6. Discussion and Recommendations

6.1. Condition of concrete elements

Inspection and diagnostic testing of the concrete elements yielded that the elements are subject to a range of defects including spalling, cracking and poor compaction. The chloride content analysis and carbonation depth measurements undertaken on samples extracted from site exceed their thresholds indicating that corrosion may have initiated in some locations. Three options are proposed for remediation the issues found in the above water reinforced concrete elements of the Hawkesbury River Underbridge:

- **Option 1** – Do Nothing and Monitor.
- **Option 2** – Concrete repairs to substructure supporting southern approach spans (Southern Abutment, Piers 1 and 2, Deck and Girders) and North East Corner of Pier 5 caisson Works to involve re-augmentation of corroded and deformed reinforcement as required and installation of sacrificial anodes at patch repair locations.
- **Option 3** – Option 2 + concrete crack and spalling repairs to all damaged concrete bridge elements and reinstatement of deteriorated render on top of caissons. Application of anti-carbonation and silane coatings to substructure supporting southern approach spans (Pilasters, Deck and Girders). Scour protection to Northern Abutment and replacement of walkway supports at Pier 1.

Due to the extent of deterioration observed it is recommended Option 3 'concrete repairs' be completed within the next 2 years for all concrete superstructure elements, and within the next 5 years for all other (substructure) concrete and steel superstructure elements.

6.2. Condition of steel elements

Inspection of the steel superstructure identified generally consistent defects across the structure, predominately surface corrosion and protective coating deterioration. Some isolated members have exhibited more severe defects such as perforations.

Remediation of the steel truss superstructure of the bridge is required to extend the service life of the bridge. These repairs ideally should be undertaken as part of the coating works program that is required, as detailed in SMEC's 'Protective Coating Assessment Report', attached in Appendix F, and discussed in Section 6.1.2. Surface preparation techniques (abrasive blasting) implemented during coating repairs works will address the surface corrosion/minor pitting observed, however this may also expose additional defects that have not been identified.

Where the surface preparation techniques expose significant pitting/perforations etc, the defects shall be repaired in conjunction with the relevant sections of MN C 10302 prior to recoating. Where strengthening works are required, again this shall be undertaken prior to recoating. Ultrasonic testing should be carried out on the cleaned and prepared metal substrate to accurately measure the element's thickness, and provide an accurate, dated benchmark for any future condition and/or load-rating assessments.

Minor defects, such as corroded bolts/rivets should be repaired prior to undertaking the coating works, as part of routine maintenance practices. Severe corrosion of specific rivets and/or connect contacts should be replaced in conjunction with the coating works, to ensure the new contact surfaces are properly and adequately prepared. These removed elements should be taken for materials testing – see Section 6.5 below for further discussion on this.

Due to the size of the structure, it is understood that the repairs will require extended timeframes for implementation. As such, it is recommended that the repairs to the northern spans of the bridge (Steel Spans 7 and 8) be undertaken as a priority, as this is where the most severe defects were identified.

It is recommended that the repair works (in conjunction with the coating works) to the bridge steelwork commence within 5 years, with minor repairs undertaken prior to that, as part of regular maintenance works. Regular cleaning of debris is also recommended, especially in the sections of the bridge below track level. All works are to be undertaken in accordance with all relevant environmental and safety regulations.

6.3. Protective coating

The extent of the surface corrosion of steel elements of the bridge is estimated to be present on 3-5% of the total surface area. The presence of coating breakdown is estimated to be on 50% of the total surface area.

A recommendation for replacement of the protective coating has been provided and comprises either a system EHB6 (high build epoxy MIO) or system PUR5 (epoxy intermediate with polyurethane finish) in accordance with AS2312.1. (Is the polyurethane coating UV stable? The bridge will be in full sun and get really hot. Polyurethane is known to yellow, and crack under UV exposure). Either system would require a total system application thickness of 325μm with a surface preparation Class 2 ½ (Sa 2 ½) in accordance with AS2312.1.

Surface preparation method Class 2 ½ (Sa 2 ½) is 'near white' blast cleaning which removes millscale, rust and foreign particles to the extent that only traces remain in the form of spots or stripes, and the cleaned surface shows varying shades of grey. Table 40 provides the recommended paint systems for complete replacement. Both systems have life to first maintenance expectance to be in excess of 25 years.

Table 40 Recommended paint replacement systems for protective coating

System	1st Coat			2nd Coat			3rd Coat		
	Type	Paint Ref No.	Nom DFT (μm)	Type	Paint Ref No.	Nom DFT (μm)	Type	Paint Ref No.	Nom DFT (μm)
EHB6	Zinc rich primer	C01 C02	75	Epoxy MIO	C13	125	Epoxy MIO	C13	125
PUR5	Zinc rich primer	C01 C02	75	High build epoxy	C13	200	Polyurethane gloss	C26	50

An alternative painting strategy for structure has been proposed requiring less stringent surface preparation and patch painting. This scheme will provide a service life of 5 to 10 years. The surface preparation for this strategy will comprise removal of loose deteriorated coating, salt and other soluble contaminants and corrosion by use of hand tools. Table 41 provides the recommended paint systems for complete patch painting.

Table 41 Recommended patch paint systems for coating repair

System	1st Coat			2nd Coat			3rd Coat		
	Type	Paint Ref No.	Nom DFT (μm)	Type	Paint Ref No.	Nom DFT (μm)	Type	Paint Ref No.	Nom DFT (μm)
-	Surface tolerant epoxy mastic primer	-	125	High build epoxy	-	150	Epoxy MIO or High build polyuret hane	-	10

6.4. Load rating

The load rating assessment results show the capacity of Spans 1 & 2 are adequate for MF loading, but have critical 300LA rating factors of 0.79 and 0.92 for the pier hogging and mid-span sagging moments, respectively.

The load rating results for the steel truss spans show that 300LA load rating produced lower results- in general- than for MF loading. The results showed that generally the top chord, stringers, cross girders and diagonals were below capacity. This is not surprising given the likely design loading at the time of the construction of the bridge, and the high theoretical design loading of the 300LA vehicle. It should be remembered that the load-rating results are for live loading on both tracks loaded simultaneously, whereas this occurs rarely in real life.

From the load rating results in can be quickly inferred that a strengthening regime to achieve a 300LA load rating is significantly more extensive than the required strengthening to achieve a MF load rating. A 300LA load rating will require the majority of members and connections to be strengthened on all spans, whilst an MF load rating will require strengthening of select members and connections. It is estimated that twice to three-times the steel tonnage would be required for strengthening to 300LA. Strengthening works for 300LA would involve all major members to ensure that stiffness is proportionately and equally increased to maintain correct load distribution throughout the structure. This is not only costly, but a time consuming process as rivets would need to be individually drilled out, new plates installed and site-drilled to match the existing fixing template, and site-fixed (using huck-bolts or similar). An alternative strengthening regime may involve bottom-hatting the cross-girders and/or stringers to increase their structural depth. The impact of this would need to be evaluated from a heritage and aesthetic perspective.

It should be noted also that the load-rating results have been provided for the instance that both tracks are loaded simultaneously. This load case occurs only rarely, and so strengthening to achieve an MF rated-load capacity will satisfactorily accommodate the actual applied live loading.

The recommendation is made that should Sydney Trains choose to proceed with a strengthening regime, and that an MF rating be targeted, as this is significantly more achievable in respect to constructability and cost. The proposed strengthening solution is to increase capacity through plating the existing structure, and connecting the additional plates through huck (blind) bolts.

Connection strengthening may be achieved through a rivet replacement scheme, again huck (blind tension) bolts may be utilised given their visually similar appearance.

6.5. Fatigue assessment

The fatigue assessment, using the EU detail category limit for normal stresses, indicates that the stringers on the longer spans have reached their fatigue life.

Further modelling which could include stringer cross-bracing and incorporating the stiffness of the rail and track system may improve the load distribution on these members, and lower the design action, and therefore slightly improve the fatigue results for these members. However, even this assessment may show that they are approaching the end of their fatigue life and should be considered for replacement. Given the aging condition of the bridge, consideration should be given now as to how fatigue can be practically managed in the on-going service life of the bridge.

Inspection of the connections did not discover any obvious signs of cracking (fatigue or otherwise) in the rivets or plate elements, though given the rivet shanks are covered by the plates they connect, any indication of fatigue cracking in the rivets would be very difficult to identify.

In order to confirm the material properties of the elements, it's recommended that plate member and rivet samples are extracted from non-critical locations and provided to a NATA approved laboratory for microscopy testing. This will assist in determine a suitable a strategy as to whether replacement of the elements on the larger spans is required.

6.6. Summary and Conclusion

SMEC has carried out several site and condition assessment inspections, which show localised spalling and corrosion of exposed reinforcement on concrete members, and minor corrosion to main steel members, and more localised moderate to severe corrosion at some crevice sites. A detailed 3D laser survey has been completed to confirm the structural general arrangement. Paint coating adhesion tests have been carried out and indicate breakdown of the coating on 50% of the surface area, which is estimated to be 52,000sqm. Diagnostic tests have been carried out on concrete samples, which indicate chloride and carbonation induced corrosion may have been initiated. The load-rating assessment has found the bridge has adequate capacity (with reduced live load factor) for MF loading, with the exception of some connections (T5-T6, T6-T7 and T1-

B2) on the 445'1" span. The fatigue assessment has found that the stringers have likely exceeded their fatigue life.

Based on these findings, SMEC recommends the following:

- repair of spalled concrete areas
- strengthening of steel members to achieve MF rated load capacity
- replacement (with like-for-like) of fatigue-life expired plates
- full cleaning and replacement of existing protective coating, carried out in conjunction with
- repair/replating of severe corrosion sites, and
- ultrasonic testing of base thickness after cleaning, prior to the application of the protective coating.

In order to manage the fatigue life of the bridge for future loading, and ensure the service life of the bridge is extended to its full potential, SMEC recommends:

- materials testing of any removed/replaced plate elements, and
- strain gauging of the structure to ascertain actual stresses experienced under actual loading.

Providing the locations are expertly identified, the latter exercise of strain-gauging has the two-fold benefit of being able to compare theoretical loading actions to actual stress effects, with the ability to calibrate this for historical (past) and future loading cases for accurate fatigue life assessments, and to provide a better estimate of the DLA, which may be lower than the value imposed by the standard.