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TOKOMARU BAY - HISTORIC WHARF RESTORATION USING MODERN MATERIALS

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SUMMARY

The Tokomaru Bay concrete wharf was constructed in the late 1930's to service the local meat freezing works. In recent years, the wharf structure has exhibited increasing levels of deterioration with extensive concrete spalling from the supporting piles resulting in its closure to the public in 2017 due to safety concerns. A community desire to preserve this piece of local history led to the development of a proposal to restore the structure. This paper gives details of a repair system developed using self-compacting concrete (SCC) and fibre reinforced plastic (FRP) rebar which was trialled successfully on two of the wharf piles in early 2021.

BACKGROUND

Tokomaru Bay is a small coastal settlement on the East Coast of New Zealand approximately 90 km north of Gisborne. The economy of the area developed from wool and meat exports. In the early 20th century a freezing works and timber wharf was constructed at Waima at the northern end of the bay. The original timber wharf was extended to cater for larger vessels and a large section 'optimistically' replaced with a concrete structure between 1937 and 1940 (Gundry 2014). Coastal freight declined following the Second World War and with the closure of the freezing works in 1952 the regular coastal service to the bay ended in 1963. The wharf continued to be used by the local community for recreational activities and was given a Category 2 heritage listing in 1984 (Pattison 2013). Over time, the wharf structure has exhibited increasing levels of deterioration with cracking and concrete spalling from the beams and supporting piles (Figure 1). It was closed to the public in 2017 due to safety concerns (Gisborne District Council 2017).

Figure 1. Cracking and Spalling of Concrete on the Wharf Structure

Local Heritage

The wharf is one of a number of structures associated with the industrial heritage of the area. A desire to preserve this and other pieces of local history prompted the establishment of the Tokomaru Bay Heritage Trust in 2015. In addition, the heritage, and its significance to the local community, were the subject of University of Waikato research projects in 2019 and 2020.

The Wharf Structure

The general structure and approximate metric dimensions of the wharf are described below based on values taken from copies of the original 1936 drawings.

The concrete section is approximately 240 m long running roughly North West - South East out to the remains of the original timber wharf head which extends a further 80 m. The structure consists of a series of trestles or bents made of pairs of inclined piles connected by a headstock. These are connected by pairs of longitudinal beams with a cast in-situ concrete deck forming a double 'T' section. The majority of the bents are spaced at 6.1 m along the wharf at the landward end out to a low tide water level of approximately 1 m a further ten bents are spaced at 5.2 m out to the junction with the timber wharf with a low tide water level of approximately 2 m.

The 'Government Standard Pattern' piles are of the 'Considère' type (Marsh 1905 cited by Reed P et al. 2008) with a 405 mm octagonal section, inclined at 1 in 6 to the vertical. The drawings show the piles as 355 mm but a note has been added that they be 'increased to 16" dia. instead of 14" dia.' Longitudinal reinforcement for the piles consisted of eight evenly distributed 22 mm plain steel bars. Circumferential steel consisted of 5 mm (No.6 gauge) wire spiral reinforcement throughout the pile length but concentrated at the top and bottom. Concrete cover to the reinforcement is shown as 40 mm. It is understood the piles were manufactured in Auckland and shipped to site.

Figure 2. Photograph Showing Profile of the Headstocks and Longitudinal Beams

The drawings show that, after installation, the concrete at the pile ends was removed and the longitudinal steel bent out to be integrated into the cast in situ headstocks. The headstocks are of rectangular cross section, 405 mm wide, varying in depth from 600 mm over the centre 900 mm, then tapering to a depth of 915 mm over the piles. (Figure 2). The tops of the piles are 2.1 m apart.

Each bent is connected by two 405 mm wide by 470 mm deep beams with a 3.7 m wide by 250mm thick cast in situ deck. The beam depths increase from 470 mm to 520 mm over a

660mm taper where the beams join the headstocks. Based on the detailing and physical inspection, the headstocks, longitudinal beams and deck were cast together. Reinforcement in the headstocks and beams ranges from 13 to 25 mm diameter with 10 mm stirrups at 200 to 300 mm centres and an indicated minimum cover of 60 mm.

The deck was reinforced with 10 mm longitudinal bars at 305 mm centres top and bottom and straight 13 mm transverse bars at 380 mm centres top and bottom alternating with cranked bars at 380 centres in the top. The deck had a 13 mm cross camber with indicated cover of 40 mm in the bottom and 75 mm in the top. A '3' Gauge' (915 mm) rail line using 28lb Darlington rail was cast into the top.

Details of the concrete mix designs were not available at the time of writing. It is worth noting that the local creek would have provided a source of fresh water. The marine aggregates in this location are not ideal for concrete although it is reported that beach sand was used in the construction of the Tologa Bay Wharf to the South (Williams 2009).

CONCRETE DETERIORATION

Concrete in its simplest form of cement, aggregate and water is a very versatile and cost effective construction material. Locally produced it can be used for a wide range of applications by relatively unskilled labour. In normal applications, when well-proportioned and constructed, concrete structures exhibit excellent durability. Even in more challenging environments it has often proven to be very resilient. Despite this, in some situations, concrete is affected by mechanical or chemical agents resulting in physical damage to the elements or breakdown of the cementing matrix.

Reinforcement Corrosion

Although it can be used in its own right for mass concrete elements, concrete's poor tensile strength limits its use unless it is combined with materials with good tensile performance such as steel. Although steel corrodes in air, the highly alkaline environment in concrete forms a stable 'passive' layer on the metal's surface. This, combined with the physical barrier provided by the surface layer of concrete, usually prevents corrosion from being a concern.

However, if the passive layer on the steel breaks down due to chemical action, corrosion can take place. Neutralisation of the passive layer can occur in the presence of acidic agents, such as atmospheric carbon dioxide (a process known as carbonation), or chlorides. Chlorides can come from a wide range of sources including: the raw materials used to make the concrete, such as sea dredged aggregates or sea water; chloride containing admixtures; or, external environmental sources such as the marine environment.

The corrosion process results in the parent steel being replaced by higher volume iron oxides. This volume increase creates bursting forces which over time crack the concrete surface layers. Aside from the visual impact, the cracking removes the physical barrier provided by the concrete allowing the corrosion reactions to continue unhindered. If untreated, this results in increased cracking and spalling as pieces of cracked concrete become detached.

Where corrosion results from carbonation, cracking and spalling is often apparent over large areas of the structure well before a significant loss of steel has occurred. In the case of chloride induced corrosion however, electrochemical currents can be generated which can create highly localised corrosion. In marine structures especially, this can be manifested as cracking, rust staining and spalling damage to structural elements in the intertidal 'splash' zone (Standards Australia 2005). Within this zone, periodic immersion keeps the concrete wet enough to maintain electrical conductivity while still allowing ingress of oxygen necessary for the corrosion process. Other parts of the structure can be affected to varying degrees depending on a range of factors including exposure, and the depth and quality of the cover concrete.

In modern concrete structures, these risks are recognised and precautions are taken to avoid or reduce the effects through appropriate design, construction and maintenance. However, in older structures, built prior to these effects being recognised or understood, this type of deterioration is quite commonplace. Because of this, repair of affected elements is frequently necessary to maintain the serviceability of the structures.

CONCRETE REPAIR

There are a range of potential approaches to treat concrete deterioration including:

- No action;
- Stabilisation - treating the structure to slow down or stop the deterioration processes;
- Cosmetic repair to restore the original appearance;
- Structural strengthening or stabilisation to reduce the risk of major structural failure;
- Restoration to an original form and function;
- Enhancement to improve the form or function to allow for a change of use, increase in structural capacity, extension of the design life or other performance requirements.

The chosen option or combination of options will be influenced by a range of factors including the underlying causes of the deterioration, the importance of the structure, the ease and potential success of any repairs, and the cost.

Repair Materials and methods

A wide range of materials are used for concrete repair, the choice usually influenced by the size of the repairs and the desired outcomes. Common materials include conventional mortars or concrete, and proprietary polymer modified cementitious systems. For special applications such as crack repair or where very good chemical resistance is required epoxy or other resin based systems can be used.

Application of repair materials can be carried out using the same methods employed for normal cement based materials. These include: hand placing, wet or dry spraying, or in situ casting using grouts, mortars or concretes depending on volume and accessibility.

Repair to damage caused by reinforcement corrosion

Where corrosion of steel is identified as a cause of the deterioration, in addition to repairing damaged concrete, the causes of the corrosion also need to be addressed. If this is not done, the corrosion mechanism will continue resulting in further deterioration.

In addition, loss of steel to corrosion may require replacement reinforcement to restore the strength of the structure. Conventional steel can be used in these cases, however if corrosion prevention measures are not applied, corrosion of the new steel or accelerated corrosion of existing steel may occur due to galvanic effect. Some protection can be provided by applying impermeable resins or anti-corrosion priming systems to the steel. In addition, many repair materials give improved protection to the reinforcement.

Where chlorides are present additional protection options may be required. These may include more corrosion resistant reinforcement such as galvanised, epoxy coated and stainless steels or FRP composites. Additionally, corrosion prevention or inhibition measures may be applied to the repair or the structure. These include: water repellent surface treatments to help the concrete to dry out and reduce its electrical conductivity; corrosion inhibiting admixtures, corrosion inhibiting surface treatments; electrochemical chloride removal and cathodic protection systems to control corrosion currents (Standards Australia 2005).

Repair of Historic Structures

Repair of historic structures can adopt similar treatment processes to those outlined above. However, there are some specific considerations which should be taken into account (New Zealand Historic Places Trust 2007):

- Work should generally match the original in terms of quality, materials and detailing;
- Repairs should generally be with original or similar materials;
- Repairs to a higher standard may be justified where life expectancy is being increased;
- Alteration should be avoided whenever possible.

It is also important to note that applying ‘new building’ design requirements can be excessive or in some cases impossible to meet with historic structures. In such cases alternative methods of analysis or ‘an observational method’ may be more appropriate to establish a suitable approach (International Council on Monuments and Sites 2003).

In essence, repairs to historic structures, in addition to providing restoration of performance and/ or functionality, should be sympathetic to the nature and significance of the structure. The interpretation and application of these recommendations to this project is discussed in more detail elsewhere (Hudd et al 2021).

PROPOSED REPAIR METHODOLOGY FOR TOKOMARU BAY

Inspection of the wharf between 2007 and 2013 attributed the observed spalling and loss of reinforcement to chloride induced corrosion due to the age of the structure in the marine environment (Tokomaru Bay Heritage Trust 2014). The effects were apparent to varying degrees throughout the structure but most obviously in the wharf piles. Based on the overall condition of the structure, repair was considered to be feasible with restoration of the piles as the highest priority. It was judged that no action would lead to increasing loss of structural integrity with failure of individual elements with either a progressive or a catastrophic collapse of the entire structure. Restoration options suggested at the time were either: complete replacement of the damaged piles; or, repair with a concrete jacket similar to the methodology employed for the Tologa Wharf Restoration (Williams 2009).

The former option had the advantage of being able to use piles of the same size and shape as the original ones thus retaining the structure's appearance. This methodology does however present significant challenges, with total replacement requiring removal of other parts of the structure to allow the new piles to be driven. Jacketing the piles, while much more feasible, was a less visually acceptable option.

At the time these recommendations were made no action was taken. However, the closure of the wharf in 2017 increased the local community’s desire for action resulting in a formal proposal for trial repairs of four of the deteriorated piles in 2019.

Trial Repair Proposal 2019

The proposed method followed conventional practice, removing the damaged concrete and steel, and strengthening the remaining core concrete with a concrete jacket reinforced with FRP rebars. The intention being to trial the method on the four piles at the landward end of the wharf.

Reinforcement

The use of FRP eliminated the risk of future corrosion and further deterioration and, importantly, was locally available. It offered higher tensile strength than steel, comparable bond strength, thermal properties and good fatigue and creep resistance. In addition, its lighter weight, approximately one quarter of the weight of equivalent steel made on-site handling much easier. This was important considering the need to locate reinforcement cages in the confined space under the wharf. The material is flexible enough to be ‘sprung’ into place, but unlike steel it cannot be bent to shape on site; any ‘bends’, including links, have to be applied at the manufacturing stage.

Replacement Concrete

The proposal included provision for the use of proprietary repair materials or site cast concrete. The use of proprietary materials, although potentially offering advantages over the site cast concrete for placement, was discounted because of cost.

The site cast concrete specification was based on the durability requirements for concrete in reinforced concrete structures in a marine environment (Exposure classification “C2” splash zone) (Standards Australia 2005). This called for a Class S40 grade with a minimum cement content of 400 kg/m^3 and a maximum water cement ratio of 0.4. These requirements, as with the New Zealand Standard for reinforced concrete (Standards New Zealand 2006), are primarily intended to provide additional protection to conventional steel reinforcement in the marine environment.

The small volumes (less than 1 m^3 per pile), uncertainty over the pouring schedule and a two hours’ drive meant ready mix concrete was impractical. Accordingly, site batched concrete was chosen and to facilitate this a 700 litre tractor mounted concrete mixer was purchased. The tractor attachment made it possible to locate it wherever it was required on site so concrete could be mixed and poured without additional handling.

The choice of site mixed concrete also presented challenges for both placing and compaction in the restricted space beneath the wharf. Conventional placing techniques required clear access with the forms open at the top. To ensure adequate compaction, the concrete would need to be placed in lifts which could only be achieved if the mould was installed in sections as the pour proceeded. Access and time constraints between tides meant this would be impractical. Pumping would overcome some of the access issues allowing the concrete to be placed to the full height of the mould but still did not address the compaction requirement. It was also recognised this would add complexity and cost.

A solution which offered to address both issues was SCC. This could be poured from the top of the wharf into the forms without pumping and did not require additional compaction after placing. In addition, typical SCC mix designs would comply with the specified concrete mix design parameters. Fortuitously, the University of Waikato, which was running student research projects on the historic structures in the area, had teaching staff with experience of SCC mix design. Using this experience a preliminary mix was developed based on a published mix design (Hudd & Edwards 2001).

The mix design had a cement content of 350 kg/m^3 with 150 kg/m^3 of limestone flour as a filler for stability and flow. To meet the concrete specification for the trial repairs, the limestone flour was replaced with fly ash. This combination not only met the given specification but also the New Zealand standard requirement for concrete in a marine environment - a minimum binder content of 450 kg/m^3 with 30 percent fly ash (Standards New Zealand 2006).

Laboratory Tests

Local aggregates were obtained, and laboratory tests were carried out at the University of Waikato using the following mix design:

General Purpose Cement	350 kg
Fly Ash	150 kg
Nuhaka black Sand	900 kg
Nuhaka 13mm Aggregate	800 kg
Water	175 litres

Sika® Viscocrete® -5-555 (NZ) superplasticiser to give the required flow characteristics

The tests were used to confirm that acceptable flow spread using an inverted slump cone, could be obtained with the mix (Figure 3). Stable flow spreads between 650 and 750 mm were achieved with water: binder ratios between 0.35 and 0.4 and superplasticiser dosed at 0.3 to 0.5 litres per 100kg of binder (Figure 3). At higher flow spreads bleeding and aggregate segregation were observed.

Figure 3. On-Site Flow Test of Trial Self-Compacting Concrete Mix

Cylinders were cast for compressive strengths testing at 7 and 28 days. Strength tests showed the mix would meet the Class 40 minimum strength requirements with 7 day strengths typically 30 - 40 MPa and 28 day strengths in excess of 50 MPa. Cylinders were water cured for three days after demoulding, then wrapped in polythene. It was felt this would be more representative of site curing although it is likely the strength performance was reduced, especially with the fly ash in the mix.

SITE TRIALS

Having obtained satisfactory performance of the mix in the laboratory, a series of site tests were carried out prior to the trial repairs to identify any issues with the proposed mix or method in practice.

These tests showed that the same mix properties were achievable on site. For simplicity, the batch size was based on whole bags of cement and fly ash, with aggregates being weighed into buckets. Without an accurate means of determining aggregate moisture content on site, the admixture dose rate was fixed and water added to give the desired flow properties.

A trial pour was carried out using a mock-up of a section of a pile. A section of the mould with reinforcement, delivery pipe and hopper was set up on the beach with the mixer at the wharf deck level. An initial attempt was unsuccessful due to poor flow through the pipe. This was attributed to the length of the delivery pipe which hadn't been pre primed. For subsequent pours, this was addressed by shortening the delivery pipe and priming it with a cement slurry. Having resolved these issues the mock pour was carried out successfully. The SCC easily filled the mould, encapsulated the reinforcement and gave an acceptable off-form finish. The satisfactory outcome of these tests gave confidence that the material and methodology could be used for the trial repairs.

Concrete Surface Preparation

Successful repairs require all damaged concrete and reinforcement to be removed from the repair area. For repairs to load bearing elements, as in this case, the load must be relieved during the repair process so the repaired section will carry load once finished. For this project steel frames were fabricated and positioned either side of the bent being repaired with legs bearing on the seafloor and hydraulic jacks positioned under the wharf beams to take the load.

Once this loads were transferred to the frame, preparation of the piles stared with removal of the damaged concrete and rebar. An additional 300 mm of sound concrete was removed below the repair area to ensure the repair had a sound surface to bond to. It was noted that all of the steel exposed during this operation, both longitudinal and spiral, was in excellent condition with little or no corrosion visible and original wire ties still present.

Reinforcement Fixing

The FRP reinforcement cages were assembled in two halves each consisting of eight 22 mm bars arranged in pairs with each bar centred 35 mm from the corner of the links (Figure 4). Links were spaced at 150 mm centres secured with plastic zip ties at each intersection. Gloves were recommended when handling the longitudinal bars as they were machined during manufacture to give a ribbed profile resulting in exposed glass fibres.

The reinforcement cage sections were positioned either side of the piles with the top ends tied to stainless steel starter bars bolted to a purpose made bracket (Figure 5). The bottom of each cage was positioned over the undamaged pile section using spacers. The sections were secured at points around the pile with short lengths of GFRP bar fixed into the substrate concrete. The two halves of the cage were tied together with additional links to give continuity around the pile.

Figure 4. FRP Reinforcement Cage Half

Figure 5. Starter Bar Connection

Once the reinforcement was installed, the sections of the mould were lowered into place and assembled around the pile. The bottom of the mould had a reduced section to match the pile profile. The position relative to the pile and the reinforcement was fixed using bolts threaded through the sides of the mould. There was a concern raised that, as the mould would be immersed in sea water prior to the concrete pour, normal oil based form release agents might become ineffective. To avoid potential issues this could create, a ‘homemade’ beeswax based form release was prepared.

Trial Concrete Pours

One of the many challenges with this project was weather and sea conditions. Clear settled conditions were required over several days to allow time to fix the reinforcement, install the mould and pour the concrete. On several occasions planned pours had to be postponed due to poor conditions. Once a suitable forecast was obtained, the reinforcement and mould were installed and preparations made for the concrete pour.

Prior to pouring, the mould was flushed with fresh water which was drained from the inlet port at the bottom of the mould. The delivery pipe was primed with a cement slurry then connected to the inlet pipe. The concrete was mixed and flow checked then discharged into the feed hopper at the upper end of the delivery pipe. Once the mould was full the inlet port was closed off and the delivery pipe removed. Using this method, two trial repairs were carried out in February 2021.

The forms remained in place for seven days, their removal, as with the concrete pours, being subject to the weather and sea conditions. After stripping the repairs were wrapped with wet hessian and plastic.

FUTURE WORK

During the course of this project a number of areas for future research were identified:

- Although the repair methodology was successful, the oversize repairs detract from the original form of the structure. Options to repair the piles to the original profile are being investigated;
- No site testing has been conducted to confirm the cause or determine the extent of the deterioration beyond what is visible, future projects are planned to address this;
- It is anticipated that some deterioration of the repairs may take place. Long-term monitoring of the repairs is required to assess and address any issues identified;
- A method needs to be devised to address the deterioration in the headstocks and longitudinal beams;
- Spalling and reinforcement corrosion is apparent on the underside of the concrete deck. Research is being undertaken to identify practical, non-intrusive, methods to address this.

CONCLUSIONS

At over 80 years old, the concrete wharf at Tokomaru Bay exhibits deterioration typical of chloride induced reinforcement corrosion in marine environments. The closure of the wharf in 2017 prompted efforts by the local community to save the structure and see it reopened. This culminated in a proposal to trial a repair methodology incorporating advanced concrete materials as the practical first stage of a restoration process. Research conducted by the University of Waikato and site trials carried out in the summer of 2020-2021 successfully demonstrated the viability of the proposed materials and method.

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