Condition assessment of corrugated steel underpasses including three-dimensional terrestrial laser scanning

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ABSTRACT

Buried Corrugated Metal Structures (BCMS) offer an efficient form of construction for road crossings and consequently over the years a number have been installed on the New South Wales (NSW) road network. Roads and Maritime Services NSW currently have approximately 70 BCMS with spans in excess of 6 m.

Roads and Maritime Services, Hunter Asset Management Division has recently undertaken a detailed assessment of six BCMS in the region. The structures range in span from 3.2 m to 11.5 m and accommodate local road, fauna, and pedestrian movements. The structures are all multi-plate construction and are either arches or horizontal elliptical tunnels. Detailed inspections of the existing structures were undertaken including extensive ultrasonic thickness testing of the exposed plates. This information was then used to inform PLAXIS soil structure modelling to assess stresses in the structures. Remedial options were then developed. Three-dimensional terrestrial laser scanning was also undertaken for each of the structures. The point cloud data from the scanning was used to assess structure geometry and serves as a baseline for ongoing monitoring of deformation and corrosion.

This paper will focus on the investigation methodology and findings. In addition, practicalities and benefits of three-dimensional laser scanning will also be discussed.

1 INTRODUCTION

Due to low cost and relatively fast construction times, Buried Corrugated Metal Structures (BCMS) have historically provided an alternative solution for crossings for state road, rail, and local government authorities. However, corrosion is a critical issue for these structures, requiring adequate structural management plans, regular inspections, and maintenance programs. Inability to do so can lead to serious and significant failures.

Routine inspections by Roads and Maritime, Hunter Asset Management identified signs of corrosion in six Roads and Maritime BCMS assets. The structures are all multi-plate construction and comprise three elliptical tunnel structures and three arch structures. The structures range in span from 3.2 m to 11.5 m and accommodate local road, pedestrian, or fauna crossings. None of the structures are for drainage.

Due to the identified issues, Roads and Maritime requested tenders to undertake detailed site inspections, condition assessments, and conceptual design of remedial options for these structures. Aurecon was engaged by Roads and Maritime for this work. Site inspections included extensive ultrasonic thickness testing of exposed steel plates. The scope of work also extended to included three-dimensional terrestrial laser scanning to accurately assess geometry, record visible corrosion, and to serve as a baseline for future monitoring. The site inspections identified trends in corrosion patterns for each of the structure types.

Based on the site data, durability, structural, and geometry assessments were undertaken for each structure. This included a comparison of measured versus estimated corrosion loss and an assessment of residual life for T44 traffic loading. Concept remedial options were then developed, assessed, and costed for the planning of future maintenance programs, together with ongoing monitoring requirements.

This paper summarises the investigation methodology and findings, identifying trends and lessons learned.

2 OVERVIEW OF STRUCTURES

Four of the assessed structures are located beneath the Pacific Motorway (M1) between Sydney and Newcastle, NSW. The two remaining structures are located beneath the Pacific Highway and a Main Road near Newcastle NSW. The structures were constructed between 1977 and 1997, with most of the M1 structures being constructed in the late seventies and early eighties.

As outlined above the structures can be categorized as either arches or horizontal ellipses. A summary of the structures is provided below.

2.1 Horizontal ellipses

All horizontal elliptical tunnels form two-lane local road underpasses beneath the M1. All horizontal elliptical tunnels also have a reinforced end collar at each end and reinforced concrete thrust beams located either side of the crown, spanning between the end collars. The plate corrugation profile for all structures comprised ribs with a pitch of 150 mm and a depth of 50 mm.

2.1.1 Local road underpass at Kangy Angy

At this location the M1 is a six-lane divided carriageway. The depth cover of fill over the structure, as indicated on the drawings is approximately 3.25 m. The structure consists of 6.35 mm (as measured on site) galvanised corrugated steel plates bolted along all edges. The bottom of the ellipse is filled to a depth of approximately 1.76 m to accommodate the roadway, pedestrian paths, and services. See Figure 1 below for a photograph illustrating the underpass when viewed from the western side.

Figure 1 Typical horizontal elliptical tunnel – Local road underpass at Kangy Angy.

The underpass is approximately 5.65 m high at the obvert, 10.7 m wide at the horizontal centre line of the ellipse, and 9.6m wide between plates at the intersection of the footpath and the plates. The length of the underpass is approximately 90 m. The underpass has been in service for 40 years, assuming it was

constructed in 1977 as noted on the available drawings. Approaches to the underpass are sealed and kerb and gutter extend through the underpass. The footpaths on either side of the local road are not concreted.

2.1.2 Local road underpass at Bushells Ridge

This underpass supports the M1 which is a four-lane divided carriageway at this location. The depth of cover of fill over the structure is approximately 1.65 m. The structure consists of 6.0 mm galvanised corrugated steel sheets bolted along all edges. The bottom of the ellipse is filled to a depth of approximately 1.65 m to accommodate the roadway, pedestrian paths, and services. The underpass is approximately 5.8 m high at the obvert, 10.35 m wide at the horizontal centre line of the ellipse, and 8.75 m wide between sheets at the intersection of the footpath and the sheets. The length of the underpass is approximately 75.5 m. The underpass has been in service for 36 years, assuming it was constructed in 1981 as noted on the available drawings. Approaches to the underpass are unsealed and kerb and gutter extends through the underpass. The footpaths on either side of the local road are not concreted. Figure 2 below shows a typical section of the structure. This cross section is similar to the other two horizontal elliptical tunnels that were assessed.

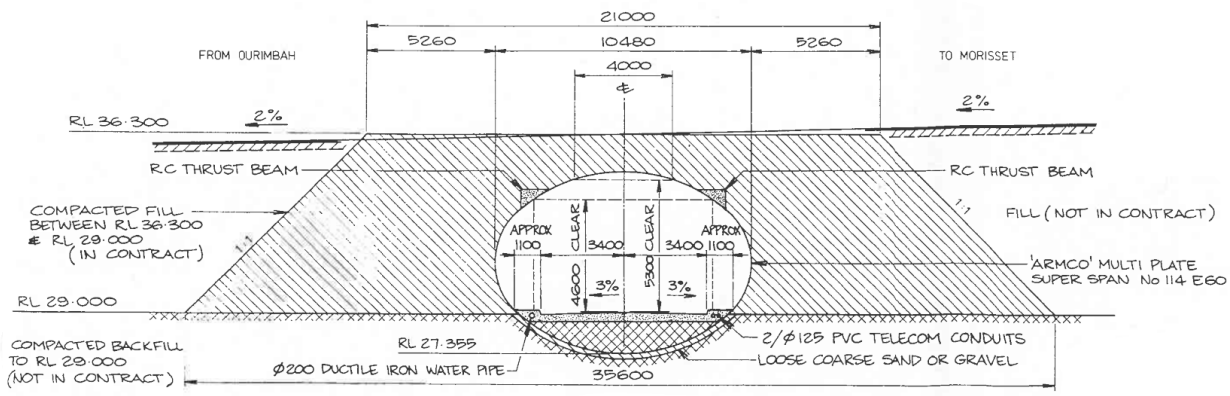

Figure 2 Typical horizontal elliptical tunnel cross section

2.1.3 Local road underpass at Somersby

At this location the M1 is a six-lane divided carriageway. The depth of cover of fill over the structure is approximately 4.4 m. The structure consists of 6.0 mm galvanized corrugated steel plates bolted along all edges. The bottom of the ellipse is filled to a depth of approximately 1.55 m to accommodate the roadway, pedestrian paths, and services. The underpass is approximately 5.65 m high at the obvert, 10.5 m wide at the horizontal centre line of the ellipse, and 9.1 m wide between plates at the intersection of the footpath and the plates. The length of the underpass is approximately 74 m. The underpass has been in service for 36 years, assuming it was constructed in 1981 as noted on the available drawings. Approaches to the underpass are sealed and kerb and gutter extends through the underpass. The footpaths on either side of the local road are concreted.

2.2 Arches

The assessed arches provide fauna or pedestrian underpasses beneath major roads and vary significantly in size and geometry. One common factor for the assessed arches is the steel plate to footing connection detail. All arches comprise a reinforced concrete footing on each side. The steel plates forming the arch are bolted to a steel channel which is cast into the reinforced concrete footing.

2.2.1 Fauna underpass at Calga

This underpass supports the M1 which is a six-lane divided carriageway at this location. The depth of cover of fill over the structure varies from approximately 3.5 m under the southbound lane to 7.3 m under the northbound lane. The structure consists of an elliptical arch with 7.0 mm galvanised corrugated steel plates bolted along all edges. The corrugation profile for this structure comprises ribs with a pitch of 150 mm and a depth of 50 mm. Both openings have a reinforced end collar. The underpass measures approximately 7.4 m high above the footing. It also has a maximum horizontal span of 11.58 m, and a horizontal span of 10.19 m at the top of the footing. Figure 3 adjacent shows a typical cross-section of the arch structure.

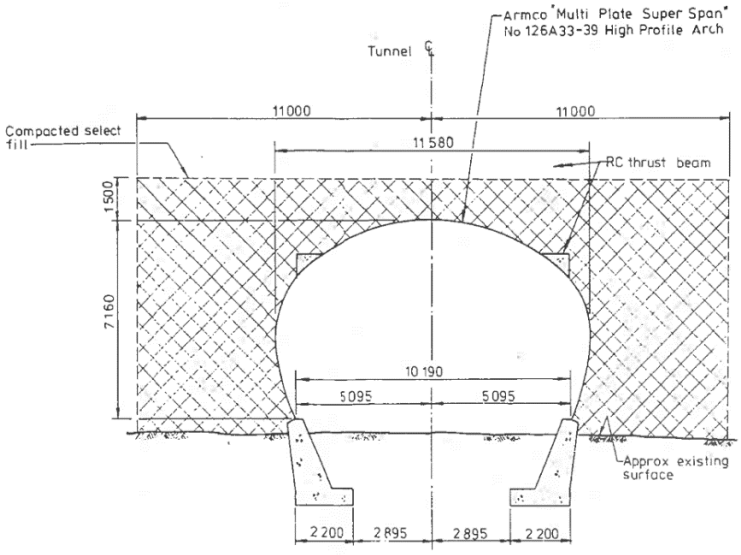

Figure 3 Typical elliptical arch cross section

The length of the underpass is approximately 86.5 m. The underpass has been in service for 35 years, assuming it was constructed in 1982 as noted on the available drawings.

2.2.2 Pedestrian underpass at Cameron Park

Figure 4 Circular arch pedestrian underpass at Cameron Park

This structure forms a pedestrian underpass beneath a three-lane Main Road at Cameron Park, near Newcastle. The cover of fill over the structure varies from 1.8 m to 2.35 m. The structure consists of a semicircular, 5.0 mm thick galvanised corrugated steel arch with sheets bolted along all edges. The sheet corrugation profile comprises ribs with a pitch of 200 mm and a depth of 55 mm. Both openings have a reinforced concrete surround. The underpass measures approximately 3.8 m high at the obvert with a clear distance between bottom sheets of approximately 8.0 m. The underpass has been in service for 24 years, assuming it was constructed in 1993 as noted on the available drawings. See Figure 4 adjacent for a photograph illustrating the underpass.

2.2.3 Fauna underpass at Taree

This structure forms a fauna underpass providing east-west access beneath Pacific Highway (HW10) near Taree. The Pacific Highway is a four-lane divided carriageway at this location. Levels taken on site

indicated that the maximum depth of fill over the structure is approximately 2.4 m above the crown. The structure comprises an elliptical arch with 3.0 mm galvanised corrugated steel sheets bolted along all edges. The sheet corrugation profile comprises ribs with a pitch of 200 mm and a depth of 55 mm.

The underpass is approximately 3.0 m high at the obvert with a clear distance between bottom sheets of approximately 2.8 m and a maximum span at mid-height of 3.2 m. The length of the underpass is approximately 60.5 m with a skylight provided at the midpoint. The underpass has been in service for 20 years, assuming it was constructed in 1997 as noted on the available drawings inspection report.

3 METHODOLOGY

A methodical inspection and testing regime was adopted for all structures with ultrasonic thickness testing being undertaken for over 50 per cent of the visible plates. This, coupled with other site-based testing, provided a data set for each structure that was used to assess the extent and trends of corrosion. Site-based testing also included three-dimensional terrestrial laser scanning.

Assessment of the structures was undertaken in terms of durability, geometry, and structural capacity, with the first two assessments helping to inform the structural assessment. This was then used to guide the development of remedial options.

In terms of durability, established standards were used for calculation of corrosion losses, due to soil side, atmospheric, and microclimate corrosion for each BCMS for the time since construction. The estimated loss was then compared with measured losses. This helped calibrate the approach as appropriate through the adjustment of corrosion categories.

The three-dimensional point cloud data was used to assess the structure geometry, checking for major deformations over the length of the structures. This was done by assessing cross sections at regular intervals (including beneath the carriageway and at the portals).

Following the above, the capacity for each structure was assessed using the geotechnical finite element analysis software PLAXIS. The analysis assessed the stresses in the structure for dead and live loading for the “as new” and “as is” (using the average measured steel plate thicknesses) conditions. The thickness of the steel plate at which the stress limit was exceeded was also calculated. Using the calculated corrosion rates a residual life was assessed for the steel plate section loss to reach the critical thickness.

The structural models were then used to develop and assess remedial options which were costed by a quantity surveyor. Options were assessed over several criteria to determine the preferred option for appropriate planning of future works. Monitoring plans and any immediate actions were also identified.

3.1 Literature review

BCMS are complex structures due to the reliance of the flexible steel members on soil structure interaction for strength. Furthermore, due to the working environment, BCMS are subject to various and significant corrosion mechanisms which affect the critical structural element. Consequently, for this project, specialist durability and soil structure interaction input were used for the assessment of the structures. Several relevant standards and guidelines were also referred to in the testing and assessment. These included:

- Australian / New Zealand Standard (AS/NZS) 2041 Buried corrugated metal structures (1), (2), (3). This is the current Australian Standard for BCMS and includes Part 1: Design methods, Part 2: Installation and Part 6: Bolted plate structures. This was used for structural and durability assessments (particularly for soil side corrosion).
- Australian Standard AS 2452.3 Non-destructive testing – Determination of thickness Part 3: Use of ultrasonic testing (4). This standard was used for guidance for the thickness testing of the plate.

- AS 3703.2 Long-span corrugated steel structures Part 2: Design and installation (5). This standard is superseded but was referred to for information.
- AS/NZS 2312.2 Guide to the protection of structural steel against atmospheric corrosion by the use of protective coatings Part 2: Hot dip galvanizing (6). This standard provided guidance with regard to the life of protective coatings for atmospheric exposure.
- AS 4312 Atmospheric corrosivity zones in Australia (7). This was used for assessment of atmospheric corrosion.
- ISO 9223 Corrosion of metals and alloys – Corrosivity of atmospheres – Classification, determination and estimation (8). This was also used for guidance for atmospheric corrosion.
- ISO 9224 Corrosion of metals and alloys – Corrosivity of atmospheres – Guiding values for corrosivity categories (9). This was also used for guidance for atmospheric corrosion.
- Austroads Guidelines for Design, Construction, Monitoring and Rehabilitation of Buried Corrugated Metal Structures (10). This is a comprehensive reference outlining the design, monitoring, and rehabilitation of BCMS.

Roads and Maritime provided work as executed drawings for all structures. For the larger structures, these also included plate layout and numbering drawings which were used for defect mapping. Level 2 inspection records for all structures were also provided. Dial Before You Dig surveys were undertaken for each site to guide remedial options (and costing). All local road underpasses were found to contain significant utilities in the footpath zones.

4 SITE INSPECTIONS

Site inspections and laser scanning works were undertaken in early 2017. Generally, one to two days were required per structure depending on the size and access. Similarly, laser scanning works also required one to two days depending on the structure. Prior to undertaking the detailed inspections, preliminary site visits were also made to each structure to aid in planning the works.

4.1 Inspection methodology

The testing scope for each structure was agreed with Roads and Maritime and included:

- Visual inspection, photographs, and defect mapping for each structure. Defects were marked on a plate layout of the structure.
- Ultrasonic thickness testing of the visible surface for cross sections at typically 5.0 m intervals along the length for the structure. For each cross-section testing was done every 1.0 m to 2.0 m. This resulted in testing of approximately 50 per cent of the exposed steel plates. Measurements were taken at mid-plate locations, away from the lapping zones. In addition to testing at regular locations as outlined above, further thickness testing was undertaken at locations which appeared to be the most affected by corrosion.
- Small cores through the steel sheets were proposed for verification of the ultrasonic thickness tests; however, this was not undertaken due to concern that this would damage any remaining protective coating.
- Galvanizing thickness tests were undertaken in locations where painting of the structure had not been undertaken.
- Terrestrial laser scanning to assess deformation and provide a benchmark survey for future monitoring was undertaken for all structures.

The materials testing of the underpass was conducted using an ultrasonic thickness tester. The thickness tester and probe were calibrated prior to testing of the underpass using a stepped wedge calibration block. All calibration was undertaken outside in the same temperature and location as the underpass to minimise errors due to temperature variations. The calibration ensured the thickness tester was calibrated to work to an accuracy of +/- 0.2 mm. Where required, the surface was prepared to reduce the surface

roughness ensuring reliable measurements were obtained. In most instances, the surface was prepared by hand with a scraper. The scraper enabled adequate surface roughness without compromising galvanising treatments. The ultrasonic thickness test unit results were further checked by measuring the plate thickness with Vernier calipers at the exposed ends in areas without corrosion.

Weather conditions including recent rain and temperature were noted at the time of the inspections.

Access for horizontal elliptical local road underpasses was relatively straightforward requiring traffic control and an Elevated Work Platform (EWP) for testing. Access for the smaller arch-shaped fauna underpasses was slightly more difficult requiring foot access through crown land. Testing was then undertaken using a small platform ladder.

The most challenging structure in terms of access was also the largest. This is the arched fauna underpass at Calga. This structure is located beneath the M1 and is surrounded by National Park on both sides of the motorway. The underpass can be accessed from a maintenance bay on the M1, but only with traffic control at set times. Access from the motorway also requires walking through steep terrain. The underpass can also be accessed from a fire trail and bushwalking track. A vehicle can be parked approximately 700-800 m from the underpass, the remaining distance is approximately 10-15 minutes by foot via the track.

The initial plan for access to this structure for testing comprised delivery of temporary scaffold to the M1 maintenance bay, which would then be carried to site and erected for use for testing. This would prove time-consuming and presented manual handling issues. After consideration of options, trials were done using the ultrasonic testing coupler mounted to a plywood template that had been cut to the corrugated profile. This was then pole mounted and used for testing. This resulted in identical results to testing by hand at close proximity. Consequently, the larger structure tested was using a small platform ladder and this methodology.

During the early stages of the project, consideration was given to using a magnetic crawler mounted thickness gauge. This type of technology is used for testing steel tanks and other industrial structures. This technology can produce a plot of the steel thickness over the length traversed. After an initial investigation, it was decided that the technology available at the time did not have the flexibility to traverse the BCMS corrugations, lap joints, and bolts without significant customization. Consequently, a manual approach was adopted. Laser scanning was undertaken using a terrestrial scanning unit (e.g. tripod mounted) and survey marks were established for future laser scanning works. Further discussion regarding laser scanning is provided in Section 5.3 below.

4.2 Inspection results and observations

The following is a summary of general results and observations for each structure type.

4.2.1 Horizontal ellipse test results and observations

Table 1 below provides a summary of the as-built and measured plate thicknesses for the horizontal ellipse structures. Comments are also provided for each structure.

Table 1: Summary of horizontal ellipse test results

Structure location	As built plate thickness (mm)	Measured plate thickness (mm) (average)	Measured plate thickness (mm) (worst case)	Comments
Kangy Angy	6.35	6.5 (top assembly) 6.0 (side)	4.0 (at the interface with ground inside underpass)	Plates on the side of the structure up to the junction with the top plate assembly varied between 6.0 mm and 7.5 mm thick. Plates on the top plate assembly (above the thrust beam) varied in thickness from 5.5 mm to 6.5 mm thick. Surface corrosion was present for the bottom 100-200 mm where the bolted plates met the footpath. The thickness of plate varied in the locations of corrosion between 4.6 mm and 7.0 mm. Surface corrosion was also present at the lap joints.
Bushells Ridge	6.0	5.9	4.0 (at the interface with ground inside underpass) Complete loss at some very localised areas (at portals)	Plates on the side and top of the structure varied between 5.0 mm and 6.1 mm. Surface corrosion was present for the bottom 100-200 mm where the bolted plate meets the footpath. The thickness of plate varied in the locations of corrosion between 4.0 mm and 6.0 mm. The ribs in four locations were either corroded through entirely or between 1.0-2.0 mm thick. This occurred near the footpath interface at the portals.
Somersby	6.0	6.0	3.5 (near bolted seams)	Plates on the side and top of the structure varied between 5.8 mm and 6.2 mm. Localised surface corrosion was present at the bolted seams. The thickness of plate varied in the locations of corrosion between 3.5 mm and 6.0 mm. The plate thicknesses at locations one to two ribs away from the visible surface corrosion measured thicknesses close to the recorded thickness for each plate, indicating that the corrosion was localised to the visible sections.

A defect plan was prepared for each structure using a plate layout with measured thickness and visible corrosion and other defects marked. A typical defect plan is shown below in Figure 5.

The inspections identified some corrosion trends across the three-horizontal ellipse BCMS. These included:

- Surface corrosion was evident at the seams of the plates, this corrosion was more prevalent at the top plates and plates above the horizontal centre line. Water was observed seeping through the plate lap joints.
- Surface corrosion was present for the bottom 100-200 mm of the exposed structure where the bolted plates meet the footpath zone. This was present for structures for which the footpath is not concreted, and earth fill is placed directly against the steel. Negligible corrosion was observed at this interface for the BCMS at Somersby which has concrete footpaths. Where present at this interface, the surface corrosion was more prevalent near the portals.
  Several inspections were undertaken during wet weather and it was identified that poor road drainage was leading to ponding in the gutter in and near the BCMS portals. This was particularly the case for the BCMS at Kangy Angy and Bushells Ridge. The passage of vehicles (even with traffic control speed restrictions) was causing ponded water to be splashed repeatedly onto the structure. The presence of mud and dirt on the BCMS walls supported this
- No visible evidence of overloading of the structures was evident (deformation, opening of plates, cracking).
- Some structures had significant vegetation (trees etc.) growing on the road batters above the structure.

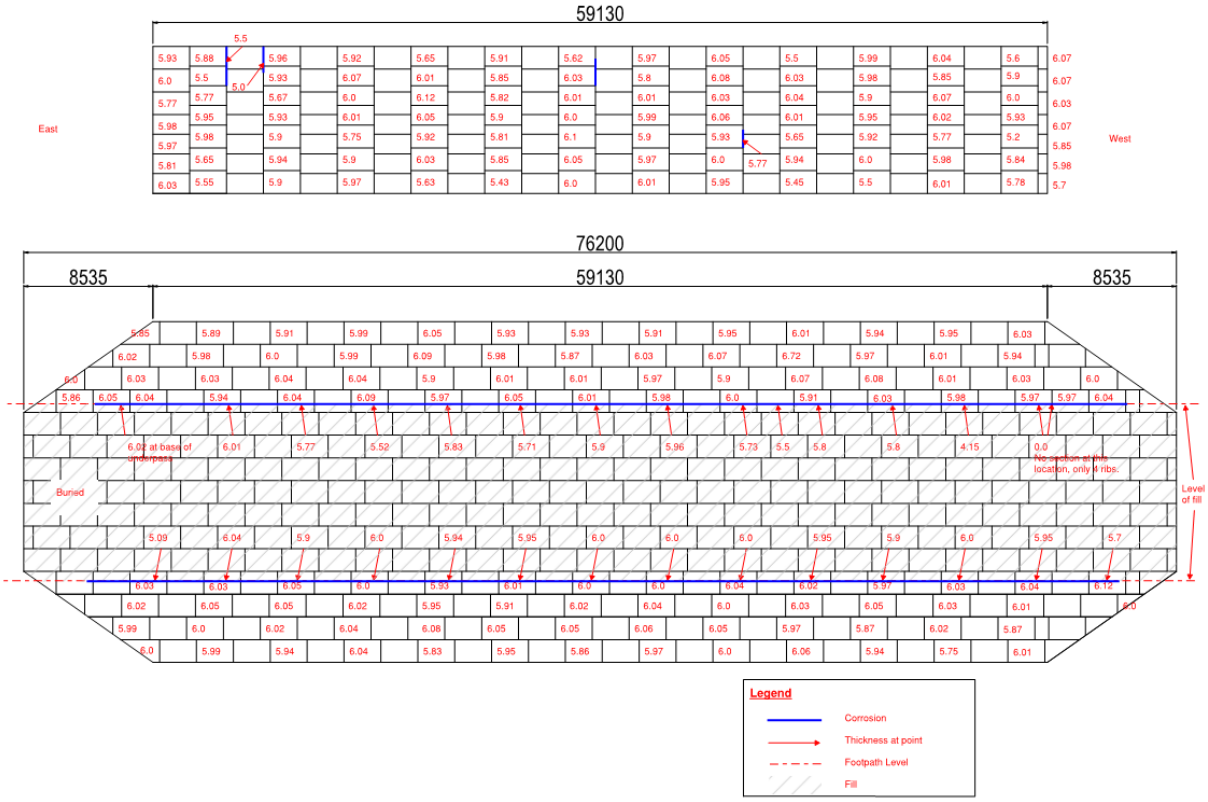

Figure 5 Defect plan for the structure at Bushells Ridge

Examples of observed corrosion are shown below in Figures 6 and 7.

Figure 6 Typical corrosion at bolted lap.

Figure 7 Path interface showing water ponding in gutter and splashed dirt and mud on steel plate.

4.2.2 Arch test results and observations

Table 2 below provides a summary of the as-built and measured plate thicknesses for the arch structures. Comments are also provided for each structure.

A defect plan was prepared for each structure using a plate layout with measured thickness and visible corrosion and other defects marked.

The inspections identified common corrosion trends across the three arch BCMS. These included:

- The most obvious trend for all arches was an increased amount of corrosion at the connection between the steel arch and the reinforced concrete footing. All arches exhibited a zone of approximately 100-150 mm increased corrosion at this interface. At this interface, corrosion was generally significantly more advanced than other locations, generally leading to localised areas with section losses of approximately 40 per cent or greater than general plate locations.
- The footing detail for all arches comprised a steel channel cast into the concrete footing. This channel was consistently observed to be either damp or holding water. Water also appeared to be seeping through the detail from the soil side. Microclimatic conditions caused by this detail are anticipated to have caused the increased corrosion.
  Corrosion at this interface was also observed to be worse for arches at Calga and Taree on the uphill side of the section. For both structures, the motorway is on a grade. Greater corrosion was observed on the side of the structure on the uphill side of the cross cross-section. This could potentially be caused by greater groundwater flows in the base of the embankment travelling downhill and being impeded by the underpass, leading to greater moisture levels on the uphill side of the cross cross-section.
- Surface corrosion was evident at the seams of the plates for the older structure (Calga) but not observed at the more recent structures (Cameron Park and Taree). Water was observed seeping through the lap joints.
- Plastic sheeting was found behind the corrugated steel plates for the Cameron Park structure. The plastic sheeting was observed through pre-existing holes in the bottom plates at the connection with the reinforced concrete footing. No sheeting was observed for the other two arch structures.
- No visible evidence of overloading of the structures was evident (deformation, opening of plates, cracking).

- Some structures had significant vegetation (trees etc.) growing on the road batters above the structure.

Table 2: Summary of arch test results

Structure location	As built plate thickness (mm)	Measured plate thickness (mm) (average)	Measured plate thickness (mm) (worst case)	Comments
Calga	7.0	6.9	4.1 at connection with footing	Plates on the side and top of the structure varied between 6.4 mm and 7.5 mm. Localised surface corrosion was present at the bolted seams. The thickness of plate varied in the locations of corrosion between 6.9 mm and 7.0 mm. Corrosion was evident for the bottom 100-150 mm where the bolted plates meet the footing. The thickness of the plate varied in the locations of corrosion between 4.1-7.0 mm.
Cameron Park	5.0	5.0	3.5 at connection with footing	Visible corrosion was present for the bottom 100-150 mm where the bolted plates meet the footing. The thickness of the plate varied in the locations of corrosion between 3.5-5.0 mm. Aside from the footing connection, minimal corrosion was identified elsewhere.
Taree	3.2	3.0 (excluding areas of complete section loss)	1 or less for approximately 18 consecutive ribs at connection with footing.	Extensive corrosion was present on the southern side for the bottom 100-150 mm where the bolted plates meet the footing. The thickness of the plate varied in the locations of corrosion between 0.0–3.0 mm. Approximately 18 ribs had a section thickness of 1.0 mm or less. Aside from the footing connection, minimal corrosion was identified elsewhere.

Examples of observed corrosion are shown below in Figures 8 and 9.

Figure 8 Corrosion at footing connection - Calga

Figure 9 Arch to footing channel connection detail at end of wing wall - Taree

5 ASSESSMENT

Following the completion of site inspection work, detailed durability, capacity, and geometry assessments were undertaken for each structure. An overview of the methodology adopted for each is provided below.

5.1 Durability assessment

BCMS are generally subjected to corrosion and abrasion. Corrosion occurs on the surface in contact with soil, and corrosion and abrasion occur on the internal surface of the buried structures. In the case where the buried structure is used as an underpass and is not subjected to flowing water for prolonged periods, abrasion is not a relevant factor affecting the durability of the buried metal structure. Consequently, soil corrosion on the external face and atmospheric corrosion on the internal face will determine the durability of the inspected structures.

5.1.1 Soil side corrosion

It is known that corrosion of metal buried in soil is affected by a wide range of parameters including the following: soil type, soil resistivity, redox potential, chloride ion concentration, sulfate ion concentration, soil pH, organic materials, sulfate reducing bacteria, sulphide presence, drainage, carbonate present, and soil condition (permeability, compaction etc.). Since all inspected structures are in select fill as part of the motorway embankments, corrosion is simplified as amounts of chloride and sulfate are restricted. This is based on the assumption that fill around and over the structures has been placed in accordance with the relevant Roads and Maritime earthworks specifications at the time of construction.

Soil side corrosion has been calculated allowing for the loss of the protective zinc coating first, followed by corrosion of the steel plate. Zinc coating thickness for each of the structures were conservatively adopted based on the relevant standard at the time of construction and the thickness of the plate, coupled with comparisons to site measurements on the atmospheric side. Based on this an adopted zinc coating of 70 microns was used for all structures except for Calga for which 85 microns was adopted due to the greater plate thickness.

AS 2041 suggest that the annual soil corrosion rate for zinc can be taken as 15 microns for the first two years and four microns for subsequent years. After consumption of the zinc coating, AS 2041 suggests that the corrosion of steel will progress at a rate of 30 microns per year. It is noted that this is conservative

when compared with AS 2159 (11) which suggests a corrosion rate of 10 microns per year as appropriate for non-aggressive backfill.

Soil side corrosion losses were then calculated based on the above approach for all structures for the number of years in service.

5.1.2 Atmospheric corrosion

Atmospheric corrosion of metals is affected by the atmospheric corrosivity which is influenced by the deposition rates of chloride and sulfur dioxide, temperature, and relative humidity as outlined in ISO 9223. It is noted that it is most likely that the atmospheric corrosivity in a tunnel underpass would generally tend to be more severe than that outside the underpass. This is based on the consideration that the sulfur dioxide concentration and the relative humidity in the underpass would generally be higher than that outside.

For the underpasses inspected, the external condition is more or less rural with low pollution and insignificant airborne marine aerosol. On this basis atmospheric corrosivity of the external environment could be assigned to C2 category as per ISO 9223 and AS 4312. However, for the purpose of internal corrosion evaluation, an atmospheric corrosivity category C3 (more severe than C2) was conservatively adopted.

The minimum service life of the zinc coating was estimated in accordance with AS 2312. After loss of the protective coating, corrosion loss of steel was estimated based on the corrosion category C3 and a first-year corrosion rate of 50 microns per year followed by corrosion based on an exponential relationship with time in accordance with ISO 9224.

5.1.3 Microclimatic factors

In the above, the influence of microclimatic factors has not been considered, particularly for internal surfaces. For an underpass, the most critical microclimatic factor is related to locations where the metal remains wet for an extended period or is kept wet “permanently”. These locations are commonly referred to ponding areas or areas exposed to water leakage/water dripping/water condensation. The corrosion rate of metals in these areas will be much higher than those estimated previously.

Prolonged wetting can bring the atmospheric corrosivity locally to category C4 and/or C5. Areas of likely prolonged wetting include:

- Junctions of galvanized steel plate with the unsealed footpath.
- The connection between the galvanized steel plate and the concrete footing.
- Leaking bolt holes and lap joints.

It is very difficult to estimate the corrosion in these areas since there is no specific information to quantify the period of wetting/leaking. The approach was based on C4 category for high-frequency wetting or C5 category for very high frequency of wetting. The inspections identified damp conditions for all culverts and rust staining on the ground near corroded locations indicating dripping water. The presence of damp conditions supported the adoption of a C4 (or higher) classification.

Based on the corrosivity category, minimum zinc coating service lives were estimated in accordance with AS 2312. After this corrosion was calculated based on ISO 9224.

Total corrosion losses were calculated for the structure ages based on a summation of the soil side, atmospheric, and microclimate corrosion losses. Based on analytical methods, corrosion losses for zones away from microclimate areas (e.g. at plate midpoints), were predicted to be greater than those measured on site. This indicates that the initial assumptions for soil side and atmospheric corrosion loss rates may have been slightly conservative. Conversely, corrosion losses for zones at microclimate areas were

predicted to be less than those measured on site. This indicated that it is very possible that a C5 category is more relevant micro climate zones in the underpass based on the observed steel loss at critical locations.

The mechanism for the observed corrosion is attributed to continued wetting and exposure to oxygen at the lap joints, footing connections, and footpath interface locations (due to the passage of vehicles). The corrosion at the lap joints is typically around the thrust beam location. It is possible that water is trapped on the thrust beam and this contributes to water ingress at these locations.

Corrosion rates are also known to increase in regions with higher stresses (12). Stresses are higher at the interface with the internal fill zone for horizontal ellipse structures, coinciding with a microclimate zone. This could also be contributing the greater observed losses in this location.

5.2 Capacity assessment

All underpass capacities were assessed using the geotechnical finite element analysis software, PLAXIS. A non-linear, two-dimensional analysis was undertaken for each BCMS considering the staged construction.

The corrugated metal structures are all installed within the road fill embankments. The available design drawings show compacted fill around all of the structures. For the purposes of analysis, it was assumed that the structures had been backfilled in accordance with the relevant Roads and Maritime specifications at the time of construction. These specifications were assumed to be comparable with the current Roads and Maritime earthworks specification R44 and consequently this was used for backfill properties.

The work as executed drawings show limited borehole data for the material beneath the structures. Where no borehole data is shown, it was assumed that the structure is founded in stiff clay. This assumption is expected to have an insignificant impact on the results of the analysis. Only the horizontal ellipse at Somersby and the arch at Calga were shown to be founded in weathered rock.

The two-dimensional PLAXIS model employed an elastic plastic Mohr-Coulomb soil model. Thrust beams were modelled for structures as per the work as executed drawings. The two-dimensional assessment considered the typical section with the most critical cover.

The underpass installation process was typically modelled in PLAXIS using an eight-stage analysis with the following steps:

- Setting up initial stress based on ground level prior to underpass construction;
- Backfill from existing ground, and construction of underpass footing;
- Construction of the corrugated underpass structure;
- Backfill to the middle level;
- Application of compaction pressure (20 kPa);
- Backfill and construction of pavement inside the tunnel;
- Backfill to the top of the embankment;
- Application of T44 highway loading in accordance with AS 5100.7 2004 (13)

Section properties for the corrugated sheeting for various thicknesses were calculated using the program Strand 7 and these properties were applied to the PLAXIS model.

The yield stress for all corrugated steel sheets was assumed to be 230 MPa. The yield stress was assumed based on the minimum value provided AS 2041-1977 (14) and AS 3703.2-1989 (5) which were the relevant standards at the time of construction.

The “as new” and “as is” capacities of the structures were assessed by comparing steel stresses for the design and measured plate thicknesses with the corrugated plate design capacity. The live loads used in

the assessment were applied in accordance with AS 5100.7-2004 (13) using T44 as the load rating vehicle. The live load cases were assessed based on the number of lanes of vehicles with load factors applied in accordance with AS 5100.7. Combined Ulimate Limit State (ULS) and Serviceability Limit State (SLS) axial and bending stresses were assessed for each structure and compared with the design capacity. Generally, for horizontal ellipse structures, the peak bending stresses occurred near the thrust beam location and the interface at the internal pavement level. Similarly, for the elliptical arches at Calga and Taree, the peak bending stresses occurred near the upper section of the arch (in similar locations to thrust beams). For the semicircular arch at Cameron Park, the peak bending stresses occurred near the footings. Plate capacities were assessed based on the thickness at the locations under consideration.

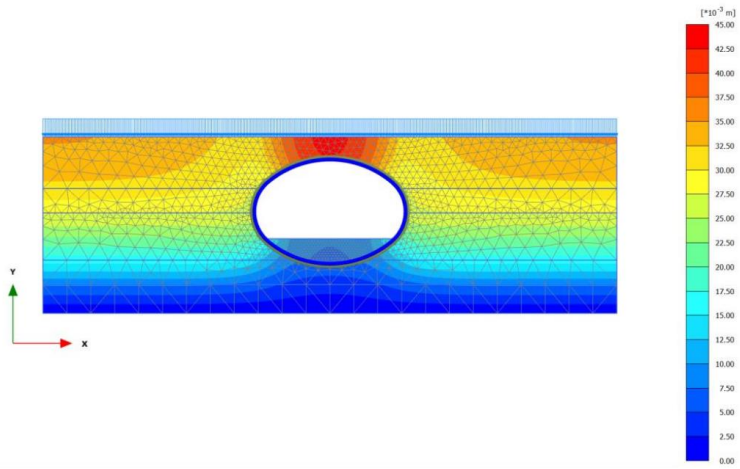

Figure 10 SLS deflection diagram from the PLAXIS model for 'as new" condition – Bushells Ridge

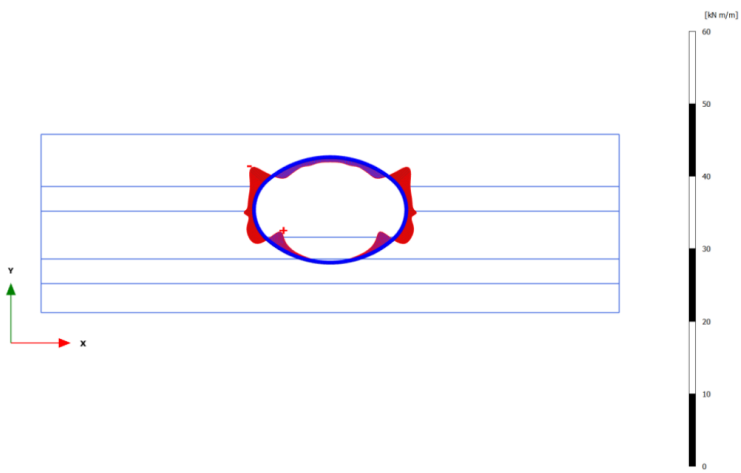

Figure 11 ULS bending diagram from the PLAXIS model for 'as new" condition – Bushells Ridge

In addition to stresses due to axial thrust and bending, capacity checks for seam strength and global buckling were also undertaken in accordance with AS 2041. Furthermore, bearing capacities of strip footings for arches were also assessed.

The results of the analysis were used for the assessment of the residual life and to guide the development of remedial actions. Output from the analysis typically comprised deflection, axial force, and bending moment diagrams. Examples of typical deflection and bending moment diagrams for a horizontal ellipse are provided above Figures 10 and 11.

5.3 Geometry assessment

A full scan of each corrugated steel underpass was undertaken using a terrestrial laser scanner to produce three-dimensional point clouds. The initial scan provides a baseline survey for future comparison but also for comparison against the original design geometry. Extraction of cross sections from the point cloud data at regular intervals can facilitate monitoring of changes in cross-section and longitudinal profile. Advantages associated with this technique include:

- A safe and comprehensive method of remote data capture that can be undertaken in ways that will not impede traffic or increase safety risks to staff.

- Terrestrial laser scanning allows for non-contact, rapid, highly accurate data sets of millions of three-dimensional points with millimetre level accuracy.
- Three-dimensional points can be draped with photographic images captured by the scanning process to deliver a spatially accurate real-world three-dimensional model of the structure.
- A significant benefit of using laser scanning for deformation analysis is that the method provides the ability to detect and quantify minor changes in the surface profile and features (e.g. visible corrosion, deformations) due to the high density of the point cloud and the imagery data.

The intent of the laser scanning was primarily to provide a baseline for future monitoring. A comparison of measured geometry against the design geometry was also undertaken. Due to the flexible nature of the structures, the relevance of this is limited as the as-built geometry may have been significantly different to the design geometry. However, it is a reasonable assumption that the difference in deformation between the lightly loaded section (at the portals) and at the heavily loaded section (the midpoint of the alignment) is primarily due to the application of surcharge and fill. Consequently, an assessment of the magnitude of deflection at the crown of the arch is possible for comparison with the PLAXIS model results and design geometry.

The data captured by the laser scans was used to produce a surface of the existing structure. Cross sections were then taken from the model at regular centres along the underpass. In addition, longitudinal sections were also undertaken at regular increments around the perimeter of the structure. The results from these sections were then compared against the original drawings for the underpass and compared against the serviceability limit state deflections calculated using the PLAXIS model. Figure 12 below illustrates the surface and section for the BCMS at Bushells Ridge.

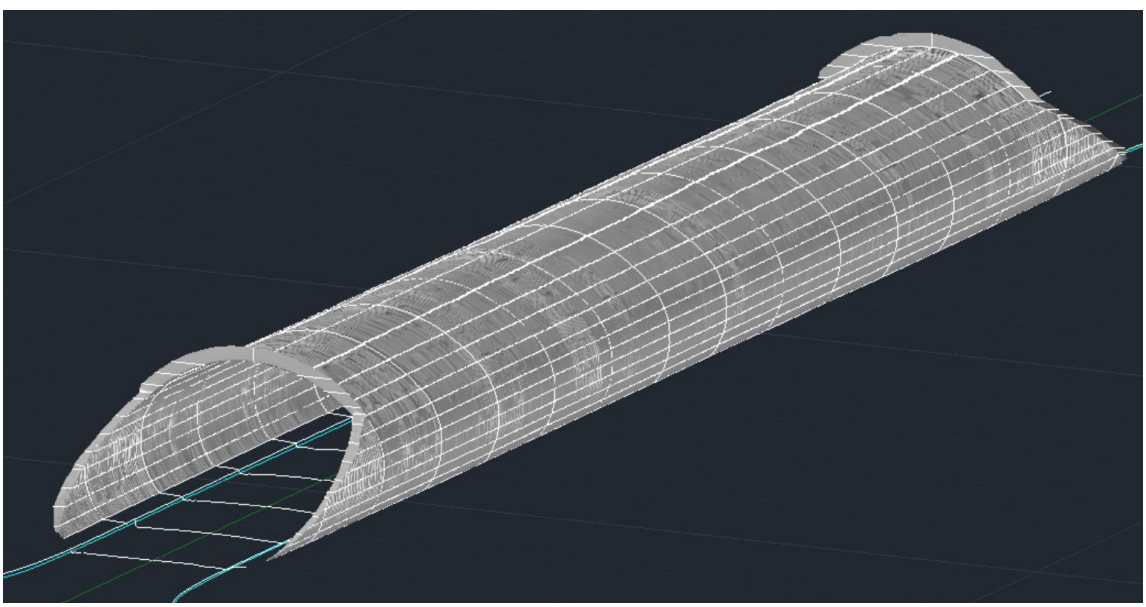

Figure 12 Rendered view with sections for BCMS at Bushells Ridge

Most structures exhibited a reduction in cross-section height from the height shown on the design drawings. However, the structures at Somersby and Calga had a greater height than the design geometry. This could be due to the pavement level being slightly higher (for Somersby), footings being located closer together than the design (for Calga) or variation in compaction during construction. Without further monitoring it is not possible to confirm if the observed deformations between the portals and most heavily loaded sections of the structures are due to structural deterioration or differences between the design and as-built geometry.

The point cloud data was draped with the photographic imagery captured during the scan to create spatial models of each structure. The spatial models clearly show the extent of corrosion. A lesson learned from this project would be to use temporary lighting to improve the colour return on the photo imagery. An opportunity for future development would be to develop routines that define surface corrosion boundaries to measure the extent of surface corrosion and assess changes between scans done at different times (e.g. growth of surface corrosion extents over a period of years). Figure 13 below shows a screenshot from the rendered spatial model for the BCMS at Somersby.

Figure 13 Screenshot from rendered spatial model at Somersby

6 REHABILITATION

An assessment of the residual life each structure was made using the measured plate thicknesses, durability, and capacity assessments. For each structure, the critical plate thickness at which the plate stress limits would be exceeded was calculated. Then corrosion parameters were adjusted to most closely match the observed corrosion. This varied depending on the structure but generally, the soil side and atmospheric corrosion loss rates were reduced, and the microclimate corrosion rate increased. For example, the soil side loss rate was reduced from 30 microns per year (as per AS 2041) to 10 microns per year (as per AS 2159 for steel in non-aggressive fill). Similarly, the atmospheric corrosivity category was reduced from C3 to C2 and the microclimate atmospheric corrosivity category increased from C4 to C5. Following the adjustments, the time required for the existing steel to reach the critical thickness was calculated to determine the residual life.

Short term remedial works that would be beneficial for extending the life expectancy were identified for all structures. These included:

- For horizontal ellipses, sealing the road pavement on the approaches (if unsealed) and improving road drainage.
- For horizontal ellipses, completion of small test pits in the unsealed footpath zones to assess if corrosion continued below ground level.
- For arches, cleaning the foundation connection channel at the interface of plates and the footing to remove dirt and debris. Furthermore, improvement of drainage near the plate to footing connection to direct water away from the channel.

- Cleaning and painting of corroded areas for several structures.
- Removal of trees and vegetation on the road embankment batters for several structures.

Preparation of ongoing monitoring plans were also recommended.

In addition to the above, more significant strengthening measures were also assessed for the purposes of planning works as the structures reach the end of their residual life. These are summarised briefly below.

6.1 Horizontal elliptical tunnels

Even with the short term remedial actions outlined above, corrosion of the structures will eventually reach the point where section capacities are close to exceedance and more significant remedial actions are required. The estimated residual life varies depending on the structure geometry, loading and observed corrosion rates. Ongoing monitoring will be employed to determine when more significant strengthening measures will need to be constructed.

Long term remedial options considered for horizontal elliptical tunnels generally comprised the three following concept options:

1. Propping the structure in the short to medium term using steel sets. This draws on tunneling construction methods whereby rolled steel sections (steel sets) are used for temporary excavation roof propping as the excavation advances. In a tunneling application, the steel sets would normally be founded on the rock in the tunnel floor. Due to insufficient founding material in this application, the steel sets for this option would need to be founded on a reinforced concrete footing at each side of the underpass, propped by a reinforced concrete pavement/footing.
2. Construction of a new in-situ concrete arch using a shotcrete lining within the existing corrugated steel underpass. The shotcrete lining would need to be founded on a cast in-situ concrete footing plinth at each footpath which would extend approximately 1500 mm above the existing footpath level. Loads from the shotcrete arch would then be transferred via the in-situ concrete footing plinth into an in-situ concrete pavement base slab. The base slab would serve as a footing and prop for the shotcrete arch.
3. This option replaces the existing corrugated steel arch structure with new precast, reinforced concrete arch crown units, supported on cast in-situ concrete footings. The precast crown units would be founded on a cast in-situ concrete footing plinth at each footpath which would extend approximately 1500 mm above the existing footpath level. Loads from the precast arch would be taken by the footing. It was also proposed to prop the two footings with a cast in-situ concrete base slab which would form the pavement for the underpass. Construction of this option was proposed to be undertaken using temporary steel slide beams to slide crown units into position, before grouting the perimeter.

Further options were discussed with specialist suppliers and remedial repair companies. New technology has allowed the rehabilitation of smaller BCMS structures using fibre reinforced polymer, slip lining, and pipe jacking. However, these applications have been used for smaller drainage culverts where access is available to the full internal perimeter of the structure. For larger, partially filled structures, this repair using these methods was not feasible.

In addition to the above, a review of the use of cathodic protection was undertaken. Internationally, there are limited examples of cathodic protection being applied to bolted BCMS and none were identified in Australia. Case studies indicated that cathodic protection, would likely not be an economical solution where the area requiring protection is large, or where electrical connectivity is questionable (e.g. bolted plates) (15). Consequently, cathodic protection was not considered a feasible option.

All options were costed and then compared in terms of cost, ongoing maintenance, constructability, and functionality. Generally, steel sets were the most cost-effective option, followed closely by shotcrete arches and then precast arches. However, steel sets were considered less favourable in terms of ongoing

maintenance. Whilst providing a durable and relatively cost-effective solution, shotcreting would also probably result in the most traffic disruption, requiring scaffold due to the size of the structures. All options would require construction of the pavement slab in two stages, possibly enabling single lane road access to be maintained during construction.

A multi-criteria assessment of the concept options generally resulted in shotcrete lining being selected as the preferred long term remedial option. Figure 14 below shows a concept sketch for the shotcrete lining option.

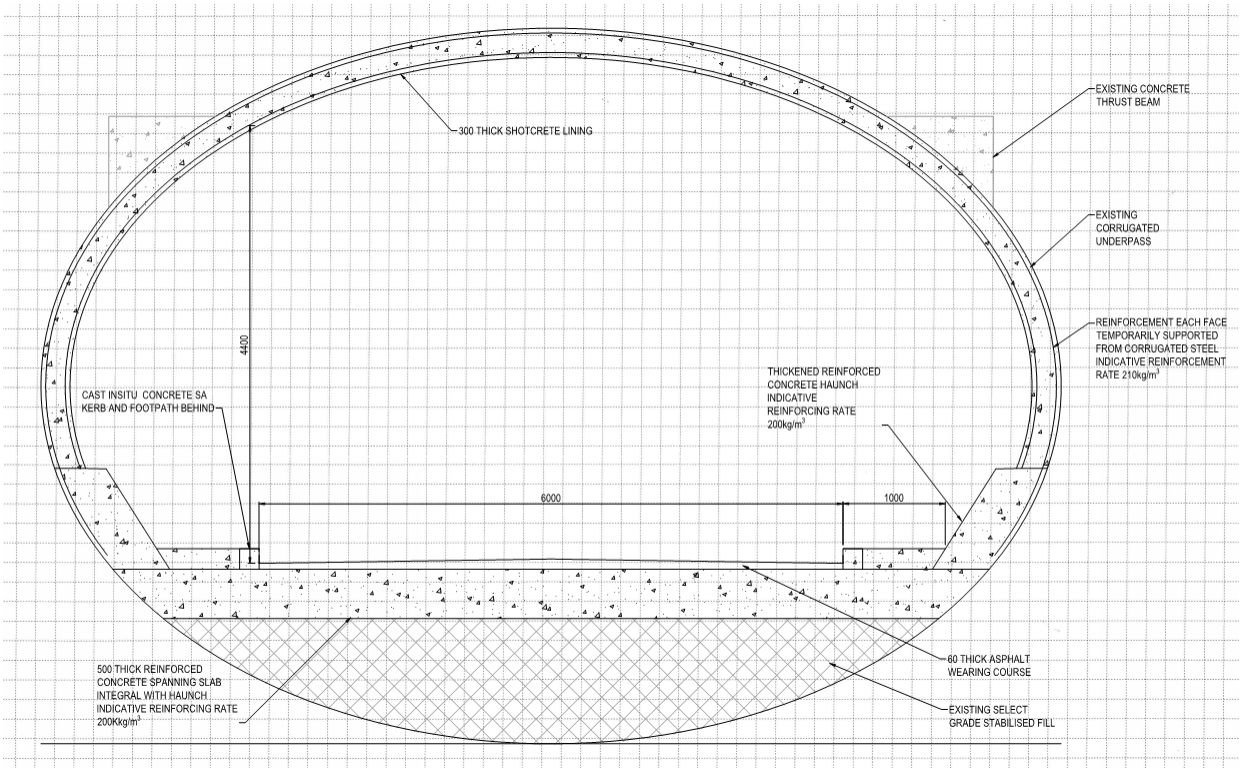

Figure 14 Concept option for shotcrete lining remedial works – Somersby

6.2 Arches

Similar to the horizontal ellipse structures, even when considering the short term remedial actions outlined above, corrosion of arch structures will continue until section capacities are close to exceedance. Again, the residual life for each structure depends on the geometry, loading and observed corrosion rate. Ongoing monitoring of the structures will help guide the requirement for installation of more significant strengthening works.

Long term remedial options considered for arch structures generally comprised the three following concept options:

1. For the smaller structures at Cameron Park and Taree, strengthening the corroded zone at the plate to footing connection was proposed as a remedial option. This would comprise welding shear studs to the corrugated sheets (above the corroded zone) to transfer the load to a cast in-situ concrete plinth which transferred the load to the existing footing.
2. Shotcrete lining was also proposed for all arch structures, with a new shotcrete arch springing from a widening to the existing concrete footing.
3. Use of precast arches or boxes (depending on the structure size) supported on cast in-situ concrete footings were also proposed. If adopted these options would require temporary steel slide beams to

jack the new precast concrete units into position. The void between the existing steel arch and the new precast concrete would then be grout filled.

As for horizontal ellipses, new technology such as fibre reinforced polymer linings were not considered due to the inability to make a connection with the arch footings. Similarly, cathodic protection was not considered for the reasons outlined above.

Costing of all options indicated that strengthening the arch to footing connection (where feasible) was the lowest cost option. Where this was not possible, shotcrete lining was the lowest cost option. As above, options were compared using a multi-criteria analysis of cost, ongoing maintenance, constructability, and functionality. This generally resulted in the concrete strengthening plinth being selected when feasible and the shotcrete arch option being selected for other cases.

7 CONCLUSION

This project completed a detailed assessment of three horizontal ellipse and three arch BCMS in the NSW Hunter region. Assessment included detailed site testing, durability, structural, and geometry assessments. In addition, three-dimensional laser scanning was undertaken for all structures, providing detailed point cloud data. This information was used to estimate the residual design life for all structures and then to develop and cost three concept remedial options for each structure before selecting a preferred option.

Key findings and lessons learned from the project included the following:

- Corrosion of the inspected horizontal ellipse structures typically occurred at the lap joints in the upper section of the structure and at the interface between the inner face and unpaved footpaths. This was attributed to poor local road drainage in the structures leading to local traffic splashing water onto the inside faces of the structures.
- Corrosion of the inspected arches typically occurred at the interface between the bolted plates and the cast in-situ reinforced concrete footing. This was attributed to microclimate conditions at this interface, exacerbated by the channel filling with dirt and debris which hold moisture.
- Laser scanning provided a safe and comprehensive method of remote data. This enabled the creation of spatially accurate models, from which sections could be cut at regular intervals for assessment of deformation. The scanning undertaken for this project will form a baseline survey for any future monitoring. Temporary lighting would benefit future scanning to improve photo imagery.
- Observed corrosion loss rates were generally lower than rates initially assessed for soil side and atmospheric corrosion at mid plate locations when using the relevant standards. Conversely, corrosion loss rates for microclimate conditions were generally higher than initially assessed in accordance with the relevant standards.
- For large horizontal ellipse structures, steel set strengthening was generally the most cost-effective remedial option followed by shotcrete lining. For arch structures strengthening of the footing to plate connection was generally the most cost-effective option, followed by construction of a shotcrete lining.
- Simultaneously undertaking a detailed assessment of several similar structures allowed for efficiencies in assessment and consistency in reporting that will assist in future planning decisions.

This project is an example of how early planning and proactive monitoring using the latest in technology best enables asset owners to actively manage the residual life of critical structures.

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Alan Michie is a Bridge Engineer at Aurecon with three years' experience in the design of bridges and civil infrastructure. Complementing his design experience is over 15 years' experience in the residential and commercial construction industry, specialising in the construction of small civil structures. Alan was the structural engineer for the project undertaking the inspections, condition assessment, development of remedial options, and reporting for the project.

Lindsay Brown is the Roads and Maritime Services NSW, Regional Bridge Engineer for the Hunter Region. During the works described in this paper, he was the region's Bridge Maintenance Planner and responsible for bridge asset management. Prior to commencing his current role in 2015, he gained seven years of bridge design experience in the Roads and Maritime Bridge New Design section, carrying out detailed new and rehabilitation design of complex structures. He previously spent five years in Bridge Evaluation and Assessment section, where he carried out heavy load assessments, bridge condition inspections and analytical assessments of existing structures. He holds a Bachelor of Engineering in Civil Project Engineering and Management with honours, from the University of Sydney.