**RTI 22-23-38**

Record 4

**The following has been released in relation to a request for information relating to the Tasman Bridge.**

This folder is for the 12 and 6 Month servicing of the Tasman Bridge

# Bridge servicing

## 12 Monthly service April 2019- Inspections and recommendations



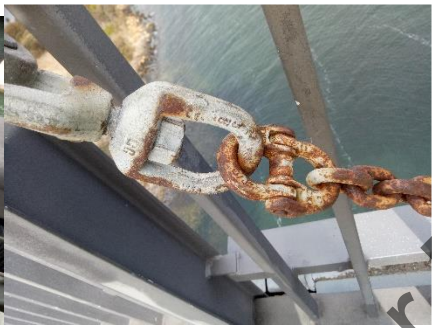
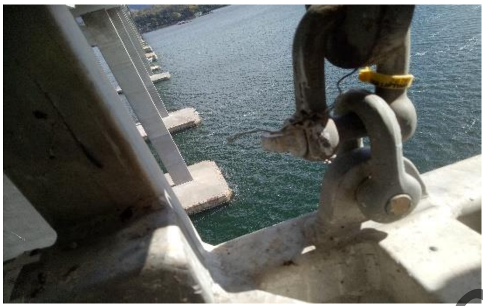
s.36 [redacted] April 2019

| INSPECTIONS | ACTIONS | CONCLUSIONS |
|---|---|---|
| 1-Inspections were done as per Fault finding program for MIG and FLG's | | |
| 2-Oil samples were taken of all gear and hydraulics | Oils checked for levels. Oil samples were inspected by William Adams. | All Hydraulic oils were deemed to be ok and monitor. Gear box oils were changed on the MIG. Will monitor now. |
| 3-Limit switches sticking | Sprayed with Lanolin | Recommend electrician to free up. |
| 4-Hydraulic hoses | Damaged due to being in weather | Recommend some need replacing. |
| 5-Non-slip tape worn and missing on access ladders (Photo 8) | Replace missing and worn tape on ladder rungs | Downer EDI to confirm with TEC to rectify issue. |
| 6-Hamerlocks on chains and shackles | Seized on chain sprayed with CRC. Some D shackles now riding off centre see photo. | Chains and hammerlocks to be replaced. Replace D shackle with Bow shackle. |
| 7-Extension arms on North & South Gantry | Arms are bent | Downer EDI to advise. |
| 8-Lynch pin missing from upstream platform gate bottom hinge | Fit new lynch pin | Downer EDI to confirm with TEC to rectify issue. |
| 9-Inspection tags missing from shackles and some shackles need to be re-moused | Inspect lifting equipment | Downer EDI to confirm with TEC to rectify issue. |
| 10-Plywood on platform has some damage | Monitor damage to ply to ensure it doesn't split | Downer EDI to monitor. |
| 11-Mesh underfoot on platform has a lot of flex in some areas under the plywood | Check platform flooring for fatigue or if welds needs to be added in for strength | Downer EDI to confirm with TEC to investigate. |
| 12-Rubber on hydraulic hoses deteriorating | Hoses to be replaced | Downer EDI to engage hydraulic technician to rectify issue. |

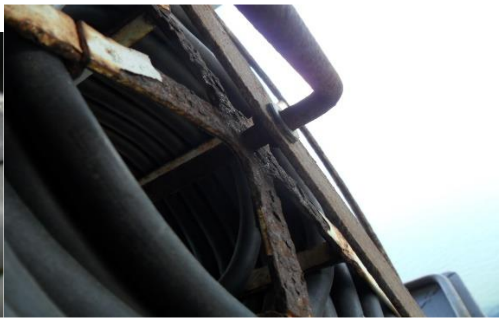
| | | |
|---|---|---|
| 13- Extension lead on Gantry | Out of date | All electrical leads need to be test and tagged in date. |
| 14-North Pendant boot | Damaged – wrapped in electrical tape. | Electrician to repair. |
| 15-Aluminium gantry has frame tied to it. Ply floor under foot. | Damaged ratchets. Ply floor unsatisfactory to be standing on. Damaged welds. | Replace ratchets. Replace ply. Repair welds to engineer's specifications. |
| 16-Covers on winch drum. | Bolts damaged | Recommend replace 6mm bolts with 8mm bolts. |
| 17-Grease on drums- Wire ropes | Were not able to grease rope fully on both drums as Downer were unable to move MIG up and down | Recommend this be done at next service. |

| | | |
|---|---|---|
| 18-Pressure gauge on power pac | Gauge was damaged missing plug. Potential for water damage to gauge | Hole for plug had Denso tape in it instead of proper plug. Recommend new gauge. |
| 19-North power real on Gantry reel | Retractable reel starting to weep oil | Cleaned the weep. Recommend monitor, long term remove reel and repair. |
| 20-Brake on MIG | Brake inspected. | Brake seems to be in good condition. |
| 21-Emergency brake | Emergency brake could not be checked as Downer were unable to operate | Recommend removing covers to inspect. |
| 22- Rigging equipment | Rigging equipment checked | TEC have engaged Taslifting to inspect and Retag all lifting and rigging equipment. |
| 23- Gantry rollers | Rollers inspected some wear | Monitor. Downer to advise TEC on replacement. |


Truckers











WARNING!
THIS EQUIPMENT MAY
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Record 5



# Department of State Growth

## On-Road Traveller System and Tasman Bridge Lane Use Management System

## Tasman Bridge Gantry Structural Capacity Assessment

June 2020

# Executive summary

This Tasman Bridge Gantry Structural Capacity Assessment has been prepared for the Department of State Growth (the Department) in order to assist in the development of options for the On-Road Traveller Information System (OTIS) and Tasman Bridge Lane Use Management System (LUMS) project (the Project).

The purpose of this report is to provide general guidance to Tenderers on the potential for the gantries to be used as part of the proposed OTIS and LUMS design. Tenderers shall undertake their own structural inspections, assessments and analysis of the gantries to confirm their adequacy to form part of their final designs. Any outcomes or recommendations included in this report must not be relied upon, as:

- The assessment is limited to the gantries themselves and the connecting members to the bridge structure.
- The Tasman Bridge itself was not included in the assessment.
- The accuracy of the assessment is based on the accuracy of the details in the available as-built drawings.
- It was assumed that all members and connections were installed as specified.
- The gantries were not physically inspected during this assessment.
- All gantries were assessed as new gantries and no reductions of the gantry sections were taken into account.
- Due to incomplete information, Gantry 6 was visually inspected with the purpose of confirming the dimensions of the gantry and no physical condition assessment being carried out.
- Fatigue assessments were limited to the connections, with available structural details including bolted connections at the base of the gantries and the bolted connections at the beam-column connections.
- Fatigue assessments of the welded connections (i.e. chords of the trusses to the end plates or sign connections) were not assessed due to incomplete information in the drawings.



Full details of the assessment are provided in Section 3, and Appendix A to Appendix D.

The assessment indicates that members of all of the gantries 1-7 and 9-12 were adequate in handling the existing loads and the additional load of 1000(w) x 1000(h) and 50 kg signs mounted on the trusses, centrally over the lanes, or on the columns of the gantries.

The top and bottom chords of the trusses of Gantry No 8 & 13 were determined to fail theoretically under ultimate load combinations. Therefore these members require additional strengthening or replacement in order for the gantries to adequately handle the load imposed on them.

The structural capacity of the connections are theoretically adequate in handling the ultimate limit state.

When assessing fatigue of bolted connections, the base connection was found to be critical for all gantries. The connections were checked for fatigue in the bolts (both in tension and shear), fatigue in the base plate in bending, and fatigue in the welds. Connection Type 5 (6-M30 Bolts 25mm Plate located in the Base of gantries 2, 4, 5, 6 and 7) was determined to fail as a result of fatigue in the bolts in tension. Connection Type 8 (4-M30 Bolts 36mm Plate located in the Base of gantry 3) was also determined to fail due to fatigue in the bolts in tension.

Beam-Column connections were found to have negligible forces acting on them due to the natural gust wind load.

Fatigue assessments of welded connections between the truss chords and the end plates were undertaken assuming 8mm fillet welds for the connections. All connection were deemed to be adequate in fatigue.

Welded connections between secondary beams and columns, and connections between the signage and the gantries were not considered. Further information is required to conduct these assessments as there is limited details relating to these connections in the reference drawings.

Based on the outcomes and limitations of this assessment, the following commentary on the results should be noted:

- Strengthening of the truss chords of Gantry No 8 & 13 is recommended if additional loading is to be imposed on these gantries. A targeted inspection is recommended to check the likely corrosion and section loss within the top and the bottom chords of the truss.
- Further targeted inspections of the connections (Type 5 & Type 8) to determine the current condition, and a detailed Finite Element (FEM) fatigue stress analysis was not undertaken. Remedial works might be required based on the inspection and the results from FEM analysis of the connections for fatigue stresses.
- Due to limited details in the reference drawings, a detailed fatigue assessment was not undertaken for the welded connections listed below. Additionally, an initial targeted inspection was not undertaken to confirm the connection details. Therefore the fatigue assessment on the welded connections is limited.

| Connection | Description | Gantry |
|---|---|---|
| Type 4 | 370x152 Beam to 254x152 RHS Column Weld | 1 |
| Type 7 | 89 CHS to End Plate Weld | 2, 6, 7, 8, 13 |
| Type 10 | 380x200 Beam to End Plate Weld | 3 |
| Type 11 | 430x180 Beam to 300x180 Column Weld | 3 |
| Type 13 | 273 CHS to End Plate Weld | 4, 5, 9, 10, 11, 12 |
| Type 16 | 203x102 RHS Beam to 254x152 RHS Column Weld | 8, 13 |
| Type 19 | Sign to Gantry Connection | 1, 2, 3, 7, 8, 13 |
| Type 21 | Column to Base Plate Weld | All |

# Table of contents



# Table index



# Figure index



# Appendices

# 1. Introduction

## 1.1 Background and key objectives

The Department of State Growth (the Department) engaged GHD to support the planning and delivery of the installation of new infrastructure, at various locations around Greater Hobart, for the providing information to travellers on the Hobart road network through the provision of a On Road Traveller Information System (OTIS), and the upgrading of infrastructure on the Tasman Bridge and Tasman Highway to accommodate a new Lane Use Management System (LUMS).

The OTIS and LUMS will improve travel times, network efficiency and safety outcomes for the Tasman Bridge and the Greater Hobart area. These interconnected initiatives are another example of the Department developing smart infrastructure and transport systems to support ongoing industry and business growth and improving community outcomes. By delivering new infrastructure at various locations within the Greater Hobart area and upgrading existing infrastructure on the Tasman Bridge and Tasman Highway, the Department will enable the safe, reliable and efficient transport of people and goods through an improved, integrated transport system.

The On-Road Traveller Information System (OTIS) and Tasman Bridge Lane Use Management System (LUMS) project (the Project) has several key objectives:

- Improved road network management system integration;
- Enhanced corridor management capability and greater visibility of prevailing traffic conditions;
- Improved network resilient to changes in traffic conditions that may arise due to crashes, breakdowns or other unplanned events;
- Improved travel times and network efficiency, through:
  - Enhanced travel time prediction reliability.
  - Enhanced traffic throughput.
  - Enhanced traveller decision making opportunities.
- Reduced vehicle emissions;
- Increased accountability for the TMC operators and the incident management teams
- Improved traffic safety; and
- Improved work, health and safety outcomes of the use and management of traffic flow systems.

This report documents a desktop assessment of existing overhead gantries and the provision of guiding parameters for the additional loads that can be tolerated.

## 1.2 Purpose of this report

The purpose of the Project is to develop a Staged Design and Construct Framework (the Framework) that will inform, and be the basis of the Specifications for, the Project Request for Tender package from which the design and construct contractor for the Project will be procured.

As shown in Figure 1-1 the Framework is informed by three elements: a Concept of Operations report; a structural assessment of the Tasman Highway Gantries; and a Traffic Assessment of the Tasman Highway. These elements will assist in the Framework appropriately balancing the need to guide the potential scope of tender submissions, while still allowing for a level of innovation.

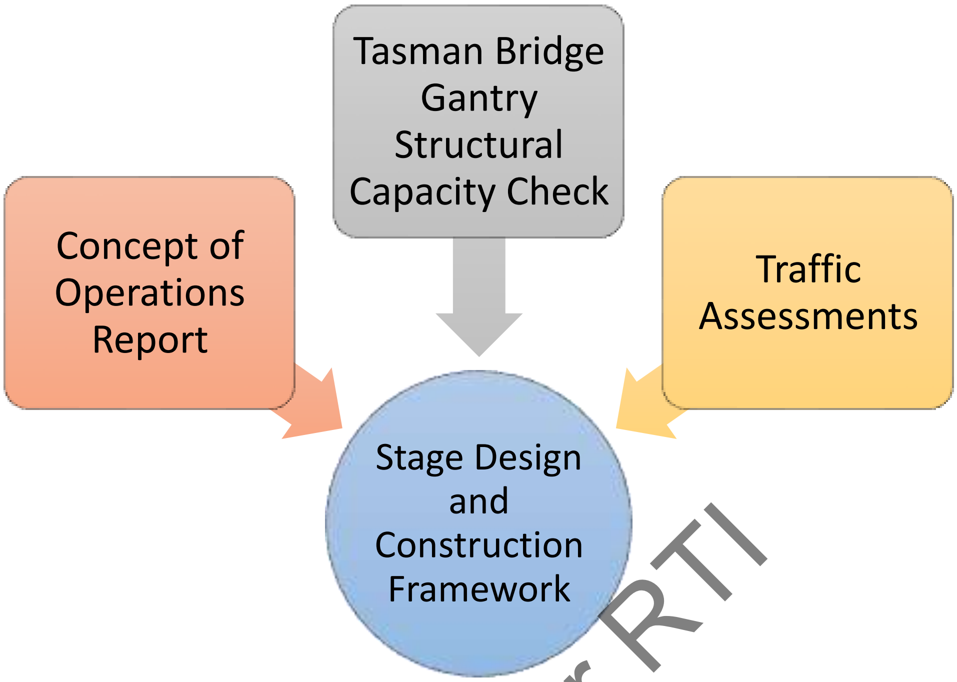


**Figure 1-1 Convergence of Framework elements**

This report, the Tasman Bridge Gantry Structural Capacity Assessment, details the results of a desktop assessment of gantries 1 to 13. The assessment has been limited to the gantries themselves, and the connecting member to the bridge structure / base. The assessment confirms the current condition of the gantries and if additional loads can be tolerated. This information will be used in the Framework to inform each gantry's ability to support any additional load from a LUMS device that may be mounted to the gantry.

## 1.3 Reference documents

This report has been prepared with reference to a range of existing State Government plans, as well as standards and guidelines including the following drawings detailed in Table 1-1 and guidelines detailed in Table 1-2.

**Table 1-1 Reference drawings**

| Drawing Number | Drawing Title |
|---|---|
| DE-341 | Reversible Lane Control Gantry No 1 Assembly Details |
| DE-342 | Reversible Lane Control Gantry No 1 Column Details |
| DE-343 | Reversible Lane Control Truss for Gantry No 1 |
| DE-409 | Tasman Highway Gantry No 1 Modifications to Existing Truss - Sheet 1 of 2 |
| DE-410 | Tasman Highway Gantry No 1 Modifications to Existing Truss - Sheet 2 of 2 |
| DE-404 | Tasman Highway Gantry No 2 Assembly Details |
| DE-405 | Tasman Highway Gantry No 2 Footing Details |
| DE-406 | Tasman Highway Gantry No 2 Trestle Details |
| DE-407 | Tasman Highway Gantry No 2 Truss Details |
| DE-408 | Tasman Highway Gantry No 2 Sign Light Attachment |
| DE-411 | Tasman Highway Gantry No 3 Assembly Details |
| DE-412 | Tasman Highway Gantry No 3 Trestle & Tower Details - Sheet 1 of 3 |

| | |
|---|---|
| DE-413 | Tasman Highway Gantry No 3 Trestle & Tower Details - Sheet 2 of 3 |
| DE-414 | Tasman Highway Gantry No 3 Trestle & Tower Details - Sheet 3 of 3 |
| DE-415 | Tasman Highway Gantry No 3 Truss Details - Sheet 1 of 2 |
| DE-416 | Tasman Highway Gantry No 3 Truss Details - Sheet 2 of 2 |
| DE-417 | Tasman Highway Gantry No 3 Signal Light Attachment |
| DE-337 | Reversible Lane Control Gantry No 4 Assembly Details |
| DE-338 | Reversible Lane Control Gantry No 4 Fabrication Details |
| DE-339 | Reversible Lane Control Gantry No 5 Assembly Details |
| DE-340 | Reversible Lane Control Gantry No 5 Fabrication Details |
| DE-349 | Reversible Lane Control Gantry No 7 Assembly and Footing Details |
| DE-350 | Reversible Lane Control Gantry No 7 Column Details |
| DE-351 | Reversible Lane Control Truss for Gantry No 7 |
| DE-352 | Reversible Lane Control Truss for Gantry No 7 – Sections |
| DE-324 | Reversible Lane Control Gantry No 8 Assembly Details |
| DE-325 | Reversible Lane Control Gantry No 8 Trestle Details |
| DE-326 | Reversible Lane Control Gantry No 8 Truss |
| DE-327 | Reversible Lane Control Anchorage Arrangement for Gantry No 8 |
| DE-356 | Reversible Lane Control Widening Gantry No 8 Assembly Details |
| DE-357 | Reversible Lane Control Widening Gantry No 8 Truss |
| DE-358 | Reversible Lane Control Widening Gantry No 8 Trestle Details |
| DE-328 | Reversible Lane Control Gantry No's 9,10,11 & 12 Assembly |
| DE-329 | Reversible Lane Control Gantry No's 9,10,11 & 12 Details |
| DE-330 | Reversible Lane Control Gantry No's 9,10,11 & 12 Anchorage Arrangement |
| DE-362 | Reversible Lane Control Widening Gantry No 9,10,11 & 12 Details |
| DE-362 A | Reversible Lane Control Widening Gantry No 9,10,11 & 12 Details |
| DE-363 | Reversible Lane Control Widening Gantry No 9,10,11 & 12 Assembly Details |
| DE-344 | Reversible Lane Control Gantry No 13 Assembly Details |
| DE-345 | Reversible Lane Control Gantry No 13 Trestle Details |
| DE-346 | Reversible Lane Control Gantry No 13 Truss |
| DE-347 | Reversible Lane Control Anchorage Arrangement for Gantry No 13 |
| DE-360 | Reversible Lane Control Widening Gantry No 13 Truss |
| DE-361 | Reversible Lane Control Widening Gantry No 13 Trestle Details |



**Table 1-2 Reference standards / guidelines**

| Australian Standard | Title |
|---|---|
| AS/NZS 1170.0 – 2002 | Structural Design Actions Part 0: General Principles |
| AS/NZS 1170.1 – 2002 | Structural Design Actions Part 1: Permanent, Imposed and Other Actions |
| AS/NZS 1170.2 – 2011 | Structural Design Actions Part 2: Wind Actions |
| AS 4100 – 1998 | Steel Structures |
| AS 5100.2 – 2017 | Bridge Design Part 2: Design Loads |
| AS 5100.6 – 2017 | Bridge Design Part 6: Steel and Composite Construction |
| AS 1275 – 1985 | Metric Screw Threads for Fasteners |
| AASHTO | Standard Specification for Structural Supports for Highway Signs, Luminaires and Traffic Signals |

## 1.4 Scope

In anticipation that new devices associated with the LUMS will be mounted on the existing gantries, an assessment was undertaken on all gantries on the Tasman Bridge. The assessment was undertaken as a desktop study in order to identify and provide the parameters for the additional loads that can be tolerated by the existing gantries including:

- Lateral and rotational forces at the base of the gantry
- Loadbearing capacity of the gantry
- Fatigue assessment

## 1.5 Limitations

The assessment is limited to the gantries themselves and the connecting members to the bridge structure. The Tasman Bridge itself was not included in the assessment. The accuracy of the assessment is based on the accuracy of the details in the available as-built drawings. It is assumed that all members and connections were installed as specified. The gantries were not physically inspected during this assessment. All gantries were assessed as new gantries and no reductions of the gantry sections were taken into account.

Due to incomplete information, Gantry 6 was visually inspected with the purpose of confirming the dimensions of the gantry. As stated earlier, a physical condition assessment was not carried out on the gantry.

Fatigue assessments were limited to the connections with available structural details including bolted connections at the base of the gantries and the bolted connections at the beam-column connections. Fatigue assessments of the welded connections (i.e. chords of the trusses to the end plates or sign connections) were not assessed due to incomplete information in the drawings.



## 1.6 Methodology

Based on the documents provided by the Department of State Growth (see section 1.3), models of the gantries were developed in the structural analysis software, SpaceGass. The wind loading applied on the gantries were calculated according to AS1170.2 and the self-weight loading was calculated based on the available drawings. The calculated factored loads were applied to the models and the induced moments, forces and reactions were analysed by SpaceGass. Hand calculations were undertaken to calculate the ULS capacity of the gantry members and connections in accordance with AS4100. These were compared against the SpaceGass analysis to determine the adequacy of the gantry in handling the applied loads. Similarly, the fatigue stresses were calculated in accordance with AASHTO guidelines (hand calculations), and compared against the demand stress from SpaceGass analysis for the average wind speed found for the site.

## 1.7 Disclaimer

This report: has been prepared by GHD for Department of State Growth and may only be used and relied on by Department of State Growth for the purpose agreed between GHD and the Department of State Growth as set out in this report.

GHD otherwise disclaims responsibility to any person other than Department of State Growth arising in connection with this report. GHD also excludes implied warranties and conditions, to the extent legally permissible.

The services undertaken by GHD in connection with preparing this report were limited to those specifically detailed in the report and are subject to the scope limitations set out in the report.

The opinions, conclusions and any recommendations in this report are based on conditions encountered and information reviewed at the date of preparation of the report. GHD has no responsibility or obligation to update this report to account for events or changes occurring subsequent to the date that the report was prepared.

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## 1.8 Report structure

The report is set out as follows:

- **Section 1 – Introduction:** provides an overview of the project objectives.
- **Section 2 – Study area:** defines the project study area.
- **Section 3 – Desktop structural assessment:** the outcomes of the structural capacity assessment.
- **Section 4 – Recommendations:** details the recommended additional strengthening required and any further investigations that will be required to confirm the outcomes of the assessment.

# 2. Study area

The project is located in Hobart, Tasmania. As indicated by Figure 2-1 below, the thirteen gantries that constitute the extent of this assessment, are located on the approximately 3 km length of the Tasman Highway/Tasman Bridge between the Eastern Shore and Hobart CBD.




Map data source: Department of State Growth, Gantry locations, 2020; DPIPWE, topographic base map, 2020

**Figure 2-1 Gantry locations**

# 3. Desktop structural assessment

## 3.1 Loading

The wind loading applied to the gantries were determined in accordance with the AS 1170.2. The wind load parameters for ULS and fatigue are considered as below:

- Region = A3
- Ultimate Regional Wind Speed V1000 = 46m/s
- Terrain Category = 2.5
- Topography Multiplier (Mt) = 1
- Shielding Multiplier (Ms) = 1
- Average Mean Wind Speed (Fatigue) = 16.1 Km/h

The assessed considered proposed signage mounted on the gantries of dimensions 1000(w) x 1000(h) and 50 kg in weight. The assessment is based on signage mounted on the trusses (centrally over the lanes), or on the columns of the gantries.

The following load combinations from AS5100 were utilised for the assessment of the gantries:

- $1.1G + 2G_{SDL} + W_U$
- $0.9G + 0.9G_{SDL} + W_UG + W_S$
- Natural Gust Wind

Where G is the dead load, $G_{SDL}$ is the superimposed dead load, $W_U$ is the ultimate wind load, and $W_S$ is the serviceability wind load. Natural gust wind is the wind load used for the fatigue assessment.

## 3.2 Assumptions

The following assumptions were made in the assessment of the gantries:

- In the case where non-standard sections were utilised and details of the material properties were unavailable, the yield strength was assumed to be 230 MPa. For standard sections which were constructed and strengthened using new steel members around 1990, a yield strength of 300 Mpa was assumed. For 89x4.88mm CHS, sections were assumed to utilise a yield strength of 350 MPa. Details of the assumptions in relation to yield stress for each member of the gantries is presented in Appendix A.
- For the purpose of determining the wind speeds, the top of the gantries were assumed to be between 30 m and 50 m above the water level, were appropriate. The entire width of the gantry was assumed to be used for signage, and the worst case wind direction was assumed to be acting normal to the signage.

## 3.3 Member capacity

Members of all of the gantries 1-7 and 9-12 were determined to be adequate in handling the existing loads and the additional load for the proposed signage with results were presented in Table 3-1. The top and bottom chords of the trusses of Gantry No 8 & 13 were determined to fail theoretically under ultimate load combinations. These members require additional strengthening or replacement in order for the gantries to adequately handle the load imposed on them.

Refer to Appendix A for detailed member capacity calculations.

**Table 3-1 Member capacity**

| Gantry ID | Asset Numbers | Member Capacity | | | Likely Failure Mode | Additional Strengthening to Support 50 kg Sign Over Each Lane |
|---|---|---|---|---|---|---|
| | | Top/Bottom Chord | Column | Secondary beam | | |
| 1 | SG 6164 | Adequate | Adequate | Adequate | N/A | Not Required |
| 2 | SG 6165 | Adequate | Adequate | N/A | N/A | Not Required |
| 3 | SG 6166 | Adequate | Adequate | Adequate | N/A | Not Required |
| 4 | SG 6167 | Adequate | Adequate | N/A | N/A | Not Required |
| 5 | SG 6168 | Adequate | Adequate | N/A | N/A | Not Required |
| 6 | SG 6169 | Adequate | Adequate | N/A | N/A | Not Required |
| 7 | SG 6170 | Adequate | Adequate | Adequate | N/A | Not Required |
| 8 | SG 6171 | Marginal | Adequate | Adequate | Biaxial bending failure | Recommended |
| 9 | SG 6172 | Adequate | Adequate | N/A | N/A | Not Required |
| 10 | SG 6173 | Adequate | Adequate | N/A | N/A | Not Required |
| 11 | SG 6174 | Adequate | Adequate | N/A | N/A | Not Required |
| 12 | SG 6175 | Adequate | Adequate | N/A | N/A | Not Required |
| 13 | SG 6176 | Marginal | Adequate | Adequate | Biaxial bending failure | Recommended |

## 3.4 Connection capacity

The connection capacities - with the current available details - were reviewed against the ULS demands. The structural capacity of the connections are theoretically adequate in handling the ultimate limit state, see Table 3-2, with connection Type 5 failing in fatigue. If the utilisation is less than or equal to 100% it indicates that the connection has sufficient capacity.

Refer to Appendix B for detailed connection capacity calculations.

**Table 3-2 Connection capacity**

| Connection | Description | Location | Utilized Gantries | ULS Utilisation | Fatigue Utilisation |
|---|---|---|---|---|---|
| Type 1 | 6-M30 Bolts 32mm Plate | Base | 1 | 30% (Adequate) | 75% (Adequate) |
| Type 2 | 8-M20 Bolts 25mm Plate | Beam-Column | 1 | 25% (Adequate) | 15% (Adequate) |
| Type 3 | Column to Base Plate Weld | Base | 1 | 55% (Adequate) | 10% (Adequate) |
| Type 4 | Secondary Beam to Column Weld | Beam-Column | 1 | N/A | N/A |
| Type 5 | 6-M30 Bolts 25mm Plate | Base | 2, 4, 5, 6, 7 | 40% (Adequate) | 200% (FAILED) |
| Type 6 | 4-M16 Bolts 12mm Plate | Beam-Column | 2, 6, 7 | 50% (Adequate) | 70% (Adequate) |
| Type 7 | 89mm CHS to End Plate Weld | Beam-Column | 2, 6, 7, 8, 13 | N/A | 40% (Adequate) |
| Type 8 | 4-M30 Bolts 36mm Plate | Base | 3 | 20% (Adequate) | 100% (Adequate) |
| Type 9 | 10-M24 Bolts 32mm Plate | Beam-Column | 3 | 25% (Adequate) | 20% (Adequate) |

| Connection | Description | Location | Utilized Gantries | ULS Utilisation | Fatigue Utilisation |
|---|---|---|---|---|---|
| Type 10 | Truss Chords to End Plate Weld | Beam-Column | 3 | N/A | < 5% (Adequate) |
| Type 11 | Secondary Beam to Column Weld | Beam-Column | 3 | N/A | N/A |
| Type 12 | 4-M24 Bolts 25mm Plate | Beam-Column | 4, 5 | 20% (Adequate) | 15% (Adequate) |
| Type 13 | 273mm CHS to End Plate Weld | Beam-Column | 4, 5, 9, 10, 11, 12 | N/A | 35% (Adequate) |
| Type 14 | 4-M30 Bolts 20mm Plate | Base | 8, 13 | 20% (Adequate) | 70% (Adequate) |
| Type 15 | 4-M24 Bolts 12mm Plate | Beam-Column | 8, 13 | 25% (Adequate) | 25% (Adequate) |
| Type 16 | Secondary Beam to Column Weld | Beam-Column | 8, 13 | N/A | N/A |
| Type 17 | 4-10mm Plate Welded Connection | Base | 9, 10, 11, 12 | 50% | 70% (Adequate) |
| Type 18 | 4-M30 Bolts 12mm Plate | Beam-Column | 9, 10, 11, 12 | 45% | 45% (Adequate) |
| Type 19 | Sign to Gantry Connection | Sign | 1, 2, 3, 7, 8, 13 | N/A | N/A |
| Type 20 | Connections of Minor Members | All | All | Negligible | Negligible |

## 3.5 Fatigue assessment

When assessing the fatigue of the bolted connections, the base connection was found to be critical for all gantries. Hence, a fatigue assessment was conducted on all bolted base connections. The assessment was calculated for the remaining service life, and summarised in Appendix D. The average wind speed for the site was determined to be 16.1 km/h (4.5m/s), obtained from the Bureau of Meteorology data (station 094029). The wind pressure acting on the gantries was determined in accordance with AASHTO, using the wind speed of 16.1 km/h.

The connections were checked for fatigue in the bolts (both in tension and shear), fatigue in the base plate in bending, and fatigue in the welds. Connection Type 5 was determined to fail as a result of fatigue in the bolts in tension, under the assumption made in the calculations. Connection Type 8 was also determined just pass due to fatigue in the bolts in tension.

Beam-Column connections were found to have negligible forces acting on them due to the natural gust wind load.

Fatigue assessments of welded connections between the truss chords and the end plates were also undertaken. 8mm fillet welds were assumed for these connections. Apart from connection Type 13, all beam to end plate welded connections were deemed to have a negligible load acting on them due to the natural gust wind load. Connection Type 13 was deemed to be adequate in fatigue given the assumptions in the calculations are accurate. Refer to Appendix B, Table B11 for detailed connection fatigue results.

Welded connections between secondary beams and columns, and connections between the signage and the gantries were not considered. Further information is required to conduct these assessments as there is limited details relating to these connections in the reference drawings.

# 4. Recommendations

The purpose of this report is to provide general guidance to Tenderers on the potential for the gantries to be used as part of the proposed OTIS and LUMS design. The results of the desktop assessment of gantries 1 to 13 indicates that some structural elements of the gantries may not tolerate the additional loads imposed upon them if new devices associated with the LUMS are mounted. Tenderers shall undertake their own structural inspections, assessments and analysis of the gantries to confirm their adequacy to form part of their final designs. Any outcomes or recommendations included in this report must not be relied upon. Therefore based on the outcomes and limitations of this assessment, the following commentary on the results should be noted:

- Strengthening of the truss chords of Gantry No 8 & 13 is recommended if additional loading is to be imposed on these gantries. A targeted inspection is recommended to check the likely corrosion and section loss within the top and the bottom chords of the truss.
- Further targeted inspections of the connections that failed (Type 5 & Type 8) to determine the current condition, and a detailed Finite Element (FEM) fatigue stress analysis was not undertaken. Remedial works might be required based on the inspection and the results from FEM analysis of the connections for fatigue stresses.
- Due to limited details in the reference drawings, a detailed fatigue assessment was not undertaken for the welded connections listed in Table 4-1 below. Additionally, an initial targeted inspection was not undertaken to confirm the connection details. Therefore the fatigue assessment on the welded connections is limited.

**Table 4-1 Connections requiring detail confirmation**

| Connection | Description | Gantry |
|---|---|---|
| Type 4 | 370x152 Beam to 254x152 RHS Column Weld | 1 |
| Type 7 | 89 CHS to End Plate Weld | 2, 6, 7, 8, 13 |
| Type 10 | 380x200 Beam to End Plate Weld | 3 |
| Type 11 | 430x180 Beam to 300x180 Column Weld | 3 |
| Type 13 | 273 CHS to End Plate Weld | 4, 5, 9, 10, 11, 12 |
| Type 16 | 203x102 RHS Beam to 254x152 RHS Column Weld | 8, 13 |
| Type 19 | Sign to Gantry Connection | 1, 2, 3, 7, 8, 13 |
| Type 21 | Column to Base Plate Weld | All |

# Appendices

# **Appendix A** – Member capacity calculations

### Table A-1 Gantry No 1

| Section | Description | About x-axis | | | | About y-axis | | | | Axial | | Bi-Axial Bending Ratio | Capacity |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | ϕMsx (kNm) | Msx* (kNm) | ϕVx (kN) | Vx* (kN) | ϕMsy (kNm) | Msy* (kNm) | ϕVy (kN) | Vy* (kN) | ϕNu (kN) | N* (kN) | | |
| 254x152x6 RHS | Main Beam, fsy=300 MPa | 108.50 | 78.61 | 493.78 | 23.50 | 75.95 | 15.56 | 295.49 | 13.41 | 975.24 | 135.40 | 0.92(*) | Adequate |
| 254x152x9.5 RHS | Column, fsy=300 MPa | 166.15 | 49.95 | 781.81 | 22.43 | 115.53 | 18.52 | 467.86 | 11.03 | 1227.22 | 160.33 | N/A | Adequate |
| 370x152 | Secondary Beam, fsy=230 MPa | 367.40 | 55.27 | 1470.53 | 68.61 | 162.36 | 15.92 | 604.11 | 8.24 | N/A | N/A | N/A | Adequate |
| 203x102x6.3 | Secondary Beam, fsy=230 MPa | 49.80 | 43.45 | 317.68 | 48.44 | N/A | N/A | N/A | N/A | N/A | N/A | N/A | Adequate |

(*) If the bi-Axial Bending Ratio is less than 1 that means the member has sufficient capacity

### Table A-2 Gantry No 2

| Section | Description | About x-axis | | | | About y-axis | | | | Axial | | Bi-Axial Bending | Capacity |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | ϕMsx (kNm) | Msx* (kNm) | ϕVx (kN) | Vx* (kN) | ϕMsy (kNm) | Msy* (kNm) | ϕVy (kN) | Vy* (kN) | ϕNu (kN) | N* (kN) | | |
| 89 Dia CHS,fsy=350 Mpa | Main Beam | 10.71 | 5.44 | 144.02 | 6.99 | N/A | N/A | N/A | N/A | 361.79 | 102.66 | 0.84(*) | Adequate |
| 375x152x12 RHS, fsy=230 Mpa | Column | 231.02 | 162.10 | 1126.74 | 24.20 | 132.80 | 8.36 | 453.08 | 7.45 | 2422.09 | 36.68 | N/A | Adequate |

(*) If the bi-Axial Bending Ratio is less than 1 that means the member has sufficient capacity

### Table A-3 Gantry No 3

| Section | Description | About x-axis | | | | About y-axis | | | | Axial | | Bi-Axial Bending | Capacity |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | ϕMsx (kNm) | Msx* (kNm) | ϕVx (kN) | Vx* (kN) | ϕMsy (kNm) | Msy* (kNm) | ϕVy (kN) | Vy* (kN) | ϕNu (kN) | N* (kN) | | |
| 2 - 380 PFC | Main Beam,fsy=300 MPa | 261.03 | 68.72 | 1231.20 | 80.08 | 242.99 | 165.25 | 1134.00 | 22.06 | 2892.33 | 96.45 | 0.98 | Adequate |
| 2 - 300 PFC | Column,fsy=300 MPa | 267.97 | 71.76 | 777.60 | 31.07 | 169.55 | 8.46 | 715.39 | 2.75 | 2492.71 | 203.72 | N/A | Adequate |
| 2 - 430x16 Plates & 2 - 180x25 Plates | Secondary Beam, fsy=230 MPa | 706.74 | 86.57 | 1117.80 | 94.97 | 362.97 | 2.67 | 1708.99 | 2.81 | N/A | N/A | N/A | Adequate |
| 254x152x6.3 RHS | Secondary Beam, fsy=230 MPa | 87.10 | 72.60 | 397.49 | 72.83 | N/A | N/A | N/A | N/A | N/A | N/A | N/A | Adequate |

(*) If the bi-Axial Bending Ratio is less than 1 that means the member has sufficient capacity



### Table A-4 Gantry No 4

| Section | Description | About x-axis | | | | About y-axis | | | | Axial | | Capacity |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | ϕMsx (kNm) | Msx* (kNm) | ϕVx (kN) | Vx* (kN) | ϕMsy (kNm) | Msy* (kNm) | ϕVy (kN) | Vy* (kN) | ϕNu (kN) | N* (kN) | |
| 273 Dia CHS | Main Beam,fsy=230 MPa | 88.34 | 18.76 | 375.05 | 10.70 | N/A | N/A | N/A | N/A | N/A | N/A | Adequate |
| 305x203x6.3 RHS | Column,fsy=230 MPa | 128.70 | 38.89 | 477.30 | 7.09 | 97.28 | 18.76 | 317.68 | 4.80 | 957.46 | 13.71 | Adequate |

**Table A-5 Gantry No 5**

| Section | Description | About x-axis | | | | About y-axis | | | | Axial | | Capacity |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | ϕMsx (kNm) | Msx* (kNm) | ϕVx (kN) | Vx* (kN) | ϕMsy (kNm) | Msy* (kNm) | ϕVy (kN) | Vy* (kN) | ϕNu (kN) | N* (kN) | |
| 273 Dia CHS | Main Beam,fsy=230 MPa | 88.34 | 20.12 | 375.05 | 11.37 | N/A | N/A | N/A | N/A | N/A | N/A | Adequate |
| 305x203x6.3 RHS | Column,fsy=230 MPa | 128.70 | 40.34 | 477.30 | 7.49 | 97.28 | 20.12 | 317.68 | 5.13 | 957.46 | 14.31 | Adequate |

**Table A-6 Gantry No 6**

| Section | Description | About x-axis | | | | About y-axis | | | | Axial | | Bi-Axial Bending | Capacity |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | ϕMsx (kNm) | Msx* (kNm) | ϕVx (kN) | Vx* (kN) | ϕMsy (kNm) | Msy* (kNm) | ϕVy (kN) | Vy* (kN) | ϕNu (kN) | N* (kN) | | |
| 89 Dia CHS | Main Beam, fsy=250 | 10.86 | 3.52 | 146.25 | 5.61 | N/A | N/A | N/A | N/A | 382.85 | 43.60 | 0.86 | Adequate |
| 375x152x12 RHS | Column,fsy=230 MPa | 290.07 | 57.02 | 1458.00 | 8.76 | 150.76 | 6.77 | 453.08 | 6.61 | 2403.16 | 26.77 | N/A | Adequate |

(*) If the bi-Axial Bending Ratio is less than 1 that means the member has sufficient capacity

**Table A-7 Gantry No 7**

| Section | Description | About x-axis | | | | About y-axis | | | | Axial | | Bi-Axial Bending | Capacity |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | ϕMsx (kNm) | Msx* (kNm) | ϕVx (kN) | Vx* (kN) | ϕMsy (kNm) | Msy* (kNm) | ϕVy (kN) | Vy* (kN) | ϕNu (kN) | N* (kN) | | |
| 89 Dia CHS | Main Beam,fsy=250 | 10.86 | 4.53 | 146.25 | 25.05 | N/A | N/A | N/A | N/A | 382.85 | 60.00 | 1 | Adequate |
| 375x152x12 RHS | Column,fsy=230 MPa | 290.07 | 190.32 | 1458.00 | 27.77 | 150.76 | 8.26 | 453.08 | 10.06 | 2403.16 | 30.93 | N/A | Adequate |
| 75x50x6 RHS | Secondary Beam,fsy=230 MPa | 5.70 | 0.94 | 97.20 | 2.72 | 4.32 | 2.37 | 64.80 | 28.36 | N/A | N/A | N/A | Adequate |

(*) If the bi-Axial Bending Ratio is less than or equal 1 that means the member has sufficient capacity

**Table A-8 Gantry No 8**

| Section | Description | About x-axis | | | | About y-axis | | | | Axial | | Bi-Axial Bending | Capacity |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | ϕMsx (kNm) | Msx* (kNm) | ϕVx (kN) | Vx* (kN) | ϕMsy (kNm) | Msy* (kNm) | ϕVy (kN) | Vy* (kN) | ϕNu (kN) | N* (kN) | | |
| 89 Dia CHS | Main Beam,fsy=350 MPa | 10.71 | 8.49 | 144.02 | 17.86 | N/A | N/A | N/A | N/A | 361.79 | 34.59 | 1.24 (*) | Failed |
| 375x152x12 RHS | Column,fsy=230 MPa | 87.10 | 36.83 | 518.46 | 21.71 | 60.93 | 8.41 | 237.87 | 7.05 | 1013.55 | 127.81 | N/A | Adequate |
| 75x50x6 RHS | Secondary Beam,fsy=230 MPa | 49.80 | 36.90 | 317.68 | 41.42 | 30.55 | 5.62 | 159.62 | 13.15 | N/A | N/A | N/A | Adequate |

(*) If the bi-Axial Bending Ratio is greater than 1 that means the member has insufficient capacity

### Table A-9 Gantry No 9, 10, 11 & 12

| Section | Description | About x-axis | | | | About y-axis | | | | Axial | | Bi-Axial Bending | Capacity |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | ϕMsx (kNm) | Msx* (kNm) | ϕVx (kN) | Vx* (kN) | ϕMsy (kNm) | Msy* (kNm) | ϕVy (kN) | Vy* (kN) | ϕNu (kN) | N* (kN) | | |
| 273 Dia CHS | Main Beam,fsy=230 MPa | 92.55 | 57.55 | 393.36 | 22.47 | N/A | N/A | N/A | N/A | 294.84 | 16.44 | 0.86 | Adequate |
| 305x203x6.3 RHS | Column,fsy=230 MPa | 93.69 | 57.55 | 477.30 | 14.24 | 75.41 | 73.67 | 317.68 | 16.44 | 989.94 | 25.20 | N/A | Adequate |

(*) If the bi-Axial Bending Ratio is less than 1 that means the member has sufficient capacity

### Table A-10 Gantry No 13

| Section | Description | About x-axis | | | | About y-axis | | | | Axial | | Bi-Axial Bending | Capacity |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | ϕMsx (kNm) | Msx* (kNm) | ϕVx (kN) | Vx* (kN) | ϕMsy (kNm) | Msy* (kNm) | ϕVy (kN) | Vy* (kN) | ϕNu (kN) | N* (kN) | | |
| 89 Dia CHS | Main Beam,fsy=350 MPa | 10.86 | 8.74 | 146.25 | 17.50 | N/A | N/A | N/A | N/A | 382.85 | 67.75 | 1.23 | Failed |
| 254x152x6.3 RHS | Columnfsy=230 MPa | 87.10 | 40.05 | 518.46 | 22.31 | 60.93 | 7.54 | 237.87 | 6.85 | 1022.94 | 153.63 | N/A | Adequate |
| 203x102x6.3 RHS | Secondary Beam, fsy=230 MPa | 49.80 | 45.43 | 317.68 | 51.23 | 30.55 | 5.58 | 159.62 | 14.08 | N/A | N/A | N/A | Adequate |
| 102x102x6.3 RHS | Secondary Beam,fsy=230 MPa | 17.94 | 10.20 | 159.62 | 19.07 | N/A | N/A | N/A | N/A | N/A | N/A | N/A | Adequate |
| 150x12 Bracing Plate | Bracing Plate, fsy=230 MPa | 13.97 | 5.42 | 223.56 | 14.33 | N/A | N/A | N/A | N/A | N/A | N/A | N/A | Adequate |

(*) If the bi-Axial Bending Ratio is greater than 1 that means the member has insufficient capacity

# **Appendix B** – Connection capacity calculations

### Table B-1 Gantry No 1

| Connection | Shear Capacity | | Tension Capacity | | Combined Actions | Weld Capacity | | Capacity |
|---|---|---|---|---|---|---|---|---|
| | ϕVf (kN) | V* (kN) | ϕNtf (kN) | Nt* (kN) | | ϕvw (N/mm) | vw* (N/mm) | |
| Base Connection | 291.00 | 25.00 | 372.50 | 102.62 | 0.08 | 1651.2 | 922.16 | Adequate |
| Beam-Column Connection | 129.33 | 27.06 | 162.68 | 43.07 | 0.11 | N/A | N/A | Adequate |

### Table B-2 Gantry No 2

| Connection | Shear Capacity | | Tension Capacity | | Combined Actions | Weld Capacity | | Capacity |
|---|---|---|---|---|---|---|---|---|
| | ϕVf (kN) | V* (kN) | ϕNtf (kN) | Nt* (kN) | | ϕvw (N/mm) | vw* (N/mm) | |
| Base Connection | 291.00 | 24.28 | 372.50 | 122.54 | 0.12 | N/A | N/A | Adequate |
| Beam-Column Connection | 82.77 | 19.80 | 104.25 | 50.03 | 0.29 | N/A | N/A | Adequate |

### Table B-3 Gantry No 3

| Connection | Shear Capacity | | Tension Capacity | | Combined Actions | Weld Capacity | | Capacity |
|---|---|---|---|---|---|---|---|---|
| | ϕVf (kN) | V* (kN) | ϕNtf (kN) | Nt* (kN) | | ϕvw (N/mm) | vw* (N/mm) | |
| Base Connection (Inner Columns) | 291.00 | 28.62 | 372.50 | 59.58 | 0.04 | N/A | N/A | Adequate |
| Base Connection (Outer Columns) | 291.00 | 31.07 | 372.50 | 82.27 | 0.06 | N/A | N/A | Adequate |
| Beam-Column Connection | 186.24 | 41.38 | 234.39 | 57.26 | 0.11 | N/A | N/A | Adequate |

**Table B-4 Gantry No 4**

| Connection | Shear Capacity | | Tension Capacity | | Combined Actions | Weld Capacity | | Capacity |
|---|---|---|---|---|---|---|---|---|
| | ϕVf (kN) | V* (kN) | ϕNtf (kN) | Nt* (kN) | | ϕvw (N/mm) | vw* (N/mm) | |
| Base Connection | 291.00 | 8.56 | 372.50 | 38.40 | 0.01 | N/A | N/A | Adequate |
| Beam-Column Connection | 186.24 | 12.36 | 234.39 | 45.74 | 0.04 | N/A | N/A | Adequate |

**Table B-5 Gantry No 5**

| Connection | Shear Capacity | | Tension Capacity | | Combined Actions | Weld Capacity | | Capacity |
|---|---|---|---|---|---|---|---|---|
| | ϕVf (kN) | V* (kN) | ϕNtf (kN) | Nt* (kN) | | ϕvw (N/mm) | vw* (N/mm) | |
| Base Connection | 291.00 | 9.08 | 372.50 | 40.51 | 0.01 | N/A | N/A | Adequate |
| Beam-Column Connection | 186.24 | 13.15 | 234.39 | 48.53 | 0.05 | N/A | N/A | Adequate |

**Table B-6 Gantry No 6**

| Connection | Shear Capacity | | Tension Capacity | | Combined Actions | Weld Capacity | | Capacity |
|---|---|---|---|---|---|---|---|---|
| | ϕVf (kN) | V* (kN) | ϕNtf (kN) | Nt* (kN) | | ϕvw (N/mm) | vw* (N/mm) | |
| Base Connection | 291.00 | 8.80 | 372.50 | 43.96 | 0.01 | N/A | N/A | Adequate |
| Beam-Column Connection | 82.77 | 9.81 | 104.25 | 27.54 | 0.08 | N/A | N/A | Adequate |

### Table B- 7 Gantry No 7

| Connection | Shear Capacity | | Tension Capacity | | Combined Actions | Weld Capacity | | Capacity V* (kN) |
|---|---|---|---|---|---|---|---|---|
| | ϕVf (kN) | V* (kN) | ϕNtf (kN) | Nt* (kN) | | ϕvw (N/mm) | ϕVf (kN) | |
| Base Connection | 291.00 | 27.78 | 372.50 | 139.30 | 0.15 | N/A | N/A | Adequate |
| Beam-Column Connection | 82.77 | 14.62 | 104.25 | 48.83 | 0.25 | N/A | N/A | Adequate |

### Table B-8 Gantry No 8

| Connection | Shear Capacity | | Tension Capacity | | Combined Actions | Weld Capacity | | Capacity |
|---|---|---|---|---|---|---|---|---|
| | ϕVf (kN) | V* (kN) | ϕNtf (kN) | Nt* (kN) | | ϕvw (N/mm) | vw* (N/mm) | |
| Base Connection | 140.24 | 0.00 | 179.52 | 36.11 | 0.06 | N/A | N/A | Adequate |
| Beam-Column Connection | 186.24 | 0.00 | 234.39 | 47.48 | 0.05 | N/A | N/A | Adequate |

### Table B-9 Gantry No 9, 10, 11 & 12

| Connection | Shear Capacity | | Tension Capacity | | Combined Actions | Weld Capacity | | Capacity |
|---|---|---|---|---|---|---|---|---|
| | ϕVf (kN) | V* (kN) | ϕNtf (kN) | Nt* (kN) | | ϕvw (N/mm) | vw* (N/mm) | |
| Base Connection | N/A | N/A | N/A | N/A | N/A | 1238.40 | 794.54 | Adequate |
| Beam-Column Connection | 291.00 | 26.17 | 372.50 | 165.02 | 0.20 | N/A | N/A | Adequate |

### Table B-10 Gantry No 13

| Connection | Shear Capacity | | Tension Capacity | | Combined Actions | Weld Capacity | | Capacity |
|---|---|---|---|---|---|---|---|---|
| | ϕVf (kN) | V* (kN) | ϕNtf (kN) | Nt* (kN) | | ϕvw (N/mm) | vw* (N/mm) | |
| Base Connection | 140.24 | 22.37 | 179.52 | 39.26 | 0.07 | N/A | N/A | Adequate |
| Beam-Column Connection | 140.24 | 17.02 | 179.52 | 42.43 | 0.07 | N/A | N/A | Adequate |

### Table B-11 Summary Fatigue capacity

| Connection | Utilisation (%) | | | |
|---|---|---|---|---|
| | Bolts in Tension | Bolts in Shear | Base Plate | Weld |
| Type 1 | 73 (Adequate) | 18.9 (Adequate) | 70.6 (Adequate) | Adequate |
| Type 5 | 212 (FAILED) | 21.2 (Adequate) | 95.1 (Adequate) | Adequate |
| Type 8 | 100 (Acceptable) | 27.5 | 66.4 (Adequate) | Adequate |
| Type 14 | 61 (Adequate) | 16.6 | 68.9 (Adequate) | Adequate |
| Type 17 | N/A | N/A | N/A | Adequate |

# **Appendix C** – Failed gantry superstructure models

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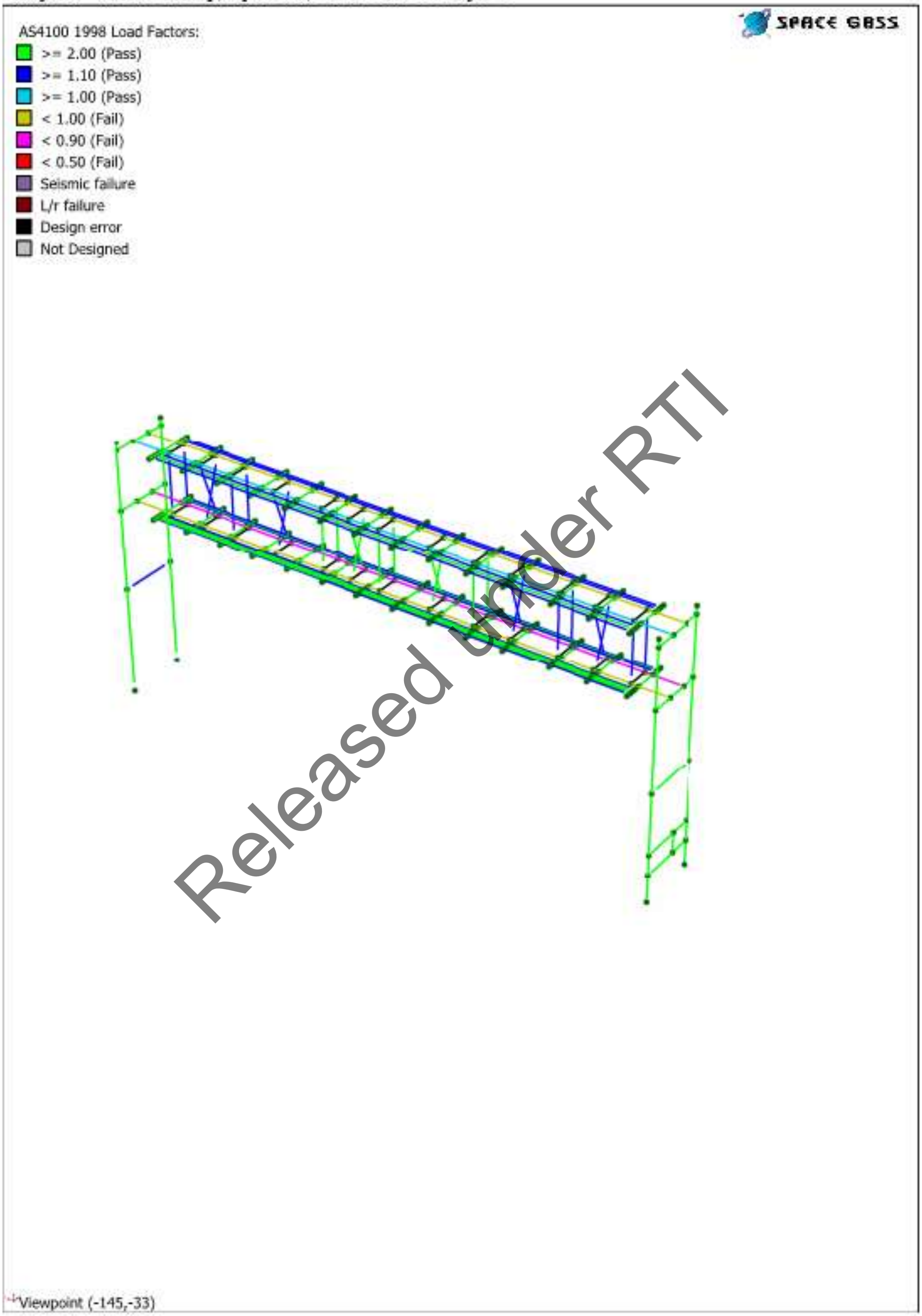

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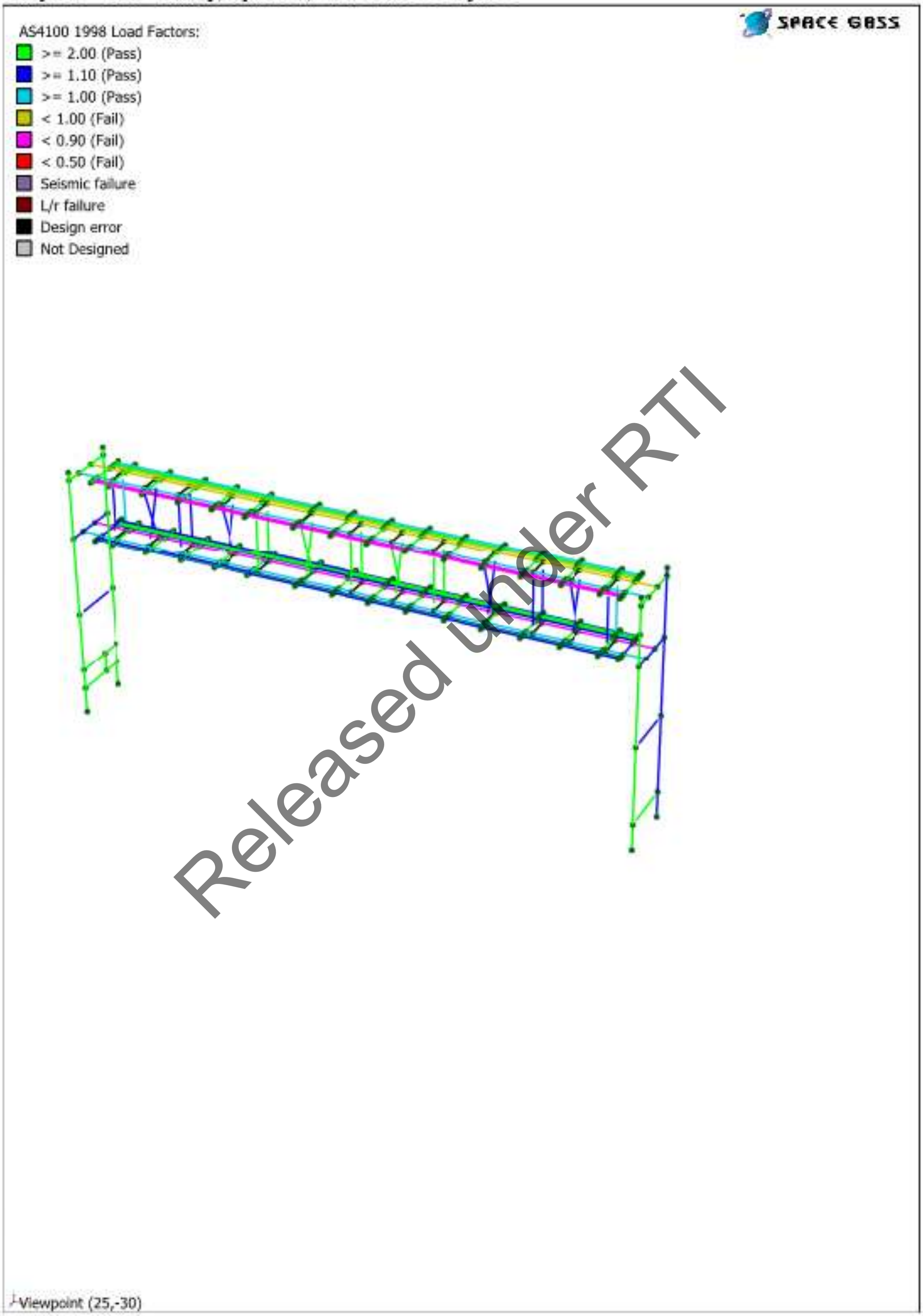
Designer: Date: Tuesday, April 28, 2020 11:09 AM Page: 1



# **Appendix D** – Year of Gantry construction

**Table D-12 Remaining Service Life by Gantry for Assessment**

| Gantry No | Structure type | Year | | Current age (years) | Total required service life (years) |
|---|---|---|---|---|---|
| | | Construction | Replacement / Modification | | |
| G1 | Superstructure | 1971 | 1987 | 33 | 80 |
| | Substructure | 1971 | 1971 | 49 | 80 |
| G2 | Superstructure | 1987 | No | 33 | 80 |
| | Substructure | 1987 | No | 33 | 80 |
| G3 | Superstructure | 1987 | No | 33 | 80 |
| | Substructure | 1987 | No | 33 | 80 |
| G4 | Superstructure | 1971 | No | 49 | 80 |
| | Substructure | 1971 | No | 49 | 80 |
| G5 | Superstructure | 1971 | No | 49 | 80 |
| | Substructure | 1971 | No | 49 | 80 |
| G6 | Superstructure | No information | No information | No information | 80 |
| | Substructure | No information | No information | No information | 80 |
| G7 | Superstructure | 1971 | No | 49 | 80 |
| | Substructure | 1971 | No | 49 | 80 |
| G8 | Superstructure | 1971 | 1990 | 30 | 80 |
| | Substructure | 1971 | 1990 | 30 | 80 |
| G9 to G12 | Superstructure | 1971 | 1990 | 30 | 80 |
| | Substructure | 1971 | 1990 | 30 (*) | 80 |
| G13 | Superstructure | 1971 | 1976 | 44 | 80 |
| | Substructure | 1971 | 1976 | 44 | 80 |

(*) Connection for gantries from No.9 to No.12 are not critical due to the superstructure connecting with the substructure through the concrete traffic barrier system.

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12527300-12936-37/https://projectsportal.ghd.com/sites/pp16_03/laneusemanagementand/ProjectDocs/12527300_STRL_REPT_Tasman Bridge Gantry Structural Capacity Assessment.docx

Document Status

| Revision | Author | Reviewer | | Approved for Issue | | |
|---|---|---|---|---|---|---|
| | | Name | Signature | Name | Signature | Date |
| A | s.36 [redacted] | | | | | |
| B | | | | | | |
| 0 | | | | | | |

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Record 7

# Tasman Bridge (B5512) Pathways Review

## Concrete Test Results and Durability of Beams

The Department of State Growth

1 July 2021

→ **The Power of Commitment**



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| **Printed date** | 30/06/2021 5:03:00 PM |
| **Last saved date** | 30 June 2021 |
| **File name** | https://projects.ghd.com/oc/tasmania3/nationalhwybridgeass/Delivery/Documents/3218213-REP-Tasman Bridge (B5512) Concrete Test Results.docx |
| **Author** | s 36 [redacted] |
| **Project manager** | s 36 [redacted] |
| **Client name** | The Department of State Growth |
| **Project name** | State Growth Bridge Assessments |
| **Document title** | Tasman Bridge (B5512) Pathways Review \| Concrete Test Results and Durability of Beams |
| **Revision version** | Rev 0 |
| **Project number** | 3218213 |



**Document status**

| Status Code | Revision | Author | Reviewer | | Approved for issue | | |
|---|---|---|---|---|---|---|---|
| | | | Name | Signature | Name | Signature | Date |
| Final | 0 | s 36 [redacted] | [redacted] | s 36 [redacted] | [redacted] | | 1/07/2021 |
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# Contents



## Table index



## Figure index



## Appendices

# 1. Introduction

## 1.1 Purpose of this report

This report provides analysis of the corrosion risk to the Tasman Bridge beams based on laboratory testing of concrete web cored samples.

## 1.2 Scope and limitations

This report: has been prepared by GHD for The Department of State Growth and may only be used and relied on by The Department of State Growth for the purpose agreed between GHD and The Department of State Growth as set out in section 1.1 of this report.

GHD otherwise disclaims responsibility to any person other than The Department of State Growth arising in connection with this report. GHD also excludes implied warranties and conditions, to the extent legally permissible.

The services undertaken by GHD in connection with preparing this report were limited to those specifically detailed in the report and are subject to the scope limitations set out in the report.

The opinions, conclusions and any recommendations in this report are based on conditions encountered and information reviewed at the date of preparation of the report. GHD has no responsibility or obligation to update this report to account for events or changes occurring subsequent to the date that the report was prepared.

The opinions, conclusions and any recommendations in this report are based on assumptions made by GHD described in this report. GHD disclaims liability arising from any of the assumptions being incorrect.

The opinions, conclusions and any recommendations in this report are based on information obtained from, and testing undertaken at or in connection with, specific sample points. Site conditions at other parts of the site may be different from the site conditions found at the specific sample points.

Investigations undertaken in respect of this report are constrained by the particular site conditions, such as the access difficulties. As a result, not all relevant site features and conditions may have been identified in this report.



## 1.3 Background

The Tasman Bridge is located across the River Derwent and was built in 1964 with the deck consisting mainly of six segmented post-tensioned I-beams with a composite slab on top. The River Derwent is considered a marine environment at this location.

GHD recently undertook an assessment of possibly widening and strengthening of the Tasman Bridge. As part of the study, GHD reviewed concrete sampling and analysis of chloride ingress and carbonation depth undertaken by State Growth in 2003 and made an assessment of durability of the bridge substructure, refer GHD Memorandum dated 10 June 2020 (attached in Appendix A).

A number of concrete core samples were taken and tested in May 2020. This report presents results of the testing and discusses the current corrosion risk to the bridge deck beams.

# 2. Investigation details

GHD obtained four 45 mm diameter concrete cores samples from the web of the deck beams slightly west of the mid-point of span 21, as shown on Figure 1, Figure 2 and Figure 3. Each core passed through the full web section. Cores were taken from the upper region of the beam webs to avoid embedded post-tensioned cables. Typical photos are shown for beam 1 in Table 1.

*Table 1 Typical investigation photos*

| Break out location | Core locations | Cores with phenolphthalein indicator |
|---|---|---|
|  |  |  |
| **Breakout** | **Breakout with phenolphthalein indicator** | **Breakout depth** |
|  |  |  |

Each end of each core was marked with "N" or "S" to designate the "North" and "South" faces of the beams.

The cores were analysed by SGS Australia for chloride and cement content and carbonation depth, with results presented in the Certificates of Test listed below:

– CoT 14973 Depth of Carbonation
– CoT 14980 Cement Content
– CoT 14979 Chloride Content

The Certificate of Tests are included in Appendix B.

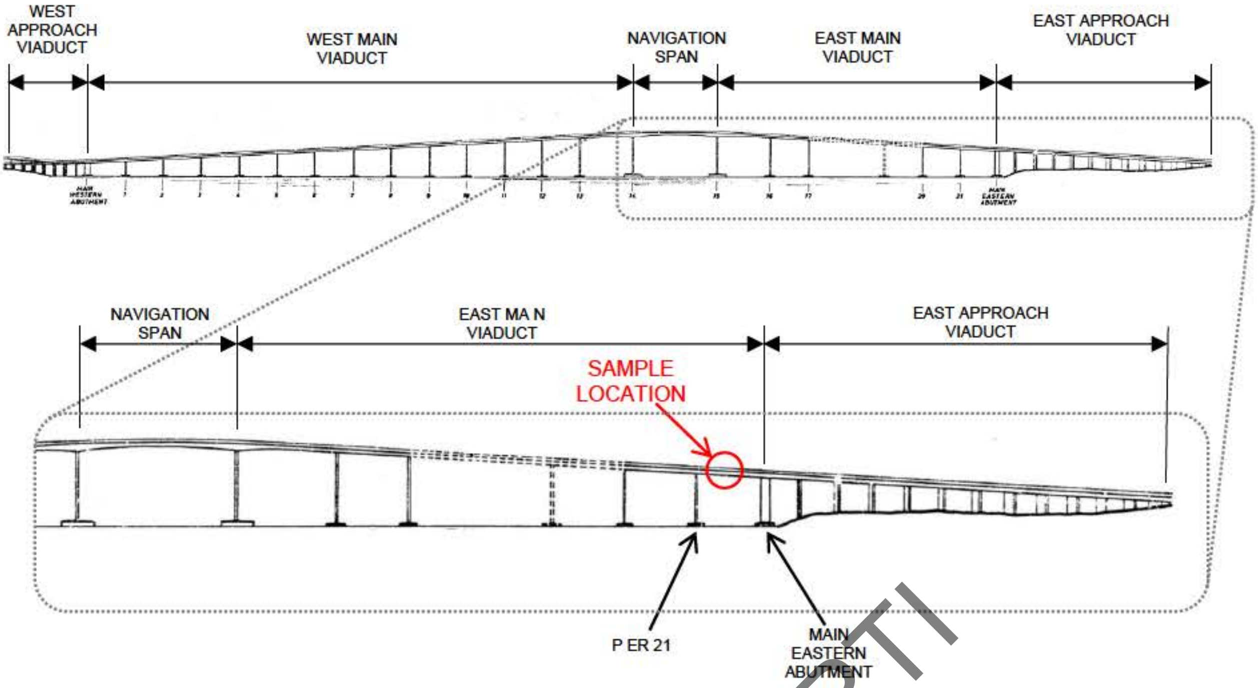


*Figure 1 Test Location*

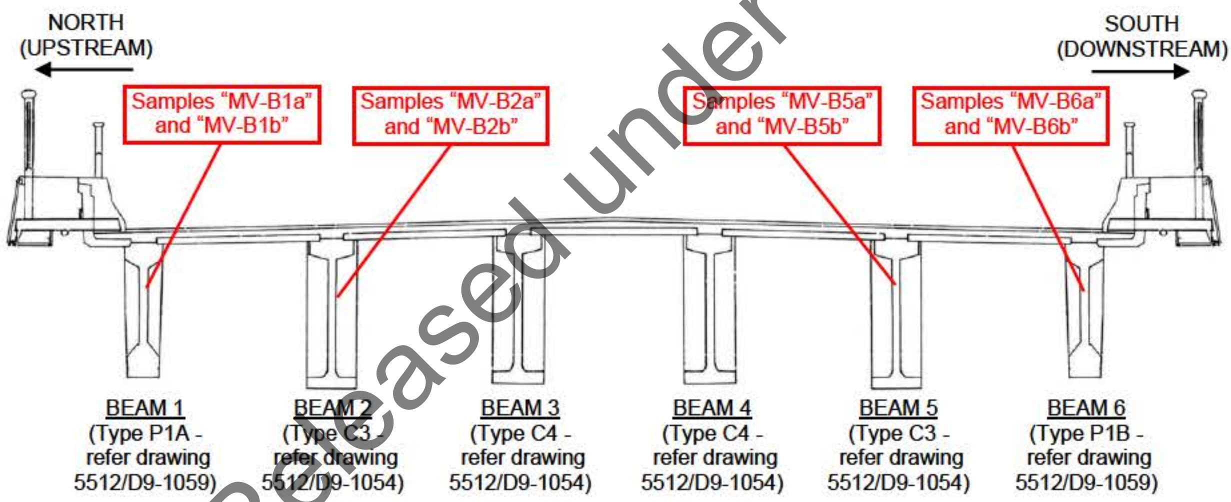


*Figure 2 Bridge cross-section, location of core samples and reference to original bridge drawings*

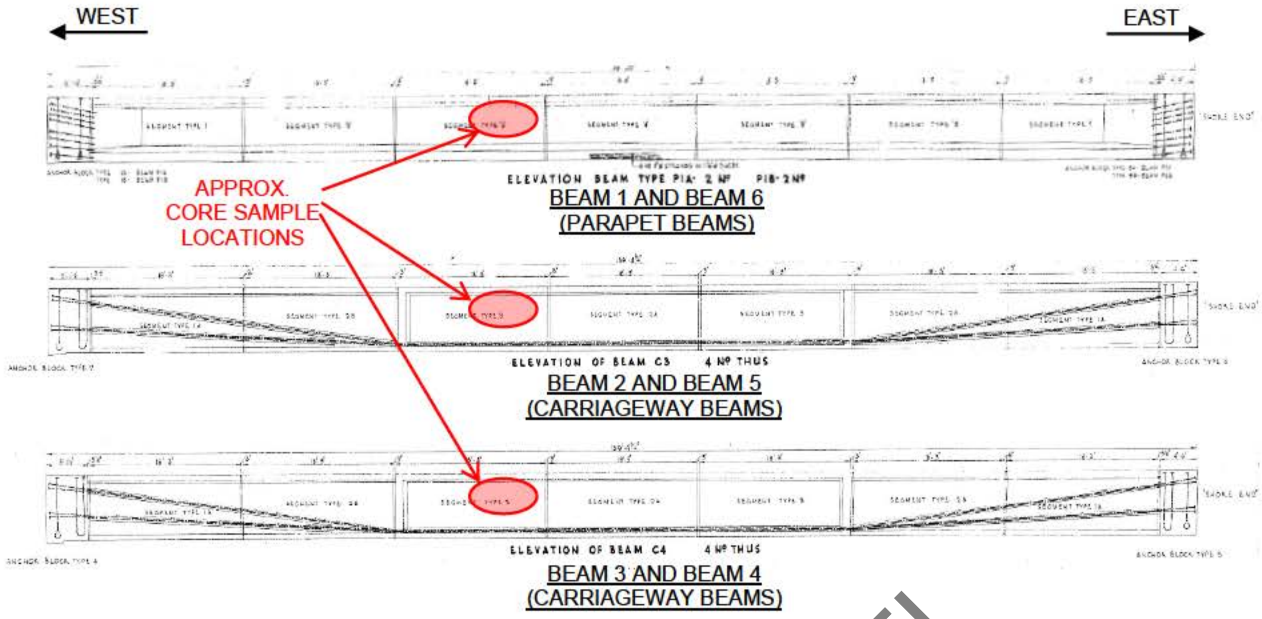


*Figure 3 Beam elevation and location of core samples (extracts from drawings 5512/D9-1059 and 5512/D9-1054)*

# 3. Results

## 3.1 General

The core photos (in the carbonation test report) indicate a dense, well-compacted concrete. No reinforcement was cut as part of the coring.

The two outer cores were ~150 mm in length and the two inner cores were ~125 mm in length. This verifies the original drawing (Drg No. 5512/AM9) that indicates the carriageway beams (beams 2-5) have a 5" thick web and the parapet beams (beams 1 & 6) have a 6" thick web.

The "As Constructed" drawings (Drg No. 5512/AM9) indicates a minimum 1" (25 mm) cover to steel for both the parapet and the carriageway beams. The "For Construction" drawings (Drg. No. 5512/D9-1051 and 5512/D9-1058) indicate the minimum cover to steel was 1¼" (32 mm) and $1^{3}/_{8}$" (35 mm) for the carriageway and parapet beams respectively. Measurements taken on site at the breakout locations suggest that the actual cover is consistent with those shown on the "For Construction" drawings.

The breakout depth was approximately 30 mm to 40 mm. The exposed reinforcement was sound with no evidence of corrosion or only light surface corrosion, although the more exposed beam faces (outside faces of the parapet beams) could not be accessed. The site carbonation depth (phenolphthalein) tests at the breakout areas and cores samples indicated negligible carbonation of the concrete. However, site test can be misleading if the surface is contaminated by coring slurry or break out dust.

## 3.2 Cement content

The average cement content for the four beams was 19% by weight of cement (wtcem.), with a low standard deviation of 0.39%/wtcem., which equates to 456 kg/m$^3$ assuming the concrete density was 2400 kg/m$^{3,}$ , and is typical for ~50 MPa concrete. The cement content and core length data is presented in Table 2.

*Table 2 Cement content and core length*

| Beam | Cement Content (%/wt sample) | Core length (Web Thickness) (mm) |
|---|---|---|
| 1 (northern parapet beam) | 18.6 | 150 |
| 2 (carriageway beam) | 19.6 | 125 |
| 5 (carriageway beam) | 19.0 | 122 |
| 6 (southern parapet beam) | 18.7 | 152 |
| **Average** | **19.0** | |
| **Standard Deviation** | **0.39** | |
| **Coefficient of Variance** | **2%** | |

## 3.3 Carbonation and chloride

The carbonation test results, and chloride test results for the 0-10 mm surface increments, are summarised in Table 3.

*Table 3 Carbonation and chloride results*

| Beam | Side | Carbonation depth measured in the laboratory (mm) | | Chloride (%/wt concrete) |
|---|---|---|---|---|
| | | Min | Max | 0-10 mm depth |
| 1 (northern parapet beam) | N | 4 | 20 | 0.02 |
| | S | 4 | 17 | 0.04 |
| 2 (carriageway beam) | N | 2 | 8 | 0.05 |
| | S | 3 | 5 | 0.04 |
| 5 (carriageway beam) | N | 4 | 8 | 0.04 |
| | S | 3 | 12 | 0.05 |
| 6 (southern parapet beam) | N | 6 | 17 | 0.04 |
| | S | 3 | 10 | 0.02 |
| | **Min** | **2** | **5** | **0.02** |
| | **Max** | **6** | **20** | **0.05** |



# 4. Discussion

## 4.1 Chlorides

The chloride content deeper than 10 mm from the surface was negligible in all cores, 0.01% by weight of concrete (wtconc.) or less.

The chloride in the 0-10 mm surface depth was in the range 0.02 to 0.05%/wtconc.

The 0.02%/wtconc. levels were measured on the two outermost beam faces, and the 0.04% or 0.05%/wtconc. on the remaining inner beam faces. This suggest that the outer surfaces that are subject to rain has a reduced chloride deposition compared to the sheltered faces.

The surface chloride levels ranged from 0.11% to 0.26 %/wtcem.

The 2003 surface chloride data (at a depth between 0 mm and 10 mm) for crossheads ranged from 0.13% to 0.66% %/wtcem., with a mean of 0.29%/wtcem. for elements between 10 m and 41 m high (excluding the value of 0.66%/wtcem. which is considered to be an outlier as it is not consistent with all of the other results)). This indicates that the recent data obtained is in line with the 2003 data, with surface chloride levels slightly lower, as might be expected as the beams are located higher than the crossheads.

It is not possible to calculate a meaningful chloride diffusion coefficient from the SGS data, as the inner chloride values are only reported to two decimal places and the chlorides are reported as either 0.01 % or <0.01%.

Overall, the chloride levels are very low and indicate chloride ingress via aerosols into uncracked concrete has been minimal over the bridge's 55-year service life.

## 4.2 Carbonation

Overall carbonation depths in laboratory tests ranged from 2 mm to 20 mm. This data is considered more accurate than the site tests.

The maximum carbonation was greater at the two outer beams than the two inner beams of the order, approximately twice the extent. This suggest different micro-environments, with the more sheltered inner beams being drier and so having a lower rate of carbonation. Typically, the carbonation rate is greatest at relative humidity's around 65% to 70%

The carbonation depths up to 20 mm are significantly higher than measured in 2003 for crossheads, which had a maximum 10 mm carbonation depth.

The carbonation coefficient at the maximum 20 mm carbonation depth is calculated as 2.7 mm/year$^{0.5}$. Based on this rate, at age of 100 years, the maximum carbonation depth is predicted to be 27.0 mm.

The observed condition of reinforcement (only light surface corrosion) at the break-out locations and the results of the carbonation tests suggest that carbonation is not a current risk in uncracked concrete but may become a risk at low concrete cover areas within the original 100-year service life to 2064. Once corrosion becomes initiated, the rate is depended on the concrete moisture content and is typically greatest in concrete subject to regular direct wetting, such as the outer faces and soffit of the outer beams. Again, this suggests greatest corrosion risk overall to the two outer beams, and lower risk to the inner beams

It may be prudent to apply a protective coating to minimise the long-term risk of carbonation-induced corrosion to the outer beams.

## 4.3 Summary

The recent sampling and testing of concrete cores complements the prior 2003 core sampling program.

The cores verify the design drawing dimensions and indicate the beam web concrete is dense, sound and likely to be of the order 50 MPa in strength.

The current corrosion risk for uncracked concrete beams of this type is low in terms of carbonation or chloride-induced corrosion.

Chloride levels were very low and in line with the 2003 DSG data.

Carbonation depths in the test cores was greater than measured in 2003, and suggests that in the longer term carbonation may become a higher corrosion risk than ingress of chlorides, in particular at areas of low cover to the outer beams. Such a risk could be mitigated through application of a protective coating system.

# Appendices

# Appendix A

## Memorandum covering the durability assessment of the beams

# Memorandum

10 June 2020

| | | | |
|---|---|---|---|
| To | [redacted] | | |
| Copy to | [redacted] | | |
| From | [redacted] | Tel | +61 3 86878125 |
| Subject | Durability Assessment of Beams | Job no. | 6137698 |

Dear Liam

## 1 Purpose

This report provides a preliminary analysis of the corrosion risk to the Tasman Bridge piers and beams, in particular the external post-tensioning strands and internal pre-tensioning strand.

## 2 Limitations

The memorandum report (report) has been prepared by GHD for the Department of State Growth and may only be used and relied on by the Department of State Growth for the purpose agreed between GHD and the  as set out in section 1 of this report.

GHD otherwise disclaims responsibility to any person other than the Department of State Growth arising in connection with this report. GHD also excludes implied warranties and conditions, to the extent legally permissible.

The opinions, conclusions and any recommendations in this report are based on assumptions made by GHD described in this report (refer section 3 of this report). GHD disclaims liability arising from any of the assumptions being incorrect.

GHD has prepared this report on the basis of information provided by the Department of State Growth, which GHD has not independently verified or checked beyond the agreed scope of work. GHD does not accept liability in connection with such unverified information, including errors and omissions in the report which were caused by errors or omissions in that information.

## 3 Existing Information

GHD has reviewed the following information:

- Original bridge design drawings (key features attached at end of memo)
- DSG Chloride Data summarised in a spreadsheet.

Information not seen:

- Reports and test certificates related to the 2003 DSG investigations.





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# 4 Background

The Tasman Bridge is located across the River Derwent and was built in 1964 with the deck consisting mainly of six I beams with a composite slab on top. The River Derwent can be considered a marine environment at this location.

GHD is currently undertaking an assessment of possibly widening and strengthening of the Tasman Bridge. GHD's analysis has used current traffic loadings and with or without deck widening. The analysis has found under certain scenarios that tensions may develop across the segmental joints. As there is no capacity to carry tension, this will mean the joints will open.

Concrete sampling and analysis of chloride ingress and carbonation depth was undertaken in 2003 that led to application of protective coatings to the lower piers.

The Department of State Growth has asked GHD to conduct a durability assessment on the bridge in particular the risk that opening of cracks could lead to corrosion of the beam prestressing strands.

## 4.1 Bridge features

The I beams were constructed using precast segments stressed together using post tensioning. The 1' diameter (25.4 mm) tendons were placed in 1.25" (32mm) diameter ducts, with the ducts grouted after stressing. However, the ducts will not have crossed the segment joints, which were nominated as 3" (76 mm) wide.

The ducts in the outer parapet beams are entirely encased within the I beams, with minimum concrete cover to the I beam reinforcement of 1" (25.4 mm).

The carriageway beams' strands are located externally terminating in end anchor blocks, restrained by L bars at holes in the 5" (127 mm) thick webs, and encased in insitu "fine" concrete.

The Navigation spans between Piers 13 and 16 have more complex beams that incorporate 9" thick concrete web stiffeners, cantilevered and suspended central beams with half joints and more complex post tensioning.

The crosshead beams are typically solid precast beams, with segmented precast units pre-stressed together laterally and longitudinally with 1.25" macalloy bars for Piers 13 to 16, the 3" gaps between units being filled with "Class A" concrete.

The Piers are hollow precast concrete units stressed together vertically with 1.25" macalloy bars located in 2.25" ducts.

The gap filling concrete is assumed placed and attain full strength before stressing, and the strand encasement concrete is assumed placed after stressing the tendons.





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## 5 DSG 2003 Data

### 5.1 Test locations

DSG obtained samples for nine of the bridge piers from the west bank to the central (Navigation) span at the following locations:

- Base of Piers
- Top of Piers (1 m below top)
- Crosshead beam (precise location not specified)

### 5.2 Chloride Data

The raw DSG 2003 chloride data is shown in Table 3 noting the data is presented a %chloride by weight of cement, indicating that the cement content was determined at the time. The chloride would have been determined by the test laboratory as weight of concrete sample. GHD has not been provided with the actual cement content.

Concrete samples were obtained at five depth increments.

Not all samples at the top of piers and in cross head beams were analysed at the time, at many locations only the two surface samples were tested. It is not known if the untested samples have been retained, although this is unlikely.

Concrete strength of cross head and columns is understood 24.5 MPa, suggesting a cement content was 280 Kg/m$^3$. This assumed cement content value used to adjust the chloride contents from by %/wt cement to by %/wt concrete. Preferably the original test data would be used in the analysis in its original %/wt concrete format.



### 5.3 Carbonation Data

The 2003 test data indicates very low carbonation to cross heads and columns, the great majority being <5 mm, and a maximum 10 mm. Provided that the reinforcement cover exceeds the minimum specified, it is expected the risk of carbonation induced corrosion is small.

No detailed analysis has been undertaken of this data.

Any additional carbonation test data and actual concrete cover data should be assessed to verify the rate of carbonation and risk of carbonation induced corrosion.

## 6 Variation of Chloride with Height

### 6.1 General

The outer 0-10 mm depth increment is an approximation of the actual surface chloride level that would be estimated in diffusion modelling. The actual values provide an average chloride content over



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the depth sampled. In some cases, the 0-10 mm depth increment can under or overestimate the actual surface chloride level depending on the exposure environment and time of wetness.

However, it will provide an indication of approximate level of saline aerosol deposition.

The 0-10 mm chloride levels have been plotted with height for the crossheads and top of columns in Figure 1 and Figure 2.

Note that preferably DSG should provide access to the raw data, test certificates, or confirmation of the cement content, this must have been measured by the test lab in 2003.

**Assumptions**

The analysis makes the following assumptions:

1. Plot of the raw data, presented as %/wt cement.
2. Data is assumed to approximate to the surface chloride level.
3. Cross head height is to base of the cross head/top of pier, calculated from design drawings.
4. Top of column samples taken 1 m below cross head/top of columns

**Data trends**

Crosshead: Mean is around 0.3%/wt cement, up to 35 m height, then above this level a wide range 0.13 % to 0.66% %/wt cement. This suggests a range of micro-climates exist at the beam level.

Top of Column: Surface chloride tends to decrease with height, with some low/high outliers, which is the expected trend for saline aerosol concentration variation. The maximum value is 0.75%/wt cement.

The maximum measured surface chloride value 0.75%/wt cement equates to 0.07 %/wt concrete.





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**Figure 1 Plot of surface chloride variation with height for crossheads**

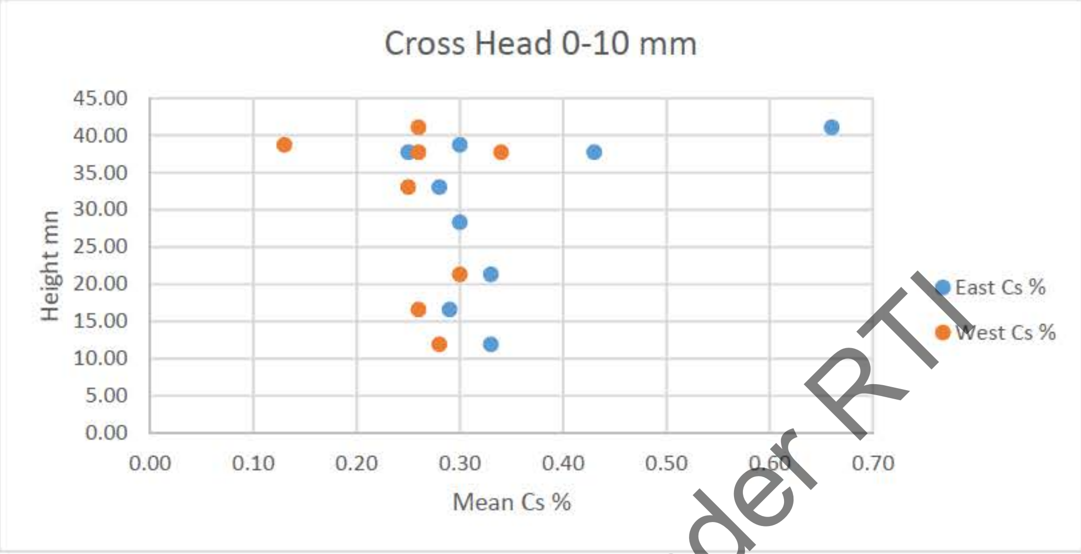
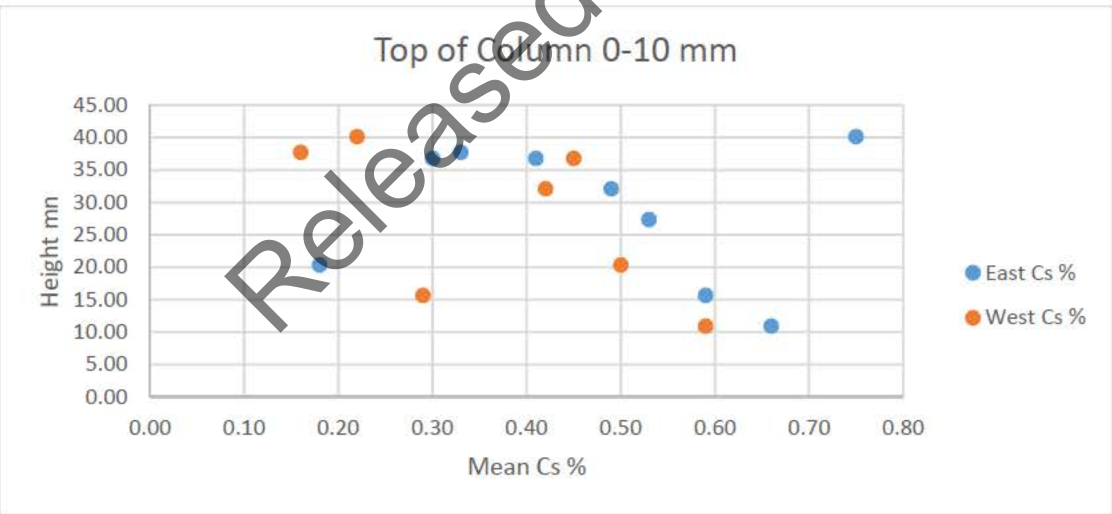



**Figure 2 Plot of surface chloride variation with height for top of piers**




## 7 Chloride Diffusion Analysis for Cross Heads

The chloride diffusion coefficient was determined for the Cross Heads 16 & 17 that have complete chloride test data, both east and west faces, using GHD's in-house curve fitting tool. The tool fits the data to Fick's Second Law of Diffusion equation using a least squares curve fitting algorithm.

The raw data was adjusted to % by wt concrete as required by the model, based on the assumed cement content, and the structure was assumed to have 39 years of exposure.





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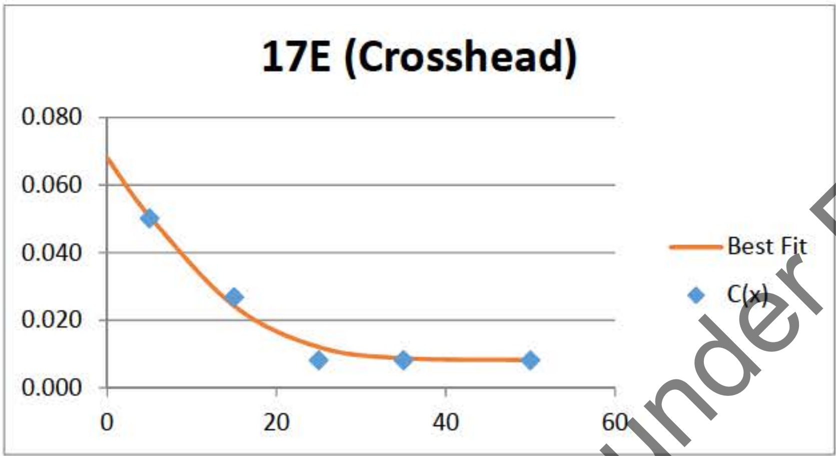
A typical plot is shown in Figure 3, confirming that although surface chloride levels are low, the ingress mechanism is consistent with a diffusion mechanism, and that chloride levels in the original concrete mix were very low around 0.01% by weight concrete. It also shows that the calculated surface chloride level is always higher than the sample increment value.

**Figure 3 Typical curve fit of chloride data**




The chloride diffusion analysis data for all four sample locations is shown in Table 1.

Both the calculated mean chloride diffusion coefficient and surface chloride levels for the main span cross head samples analysed are very low:

- Mean Dc 0.071 x $10^{-12}$ $m^2/s$
- Mean Cs 0.046 wt. %/concrete (0.395 %/wt cement)

It is noted that other pier locations had higher surface chloride levels, but only the first two depth increments were analysed, so this data cannot be used to curve-fit estimate the chloride diffusion coefficient to gain a larger, more statistically significant sample size.

**Table 1 Chloride diffusion coefficient analysis for Piers 16 and 17**

| Location<br>Depth range (mm) | | | 17E<br>Crosshead | 17W<br>Crosshead | 16E<br>Crosshead | 16W<br>Crosshead | Average | stdev | CoV |
|---|---|---|---|---|---|---|---|---|---|
| 0 | to | 10 | 0.050 | 0.030 | 0.035 | 0.015 | | | |
| 10 | to | 20 | 0.027 | 0.008 | 0.008 | 0.008 | | | |
| 20 | to | 30 | 0.008 | 0.004 | 0.004 | 0.004 | | | |
| 30 | to | 40 | 0.008 | 0.004 | 0.004 | 0.004 | | | |
| 40 | to | 60 | 0.008 | 0.004 | 0.004 | 0.004 | | | |
| Cs | | wt. %/concrete | 0.07 | 0.04 | 0.05 | 0.02 | 0.05 | 0.02 | 45% |
| De | | x $10^{-12}$ $m^2/s$ | 0.07 | 0.05 | 0.04 | 0.11 | 0.07 | 0.03 | 45% |



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# 8 Chloride Corrosion Risk – Uncracked Concrete

The general risk of corrosion initiation in uncracked concrete has been assessed using GHD's in-house probabilistic chloride model, as originally developed for McGees Bridge in 2000. The model takes in to account expected variability of the input parameters and represents the general risk that corrosion might initiate and propagate.

At present there is no site test data for either the main beam precast concrete or the insitu encasement concrete. The precast beams are understood higher strength than the cross heads and are expected to have a lower chloride diffusion coefficient. No analysis has been completed for the precast beams.

## 8.1 Assumptions

The chloride threshold for corrosion activation was taken as 50% of mild steel value of 0.4%/wt cement, i.e. 0.2%/wt cement, adjusted for expected cement content.

### *8.1.1 Crossheads*

The measured diffusion coefficient for the crosshead concrete in Section 7 was adjusted back to an estimated 28-day value using an in-house algorithm to use in the model, which then uses a 100 year service life, noting the current bridge age is 55 years, hence 45 years remaining.

The assumed model inputs and variability for the crosshead beam, based on the 2003 data and experience, are listed in Table 2.

The model assumes 25 mm mean cover (range 13 mm to 37 mm) as corrosion problems are most likely at construction defects such as low cover, and noting the design cover for the piers and cross head concrete was 3" (75 mm).

The model is also expected to provide insight to the possible performance of beams, which have a design cover of 25 mm but higher quality concrete than the cross heads.

### *8.1.2 Insitu Concrete*

A preliminary estimate is made for the insitu concrete, based on the crosshead analysis, with amended input values to those in Table 2 as follows:

- Upper level surface chloride level (0.09 %/wt concrete, is x 2 the cross head mean value)
- Chloride diffusion coefficient x 3 to take into account insitu nature of the concrete ($0.216 \times 10^{-12}$ $m^2/s$)

It is emphasised that these assumptions might not be correct and must be verified through site testing.





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**Table 2 Preliminary probabilistic Chloride Model Inputs – Cross Head**

| Item | | Value | CoV | Unit |
|---|---|---|---|---|
| Chloride diffusion coefficient | Dref | 7.20E-14 | 45% | m2/s |
| Background chloride concentration | **$C_o$** | 0.01 | | wt. % concrete |
| Surface chloride concentration | **$C_s$** | 0.05 | 45% | wt. % concrete |
| Corrosion activation threshold | **$C_{act}$** | 0.023333 | 15% | wt. % concrete |
| Relative concrete age in years | | 0 | | years |
| Relative concrete age in days | | 28 | | days |
| | **$t_{ref}$** | 0.08 | | years |
| Maximum projected age | **$t_{max}$** | 100 | | years |
| Cover depth | **$x_{cover}$** | 25 | 24% | mm |
| Bar diameter | **dia** | 13 | | mm |
| Binder blend rate | **Blending** | 0 | | wt. % total binder content |
| Binder content | **Content** | 280 | | kg/m$^3$ |
| Inhibitor | **Inhibitor** | 0 | | lts/m$^3$ |
| Concrete age factor | **m** | 0.2 | | |
| Target Compressive Strength (equivant cylinder) | **Fc** | 25 | | MPa |
| Corrosion to cracking | **Xc** | 29 | 10% | |

## 8.2 Corrosion Prediction Model Results

The model results shown in Figure 4 suggests that the cross head beams are at low risk of corrosion initiation or propagation over the remaining 45 year service life, in particular as the design cover is significantly higher than modelled.

The model results shown in Figure 5 suggests corrosion might initiate in the encased strands at very low cover areas, whereas corrosion should not initiate at mean or greater cover over a 100 year life.





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**Figure 4 Precast Cross Head Corrosion Prediction Model**

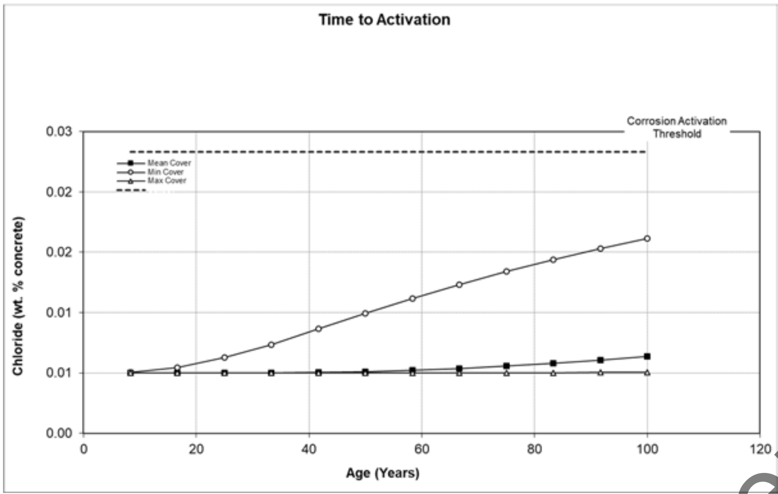
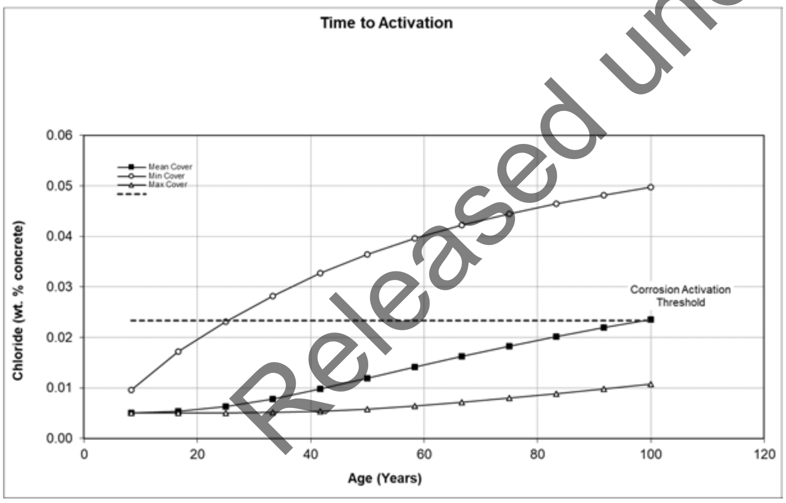



**Figure 5 Encasement Concrete Corrosion Prediction Model**






# 9 Opinion

## 9.1 Corrosion risk in uncracked concrete

The 2003 data indicates that the saline aerosols have resulted in low salt contamination of the concrete. Modelling of the data suggests that the risk of corrosion initiation and propagation has been and will remain low in the uncracked cross head concrete and most likely the infill/encasement concrete other than at very low reinforcement cover, or if the actual chloride ingress resistance of the insitu concrete is significantly worse than the precast concrete.



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## 9.2 Corrosion risk at cracks

The corrosion risk at cracks is a complex issue.

All available chloride models assume diffusion through uncracked concrete. A crack can provide a direct path for chlorides or $CO_2$ gas to diffuse along, hence more rapid ingress. However, the crack might become filled with corrosion product that stifles corrosion rates. Guidance in publications on the subject can be contradictory.

GHD has analysed chloride migration away from the walls of a crack in a seawater splashed deck slab, and demonstrated significant migration laterally from the crack surfaces is possible. In addition, it has seen evidence of salt concentration effects for slightly saline water leaking through cracked tunnel liners, and at the top of roof support columns in raw water storage tanks with only 300-400 ppm chloride (within drinking water quality guidelines), that in both cases led to chloride induced corrosion.

In the case of possible cracks in the elevated bridge deck beams, the concrete is exposed to saline aerosols that, whilst low in concentration, have a demonstrated ability to migrate under a diffusion mechanism into the concrete, as shown in this data assessment. The process is expected controlled by time of wetness on the concrete surface. Photos of the beam flange soffits shows white water marks indicating moisture forms on the beams from time to time through rain or condensation. Also noting that salts are hygyroscopic and so can attract moisture. In a wetting/drying environment, chlorides may be drawn in to the concrete though an absorption mechanism. This would also occur at cracks, even hairline cracks, which often absorb and hold moisture, and would be a higher risk if cracks are opening dynamically.

It is also noted that stressed steel has a much lower corrosion initiation level than mild steel, typically it is assumed 50% that of mild steel, so is a higher risk of corrosion initiation at low chloride levels.

## 9.3 Key Issues

The quality of the stressed strand's insitu placed concrete encasement is not known and is critical to the bridge durability and hence integrity. It is anticipated that the quality of this insitu concrete is lower than the precast beam, cross head or pier segments.

Risks for the insitu placed concrete include:

- Carbonation leading to general corrosion of strands
- Chloride ingress generally through the insitu concrete that is likely to have greater permeability than the precast concrete, leading to general pitting corrosion of strands
- Chloride ingress at shrinkage cracks, either between the encasement and beams, or at flexural cracks at the joints, leading to accelerated pitting corrosion of strands and potential catastrophic failure.

Overall it is recommended that the post tensioned beams should not be allowed to go into tension that would allow cracks to open in the slightly saline atmospheric environment.





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The actual chloride ingress profile, carbonation depth, cover and cement content data for the insitu gap filling and strand encasement concrete should be determined through very careful drill dust sampling and site tests. If practical, inspection of strands for any evidence of corrosion at the joint interfaces would be of interest.

Obtaining similar data on the precast beams would also enable verification of their current and future corrosion risks.

Regards

s 36 [redacted]





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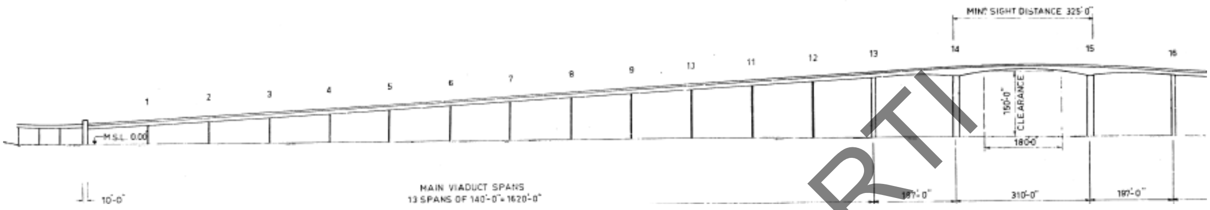
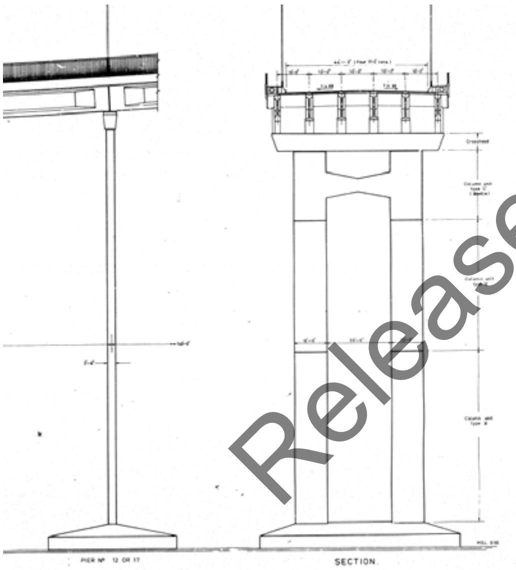
# Memorandum

## 10 Drawings

### Figure 6 Bridge elevation with Pier numbers




### Figure 7 Typical Pier Elevations




### Figure 8 Typical deck section

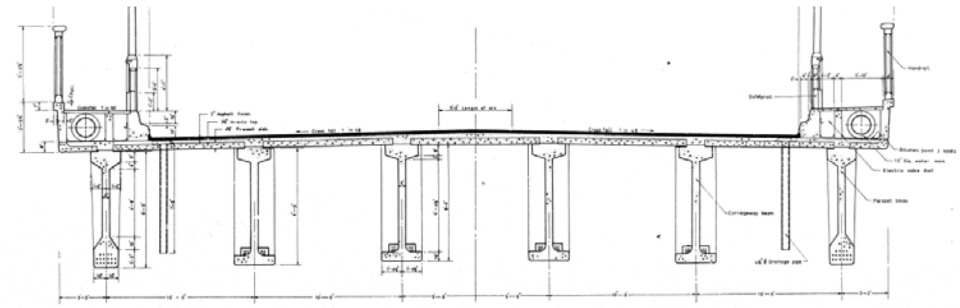






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**Figure 9 Post tensioned beam**

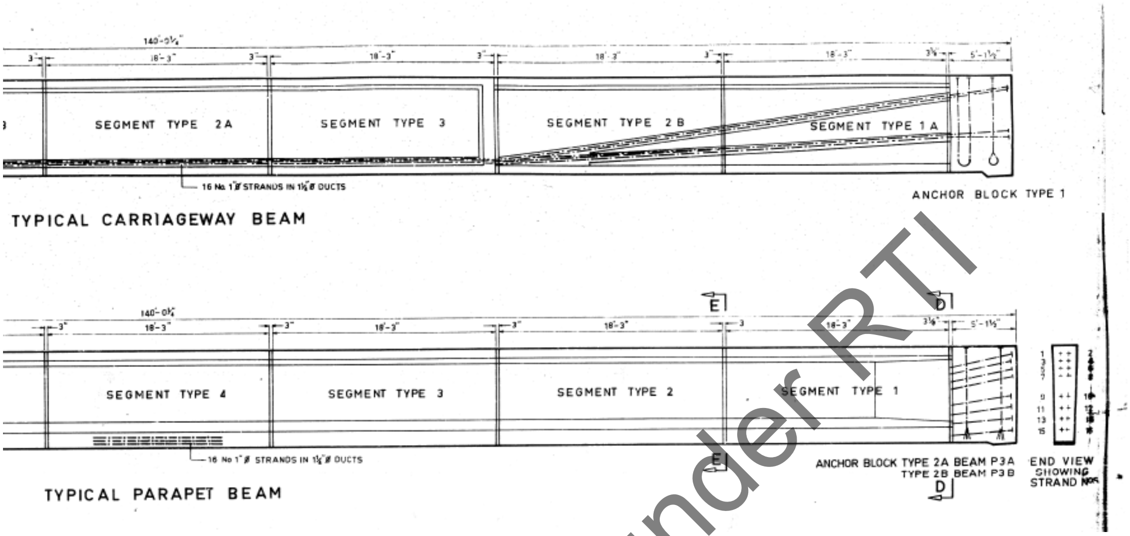


**Figure 10 Post tensioned beam sections**

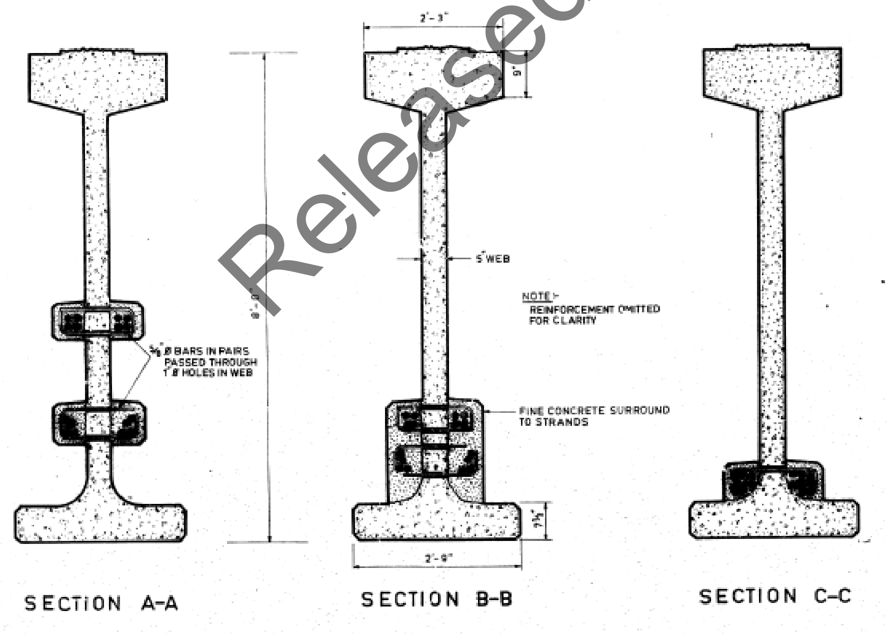






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**Figure 11 Pre-tensioned beam**

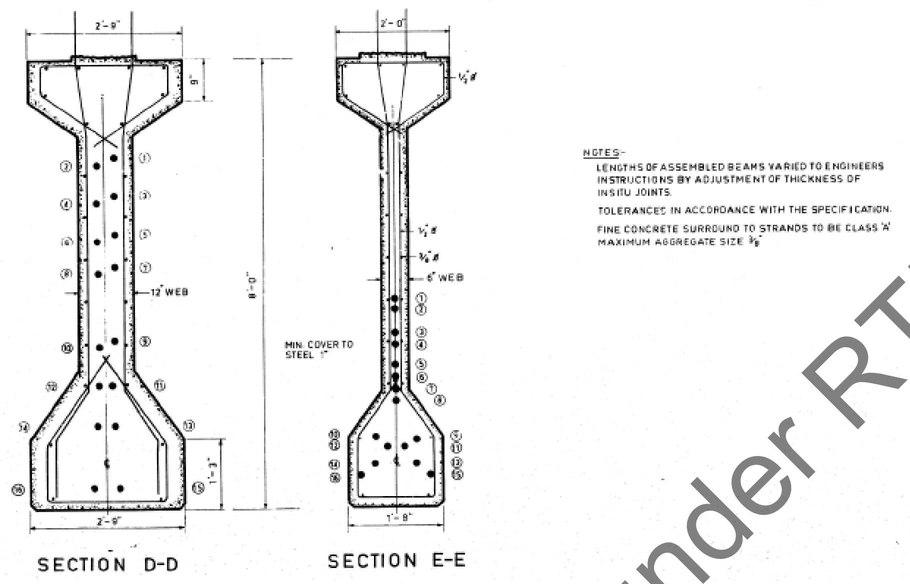


**Figure 12 Navigation Span - Elevation of post tensioned beams**

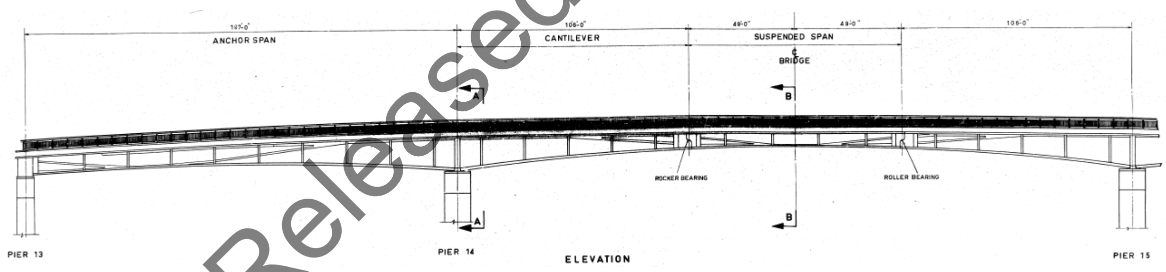






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**Figure 13 Navigation Span - Post tensioned beam details**

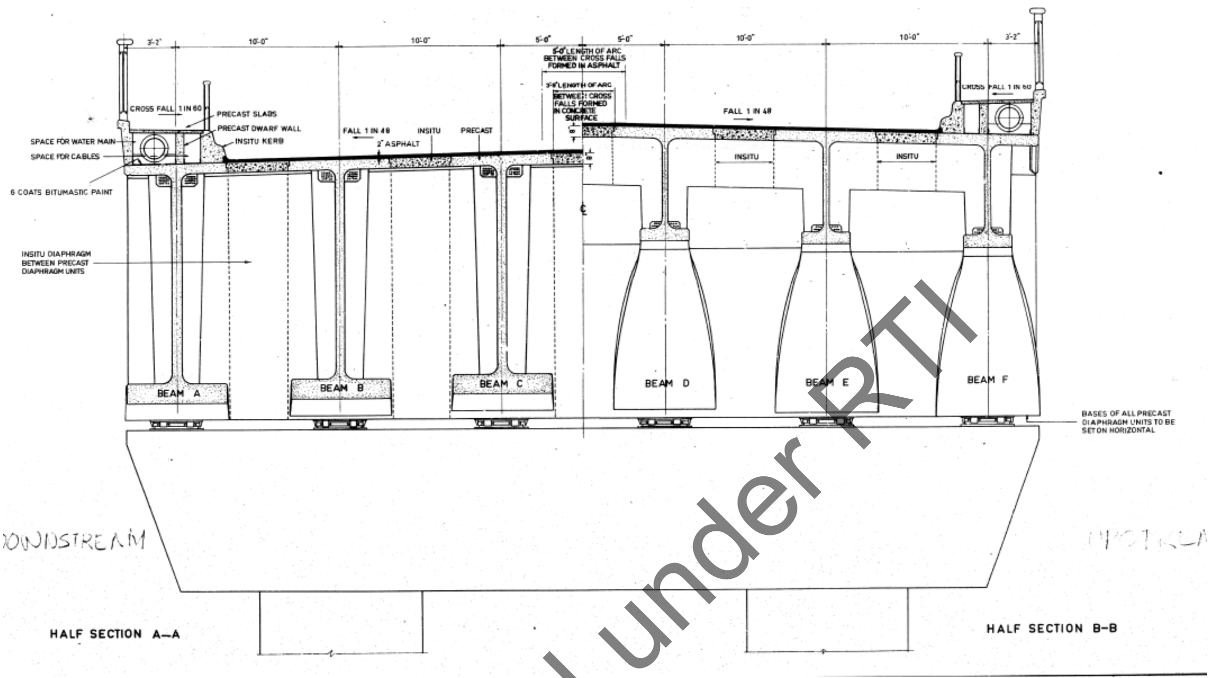


**Figure 14 Cross head beam details**

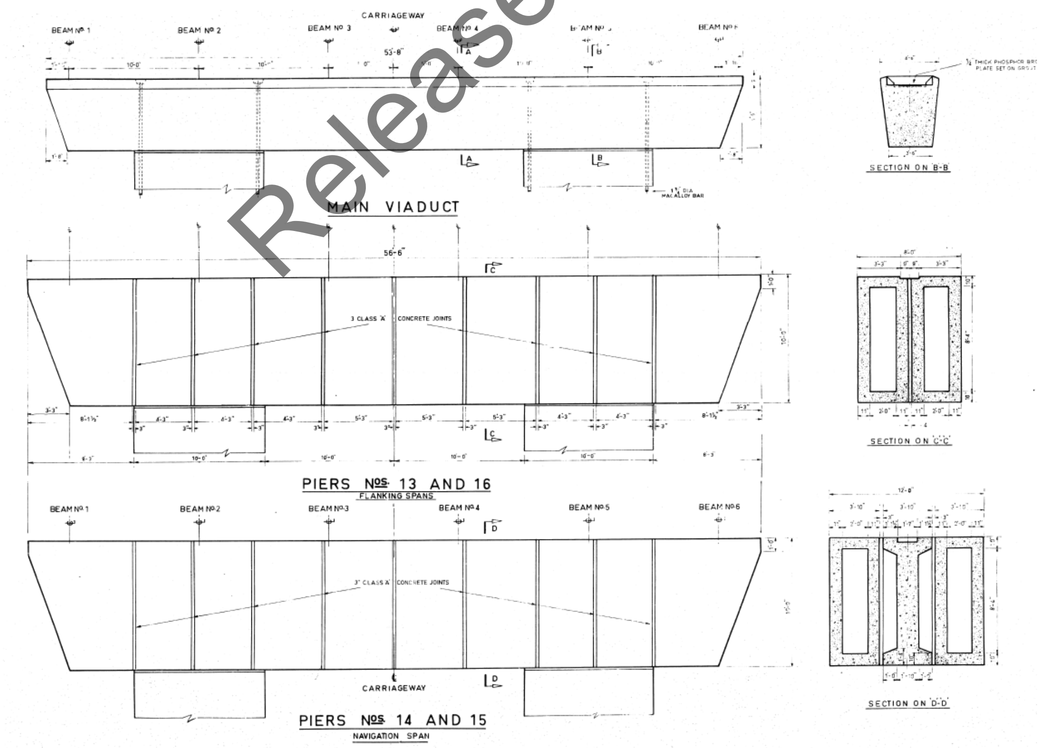






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**Table 3 DSG 2003 Chloride Data (% by weight of cement)**

| Pier Number | | | 17 | | 16 | | 14 | | 12 | | 10 | | 8 | | 5 | | 3 | | 1 | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Crosshead | Face | | East | West | East | West | East | West | East | West | East | West | East | West | East | West | East | West | West A# | West B# |
| | Depth, mm | 0 - 10 | 0.43 | 0.26 | 0 30 | 0.13 | 0.66 | 0.26 | 0.25 | 0.34 | 0.28 | 0.25 | 0.30 | | 0.33 | 0.30 | 0.29 | 0.26 | 0.33 | 0.28 |
| | | 10 - 20 | 0.23 | 0.07 | 0 07 | 0.07 | 0.11 | 0.07 | 0.07 | 0.07 | 0.07 | 0.07 | 0.08 | | 0.09 | 0.03 | 0.05 | 0.07 | 0.07 | 0.11 |
| | | 20 - 30 | 0.07 | 0.03 | 0 03 | 0.03 | | | | | | | | | | | | | | |
| | | 30 - 40 | 0.07 | 0.03 | 0 03 | 0.03 | | | | | | | | | | | | | | |
| | | 40 - 60 | 0.07 | 0.03 | 0 03 | 0.03 | | | | | | | | | | | | | | |
| Top of Column | Face | | East | West | East | West | East | West | East | West | East | West | East | West | East | West | East | West | East | West |
| | Depth, mm | 0 - 10 | 0.30 | | 0 33 | 0.16 | 0.75 | 0.22 | 0.41 | 0.45 | 0.49 | 0.42 | 0.53 | | 0.18 | 0.50 | 0.59 | 0.29 | 0.66 | 0.59 |
| | | 10 - 20 | 0.16 | | 0 07 | 0.07 | 0.08 | 0.05 | 0.13 | 0.03 | 0.08 | 0.07 | 0.04 | | 0.11 | 0.14 | 0.11 | 0.03 | 0.07 | 0.09 |
| | | 20 - 30 | 0.07 | | 0 03 | 0.03 | | | | | | | | | | | | | | |
| | | 30 - 40 | 0.03 | | 0 03 | 0.03 | | | | | | | | | | | | | | |
| | | 40 - 60 | 0.03 | | 0 03 | 0.03 | | | | | | | | | | | | | | |
| Base of Column | Face | | East* | West* | East | West | South | West | South | West | South | West | East | West | South | West | South | West | South | West |
| | Depth, mm | 0 - 10 | 0.10 | 0.07 | 3.71 | 0.23 | 0.16 | 0.21 | 1.89 | 0.50 | 1.97 | 0.29 | 3.75 | | 0.95 | 0.17 | 1.10 | 0.16 | 0.63 | 0.30 |
| | | 10 - 20 | 0.20 | 0.13 | 2 53 | 0.10 | 0.01 | 0.05 | 0.81 | 0.13 | 0.91 | 0.09 | 1.33 | | 0.37 | 0.07 | 0.54 | 0.03 | 0.25 | 0.22 |
| | | 20 - 30 | 0.07 | 0.07 | 0.76 | 0.03 | 0.01 | 0.03 | 0.39 | 0.11 | 0.26 | 0.03 | 0.84 | | 0.09 | 0.03 | 0.26 | 0.03 | 0.16 | 0.13 |
| | | 30 - 40 | 0.03 | 0.03 | 0 33 | 0.03 | 0.01 | 0.01 | 0.12 | 0.07 | 0.13 | 0.01 | 0.51 | | 0.03 | 0.01 | 0.12 | 0.01 | 0.08 | 0.08 |
| | | 40 - 60 | 0.03 | 0.03 | 0 07 | 0.03 | 0.01 | 0.01 | 0.05 | 0.01 | 0.09 | 0.01 | 0.22 | | 0.05 | 0.01 | 0.03 | 0.01 | 0.03 | 0.03 |





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# Appendix B

## Laboratory test certificates

Client: GHD Pty Ltd
2 Salamanca Square
Hobart TAS 7000

**Your Reference:** State Growth Bridge Assessments, Job No. 3218213

**Our Reference:** JN 20-10-159

# Certificate of Test No. 14980

**Sample:** Concrete Core Samples

**Date Received:** 5th June 2020

**Date Tested:** 2nd – 6th July 2020

**From:** Main Viaduct (Span 21 – East Abutment)

**Description & Condition:** 4 -off nominal 45 mm diameter concrete core samples

**Test Description:** **Cement Content by Calcium Oxide**

**Sample Preparation:**

Sample crushed and pulverised to pass 125 µm sieve.

**Test Method:**

Cement content by calcium oxide determination in accordance with BS 1881: Part 124: 2015 "Methods for Analysis of Hardened Concrete" Section 6.4.



| | | | |
|---|---|---|---|
| [redacted] | 06/07/2020 | [redacted] | 07/07/2020 |
| Tested By | Date | Authorised Signatory | Date |
| [redacted] | | [redacted] | |

**Test Results:**

| SGS Lab No. | Client No. | Sample Location | CaO Content % Wt Sample | Cement Content % Wt Sample |
|---|---|---|---|---|
| P47696 | MV B1b | Beam 1 – Sample b | 11.9 | 18.6 |
| P47697 | MV B2b | Beam 2 – Sample b | 12.6 | 19.6 |
| P47698 | MV B5b | Beam 5 – Sample b | 12.2 | 19.0 |
| P47699 | MV B6b | Beam 6 – Sample b | 12.0 | 18.7 |

Note: Assumed 64.0% CaO in cement and 0.0% soluble CaO in aggregates.

| | |
|---|---|
| **Client:** | GHD Pty Ltd<br>2 Salamanca Square<br>Hobart TAS 7000 |
| **Your Reference:** | 3218213 – State Growth Bridge Assessments |
| **Our Reference:** | JN 20-10-159 |

# Certificate of Test No. 14973

| | |
|---|---|
| **Sample:** | Concrete Core Samples |
| **Date Received:** | 5th June 2020 |
| **Date Tested:** | 16th June 2020 |
| **From:** | Tasman Bridge – Main Viaduct (Span 21 – East Abutment) |
| **Description & Condition:** | 4 -off nominal 45 mm diameter concrete core samples |

**Test Description: Depth of Carbonation**

**Sample Preparation:**

Tested on freshly cored samples.

**Test Method:**

Main Roads WA test method WA 620.1 "Carbonation of Concrete".

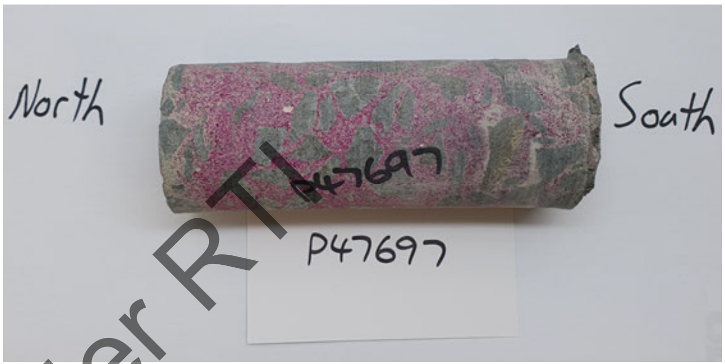


| | | | |
|---|---|---|---|
| [redacted] | 16/06/2020 | [redacted] | 07/07/2020 |
| Tested By | Date | Authorised Signatory | Date |
| [redacted] | | [redacted] | |

**Test Results:**

**SGS Lab No.: P47696**
**Client No.: B1b**
**Depth of Carbonation: North end 4-20 mm; South end 4-17 mm**
**Comments: Length – 150 mm**


**SGS Lab No.: P47697**
**Client No.: B2b**
**Depth of Carbonation: North end 2-8 mm; South end 3-5 mm**
**Comments: Length – 125 mm**




**SGS Lab No.: P47698**
**Client No.: B5b**
**Depth of Carbonation: North end 4-8 mm; South end 3-12 mm**
**Comments: Length – 122 mm**


**SGS Lab No.: P47699**
**Client No.: B6b**
**Depth of Carbonation: North end 6-17 mm; South end 3-10 mm**
**Comments: Length – 152 mm**

SGS

| | |
|---|---|
| **Client:** | GHD Pty Ltd<br>2 Salamanca Square<br>Hobart TAS 7000 |
| **Your Reference:** | State Growth Bridge Assessments, Job No. 3218213 |
| **Our Reference:** | JN 20-10-159 |

# Certificate of Test No. 14979

| | |
|---|---|
| **Sample:** | Concrete Core Samples |
| **Date Received:** | 5th June 2020 |
| **Date Tested:** | 16th – 22nd June 2020 |
| **From:** | Tasman Bridge, Main Viaduct (Span 21 – East Abutment) |
| **Description & Condition:** | 4 -off nominal 45 mm diameter concrete core samples |

**Test Description:** **Acid Soluble Chloride Content**

**Sample Preparation:**

Sub-samples removed from cores by dry diamond saw, pulverised to pass 125 µm sieve prior to analysis.

**Test Method:**

Chloride content in accordance with BS 1881: Part 124: 2015 "Methods for Analysis of Hardened Concrete" Section 12.1, except titration by potentiometric method.



s 36 [redacted] 22/06/2020 s 36 [redacted] 07/07/2020

Tested By Date Authorised Signatory Date

s 36 [redacted] s 36 [redacted]

**Test Results:**

| SGS Lab No. | Client No. | Sample Location | Depth (mm) | % $Cl^-$ by Weight of Concrete |
|---|---|---|---|---|
| P47692 | MV B1a | Beam 1 – Sample a | 0-10 | 0.02 |
| | | | 10-25 | < 0.01 |
| | | | 25-40 | < 0.01 |
| | | | 40-60 | < 0.01 |
| | | | 60-90 | 0.01 |
| | | | 90-110 | < 0.01 |
| | | | 110-125 | < 0.01 |
| | | | 125-140 | 0.01 |
| | | | 140-150 | 0.04 |
| P47693 | MV B2a | Beam 2 – Sample a | 0-10 | 0.05 |
| | | | 10-25 | 0.01 |
| | | | 25-40 | 0.01 |
| | | | 40-55 | < 0.01 |
| | | | 55-70 | 0.01 |
| | | | 70-85 | < 0.01 |
| | | | 85-100 | < 0.01 |
| | | | 100-115 | < 0.01 |
| | | | 115-125 | 0.04 |
| P47694 | MV B5a | Beam 5 – Sample a | 0-10 | 0.04 |
| | | | 10-25 | 0.01 |
| | | | 25-40 | 0.01 |
| | | | 40-55 | < 0.01 |
| | | | 55-70 | < 0.01 |
| | | | 70-85 | < 0.01 |
| | | | 85-100 | < 0.01 |
| | | | 100-115 | < 0.01 |
| | | | 115-125 | 0.05 |

Certificate No. 14979

**Test Results:**

| SGS Lab No. | Client No. | Sample Location | Depth (mm) | % $Cl^-$ by Weight of Concrete |
|---|---|---|---|---|
| P47695 | MV B6a | Beam 6 – Sample a | 0-10 | 0.04 |
| | | | 10-25 | 0.01 |
| | | | 25-40 | < 0.01 |
| | | | 40-60 | < 0.01 |
| | | | 60-90 | < 0.01 |
| | | | 90-110 | 0.01 |
| | | | 110-125 | < 0.01 |
| | | | 125-140 | 0.01 |
| | | | 140-150 | 0.02 |

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→ The Power of Commitment

Record 9

# FLG Plant Inspection

INSPECTION REPORT FOR STORNOWAY

Project No: 4091.007
3 May 2021



COVA

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# REPORT FOR STORNOWAY

## FLG Plant Inspection

### DOCUMENT ISSUE AUTHORISATION

**PROJECT:** FLG Plant Inspection
**PROJECT NO:** 4091.007
**AUTHOR:** s.36 [redacted]

| DATE | PURPOSE OF ISSUE/NATURE OF REVISION | REV | REVIEWED BY | ISSUE AUTHORISED BY |
|---|---|---|---|---|
| 3 May 2021 | Issue to client | 0 | DF | RJH |
| | | | | |

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**COVA Thinking Pty Ltd**
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**ACN 117 492 814 ABN 24 117 492 814**

Telephone: (03) 6212 4400
Email: cova@covathinking.com

## 1. INSPECTION AND TESTING CERTIFICATE

**1. Title:** Inspection and Testing certificate for Tasman Bridge Maintenance Inspection Gantry

**2. Equipment data:**

Registration No.: T30525

Identification No.: Tasman Bridge Feature Lighting Gantry (FLG)

Manufacturer's name: EMTEC Pty Ltd

Manufacturer's address: Suite 5, 40 Molle St, Hobart, TAS 7000

Telephone No.: (03) 62124400

Facsimile No.: (03) 62124475

Owner's name: Department of State Growth

Address: 10 Murray St, Hobart, TAS 7000

Telephone No.: (03) 6233 9406

Facsimile No.: (03) 6233 8696

Maximum rated capacity: 210 kg (Tower platforms)



**3. Inspection**

| Items Inspected | Satisfactory | Notes |
|---|---|---|
| Platform | ✓ | |
| Records | | |
| Operating Manuals | | |
| Maintenance Manuals | | |
| Guarding | ✓ | |
| Safety gear | | |
| - Signage | ✓ | |
| - Lanyards points | ✓ | |
| - Trailing electric cables | ✓ | *a |
| - Wheels | ✓ | |
| Brakes | ✓ | |
| Guide rollers | ✓ | |
| Lubrication | ✓ | |
| Levelling wheel mechanism | ✓ | |
| HPP | ✓ | |
| Hydraulic hoses | ✓ | |

| | | |
|---|---|---|
| Drive coupling | ✓ | |
| General Electrical | ✓ | |
| Access | ✓ | |
| Clearances | ✓ | |
| Gates and latches | ✓ | |
| Protective meshing | ✓ | |
| Safety requirements for personnel access and egress | ✓ | |
| Limit switches | ✓ | |
| Isolation | ✓ | |
| Pendant controls | ✓ | *b |
| Structure and connections | ✓ | |

**4. Repairs**

| Description | Timeframe |
|---|---|
| *a – Trailing cables have been tested and tagged. One is in date, due for next inspection on 9/6/21. The other has been filled out incorrectly and should be retested and tagged immediately. | **Immediately** |
| *b –The outer cable sheath of both the pendant controllers has pulled free from the gland nut and been temporarily patched up with electrical tape. This will eventually lead to water ingress and failure of the pendant or electrical plug. Re-terminate and/or repair the cable/gland immediately. | **Immediately** |



**5. Tests**

- HPP
- Long travel drives & brakes
- Pendant controller
- E-stops and isolators

**6. Test site/station detail:**

On the Feature Lighting Gantries, Tasman Bridge, Hobart.

**7. Certification:**

a. The general condition of the machine is good with some components requiring some attention and maintenance.

b. Remarks/Repairs: (see section 4)

**Recommendation:**

I recommend that:

i. The following components of the equipment be re-examined and tested as appropriate upon completion of the prescribed maintenance:

**– All items listed under Section 4 – Repairs.**

ii. The following components of the equipment be re-examined and tested as appropriate on or before the date indicated for the item:

Annual checks of:
– Trailing cables
– The lanyard lines
– Switchgear
– Fire Extinguishers
– First Aid kits

iii. This equipment be re-examined and tested on or before the 18th February 2022

c. The examination was a routine, annual inspection.

d. **I hereby certify that I have examined and tested as appropriate, the Tasman Bridge Feature Lighting Gantries (FLG) and I find that the equipment is in a satisfactory condition for safe use.**

e. Number of sheets attached: 8

f. Competent persons details:

Name: [redacted]

Address: Suite 5, 40 Molle St, Hobart, TAS 7000

Telephone No.: (03) 62124488

Facsimile No.: (03) 62124475

Qualifications: Mechanical Engineer, Cert IV in Advanced TIG Welding, Working at Heights, EWP Licence

Relevant experience: Design, drafting and commissioning of large Bulk Materials Handling Machines such as conveyors, stackers, shiploaders, and various industrial machinery, and Authorised Inspector of Building Maintenance Units in Tasmania.

g. Signature:

[redacted]

Date: 3rd May 2021

# APPENDIX A
# PHOTOS

# 1 Appendix A – Photos


Fig. *a(i)


Fig. *a(ii)

Fig. *b(i)

Fig. *b(ii)

Record 10

# MIG Plant Inspection

INSPECTION REPORT FOR STORNOWAY

Project No: 4091.007
3 May 2021



COVA

## DOCUMENT ISSUE AUTHORISATION

**PROJECT:** MIG Plant Inspection
**PROJECT NO:** 4091.007
**AUTHOR:** s.36 [redacted]

| DATE | PURPOSE OF ISSUE/NATURE OF REVISION | REV | REVIEWED BY | ISSUE AUTHORISED BY |
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| 3 May 2021 | Issue to client | 0 | DF | RJH |
| | | | | |

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**2. Equipment data:**

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Identification No.: Tasman Bridge Maintenance Inspection Gantry (MIG)

Manufacturer's name: EMTEC Pty Ltd

Manufacturer's address: Suite 5, 40 Molle St, Hobart, TAS 7000

Telephone No.: (03) 62124400

Facsimile No.: (03) 62124475

Owner's name: Department of State Growth

Address: 10 Murray St, Hobart, TAS 7000

Telephone No.: (03) 6233 9406

Facsimile No.: (03) 6233 8696

Maximum rated capacity: 510 kg (Gantry)
210 kg (Tower platforms)
90 kg (Extension platforms)



**3. Inspection**

| Items Inspected | Satisfactory | Notes |
|---|---|---|
| Platforms | ✓ | *a |
| Records | ✓ | |
| Operating Manuals | ✓ | |
| Maintenance Manuals | ✓ | |
| Guarding | ✓ | |
| Safety gear | | |
| - Signage | ✓ | |
| - Harnesses/Lanyards | ✓ | |
| - Fire Extinguishers | ✓ | |
| - First Aid kit | ✓ | |
| Trailing electric cables | ✓ | *b |
| Long travel chains | ✓ | *c |
| Long travel chain hoist | ✓ | |
| Long travel clamp brakes | ✓ | |
| Chain wheel idlers | ✓ | |

| | | |
|---|---|---|
| Guide rollers on bogies | ✓ | |
| Bogie wheels | ✓ | |
| Reaction castors | ✓ | |
| Guide rollers on handrail | ✓ | *d |
| Lubrication | ✓ | |
| Levelling cylinders | ✓ | *e |
| Levelling cylinder pins | ✓ | |
| Winch drives (gearbox & Brake motor) | ✓ | |
| Winch drum | ✓ | |
| Winch ropes (visual only) | ✓ | *f |
| Winch brakes | ✓ | |
| HPP | ✓ | |
| Hydraulic hoses | ✓ | *g |
| Electrical system | ✓ | |
| Wiring diagram | ✓ | |
| Indicator lamps | ✓ | |
| Access | ✓ | |
| Clearances | ✓ | |
| Gates and latches | ✓ | |
| Protective meshing | ✓ | |
| Safety requirements for personnel access and egress | ✓ | |
| Limit switches | ✓ | *h |
| Isolation (E-stops) | ✓ | |
| Wire Ropes | ✓ | |
| Guidance system | ✓ | |
| Controls | ✓ | |
| Electrical cables and wiring | ✓ | *i |
| Dummy plugs | ✓ | |
| Structure and connections | ✓ | |
| Communication systems | ✓ | |

## 4. Repairs

| Description | Timeframe |
|---|---|
| *a – The plywood covering, over the aluminium mesh floor of the gantry, is weathered and starting to break up. Consider replacing or removing the plywood now that the major bridge maintenance works have been completed. | **12 months** |
| *b – Trailing cables are tested, tagged and in date, due for next inspection on 9/6/21 | **12 months** |
| *c – Long travel chain anchor shackles are tested, tagged and in date, due for next inspection in April 2021<br>– The rusty hammerlock links between the long travel chains and the swivels have been replaced. | **3 months** |
| *d – The guide rollers that run against the handrail were previously rubber lined. These rollers have been replaced with hard plastic rollers. These rollers damage the paint on the handrail which will lead to future corrosion issues. Monitor and replace the hard plastic rollers with rubber (or polyurethane) tyred alternatives as per the original design if paint damage becomes excessive. | **24 months** |
| *e – The leveling cylinder bypass valves on the upriver gantry were very stiff to operate (nearly seized). Lubricant spray was worked into the taps, but future disassembly and service may be required. | **12 months** |
| *f – There was some surface corrosion noted on the wire rope loop around the thimble. Recommend clean and relubricate | **Immediately** |
| *g – Deterioration of the outer jacket of some hydraulic hoses was noted on both towers. Continue to monitor hoses and replace worn/deteriorated hosed before leaks occur. | **12 months** |
| *h – The large NHP limit switches for the gantry hoist working and final limits are inoperable. Recommend immediate repair or replacement. Care must be taken to setup and recommission switches if they are moved or adjusted in any way. | **Immediately** |
| *i – Some of the electrical wiring runs through PVC pipe sleeves under metal 'P' clips. Recommend tidying up electrical wiring to prevent damage | **12 months** |



## 5. Tests

- HPP
- Pendant controller
- E-stops and isolators
- Hoisting, brakes & limits
- Long Travel & brakes

## 6. Test site/station detail:

On the Maintenance Inspection Gantry, Tasman Bridge, Hobart.

**7. Certification:**

a. The general condition of the machine is good. There are several components that require immediate attention and maintenance to ensure continued reliable and safe operation.

b. Remarks/Repairs: (see section 4)

**Recommendation:**

I recommend that:

i. The following components of the equipment be re-examined and tested as appropriate upon completion of the prescribed maintenance:

**– All items listed under Section 4 – Repairs.**

ii. The following components of the equipment be re-examined and tested as appropriate on or before the date indicated for the item:

Annual checks of:
- The long travel chains and shackles
- The winch ropes
- Trailing cables
- The lanyard lines
- Switchgear
- Fire Extinguishers
- First Aid kits

iii. This equipment be re-examined and tested on or before the 18th February 2022

c. The examination was a routine, annual inspection.

**d. I hereby certify that I have examined and tested as appropriate, the Tasman Bridge Maintenance Inspection Gantry (MIG) and I find that the equipment is in a satisfactory condition for safe use, provided the inoperable hoist limits are rectified immediately and attention to maintenance listed in section 4.**

e. Number of sheets attached: 9

f. Competent persons details:

Name: [redacted]

Address: Suite 5, 40 Molle St, Hobart, TAS 7000

Telephone No.: (03) 62124488

Facsimile No.: (03) 62124475

Qualifications: Mechanical Engineer, Cert IV in Advanced TIG Welding, Working at Heights, EWP Licence

Relevant experience: Design, drafting and commissioning of large Bulk Materials Handling Machines such as conveyors, stackers, shiploaders, and various industrial machinery, and Authorised Inspector of Building Maintenance Units in Tasmania.

g. Signature:

[redacted]

Date: 30th April 2021

# APPENDIX A
# PHOTOS

1 Appendix A – Photos

Fig. *b

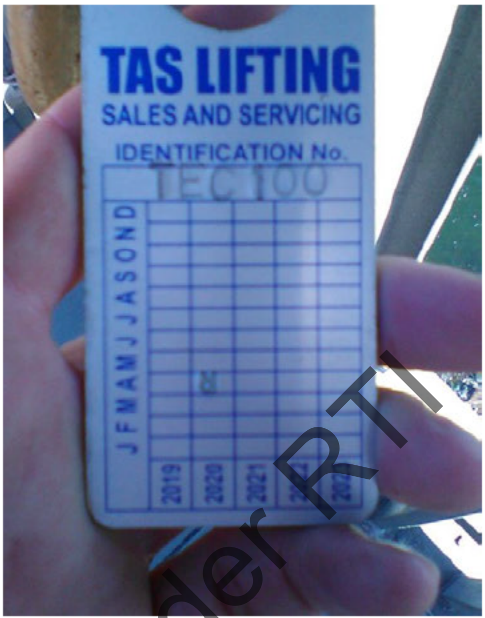


Fig. *c

Fig. *d

Fig. *f

Fig. *g

Record 13

# FLG Plant Inspection

INSPECTION REPORT FOR STORNOWAY

Project No: 4091.009
12 May 2022



COVA

COVA



## DOCUMENT ISSUE AUTHORISATION

**PROJECT:** FLG Plant Inspection
**PROJECT NO:** 4091.009
**AUTHOR:** s.36 [redacted]

| DATE | PURPOSE OF ISSUE/NATURE OF REVISION | REV | REVIEWED BY | ISSUE AUTHORISED BY |
|---|---|---|---|---|
| 12 May 2022 | Issue to client | 0 | DF | RJH |
| | | | | |



COVA Thinking Pty Ltd
Suite 5, 40 Molle St, Hobart, TAS 7000 AU
ACN 117 492 814 ABN 24 117 492 814

Telephone: (03) 6212 4400
Email: cova@covathinking.com

## 1. INSPECTION AND TESTING CERTIFICATE

1. **Title:** Inspection and Testing certificate for Tasman Bridge Maintenance Inspection Gantry

2. **Equipment data:**

Registration No.: T30525

Identification No.: Tasman Bridge Feature Lighting Gantry (FLG)

Manufacturer's name: EMTEC Pty Ltd

Manufacturer's address: Suite 5, 40 Molle St, Hobart, TAS 7000

Telephone No.: (03) 62124400

Facsimile No.: (03) 62124475

Owner's name: Department of State Growth

Address: 10 Murray St, Hobart, TAS 7000

Telephone No.: (03) 6233 9406

Facsimile No.: (03) 6233 8696

Maximum rated capacity: 210 kg (Tower platforms)

3. **Inspection**

| Items Inspected | Satisfactory | Notes |
|---|---|---|
| Platform | ✓ | *a |
| Records | ✓ | |
| Operating Manuals | ✓ | *b |
| Maintenance Manuals | ✓ | *c |
| Guarding | ✓ | |
| Safety gear | | |
| - Signage | ✓ | |
| - Lanyards points | ✓ | |
| - Trailing electric cables | ✓ | *d |
| - Wheels | ✓ | *e |
| Brakes | ✓ | |
| Guide rollers | ✓ | *f |
| Lubrication | ✓ | |
| Levelling wheel mechanism | ✓ | |
| HPP | ✓ | |
| Hydraulic hoses | ✓ | |

| | | |
|---|---|---|
| Drive coupling | ✓ | |
| General Electrical | ✓ | |
| Access | ✓ | |
| Clearances | ✓ | |
| Gates and latches | ✓ | |
| Protective meshing | ✓ | |
| Safety requirements for personnel access and egress | ✓ | |
| Limit switches | ✓ | |
| Isolation | ✓ | |
| Pendant controls | ✓ | |
| Structure and connections | ✓ | |

*b , *c – OAM manuals are part (included in) the MIG OAM stored on the machine.

**Repairs**

| Description | Timeframe |
|---|---|
| *a – The hatches on the platform are weathering heavily around the edges. The main panel feels sound but replacement is necessary asap. | **3 months** |
| *b – Trailing cables were tested, tagged but out of date, they were due for inspection on 22/1/22. There were some inconsistencies with the tags suggesting some items were missed or tags had detached. | **Immediately** |
| *c – Polyurethane wheels are showing signs of crazing and weathering. Monitor and replace as necessary. | **12 months** |
| *d – The white plastic guide rollers are showing signs of wear and weathering. Monitor and replace as necessary. | **12 months** |

**4. Tests**

- HPP
- Long travel drives & brakes
- Pendant controller
- E-stops and isolators

**5. Test site/station detail:**

On the Feature Lighting Gantries, Tasman Bridge, Hobart.

**6. Certification:**

a. The general condition of the machine is good with some components requiring some attention and maintenance.

b. Remarks/Repairs: (see section 4)



**Recommendation:**

I recommend that:

i. The following components of the equipment be re-examined and tested as appropriate upon completion of the prescribed maintenance:

– **All items listed under Section 4 – Repairs.**

ii. The following components of the equipment be re-examined and tested as appropriate on or before the date indicated for the item:

Annual checks of:
– Trailing cables
– The lanyard lines
– Switchgear
– Fire Extinguishers

– First Aid kits

iii. This equipment be re-examined and tested on or before the 17th March 2023

c. The examination was a routine, annual inspection and for the purposes of re-Registration of Plant.

**d. I hereby certify that I have examined and tested as appropriate, the Tasman Bridge Feature Lighting Gantries (FLG) and I find that the equipment is in a satisfactory condition for safe use.**

e. Number of sheets attached: 8

f. Competent persons details:

Name [redacted]

Address: Suite 5, 40 Molle St, Hobart, TAS 7000

Telephone No.: (03) 62124488

Facsimile No.: (03) 62124475

Qualifications: Mechanical Engineer, Cert IV in Advanced TIG Welding, Working at Heights, EWP Licence

Relevant experience: Design, drafting and commissioning of large Bulk Materials Handling Machines such as conveyors, stackers, shiploaders, and various industrial machinery, and Authorised Inspector of Building Maintenance Units in Tasmania.

g. Signature:

[redacted]

Date: 11th May 2023

# APPENDIX A
# PHOTOS

## 1 Appendix A – Photos

Fig. *a


Fig. *d

Fig. *e

Fig. *f

Record 14

# MIG Plant Inspection

INSPECTION REPORT FOR STORNOWAY

Project No: 4091.009
12 May 2022

## DOCUMENT ISSUE AUTHORISATION

**PROJECT:** MIG Plant Inspection
**PROJECT NO:** 4091.009
**AUTHOR:** s.36 [redacted]

| DATE | PURPOSE OF ISSUE/NATURE OF REVISION | REV | REVIEWED BY | ISSUE AUTHORISED BY |
|---|---|---|---|---|
| 12 May 2022 | Issue to client | 0 | DF | RJH |
| | | | | |

This document has been prepared in accordance with the scope of services agreed upon between **COVA Thinking Pty Ltd** (COVA) and the Client. To the best of COVA's knowledge, the document presented herein represents the Client's intentions at the time of printing of the document. However, the passage of time, manifestation of latent conditions or impacts of future events may result in the actual contents differing from that described in this document. In preparing this document COVA has relied upon data, surveys, analysis, designs, plans and other information provided by the client, and other individuals and organisations referenced herein. Except as otherwise stated in this document, COVA has not verified the accuracy or completeness of such data, surveys, analysis, designs, plans and other information.

No responsibility is accepted for use of any part of this document in any other context or for any other purpose by third parties.

This document does not purport to provide legal advice. Readers should engage professional legal advisers for this purpose.

**COVA Thinking Pty Ltd**
Suite 5, 40 Molle St, Hobart, TAS 7000 AU
**ACN 117 492 814 ABN 24 117 492 814**

Telephone: (03) 6212 4400
Email: cova@covathinking.com

## 1. INSPECTION AND TESTING CERTIFICATE

**1. Title:** Inspection and Testing certificate for Tasman Bridge Maintenance Inspection Gantry

**2. Equipment data:**

Registration No.: T30525

Identification No.: Tasman Bridge Maintenance Inspection Gantry (MIG)

Manufacturer's name: EMTEC Pty Ltd

Manufacturer's address: Suite 5, 40 Molle St, Hobart, TAS 7000

Telephone No.: (03) 62124400

Facsimile No.: (03) 62124475

Owner's name: Department of State Growth

Address: 10 Murray St, Hobart, TAS 7000

Telephone No.: (03) 6233 9406

Facsimile No.: (03) 6233 8696

Maximum rated capacity: 510 kg (Gantry)

210 kg (Tower platforms)

90 kg (Extension platforms)



**3. Inspection**

| Items Inspected | Satisfactory | Notes |
|---|---|---|
| Platforms | ✓ | *a |
| Records | ✓ | *b |
| Operating Manuals | ✓ | |
| Maintenance Manuals | ✓ | |
| Guarding | ✓ | |
| Safety gear | | |
| - Signage | ✓ | |
| - Harnesses/Lanyards | ✓ | |
| - Fire Extinguishers | ✓ | |
| - First Aid kit | ✓ | |
| Trailing electric cables | ✓ | *c |
| Long travel chains | ✓ | |
| Long travel chain hoist | ✓ | |
| Long travel clamp brakes | ✓ | |
| Chain wheel idlers | ✓ | |

| | | |
|---|---|---|
| Guide rollers on bogies | ✓ | |
| Bogie wheels | ✓ | |
| Reaction castors | ✓ | |
| Guide rollers on handrail | ✓ | |
| Lubrication | ✓ | |
| Levelling cylinders | ✓ | |
| Levelling cylinder pins | ✓ | |
| Winch drives (gearbox & Brake motor) | ✓ | |
| Winch drum | ✓ | |
| Winch ropes (visual only) | ✓ | *d |
| Winch brakes | ✓ | |
| HPP | ✓ | |
| Hydraulic hoses | ✓ | |
| Electrical system | ✓ | |
| Wiring diagram | ✓ | |
| Indicator lamps | ✓ | |
| Access | ✓ | |
| Clearances | ✓ | |
| Gates and latches | ✓ | |
| Protective meshing | ✓ | *e |
| Safety requirements for personnel access and egress | ✓ | |
| Limit switches | ✓ | *f |
| Isolation (E-stops) | ✓ | |
| Wire Ropes | ✓ | |
| Guidance system | ✓ | |
| Controls | ✓ | |
| Electrical cables and wiring | ✓ | *g |
| Dummy plugs | ✓ | |
| Structure and connections | ✓ | |
| Communication systems | ✓ | |

## 4. Repairs

| Description | Timeframe |
|---|---|
| *a – The plywood covering, over the aluminium mesh floor of the gantry, is weathered and starting to break up. Consider replacing or removing the plywood now that the major bridge maintenance works have been completed. | **3 months** |
| *b – Logbook being kept on previous Downer sheet. Rod Lovell to update on Stornaway sheets. | **3 months** |
| *c – Trailing cables were tested, tagged but out of date, they were due for inspection on 22/1/22. There were some inconsistencies with the tags suggesting some items were missed or tags had detached. | **6 months** |
| *d – There was some surface corrosion noted on the wire rope loop around the thimble. Recommend clean and relubricate | **Immediately** |
| *e – Parts of the plastic mesh guarding installed on the gantry is weathered and frayed. Recommend removal now that major works have been completed. | **3 months** |
| *f – The large NHP limit switches for the gantry hoist working and final limits are operable but quite stiff. Recommend lubrication, repair or replacement. They activate but may not reset.<br>The anti-crab limit switches are also very stiff. Recommend lubrication, repair or replacement.<br>ATTENTION: Care must be taken to setup and recommission switches if they are moved or adjusted in any way. | **Immediately** |



## 5. Tests

- HPP
- Pendant controller
- E-stops and isolators
- Hoisting, brakes & limits
- Long Travel & brakes

## 6. Test site/station detail:

On the Maintenance Inspection Gantry, Tasman Bridge, Hobart.

## 7. Certification:

a. The general condition of the machine is good. There are few components that require immediate attention and maintenance to ensure continued reliable and safe operation.

b. Remarks/Repairs: (see section 4)

**Recommendation:**

I recommend that:

i. The following components of the equipment be re-examined and tested as appropriate upon completion of the prescribed maintenance:

   **– All items listed under Section 4 – Repairs.**

ii. The following components of the equipment be re-examined and tested as appropriate on or before the date indicated for the item:

   Annual checks of:
   – The long travel chains and shackles
   – The winch ropes
   – Trailing cables
   – The lanyard lines
   – Switchgear
   – Fire Extinguishers
   – First Aid kits

iii. This equipment be re-examined and tested on or before the 17th March 2023

c. The examination was a routine, annual inspection and for the purposes of re-Registration of Plant.

**d. I hereby certify that I have examined and tested as appropriate, the Tasman Bridge Maintenance Inspection Gantry (MIG) and I find that the equipment is in a satisfactory condition for safe use, with attention given to maintenance listed in section 4.**

e. Number of sheets attached: 9

f. Competent persons details:

   Name: s36 [redacted]

   Address: Suite 5, 40 Molle St, Hobart, TAS 7000

   Telephone No.: (03) 62124488

   Facsimile No.: (03) 62124475

   Qualifications: Mechanical Engineer, Cert IV in Advanced TIG Welding, Working at Heights, EWP Licence

   Relevant experience: Design, drafting and commissioning of large Bulk Materials Handling Machines such as conveyors, stackers, shiploaders, and various industrial machinery, and Authorised Inspector of Building Maintenance Units in Tasmania.

g. Signature:

s.36 [redacted]

Date: 11th May 2022

# APPENDIX A PHOTOS

# 1 Appendix A – Photos

Fig. *a

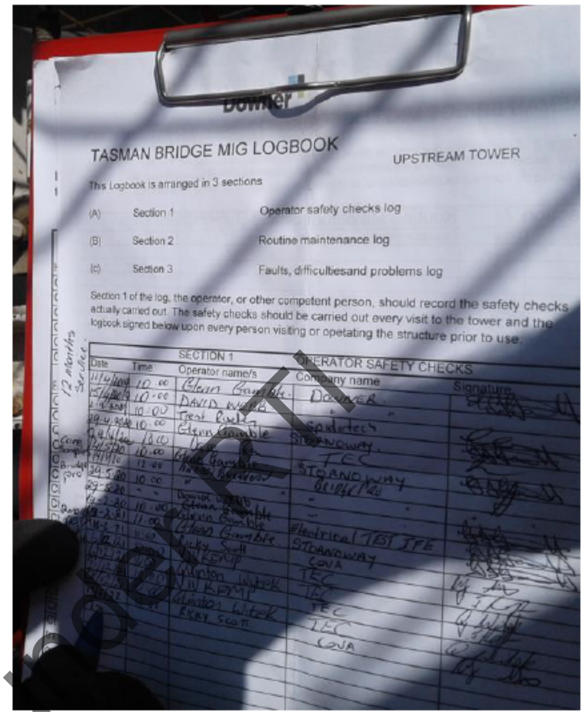


Fig. *b

Fig. *c


Fig. *d

Fig. *e

Fig. *f

[redacted]

2 Salamanca Square,
Hobart, Tasmania 7000
Australia
www.ghd.com

**Your ref:** RB00529
**Our ref:** 12603928

**06 February 2023**

[redacted]
**Stornoway Maintenance Pty Ltd**
**1-37 Tasma Street**
**North Hobart TAS 7002**

**Tasman Bridge Truck Impact Damage Inspection**

Dear [redacted]



## Background

Following a truck rollover on the western approach viaduct of the Tasman Bridge (B5512) on 16 January 2023, the Department of State Growth (State Growth) engaged Stornoway Maintenance Pty Ltd (Stornoway) to conduct an inspection of the bridge deck where the impact occurred to identify any structural damage that may have occurred.

Stornoway subsequently requested GHD Pty Ltd (GHD) to complete the structural inspections on their behalf, with Stornoway providing all labour, equipment and materials required to facilitate the inspections.

## Purpose

Purpose of this letter is to outline GHD's inspection observations and subsequent recommendations in relation to the Tasman Bridge inspection as outlined in the scope of works.

## Scope of Works

The following scope of works were undertaken by GHD as part of this inspection:

– Visual inspection of the underside of the Tasman Bridge girders between the western approach piers E and F
– Visual inspection of the top surface of the Tasman Bridge deck between the western approach piers E and F
– Preparation of a brief letter including the following:
  - A crack map identifying the locations of any identified cracks or damage within the vicinity of the incident.
  - Photographic record of all identified defects

- Description of the inspection methodology and the resulting outcomes
- Recommendations for further investigations, monitoring, or repairs as deemed appropriate.

# Limitations

In preparing this letter, GHD excludes the following:

- Inspection of any structural elements outside of the concrete deck and girders and the immediate vicinity of the incident.
- Any testing (non-destructive or otherwise) of the concrete deck
- Structural analysis or assessment

# Disclaimer

This letter has been prepared by GHD for Stornoway and may only be used and relied on by Stornoway for the purpose agreed between GHD and Stornoway as set out in this letter.

GHD otherwise disclaims responsibility to any person other than Stornoway arising in connection with this letter. GHD also excludes implied warranties and conditions, to the extent legally permissible.

The services undertaken by GHD in connection with preparing this letter were limited to those specifically detailed in the letter and are subject to the scope limitations set out in the letter.

The opinions, conclusions and any recommendations in this letter are based on conditions encountered and information reviewed at the date of preparation of the letter. GHD has no responsibility or obligation to update this letter to account for events or changes occurring subsequent to the date that the letter was prepared.

The opinions, conclusions and any recommendations in this letter are based on assumptions made by GHD described in this letter. GHD disclaims liability arising from any of the assumptions being incorrect.

# Inspection

## Underside of the Beams

GHD attended site on 25 January 2023 to conduct the inspection of the underside of the girders. The inspection was undertaken up close from an Elevated Work Platform (EWP). The EWP was supplied and operated by Stornoway and was set up between Piers D and E on the western bank. The main focus of the inspection was to locate the defects that were present on the underside of the beams, in the region where the truck had impacted the bridge.


*Figure 1 General view of the bridge between Piers D, E and F*

Initially, the underside of the superstructure was inspected between Piers D and E. No defects were noted in this location other than some minor surface cracking that was noted on the second most northern beam, towards Pier E, refer Figure 2. This cracking was likely to have been present for some time due to the calcite residue (autogenous healed) present along the crack. The EWP could not reach the crack to measure the crack width, however it was determined visually that the extent of the cracking at this location is likely to be minor.

*Figure 2 Cracking at second beam from the north between Piers D and E*

*Figure 3 General view of the condition of the soffit of the bridge deck and girders between Piers D and E*

Runoff from the bridge was noted to have stained some sections of the Pier E headstock, refer Figure 4. Some of the staining visually appears similar to vertical cracking. However, it was confirmed during the inspection that there was no cracking on surface of the headstock. The staining suggests that the joint in the roadway is not completely sealed.

*Figure 4 Staining on the eastern face of the headstock of Pier E*

Subsequently, the underside of the beams between Piers E and F were inspected utilising the EWP. Due to difficulties setting up the EWP between Piers E and F, the inspection of this location was conducted while the EWP was still set up between Piers D and E. This limited the area that the EWP could reach. However, it was possible to inspect majority of the underside of the potential contact area up close.

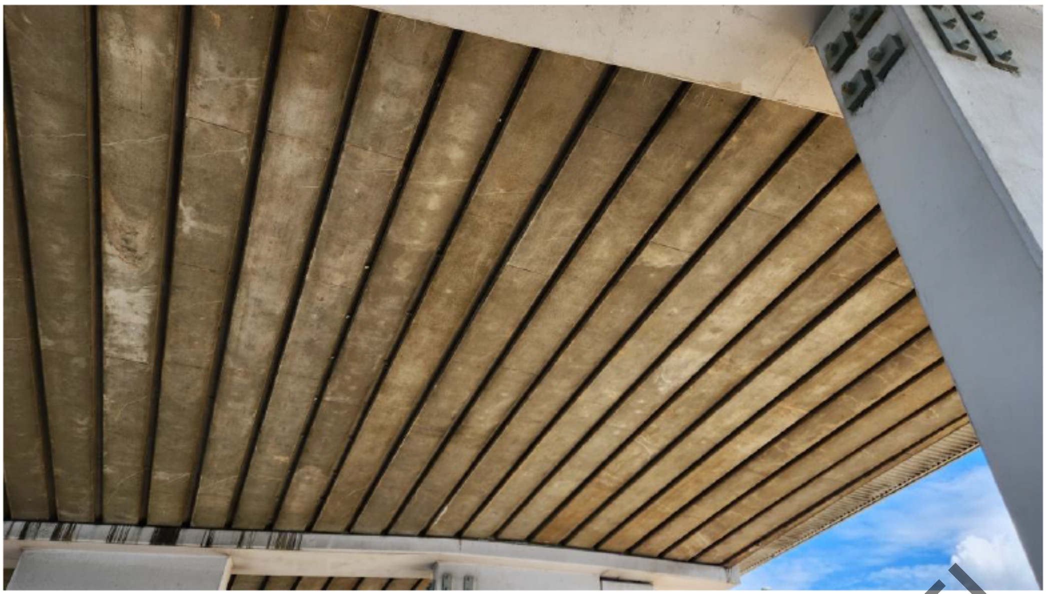

*Figure 5 General view of the condition of the soffit of the bridge deck between Piers E and F*

No defects were noted on this section of the bridge, refer Figure 5. A white runoff substance from the bridge surface was noted to have left marks on the underside of the beams. It was not possible to physically reach all of the stains, however, no cracking was noted at the locations that were accessible.

Chalk marks numbering the beams suggest there has been an inspection that has been conducted on this section of the bridge previously. It may be possible to review findings from the previous inspection to compare the condition of the beams. However, due to the minimal number of defects encountered on the beams, this is not essential.

No crack maps were developed due to the lack of defects noted on the beams.

## Top Surface of the Bridge

GHD attended site on 01 February 2023 to conduct the inspection of the top surface of the deck. The inspection was undertaken by foot with the Stornoway maintenance crew undertaking the traffic management and barrier repairs. As part of the traffic management, the second lane from the north was completely closed, with the two lanes either side of this lane intermittently closed to conduct the inspection and repairs.



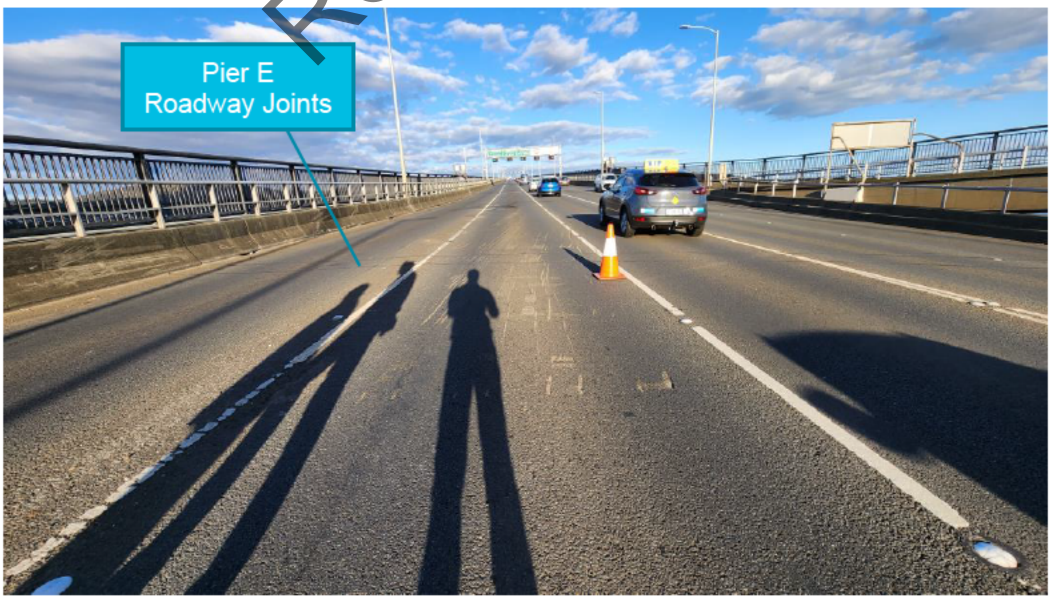


*Figure 6 General view of the damage to the asphalt*

As part of the inspection, the asphalt on the deck was not removed. Hence, it was not possible to inspect the top surface of the concrete deck. The inspection was limited to a visual inspection of the damage on the asphalt and the barrier.

The damage to the asphalt was limited to the three northern lanes of the bridge, with the majority of the damage being concentrated on the second lane from the north. The damage extended roughly from 1.5 m east of Pier F to Pier D, with majority of the damage located either side of Pier E. Pier locations were determined from the joints on the surface.


*Figure 7* *Gauges in the asphalt*

The gouges caused by the truck were found to have a maximum depth of roughly 15mm. This suggests that the damage is likely contained within the asphalt and has not reached the concrete deck. The damage appears to be superficial with no evidence indicating that the structural capacity of the bridge has been affected. It may be worth patching up the damage for aesthetic and functional purposes, however, from a structural perspective, no repairs are required.



*Figure 8* *Depth of the gouges in asphalt*

The impact from the truck was found to have cracked the base of the end post on the barrier. The hold down bolts for this post and the remainder of the barrier appeared to be in serviceable condition. Stornoway arranged for the cracking in the barrier post to be welded as part of the barrier repair.


*Figure 9* *Cracking at the base of the barrier post*

# Recommendations

As no damage was noted on the underside of the beams, and only superficial damage on the surface of the bridge, it can be concluded that the truck rollover is unlikely to have affected the load carrying capacity of the bridge. Therefore no major remediation works are required following the truck rollover. However, it would be prudent to patch the gouges in the asphalt for functional purposes. It's possible that some grooves may cause damage to the tires on vehicles, particularly on motorbikes.

Regards

[redacted]