areas, the abutment was not supported mid span by the underlying rock revetment and was spanning between piles. No clear indications of the shear connection between pile and abutment was evident.
There was concern that to reinstate the rock slope above the existing breakwater would effectively bury a latent defect that would eventually collapse leading to excessive settlements of the roadway.

To confirm the integrity of the abutment, a load test was undertaken on a single span of the abutment by parking the excavator tracks on the abutment and extending the arm/bucket out. No cracking, spalling or movement in the abutment was visually observed during this exercise.
Despite the load test, a risk remains that the rock fill now underpinning the abutment may wash out, and a deteriorated abutment would one day fail structurally, leading to settlement of the roadway. However, the residual risk of this action occurring is considered to be no worse than the general risk regarding core wash-out and settlement of the rock armour.

3.2.2 Works to Breakwater Slopes

The new rock armour (light brown colour) has been placed to an estimated angle of around 1 vertical to 3 horizontal, which provides a more stable slope than the existing breakwater, which appears in places to have been laid to 1 vertical to 1 horizontal (or steeper in places).

The rock armour visually appears to be specification compliant, which called for rocks between 0.5t and 2t in weight, but at least 50% by weight being lumps over 1t.

The new rock armour is believed to be one rock layer thick, whereas typically rock armour slopes are laid two layers thick. From the previous McMahon daily reports, it was observed that rock armour already existed under the seaward half of the Jetty wharf structure, but the remaining, landward half comprised mainly core (Type 2) rock at the surface. Hence the more seaward half of the jetty is likely to comprise two rock armour layers and the more landward half just one-layer thick.

Given the shallower slope of the new breakwater slope and the armour thicknesses noted above, it is deemed that the rock armour slopes are acceptable for use. It is recommended that RIA should inspect the new slopes after the first storm events and act accordingly, as would be expected for all rock armour breakwaters.

Refer to Figures below for views of the new rock armour slopes to the Army Jetty breakwater.
Figure 3: New rock armour slope, view seaward
Figure 4: New rock armour slope, view landward

It can be seen from Figure 5 below that the new rock armour slopes (light brown colour) are laid at a more acceptable 1 in 3 slope when compared with the old pre-existing rock armour slopes beyond, which are laid at an estimated slope of 1 in 1 or steeper.
Figure 5: New rock armour (light brown rock) in foreground and old rock armour slope beyond (dark colour)

3.2.3 Cutting off Wharf Piles

Though not photographed on the day of the survey, it could be visually observed that the most seaward rows of wharf piles appear to have been cut off at seabed level, which is as per the IFC drawings. Figure 6 shows a photograph from McMahon daily record report showing the cut off levels of the most seaward row of piles.
3.2.4 Works to Roadway on Crest of Breakwater

Crushed limestone surfacing was laid to the crest of the refurbished breakwater to achieve a minimum trafficable width of 4m, refer to Figure 7, Figure 8 and Figure 9.

The works have been completed satisfactorily, with a good, densely compacted trafficable surface. The surfacing is edged to the south with limestone rocks abutting new rock armour stones and to the north by the top of the abutment wall form the old Wharf Structure.

The surfacing should be inspected as part of RIA’s regular maintenance and inspection plans, but additionally after its first storm to review the performance of the surfacing and edge protection.

There is a small amount of damage to the top of the wall, which may have been pre-existing. RIA is organising making good to this wall.
Figure 7: Limestone surfacing to crest of breakwater, view to seaward

Figure 8: Limestone surfacing to crest of breakwater, view to landward
3.3 SURVEY OUTCOMES AND RECOMMENDATIONS

It is WGA’s opinion that the works are deemed to have reached satisfactory completion by McMahon and the site is deemed safe for public access, noting however:

- Rock armour slopes intrinsically present obvious dangers to the public with regards to trip hazards, irregular stepping surfaces and large gaps between stones.

- With the new rock armour slopes such as this one, it is expected that there will be some “settling-in” required for rocks until such time as they have been exposed to wave action, which would tend to stabilise the rocks. Additional care should be taken in these early stages.
• Whereas typically rock armour slopes are laid two layers thick, it is noted that in places the rock armour is one layer thick, (to the southern half of the refurbished slopes). Owing to the shallower slopes of the new rock armour, this is considered a low likelihood of occurrence. It is recommended that RIA increase the frequency of visual inspections during the first periods of winter storms to observe the behaviour of the new rock slope and act accordingly.

• A risk remains that the rock fill now underpinning the abutment may wash out, and a deteriorated abutment would one day fail structurally, leading to settlement of the roadway. However, the residual risk of this action occurring is considered to be no worse than the general risk regarding core wash-out and settlement of the rock armour.

• Thus, the new works present, on average, the same level of danger to the public as the rest of the Army Jetty Breakwater if the public seek to cross the new armour stones.

WGA recommend the following:

1. RIA to request Quality and As Built documentation from McMahon, including rock armour supply details.

2. The Army Jetty breakwater is inspected after exposure to its first storm event and a survey undertaken and record kept of the movement of the armour repairs planned accordingly.

3. That particular line items are inserted in RIA’s 2019 budget for increased maintenance inspections and repairs, owing to these being new works and thus expected to be subject to initial movement of the rock armour and crushed limestone surfacing.

4. That any further improvements or alterations to the Army Jetty Breakwater be included in the Greater Army Jetty development plans, thus avoiding any over-capitalisation.
APPENDIX A

IFC CONSTRUCTION DRAWINGS
EXISTING JETTY PLAN - OPTION 1

EXISTING JETTY PLAN - OPTION 1 - STAGE 1

EXISTING JETTY - SECTION A

EXISTING JETTY - OPTION 1 - STAGE 1 - SECTION A

CLASS 1 ROCK

INDIVIDUAL CLEAN LIMESTONE FROM 0.5 TON TO 2 TON IN WEIGHT WITH MIN.
BY WEIGHT BENDING LUMPS OVER 1 TON.

CLASS 2 ROCK

LUMPS UP TO 1 TON WITH MAXIMUM 35% BY WEIGHT BEING ABLE TO
PASS THROUGH A 8 IN NO. 8 SCREEN.

NOTES

- REMOVAL SEQUENCE SHOWN IS INDICATIVE. CONTRACTORS MIGHT STATEMENT TAKES PRIORITY OVER SEQUENCE. THESE DRAWINGS
  HAVE BEEN PREPARED TO SHOW THE ESTIMATED REMEDIAL WORK REQUIRED FOR THE BREAKWATER POST-DEMOLITION OF THE FIXED
  WARP. THE DESIGN INTENT FROM WGA IS THAT THE BREAKWATER DESIGN ROCK WITNESS PROFILES, SECTION, DETAILS AND RISK SIZING
  MATCHES THE JETTY CONSTRUCTION DESIGN FOR THE ARMY JETTY. THE SCOPED WGA'S WORK DOES NOT REQUIRE DESIGN VALIDATION OF
  THE ORIGINAL DESIGN FOR THE BREAKWATER. HENCE NO DESIGN RESPONSIBILITY IS TAKEN BY WGA ON THE DESIGNS PRESENTED.

DRAFT

ENG. CHKD.

ROTTNEST ISLAND ARMY JETTY

ROTTNEST ISLAND

THOMSON BAY

JETTY DEMOLITION DETAILS - OPT 1 SHT 1
1. CUT CONCRETE ENCLOSE SUPPORT BEAMS.
2. DEMOLISH TIMBER FENCING.
3. BRACING TO REMOVTI SUPPORT PILES.
4. WHERE BRACING NOT PRESENT, ALTERNATIVE BRACING/PROP TO BE INSTALLED.

EXISTING JETTY PLAN - OPTION 1 - STAGE 2

1. CUT PILES.
2. CUT BRACING.
3. REMVO PILES AND BRACING.

EXISTING JETTY PLAN - OPTION 1 - STAGE 3

NOTES:
- EXECUTION SEQUENCE SHOWN IS INDUCTIVE; CONTRACTOR'S METODOLOGY STATEMENT TAKES PRIORITY.
- THESE DRAWINGS HAVE BEEN PREPARED TO SHOW THE ESTIMATED REMOVAL WORK REQUIRED FOR THE ROCK GROINE POST-DESTRUCTION OF THE PILES.
- THE DESIGN INTENT FROM WGA IS THAT THE BREAKWATER DESIGN ROCK ARMOUR PROFILES, SIZING, DETAILS AND ROCK SIZE MATCH THE R/C CONSTRUCTION DRAWING FOR THE ARMY JETTY. THE Scope OF WGA'S WORK DOES NOT INCLUDE DESIGN VALIDATION OF THE ORIGINAL DESIGN FOR THE BREAKWATER ROCK ARMOUR RESPONSIBILITY IS TAKEN BY WGA ON THE DESIGN PREPARED.
1. Lay new crushed rock backfill to alignment.
2. Place new class 1 rock.
3. Lay new consolidated limestone and make good existing road surface.
4. Make good rock, normal 3m length.
5. Make good timber and consolidated limestone, normal 3m wide by 3m length.

EXISTING JETTY PLAN - OPTION 1 - STAGE 4

NOTE: ALL DIMENSIONS APPROXIMATE

EXISTING JETTY - OPTION 1 - STAGE 4 - SECTION A

APPENDIX QUARTERS FOR JETTY DEMOSSION ONLY

NOTE: DEMOLITION SEQUENCE SHOWN IS INDICATIVE. CONTRACTORS METHOD STATEMENT TAKES PRECEDENCE OVER SEQUENCE. THESE DRAWINGS HAVE BEEN PREPARED TO SHOW THE ESTIMATED REMOVAL WORK REQUIRED FOR THE ROAD DECK POST-DEMOLITION OF THE PIER.


ROTTNEST ISLAND ARMY JETTY
THOMSON BAY

JETTY DEMOLITION DETAILS - OPT 1 SHT 3

DOCUMENT NUMBER

WGA181787-DR-MAF0012

ENG. CHK'D

WGA
EXISTING JETTY PLAN - OPTION 2 - STAGE 1 - SECTION A

SECTION A - EXISTING

CLASS 1 - ROCK

INDIVIDUAL CLEAN LIMESTONE FROM 0.5 TON TO 2 TON IN WEIGHT WITH 50% BY WEIGHT BEING LUMPS OVER 1 TON.

CLASS 2 - ROCK

LUMPS UP TO 1 TON WITH NOT MORE THAN 10% BY WEIGHT BEING ABLE TO PASS THROUGH A RIS IN A SCREEN.

NOTES:

DESTRUCTION SEQUENCE SHOWN IS INDICATIVE. CONTRACTORS METHOD STATEMENT TAKES PRECEDENCE OVER SEQUENCE. THESE DRAWINGS HAVE BEEN DRAWN TO SHOW THE ESTIMATED REMEDIAL WORK REQUIRED FOR THE ROCK GROIN POST-DEMOLITION OF THE PILE BREAKWATER. THE DESIGN INTENT FOR WGA IS THAT THE BREAKWATER DESIGN COVER PROFILES, CROSS SECTIONS, CROSS SECTIONS AND ROCK SIZING MATCH THE NOT CONSTRUCTION DRAWINGS FOR THE ARMY JETTY. THE SCOPE OF WGA'S WORK DOES NOT INCLUDE DESIGN VALIDATION OF THE ORIGINAL DESIGN FOR THE BREAKWATER WAGE AS DESIGN RESPONSIBILITY IS TAKEN BY WGA ON THE WORK PRESENTED.
NOTES:
- CONSTRUCTION SEQUENCE SHOWN IN INDICATIVE. CONTRACTORS' METHOD STATEMENT TAKES PRECEDENCE OVER SEQUENCE. THESE DRAWINGS HAVE BEEN PREPARED TO SHOW THE ESTIMATED REMEDIAL WORK REQUIRED FOR THE ROCK GRUNGE POST-DESTRUCTION OF THE PILED WHARF. THE DESIGN INTENT FROM WGA IS THAT THE BREAKWATER DESIGN, ROCK ARMOUR PROFILES, SECTIONS, DETAILS AND ROCK SIZING MATCHES THE 1911 CONSTRUCTION DRAWINGS FOR THE ARMY JETTY. THE SCOPE OF WGA'S WORK DOES NOT INCLUDE DESIGN VALIDATION OF THE ORIGINAL DESIGN FOR THE BREAKWATER. NO DESIGN RESPONSIBILITY IS TAKEN BY WGA ON THE DESIGNS PRESENTED.
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