Bridge Over Cooks River at Tempe – Sydney NSW
Emergency Underpinning of Piers

R.Oates (RTA), L. Prasad (RTA), W.Stalder (RTA)

SYNOPSIS

Underwater inspections during April 2002, revealed that the driven prestressed concrete piles of the existing 42 year old, 3 span, 100m long, 6 lane bridge across Cooks River at Tempe were in extremely poor condition due to a phenomena known as Delayed Ettringite Formation. The bridge is on the Princes Highway, a major heavy vehicle route in Sydney with an AADT of 55000 vehicles. A bridge closure would have caused great inconvenience to the travelling public and heavy transportation industry in the southern part of the City.

The Bridge Section of the Roads and Traffic Authority of NSW (RTA) developed an innovative concept for supporting and jacking each pier without the need for closing the bridge to traffic. Both pilecaps were supported on steel yoke beams held up by stressing bars attached to steel transverse beams located on top of new bored piles. The supporting system was carefully designed to eliminate the need for even minor demolition of any part of the existing structure during installation and allowed for future encapsulation into permanent works.

The project was successfully completed through the joint effort of RTA's in-house resources and the expeditious contractors, Waterway Construction. Constraints were numerous and included conducting the design and construction concurrently, lack of redundancy in the structure, night construction, safety of the travelling public, safety of workmen, monitoring and control of traffic, monitoring of bridge movements, environmental issues, construction machinery loadings and weather.

This paper provides an overview of the project with emphasis on how the issues encountered in design and construction of the emergency underpinning of the piers, were resolved.

1 BRIDGE DESCRIPTION

Built during 1960, the Bridge over Cooks River at Tempe is a 91.4m long, 3 span, pre-cast, post-tensioned I girder bridge consisting of 40.235m central span and 25.60m flank spans. The 27.35m drop-in unit in the central span is made up of 3 precast segments of 9.14m each. Typically the 9 girders are transversely spaced at 3.05m centres. The bridge was originally fixed at the abutments and one movement joint was provided at the stepped joint in the central span. The bridge was designed to carry HS-20 loading and is currently on a B-double route.

The bridge is located in a highly aggressive environment and is very close to the sea.

The abutments are reinforced concrete walls supported on a group of vertical and raked pre-cast driven piles. The piers are reinforced concrete walls supported on single rows of 28 pre-cast, prestressed, octagonal driven piles. The geotechnical design capacity of the octagonal piles is 500 kN. The piers act as flexible piers transferring the majority of longitudinal forces to the abutments.

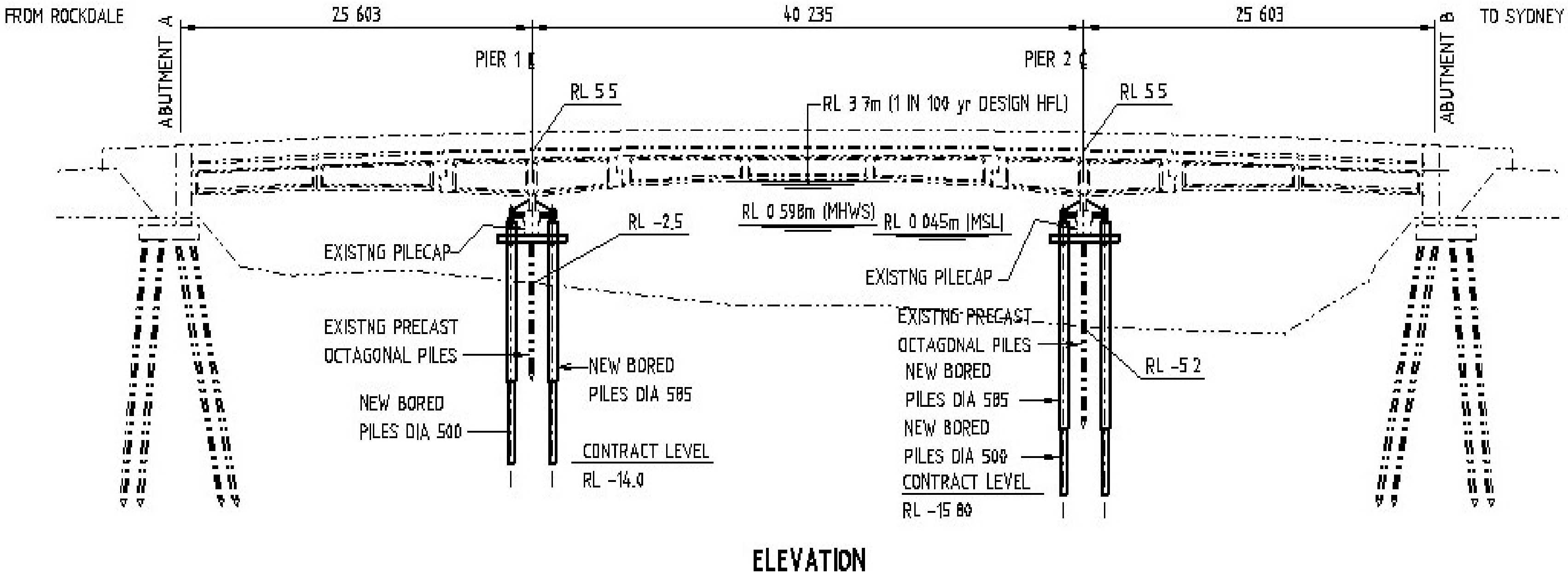

Figure 1: Elevation of the Bridge over Cooks River at Tempe showing the existing piles and the rehabilitation scheme.

The bridge has two footways and carries two major services viz; 750 diameter water and 450mm diameter gas pipes, under each footway.

2 GENESIS OF THE PROBLEM

The bridge was constructed with concrete skirts surrounding the juncture of the piles and pilecap. With this arrangement it was not possible to inspect the piles in the tidal zone. An underwater inspection during April 2002 indicated a possible problem that was confirmed when the skirts were cut off. The subsequent inspections revealed that the existing piles under the piers had severely deteriorated and the concrete in the piles had become porous and brittle. Divers inspecting the piles found they could easily pull out by hand chunks of concrete. Most pile reinforcement was exposed and severely corroded. One pile had disappeared completely. A detailed underwater inspection was then undertaken as a result and the condition of each pile was videographed, mapped, and damage classified.

Figure 2 shows the typical extent of deterioration in the pre-cast concrete piles and the estimated strength of the concrete in the different zones within the cross section.

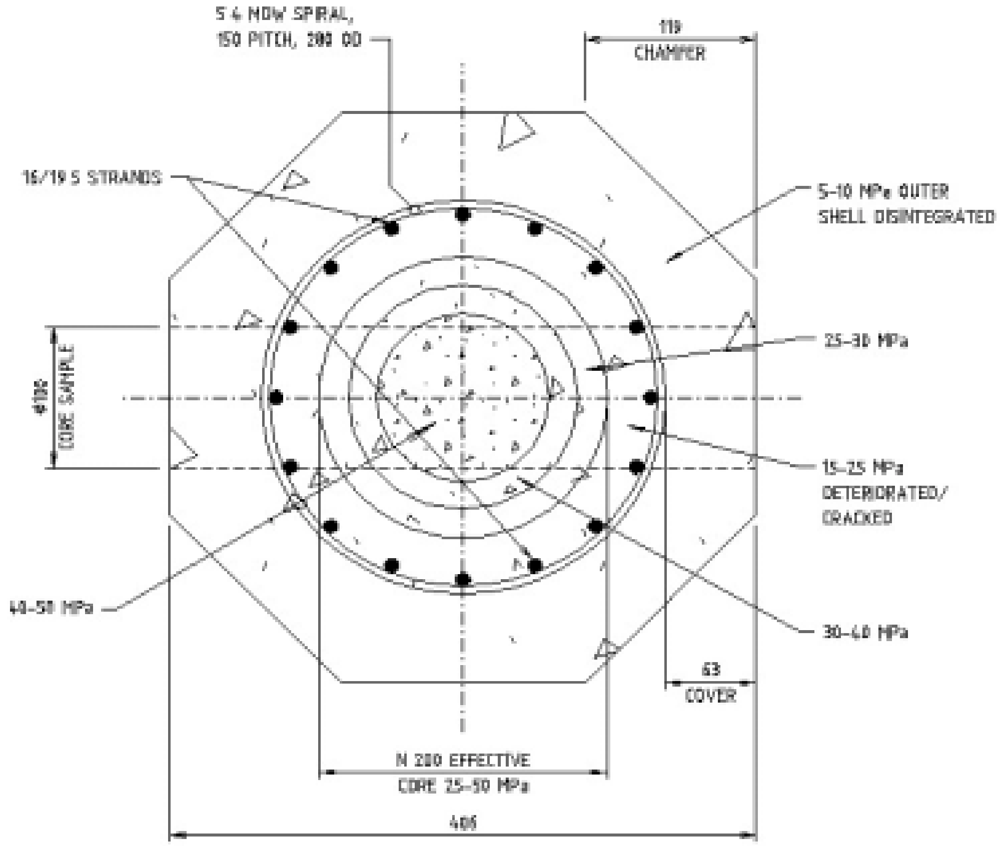

Fig 2: Estimated strengths of extensive zones of damage within the cross section of existing piles

The reason for such extensive damage was attributed to a phenomenon known as Delayed Ettringite Formation (DEF). Microscopic examination of the cores extracted from Pile 76 at Pier 2, indicated that the deterioration mechanism was initially caused by DEF followed by attack of the cement matrix by magnesium. This is a most unusual phenomenon and there are very few examples in Australia and very few bridge related examples worldwide.

2.1 Delayed Ettringite Formation

Ettringite is a scarce hydrated aluminium sulphate. The primary formation of ettringite in the initial stages of hydration is seen as beneficial because it enables the setting regulation; a damaging role is often attributed to the ettringite formation in hardened concrete. Damage from ettringite formation was first identified in heat-treated, pre-cast concrete elements, which during use had been exposed to open-air, weathering, with frequent wetting. The concretes affected were mainly high-grade concretes of high strength and low porosity. [Stark, Bollmann et.al (1)].

G.M.Idorn Consult (2), concrete technology specialists, defined Delayed Ettringite Formation, DEF, as the formation of ettringite and associated expansion observed after heat curing at too high a temperature. They reported on a large number of experiments that have shown that pastes, mortars and concrete exposed during the hardening process to high temperatures, exhibit expansion and cracking when subsequently exposed to moist conditions. G.M. Idorn (2) explains the phenomenon as follows.

Ettringite becomes unstable at temperatures exceeding 60-70^{o}C. The result of heat treatment at or above these temperatures is

(a) Decomposition of the ettringite already formed.
(b) No further formation of ettringite.

The instability of ettringite seems to be related to the amount of alkali. The higher the alkali content, the lower the temperature limits. Subsequently, during the storage or service at ambient temperature and moisture, ettringite slowly forms followed by a homogeneous expansion of the paste, which ultimately results in cracking of the hardened concrete. DEF is considered a form of internal sulphate attack.

On a microstructural level, DEF results in gaps completely surrounding aggregate particles. The width of the gaps may be more or less filled with ettringite crystals.

2.1.1 Issues arising out of DEF on the piles supporting the piers.

- Wide spread cracking and spalling, with associated major corrosion of steel pre-stressing tendons and reinforcement, resulting in loss of integrity of the pile mass. Significant loss of concrete and reinforcement from a number of piles.
- Loss of material integrity means that load capacity calculations are irrelevant.
- Assuming stress calculations if made are deemed valid, a live load factor between 1.2 and 1.3 for T44 loading will result. This is unacceptable since the lowest live load factor allowed for older bridges in good condition, with an impeccable performance history and a rigorous monitoring regime provided, is 1.6.
- Advice from Trin Cao (Concrete technology specialist, formerly CSIRO, now Connell Wagner) and literature is that the history of the deterioration mechanism is likely to be quite recent and therefore occurring at a relatively rapid rate (say, over last four to six years- not over the last 40 years).
- Failure will be sudden and probably catastrophic - very little warning.

- Risk of failure is related to vehicle mass, self weight of bridge superstructure, load shedding and rate of deterioration.
- While unlikely that failure would have occurred in the following few months, the phenomenon is so unpredictable, and with so little experience –on which to base a judgement; the risk of failure (related consequences) was considered not acceptable.

3. RECOMMENDED MEASURES

Given the high AADT of the bridge, road closure was not considered as a feasible option. After a thorough review of all the issues specific to the bridge it was agreed to undertake the following measures.

(i) Impose and rigorously enforce a load limit of 15 tonne on the structure immediately. The load limit of 15 tonne was a result of a study of the condition of the concrete material in pile, traffic count data and the use of advanced mathematical techniques of risk and probability (reliability assessment). The imposed load limit minimised the inconvenience to almost 80% of the current users.

(ii) Install continuous on-line monitoring system for the bridge piers to detect any movements.

(iii) Develop and design an underpinning system without the need to demolish any structural component.

(iv) Improve redundancy in the structure by closing up deck joints.

3.1 Reliability Assessment

In order to impose a load limit, the capacity of the existing piles in the deteriorated condition had to be assessed. The usual methods of assessment using current bridge design codes were considered unsuitable since such codes were developed for the purpose of new designs and contain conservative assumptions. Reliability methods were, therefore, adopted to quantify the risk associated with the predicted failure (crushing of un-reinforced concrete) by assessing parameters such as the compressive strength of concrete and the occurrence of maximum live load (all 6 lanes occupied by 15 tonne vehicles). The reliability of the bridge was then estimated accounting for previous load history, the load limit of 15 tonnes, and a probabilistic model of residual strength. The acceptable safety index was taken to be approximately 3-3.5.

The available data about the strength of concrete in the existing piles was extremely limited ie., assessed from only a few core samples taken from existing piles. Traffic count data, collected over a period of one week, was also used in the analysis.

The safety index β (ratio of mean value (μ_G) and standard deviation (σ_G) of the limit state function, $G=R-S$) was chosen as 3-3.5

Where

R = Resistance
S = Load effect

Prof Stuart Reid (3) from the University of Sydney carried out the reliability analysis of the axial load capacities of pile based on the modelled strength for a group of seven piles (considering effective load sharing). The strength model accounted for modelling uncertainties, random variations and estimated strength correlation effects. The strength distribution was estimated from simulation results. Prof. Reid (3) made the following recommendations.

- An assessment using an updated reserve strength distribution accounting for the "proof loading" associated with the uncertain heavy traffic loads prior to the introduction of the 15 tonne load limit, showed the safety index was around 3.6
- A subsequent assessment carried out to evaluate the effect of the continuing strength degradation over a further period of 3 months showed that the projected value of safety index fell to 1.6 over that period.
- The results of the reliability assessments confirmed the appropriateness of the RTA response (including the imposition of a 15-tonne load limit) and confirmed the need to complete the repairs as quickly as possible.

3.2 Continuous Online Monitoring

It was decided to install continuous online monitoring of the bridge for the entire duration of the project. The purpose for installing online monitoring was:

- To detect and measure any movements that may occur in piers given the condition of the existing piles.
- To monitor the vibrations in the existing pile caps whilst driving the new piles for the underpinning works.

Infratech Systems and Services (4) were commissioned to install, operate and maintain the monitoring system. Four 50mm range displacement transducers were installed to measure the vertical displacement of the pile cap and four 25mm range displacement transducers were installed to measure lateral movements. The transducers used a spring and wire system to measure displacement between two points.

Timber piles were driven approximately 1m upstream of each pier and the transducers were referenced to these piles.

An alarm was set up to be triggered by sudden changes in deflections. The threshold movements of 3mm and 1.5mm on the vertical and lateral transducers respectively were used. When the monitor triggered, data was recorded for 10 seconds and a SMS text message was sent to key personnel involved on the project.

For monitoring vibration an array of four triaxial geophones were installed. The geophones were so orientated so that the radial channels measured vibrations along the bridge, the transverse channels measured vibrations across the bridge, and the vertical channels measured vertical vibrations.

The vibration levels from traffic generally ranged up to 2mm/sec. The values above this were believed to be associated with works to remove the concrete skirts from the sides of the pile cap. Examination of a recording at 14mm/sec was consistent with an accidental impact on the pile cap from a barge. A review of the monitoring results indicated that during driving of pile casings the vibrating hammer produced a lower level of vibration compared to the drop hammer.

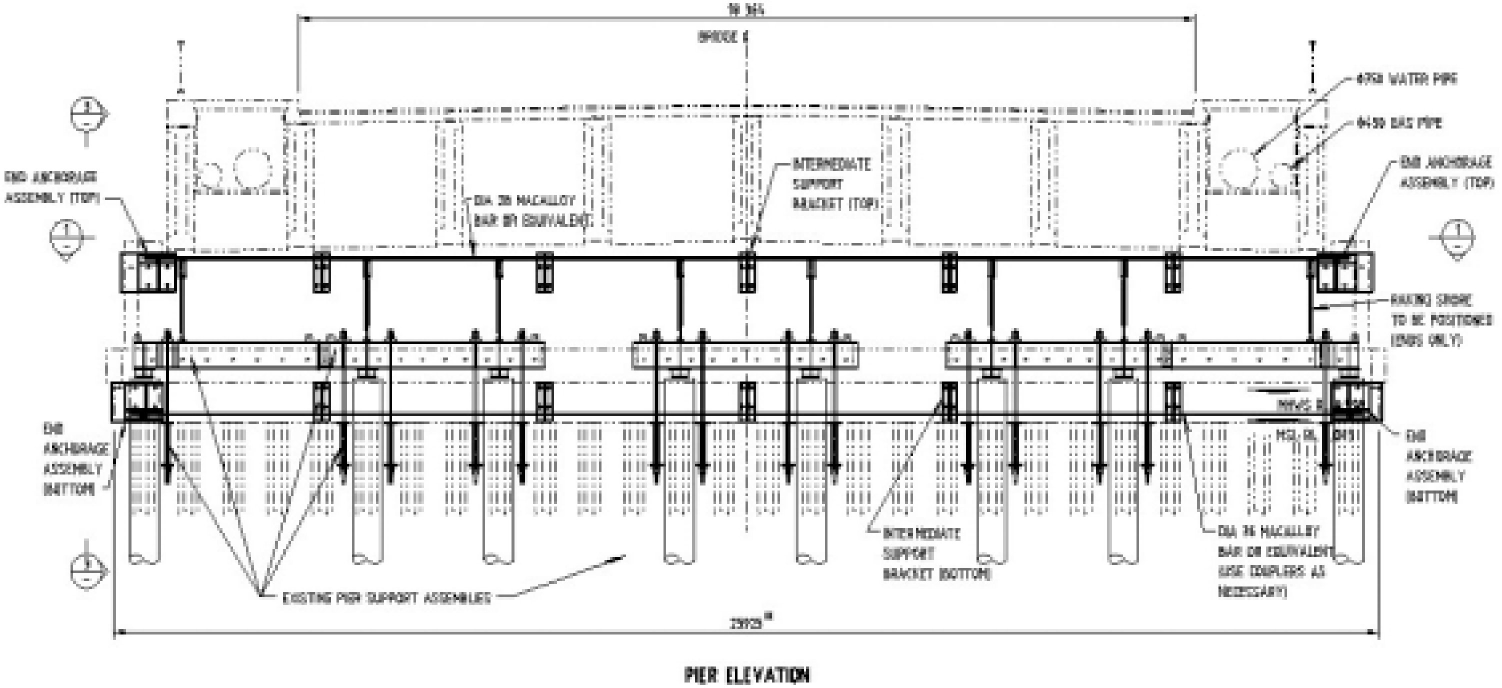

Figure 3: Elevation of the pier and the supporting arrangement

Based on the data recorded during construction, no significant movements of the substructure were detected.

3.3 Development and Design of Underpinning System

The structural system for underpinning the piers consists of 8 pairs of new piles alongside each pier and a system of structural steel support and yoke beams. The location of existing piles was surveyed and the layout for the new piles chosen so that the yoke beams could be easily installed in the gaps between the existing octagonal piles. A pile configuration to suit readily available steel casing (610 mm outside diameter) was selected and each pile casing was driven to rock through access holes created in the deck. Rock sockets under each pile were then excavated and after cleaning, reinforcement cages were installed and concrete poured to required levels.

The support beams were installed in the transverse direction to the axis of the bridge and supported on stools sitting on the new piles. The yoke beams, installed in the longitudinal direction to the bridge were pulled up under the pile caps to transfer the loads to the new piles using 50mm VSL stress bars. Groups of stress bars were linked together to counterbalance loading on the piles and were stressed simultaneously to a level equivalent to 80% of self-weight of the bridge.

The structural system was so designed that most of the structural steel components could be easily removed after a new concrete pile cap is cast around the pier and over the new piles. Controlled release of the stress in the VSL bars after will also ensure that the load path to the new piles is established.

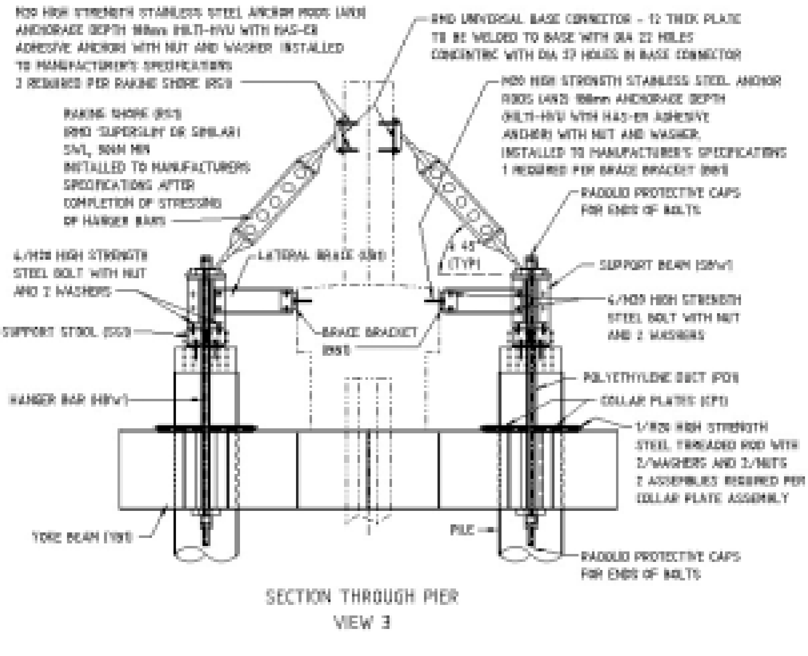

Figure 4: Section through pier showing arrangement of supporting system

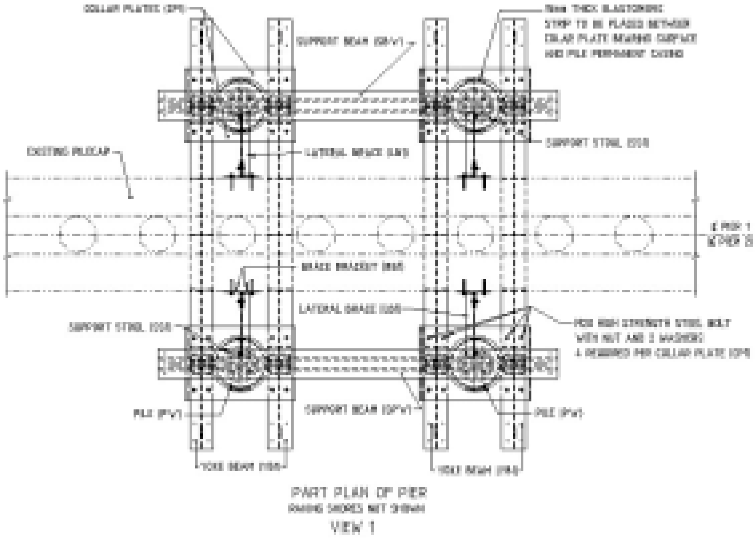

Figure 5: Part plan of the supporting system for the pier

The major service pipes viz., 750mm diameter water and 450mm diameter gas pipes underneath the footways and Telstra cables in the river bed imposed severe constraints on the pile layout. No piles could be provided through the deck between the outer girders on each side of the bridge due to presence of these services. Low headroom also prevented driving from underneath the structure. This increased the distance between the supports for the pier in the end regions. Consequently, the pier wall was analysed as a continuous deep beam spanning between the adjacent yoke beams. Lack of capacity in these regions was overcome by installing external post tensioned stress bars systems.

An attempt was made to design and use Carbon Fibre Reinforced Polymer (CFRP) strips externally to increase the tension capacity at the bottom of the pier wall. This method of strengthening was rejected after investigations from SAVCOR (5) showed that the concrete substrate did not have sufficient strength for this type of an application.

3.4 Improving Redundancy

To develop a secondary load path should a partial failure of a pier occur, the gaps between the ends of the main girders and the abutment back walls were packed out with steel plates and the steel finger expansion joint at the southern end of the drop in span was plated over. Initially it was intended to grout up the expansion gap under the fingerplates but access was poor and removing the grout after underpinning the piers would have been very difficult. Furthermore the gap was already packed solid with 40 odd years accumulation of rubbish. The purpose of filling in the gaps was to develop sufficient arching action in the overall structure so that, should a pier begin to fail, it would allow sufficient time to stop traffic and remove work crews from under the bridge.

4. SUMMARY AND CONCLUSIONS

The authors draw the following conclusions from their experience on the project

(i) It becomes very important to keep some degree of redundancy in the structure while designing bridge components that are not easily available for inspection, in particular, while designing foundations. Lack of redundancy for an important element in the load path such as the piles is what made this project extremely critical.

(ii) Underwater inspections are important. The level of cleaning of the marine growth during inspections needs to be clearly specified and followed. This helps in earlier detection of deterioration and damage. Comprehensive guidelines for underwater inspections also need to be developed for pile structural assessments.

(iii) When assessing the capacity of structures built to older design codes the application of reliability assessment techniques offer a viable alternative to the adoption of load and capacity factors specified in the current Bridge Design Code.

(iv) The successful completion of the project demonstrated that road closure is not the only option available and other risk alleviation methods can be carefully devised and enacted.

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6. ACKNOWLEDGEMENT

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7. DISCLAIMER

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