Appendix A

Inspection and Assessment Report

Cloddach Bridge, Elgin

March 2022

FAIRHURST

CONTROL SHEET

CLIENT: **The Moray Council**

REPORT TITLE: **Inspection for Assessment Report**

PROJECT REFERENCE: **140163F – Cloddach Bridge, Elgin**

Issue and Approval Schedule: 3

ISSUE 3 **FINAL**	Name	Signature	Date
Prepared by	R de Groot		10/3/22
Reviewed by	S McLaren		10/3/22
Approved by	E Halkon		10/3/22

Revision Record:

Issue	Date	Status	Description	By	Chk	App
0	10-03-2022	Draft	Initial Draft submission			
1	21-03-2022	Draft	Update incorporating checker results	RCG	EH	EH
2	30-3-22	Final	Final Issue	EH	EH	EH
3	23/5/22	F1	Addition risk information added	EH	EH	EH

This report has been prepared in accordance with procedure OP/P02 of the Fairhurst Quality and Environmental Management System.

Contents

Appendices

Executive Summary

Fairhurst was appointed by Moray Council to carry out an inspection and structural assessment of the Cloddach Bridge near Elgin. The bridge is a simply supported three span structure of steel beam and concrete jack arched slab construction.

The Inspection for Assessment was undertaken on the 14th of February 2022. The inspection of the structure found it to be in a poor condition. Significant deterioration of the steelwork was noted and large areas of scour near the structure supports.

Following the Inspection for Assessment of the bridge, a quantitative structural assessment of the bridge deck was undertaken in accordance with the Design Manual for Roads and Bridges.

The assessment found the bridge to be adequate for footway loading. The capacity of the bridge is limited by the strength of the outer girders in bending, and is based on a minimum measured thickness of steel at midspan during the inspection. The bending capacity of the inner girders was found to be limited to 3T Assessment Live Loading.

It is recommended that the bridge remains closed to vehicle traffic, although with the implementation of bollards along with regular monitoring, use by pedestrians and cyclists could be allowed in the short term. In the longer term a demolition and/or full replacement of the bridge is recommended.

1 Introduction

Cloddach Bridge is a three span structure carrying a single carriageway road over the River Lossie. The bridge is located on an unnamed road to the west of the B9010, south of Elgin.

The bridge comprises three simply supported spans of approximately 7m. Each span is formed from 7 No. steel beams at approximately 715mm centres. A concrete jack arched slab spans between the steel beams with a corrugated steel shuttering to the underside. The substructure includes mass concrete abutments and intermediate mass concrete piers.

A Special Inspection was undertaken by Moray Council in January 2022 in which concerns were raised that the condition of the structure had deteriorated significantly since the previous inspection, 2 years earlier. Fairhurst was appointed by Moray Council to carry out an inspection and structural assessment of the existing bridge structure to determine its capacity to carry vehicle and pedestrian loadings.

2 Structure Identification and Location Plans

General structure information is given in Table 1 below.

Table 1: General Bridge Record Information

Item Ref	Data
Bridge Name	Cloddach Bridge
Bridge Number	C2E/20
Location	Elgin, Moray, UK
OS Grid Ref.	E: 320174, N: 858396
Class	River Crossing – Overbridge
Function	Supports a single carriageway over the River Lossie
Form	Three span simply supported structure
Type	Steel beams with concrete jack arch
Designed by	Unknown
Built by	Unknown
Date of Construction	Approximately 1905
Owner	The Moray Council
Substructure	Mass concrete abutments and mass concrete intermediate piers
Superstructure	Steel beams with concrete jack arch
Span	Spans denoted from west to east. Clear Span 1: 6.688m Clear Span 2: 6.689m Clear Span 3: 6.668m
Carriageway Width	3.881m
Skew Angle	N/A

This report is carried out in accordance with current bridge inspection practice, in particular, CS 450, the Inspection Manual for Highway Structures and the Transport Scotland Inspection Manual.

2.1 Historical Inspection Records and Other Information Made Available

Title	Published By	Dated
Cloddach Bridge - Inspection Report	Arch Henderson	27.09.1995
Cloddach Bridge - Assessment Calculations	Grampian Regional Council	06.02.1996
Cloddach Bridge - Inspection Report	Arch Henderson	17.07.1997
Cloddach Bridge - Assessment Calculations	Moray Council	26.07.2000
Cloddach Bridge - Principal Inspection Report	Moray Council	26.09.2019
Cloddach Bridge - PI Defect Sketch	Moray Council	26.09.2019
Cloddach Bridge - Load Review Calculations	Moray Council	17.10.2019
Cloddach Bridge - Special Inspection Report	Moray Council	28.01.2022

2.2 Location Plan

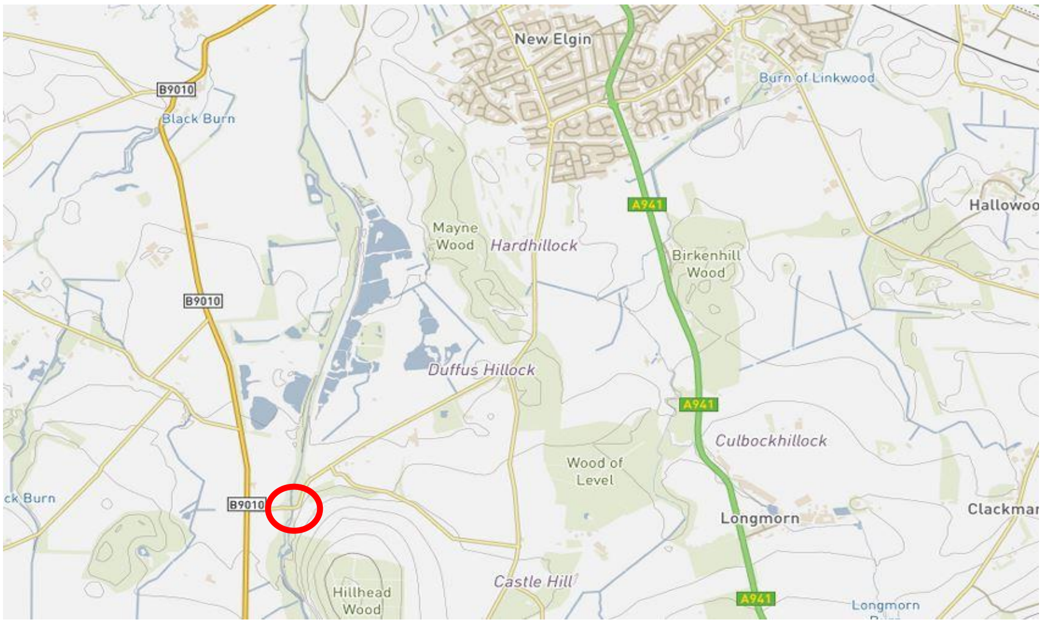


Figure 1: Bridge Location Plan

3 Inspection

An Inspection for Assessment of the entire bridge structure was undertaken by Fairhurst on the 14th of February 2022. Photographs are available in Appendix A.

3.1 General

A dimension and condition survey was undertaken to the bridge structure using hand measuring tools.

The inspection of high level areas within the end spans was undertaken using multiple tower scaffolds. No access at high level could be obtained to the underside of the central span due to the scour and fast flowing water to the invert below this span.

Power tools were used to clean back areas of existing steelwork to allow for ultrasonic thickness measurements to be obtained.

3.2 Intrusive Investigations

No intrusive investigations were undertaken as part of this assessment. The assessment is based on historic material strength values and material properties. The results from previous intrusive investigations are available and will be used to calculate the level of fill on top of the bridge. Evidence of previous concrete cores and steelwork sampling was also noted on site.

4 Inspection for Assessment

4.1 General

Cloddach Bridge is a three span bridge comprising simply supported steel beams with transverse concrete jack arched slab. The bridge carries a single carriageway for vehicles over the River Lossie.

Each span consists of 7 No. primary steel beams at 715mm centres acting compositely with a concrete jack arched slab. Each jack arch has an approximate clear span of 540mm and rise of 135mm with a corrugated steel shuttering to the underside. At quarter points of each span steel flat bar transverse ties are in place between the central five primary beams. These are hooked to the bottom flange of the primary steel beams with no other form of fixing evident.

Each primary steel beam has a clear span of approximately 6.6m between supports points. The width of the structure, between the parapets, is 4.3m with a carriageway width of approximately 3.8m.

Mass concrete abutments form the supporting substructure to the east and west with intermediate mass concrete piers forming the supporting substructure at third points of the total bridge span. Intermediate piers have triangular cutwaters both upstream and downstream. The upstream face of the cutwaters are finished with a flat plate, assumed to provide protection against potential impact damage. Where visible the concrete was noted to comprise rounded river gravel aggregate, indicating a low grade. All abutments and piers are rendered and scribed to give the impression of coursed masonry.

Mass concrete wing walls form the approach to the bridge from the east and west, with painted steel posts and rails forming the parapet to the bridge structure. Brace members tie the parapet to the underside of the bridge structure. The parapet structure, in its current form, is non-compliant and likely inadequate for vehicle restraint.

Principal inspections of the bridge were carried out in 1995, 1997 and 2019. Following the Principal Inspection in 2019, a revised vehicle weight limit of 3.0 tonnes, as well as vehicle height limit, was introduced to the bridge with overhead barriers installed to the east and west to deter non-compliant vehicles from crossing.

The following paragraphs will provide a summary of the defects to the main structural elements of the bridge structure following the Inspection for Assessment. The more detailed defects schedule can be found on drawing no. 140163F/01 and 140163F/02, included in Appendix B of this report.

4.2 Superstructure

The superstructure is in poor condition overall. The following paragraphs summarise the most significant defects to the main elements. The locations and extents of all the defects can be found in the defects schedule drawings included in Appendix B herein.

<u>Primary Steel Beams (PB1 – PB7)</u>

- There is widespread corrosion, delamination and layered rust build-up to entire length of exposed web and flange of PB1 and PB7 over all three spans as noted on defect schedule (Photograph 1, 2 & 3).

- There is widespread corrosion delamination and layered rust build-up to entire length of visible bottom flange of PB2 and PB6 over all three spans (Photograph 4).
- Visible section loss is present to flange of PB1 and PB7 over all three spans. Flange measuring 4-6mm at toe and, on average, 12.4mm at quarter point of overall flange width (Photograph 5 & 6).
- Widespread calcite stalactites are present to bottom flange of PB1 – PB7 indicating extensive water ingress through deck structure (Photograph 7 & 8).
- There is general moderate surface rust and pitting to soffit of bottom flange of internal beams (Photograph 9).
- The flaking paint to soffit of bottom flange of PB4 – PB6 in Span 1. Is indicative of condition where paint coating was only partially intact throughout all three spans. No paint coating was generally visible to PB1 and PB7 as rust layering and delamination was too extensive (Photograph 10 & 11).

Note that only the soffit of the bottom flange of PB2 – PB6 could be observed during the inspection due to the nature of the construction.

Concrete Jack-Arch Structure

- There is surface rust, delamination and corrosion to the corrugated steel shuttering to the soffit of the structure. This is widespread through the structure but predominantly found between PB1 – PB2 and PB6 – PB7 within each span (Photograph 12 & 13).
- Typical localised surface rust is present at the junction between the corrugated steel shuttering and primary steel beam bottom flange. This rust is also prevalent throughout the entire structure (Photograph 7 & 14).
- Localised failure of corrugated steel shuttering was observed at junction with PB3 within Span 1 (Photograph 15 & 16).
- The mass concrete to the jack arched slab is honeycombed with visible areas of rounded river gravel aggregate indicating the poor quality of the original concrete. (Photograph 17).

Note that the concrete jack arched slab could not be inspected across the entire structure due to presence of the corrugated steel shuttering.

Transverse Ties

- There is typical layered surface rust and delamination to all transverse ties. Areas with extensive corrosion were measured to be approximately 30mm deep with true section depth taken as 10mm within less corroded areas (Photograph 18).

Parapet and Carriageway

- Typical weathering with surface rust and failure of coating system to parapet rails and upstands has been noted (Photograph 19 & 20).
- Typical cracks are propagating from the junction between the steel parapet rails and the mass concrete approach wing walls (Photograph 21).
- There is widespread hairline cracking to approach wing walls (Photograph 22 & 23).

- There is evidence of vehicle collision to north-east approach wing wall with spalled concrete and widespread cracking to wall and cope (Photograph 24 & 25).
- A band of vegetation growth was observed within the drainage channel to the carriageway edge (Photograph 26 & 27).
- Typical drainage outlets are situated to north and south of bridge at mid-point of each span. One of the drainage outlets was cleared for the photograph but the remaining outlets were generally blocked by silt and vegetation throughout structure (Photograph 28).
- There is widespread corrosion, delamination and layered rust build-up to lower section of parapet brace member. A paint coating is visible to the upper section but appears to stop and is not visible to lower section (Photograph 29, 30 & 31).
- Vegetation is present due to nesting birds resting on parapet brace to east and west of Span 2 (Photograph 32).

4.3 Substructure

Foundations

The foundations could not be observed during the course of the inspection.

Invert

- There is a large scoured pool in the mass concrete invert below Span 2 with fast flowing water. The cut is located approximately 2.5m from east pier and approximately 2m from west pier. The extent of scour below invert is unclear due to the water level (Photograph 33). The fast flow and level drop associated with this scoured area is likely to be exacerbating the scour problem.
- There is an accumulation of vegetation to the upstream cutwater of Pier 2 (Photograph 33 & 34).

Abutments

- Graffiti is present along the entire face of both west and east abutments.
- A vertical crack emanating from the support position of PB5 to approximately 950mm above invert level to west abutment. The crack width was measured to be approximately 1mm (Photograph 35).
- A historic crack monitor was identified across vertical crack which is in a state of disrepair. There is no evidence of when crack monitor was installed (Photograph 35).
- There is significant scour to the render at base of west abutment (Photograph 36).
- Damp and algae staining is present to the face of the west abutment below the support positions of PB1 & PB7 (Photograph 37).
- Two 100mm diameter cores were identified to face of west abutment extending approximately 700mm back into structure (Photograph 37).
- There is significant calcite and algae staining to the face of east abutment below the support positions of steel beams (Photograph 38 & 39).

- Significant concrete spalling is present along top of east abutment between beams in each bay. The aggregate is exposed and loose when disturbed (Photograph 40).
- There is a significant concrete spalled area to south-east corner of east abutment leaving approach wing wall above partially unsupported (Photograph 41 & 42).
- There is scour to render at base of east abutment with exposed aggregate (Photograph 43).
- Typical mature vegetation growth is present to the east and west abutment (Photograph 44 & 45).

Intermediate Piers

- Graffiti is present to both faces of each intermediate pier.
- There is a horizontal crack propagating full width of both the western and eastern face of Pier 1. The height of the crack varies between 1.3m and 1.6m above invert level. The width of the crack varies along length but is less than 1mm at all times (Photograph 46, 47 & 48).
- A horizontal crack propagates the full width of both western and eastern face of Pier 2. The height of crack varies between 1.3m and 1.6m above invert level. The width of crack varies along the length but is less than 1mm at all times (Photograph 49, 50 & 51).
- Typical calcite staining is present to faces of both Pier 1 and 2 below support positions of steel beams (Photograph 48 & 51).
- Typical concrete spalling and cracking with vegetation growth is present to the decorative cutwater capping (Photograph 52 & 53).

4.4 Services

A single pipe was located to the south of the structure fixed to the parapet and approach walls using bracket connections. The connections are in a reasonable condition. The pipework was cracked in places and it was unclear from inspection whether services currently utilise the pipe run.

4.5 Waterproofing

The waterproofing, if present, is buried and was therefore not inspected. Owing to the significant water ingress through the bridge deck any waterproofing that is currently in place can be said to have failed.

5 Assessment

5.1 General

The assessment of the structure was undertaken in accordance with the Design Manual for Roads and Bridges (DMRB) and British Standards, in particular the documents in Table 3 below.

Table 2: Assessment Standards

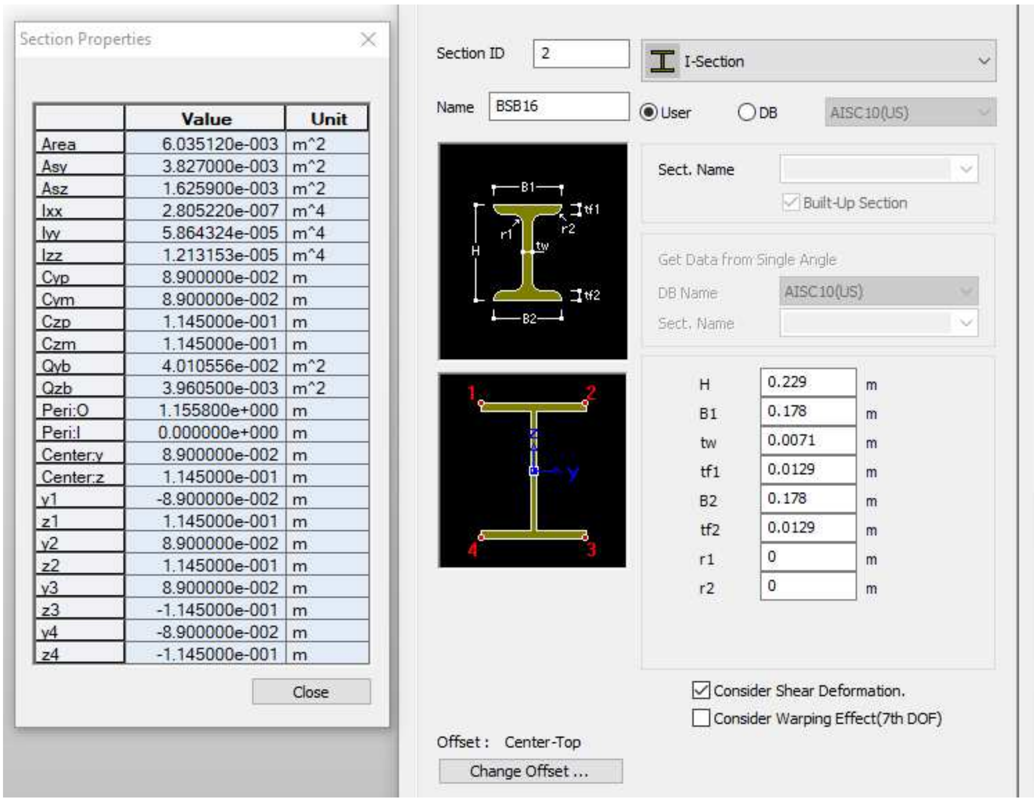
Ref	Title	Notes
CS 454	Assessment of Highway Bridges and Structures	
CS 456	The Assessment of Steel Highway Bridges and Structures	(read in conjunction with BS5400-3:2000 Steel, Concrete and Composite Bridges – Part 3:Code of Practice for design of Steel Bridges)
CS 459	The assessment of bridge substructure, retaining structures and buried structures.	

5.2 Material Strength & Assumptions

Section and material properties are based on suggested values in the Historical Structural Iron and Steel handbook BCSA 61/19, based on an estimated year of construction in 1905. It is noted that the Historical Structural Iron and Steel Sections document does not provide a yield strength for steels prior to 1948. For the purpose of the analysis calculation (which uses yield values), a comparative check was done against later steels and the same ratio for ultimate to yield strength was used to determine approximate yield values for the steel in the structure. This resulted in a yield stess of 230N/mm^2 for the steelwork.

For the purpose of capacity calculation, reduced section sizes are used based on measured values taken during the inspection for assessment for both internal and external beam types. Rust laminations were removed locally in order to take measurements of the residual steel sections.

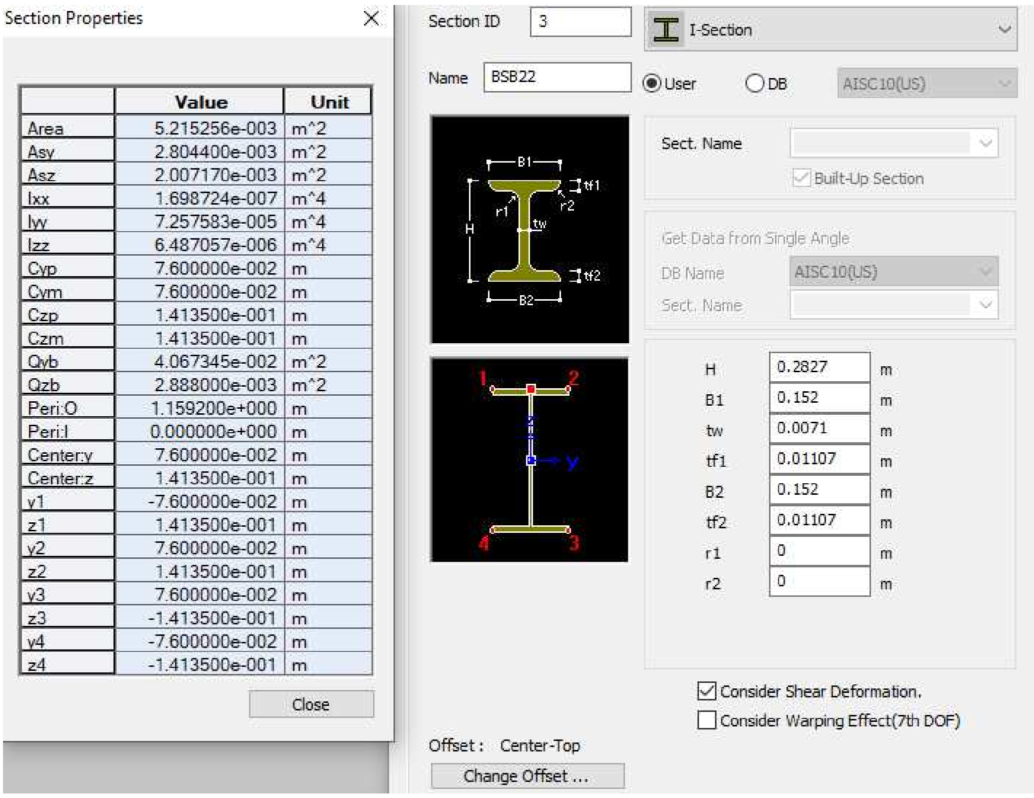
Due to the difficulty of measuring steel thicknesses of the web and top flange of the inner and outer beams, a similar level of corrosion is assumed throughout, based on the amount of corrosion that has taken place on the exposed flange. This is likely to be conservative for the assessment. Section properties for the inner beams and outer beams as used in calculation for capacity can be found in **Figures 2 and 3** respectively.



Section Properties

	Value	Unit
Area	6.035120e-003	m^2
Asy	3.827000e-003	m^2
Asz	1.625900e-003	m^2
Ixx	2.805220e-007	m^4
Iyy	5.864324e-005	m^4
Izz	1.213153e-005	m^4
Cyp	8.900000e-002	m
Cym	8.900000e-002	m
Czp	1.145000e-001	m
Czm	1.145000e-001	m
Qyb	4.010556e-002	m^2
Qzb	3.960500e-003	m^2
Peri:O	1.155800e+000	m
Peri:I	0.000000e+000	m
Center:y	8.900000e-002	m
Center:z	1.145000e-001	m
y1	-8.900000e-002	m
z1	1.145000e-001	m
y2	8.900000e-002	m
z2	1.145000e-001	m
y3	8.900000e-002	m
z3	-1.145000e-001	m
y4	-8.900000e-002	m
z4	-1.145000e-001	m

Figure 2: Section Properties Inner Beam (BSB 16)



Section Properties

	Value	Unit
Area	5.215256e-003	m^2
Asy	2.804400e-003	m^2
Asz	2.007170e-003	m^2
Ixx	1.698724e-007	m^4
Iyy	7.257583e-005	m^4
Izz	6.487057e-006	m^4
Cyp	7.600000e-002	m
Cym	7.600000e-002	m
Czp	1.413500e-001	m
Czm	1.413500e-001	m
Qyb	4.067345e-002	m^2
Qzb	2.888000e-003	m^2
Peri:O	1.159200e+000	m
Peri:I	0.000000e+000	m
Center:y	7.600000e-002	m
Center:z	1.413500e-001	m
y1	-7.600000e-002	m
z1	1.413500e-001	m
y2	7.600000e-002	m
z2	1.413500e-001	m
y3	7.600000e-002	m
z3	-1.413500e-001	m
y4	-7.600000e-002	m
z4	-1.413500e-001	m

Figure 3: Section Properties Outer Beam (BSB 22)

5.3 Loading

5.3.1 Permanent Loads

The following permanent loads have been considered:

(i) Dead Load

(ii) Superimposed Dead Load

Permanent actions acting on the structure were determined in accordance with CS 454 of the DMRB.

5.3.2 Live Loads

Snow loads and wind loads were ignored. Thermal effects were also ignored as they are unlikely to be critical actions for this type of structure.

Vehicle loads have been applied in accordance with CS 454 and included 3 tonne, 7.5 tonne, 18 tonne, footway and Group 1 Fire Engines and Group 2 Fire Engine live loads.

Pedestrian live loading applied to the full width of the bridge deck was also considered in the assessment.

5.4 Analysis

5.4.1 Modelling

As the structure is composed of 3 simply supported spans, the analysis uses a worst case single span to represent the bridge as a whole. An extract of the visualised model can be seen in Figure 4.The modelling of the structure was undertaken in proprietary finite element analysis software using the stiffness matrix method. Line beam elements to represent the beams were combined with plate finite elements to represent the deck and jack arches in the model. Soffit ties have been ignored for the purposes of this analysis.

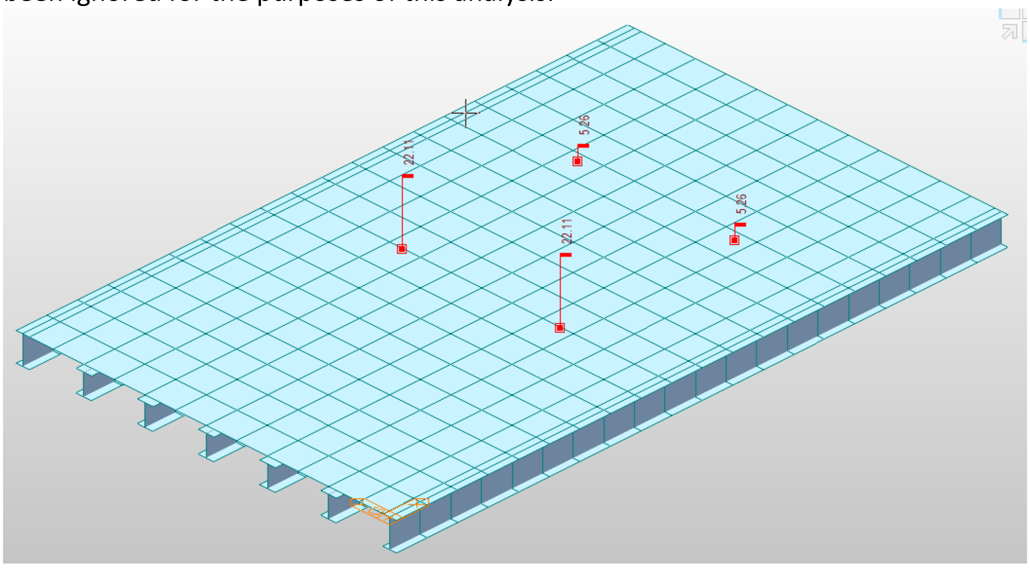


Figure 4: Vehicle load application (3 Tonne)

No information is available on the quality of the concrete, presence of reinforcement or presence of any shear connectors onto the beams. The concrete deck in combination with the jack arches are assumed to take a role only in distributing the load to the beams and has been assessed qualitatively as adequate in this role.

Original section sizes have been used to generate a conservative value for the self-weight of the beams. Considerable corrosion has taken place throughout the entirety of the structure, although this is not consistent across the different spans and the assessment is therefore conservative. The original, gross section sizes have been included in the model.

5.4.2 Analysis

Analysis of the structure has been completed using proprietary finite element analysis software, hand calculation and excel spreadsheets. Extracts from the model can be seen in Figures 5 to 7. The model is showing that there is some distribution transversely, and that the model is behaving as expected. The magnitudes of the load effects observed in the model have been verified and confirmed using hand calculations.

Corroded section properties assumed for derivation of bending moment capacity are as summarised below:

- For shear calculation, webs in the inner beams are considered as corroded to 6mm thick.
- For shear calculation, webs in outer beams are considered affected by corrosion, and are assumed to be 7.1mm thick, as noted in the worst measured section near supports
- Residual tension flange thickness for inner beams has been taken as 12.9mm from the original 15mm. Thinner sections of flange are recorded, but these are noted near the hogging points where tension in the exposed flange is negligible.
- Outer beam moment calculations assume a remaining flange thickness of 9.9mm from the original 15mm, as measured at midspan of the northernmost beam on the East span. This is assumed equal for both tension and compression flange.

The presence of concrete around the beam and the concrete deck means that any potential for lateral movement in the compression flanges, and therefore buckling effects, will be ignored for assessment purposes.

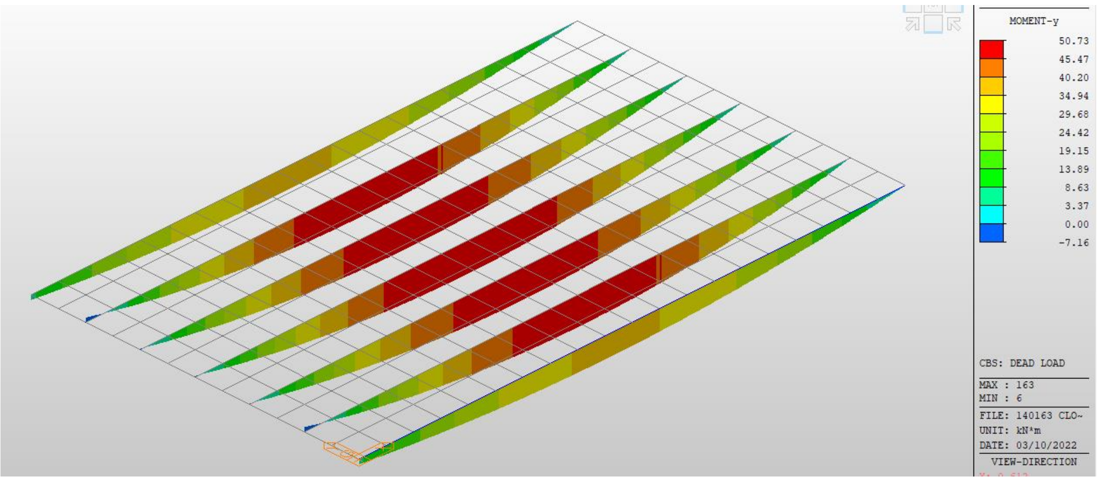


Figure 5: Dead Load beam bending moment diagram

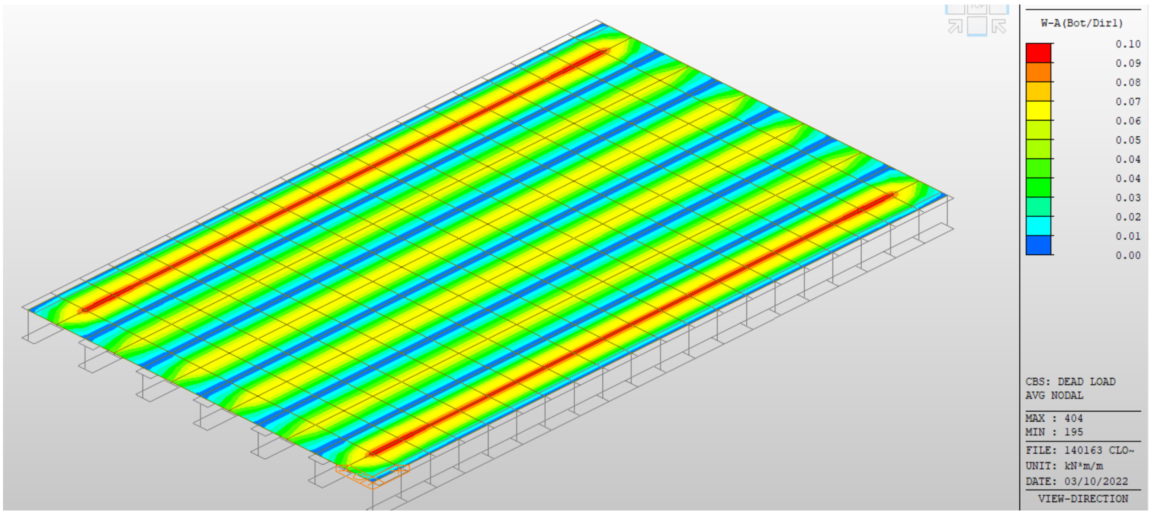


Figure 6: Dead Load slab bending moment diagram

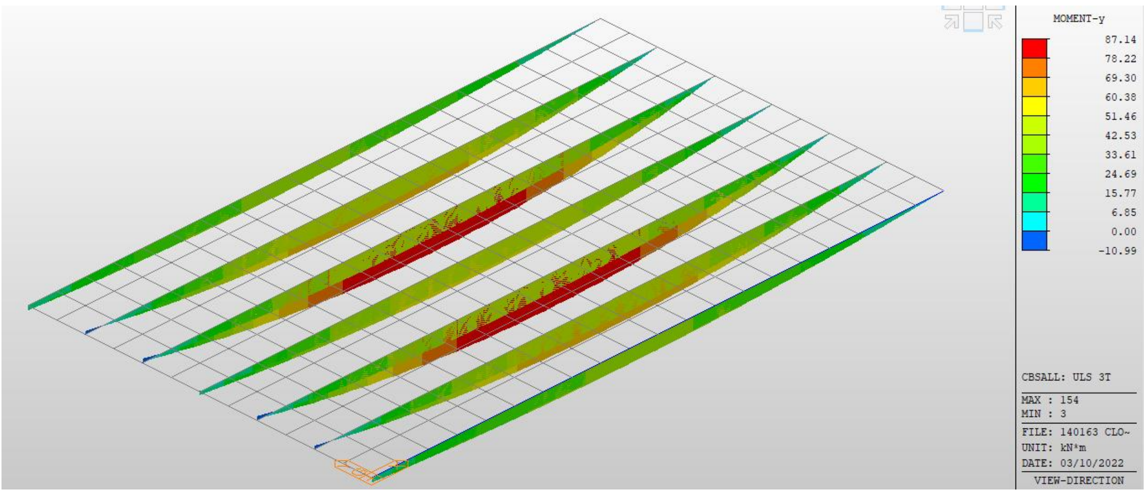


Figure 7: Vehicle load Beam Diagram (3 Tonne)

5.5 Substructure

The substructure has been assessed qualitatively based on the inspection and record information available. The scour evident to the river bed and abutment areas is of concern, particularly as it was not possible to determine the extents of the scoured area and there is a possibility that the undermined area extends beyond and underneath the structure supports.

The mass concrete abutments and piers are in a poor condition. There are significant defects and little evidence that the original construction workmanship was of good quality. It is not considered that the substructure would be suitable for reuse as part of a superstructure replacement scheme without major repair works. . The substructure has been qualitatively assessed to have a capacity limited to that of the deck.

6 Results and Discussion

6.1 Superstructure Results

The following is the summary of the results and rating for the two major elements.

Table 3: Summary of Results for All Elements

Element	Governing Load Effect	Utilisation	Result	Rating
Inner Beams	Bending Moment	95%	PASS	Group 2 Fire Engines
Edge Beams	Bending Moment	95%	PASS	Footway/ Pedestrian

The assessment indicates that the structure is inadequate for all vehicle loading. The critical element was found to be the outer girders, which when assessed using a conservative residual thickness as measured on site, indicated that the structure cannot safely support its own weight.

The corrosion loss to the inner girders was less severe, and the resulting assessment rating to these elements would allow the passage of 3T vehicles and Group 2 Fire Engines. In reality there is no way to restrict the distribution of loading between the inner and outer beams, and therefore the capacity of the bridge is limited by the outer beams.

The residual steelwork thicknesses measured on site were variable. The section used to determine the assessed capacity was based on measured section thicknesses from the midspan area. There were other areas of the flange measured which were found to be more severely corroded, however, the midspan values have been used in order to provide the capacity coincident to the worst case bending effects. Thinner flange sections were generally observed nearer to the supports, where load effects would be significantly less.

6.1.1 Inner Beams

The analysis has shown that the inner beams passed for loads up to Group 2 Fire Engines Assessment Live Load, which represents a small fire service vehicle.

Table 4: Detailed Summary of Results for Inner Beams

Element	Load Effect	Bending Utilisation	Rating
Inner Beam	Dead Load	52%	Pass
Inner Beam	3T ALL	88%	Pass
Inner Beam	Pedestrian Live Load	79%	Pass
Inner Beam	G2FE ALL	95%	Pass
Inner Beam	7.5T ALL	126%	Fail
Inner Beam	G1FE ALL	138%	Fail
Inner Beam	18T ALL	194%	Fail

6.1.2 Outer Beams

The analysis has shown that the outer beams are adequate for pedestrian live load, and therefore should be closed to any vehicle traffic.

Table 6: Detailed Summary of Results for Outer Beams

Element	Load Effect	Utilisation	Rating
Outer Beam	Dead Load	62%	Pass
Outer Beam	3T ALL	105%	Fail
Outer Beam	Pedestrian Live Load	95%	Pass
Outer Beam	Group 2 Fire Engines ALL	114%	Fail
Outer Beam	7.5T ALL	151%	Fail
Outer Beam	Group 1 Fire Engines ALL	165%	Fail
Outer Beam	18T ALL	232%	Fail

The assessment was completed using measured section properties at the worst measured point in the midspan of outer beams.

The overall assessment result is considered to be conservative as in reality a certain amount of re-distribution is possible between the beams, given the relatively small span between the two beams, and the sheet metal which is present under the jack arch. It is noted that the benefits of composite working have not been considered between the beams and the concrete the natural friction between the steel and concrete will allow for some limited transfer of load between the two materials.

7 Options

The bridge analysis indicates that the structure is inadequate for dead loading. A short term full closure is therefore recommended as an initial step.

The scour evident to the river bed and abutment areas is of concern and it would be recommended that no vehicles are allowed to approach or cross the bridge substructure.

The nature of the failure being related to the edge beam means that there is some scope for considering limited ongoing use for pedestrians and cyclists, although this would be in conjunction with regular monitoring and repair works.

All costs are indicative only and do not include any allowance for VAT, risk and inflation.

7.1 Option 1 – Stop-Up and Monitor

This option would be a 'do minimum' approach, and measures would include;

- Installation of bollards and signage on the approach to the structure to prevent vehicle access.
- Road Order to legally 'stop up' the road.
- Maintain access for pedestrians and cyclists
- Ongoing general inspections on a monthly basis, and after heavy rainfall, to monitor the condition of the steelwork and scour.
- Monitoring of the measured flange thicknesses every three months to ensure residual thickness does not reduce by more than 2mm.

Without further measures and subject to ongoing inspections, this could allow the bridge to be used by pedestrians and cyclists for a further 2 years. After this point repairs and refurbishment would be recommended.

Allowing £1000 /month for ongoing inspections and £15,000 for installation of bollards the required budget for this option is estimated to be around £50,000 over the next 2 years. After this time, a further quantitative assessment should be undertaken to re-establish the capacity of the structure.

7.2 Option 2 – Stop Up and Demolish

This option would also require monitoring of the structure if it was to be used in the short term. Recommended measures would include;

- Installation of bollards and signage on the approach to the structure to prevent vehicle access.
- Road Order to legally 'stop up' the road.
- Maintain access for pedestrians and cyclists on a temporary basis.
- Ongoing general inspections on a monthly basis, and after heavy rainfall, to monitor the condition of the steelwork and scour.

- Monitoring of the measured flange thicknesses every three months to ensure residual thickness does not reduce by more than 2mm.

Without further measures and subject to ongoing inspections, this could allow the bridge to be used by pedestrians and cyclists for a further 2 years to allow a scope and budget for demolition to be developed.

Demolition could either involve removal of the superstructure, or removal of both the substructure and superstructure. Limiting the demolition works to the superstructure would have benefits in minimizing costs and programme duration as works within the watercourse including associated licensing requirements from SEPA would be reduced.

However, leaving the existing piers in place would result in the council retaining liability for these elements. As the scour issues associated with the piers would remain, there is a potential that the substructure could collapse and block the watercourse, with associated implications for exacerbation of local flooding effects.

Allowing £1000 /month for ongoing inspection and £25,000 for installation of bollards the required budget for this option is estimated to be around £50,000 over the next 2 years.

Demolition of the superstructure is likely to cost in the region of £40,000. The inclusion of the substructure would increase the likely budget costs to approximately £120,000.

7.3 Option 3 – Repairs to Allow Ongoing Pedestrian and Cyclist Use

This option would be to undertake repair and refurbishment to allow the bridge to be safely used in the longer term by pedestrians and cyclists

- Installation of bollards and signage on the approach to the structure to prevent vehicle access.
- Road Order to legally ‘stop up’ the road.
- Undertake scour survey and undertake design of river bed training/repairs.
- Prepare a scope of works for a contractor to include grit blasting and repainting of all steelwork, sampling and detailed measurement of steelwork sections following grit-blasting.
- Update the assessment based on more accurate steelwork measurements in order to determine any strengthening requirements and prepare a scope to overplate the flanges of the outer girders.

There is an element of risk with this option, as grit blasting the steelwork may reveal further areas of deterioration that could not be observed previously. River bed surveys and scour repairs will also require input from SEPA that would extend any durations associated with the option and increase costs.

The budget cost for this option is considered to be in the region of £250,000, with a likely extension to service life of approximately 10 years.

7.4 Option 4 – Repairs to Allow Vehicle Use

The overall condition of the bridge, quality of concrete observed and scour issues suggest that there will be little benefit in undertaking an extensive repair works scheme. A full refurbishment scheme to allow safe vehicle use would involve: Bridge strengthening would allow the bridge to be used by normal traffic up to 40t.

- Scour survey and development of extensive river bank protection and repairs.
- As the option involving the most work in the river, this would require extensive liaison with SEPA. Licensing requirements would likely be highly restrictive.
- Development of strengthening scheme to the structure including cleaning, overplating/replacement and painting to all steel beams.
- Poor quality existing concrete is likely to result in repair works being more significant than indicated by visual inspection.
- Removal of concrete jack arches and replacement with new structural slab.
- Installation of a new vehicle compliant parapet.

The budget cost for this option would be 1,750,000 with an estimated extension to service life of up to 50 years.

It is noted that there is a significant amount of risk associated with this option as the condition of the structural elements retained is not known in full. The extent of repairs required to extend the life to this extent is likely to be very significant. For example, any breaking out of the concrete substructure is likely to reveal additional deterioration, and when concrete quality is poor any breaking out can be difficult to control on site. This may result in the full substructure being essentially reconstructed in situ. There is a similar risk with the superstructure. The resulting structure would still be limited in terms of capacity for heavy vehicles and ongoing durability due to the ongoing risk associated with any retained parts of the original structure.

This option also comes with the additional complexity of needing to ensure the temporary stability of the existing structural elements during any refurbishment works. This could require extensive temporary works and highly constrained sequencing which is likely to increase costs.

This option is therefore not recommended to be taken forward.

7.5 Option 5 – Demolition and Replacement

Any full replacement of the structure would be recommended to be a single span structure, possibly of steel composite or prestressed concrete beam construction.

Based on recent similar projects, the demolition of the existing structure and provision of a new bridge with 120 year design life would require a total budget of approximately £2,000,000.

8 Conclusions

It is recommended that the bridge remains closed to all vehicle traffic.

It is considered that with the installation of bollards and the implementation of a regular monitoring regime, that the bridge could be continued to be used by pedestrians and cyclists.

In the longer term a demolition and/or full single span replacement of the structure would be recommended.

Appendix A
Inspection Photographs

Photograph 1: Widespread corrosion, delamination and layered rust build-up to entire length of exposed web and flange of PB1 & PB7 over all three spans.

Photograph 2: Widespread corrosion, delamination and layered rust build up to entire length of exposed web and flange of PB1 & PB7 over all three spans.

Photograph 3: Widespread corrosion, delamination and layered rust build up to entire length of exposed web and flange of PB1 & PB7 over all three spans.

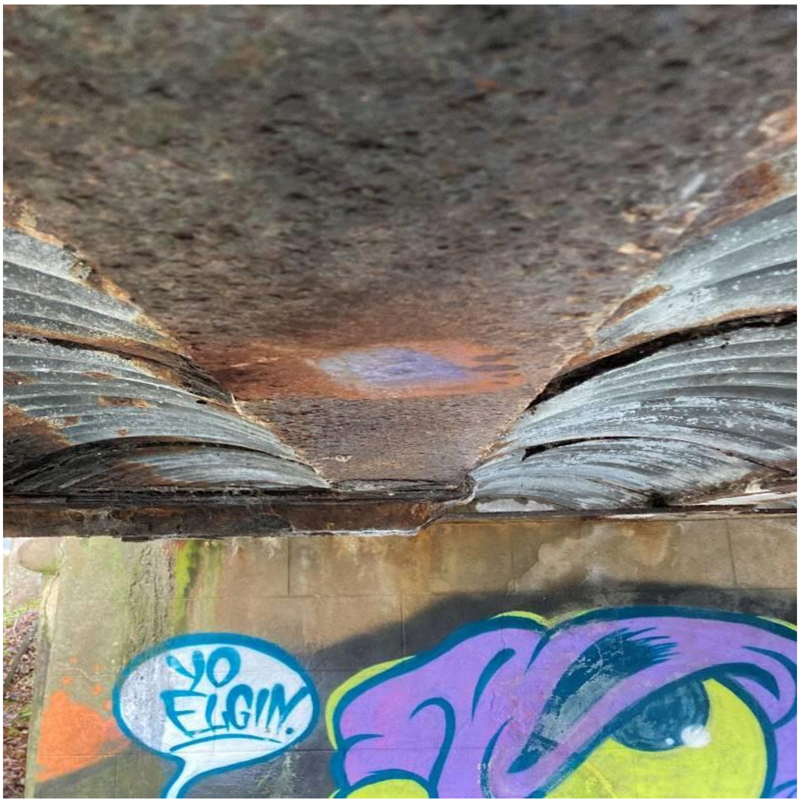
Photograph 4: Widespread corrosion, delamination and layered rust build up to entire length of visible bottom flange of PB2 & PB6 over all three spans.

Photograph 5: Visible section loss to flange of PB1 & PB7 over all three spans. Flange measuring 4-6mm at toe and, on average, 12.4mm at quarter points of overall flange width.

Photograph 6: Widespread corrosion, delamination and layered rust build up to entire length of exposed wed and flange of PB1 & PB7 over all three spans.

Photograph 7: Widespread calcite stalactites to bottom flange of PB1 – PB7 indicating extensive water ingress through deck structure. Typical localised surface rust at junction between corrugated steel shuttering and steel beam bottom flange.

Photograph 8: Widespread calcite stalactites to bottom flange of PB1 – PB7 indicating extensive water ingress through deck structure.



Photograph 9: Typical defect to internal beams - moderate surface rust and pitting to soffit of bottom flange (PB6, Span 1 shown).

Photograph 10: Flaking paint to soffit of bottom flange of PB4-PB6 in Span 1. Indicative of condition where paint coating was partially intact throughout all three spans.

Photograph 11: Flaking paint to soffit of bottom flange of PB4-PB6 in Span 1. Indicative of condition where paint coating was partially intact throughout all three spans.

Photograph 12: Surface rust, delamination and corrosion to the corrugated steel shuttering forming the soffit of the structure. Predominant in arches between PB1 - PB2 and PB6 - PB7.

Photograph 13: Surface rust, delamination and corrosion to the corrugated steel shuttering forming the soffit of the structure. Predominant in arches between PB1 - PB2 and PB6 - PB7.

Photograph 14: Typical localised surface rust at junction between corrugated steel shuttering and steel beam bottom flange.


Photograph 15: Localised failure of corrugated steel shuttering at junction with PB3 within Span 1.


Photograph 16: Localised failure of corrugated steel shuttering at junction with PB3 within Span 1.

Photograph 17: Mass concrete jack arched slab visible with rounding river gravel aggregate indicating a low grade concrete mix.

Photograph 18: Typical layered surface rust and delamination to all transverse ties.

Photograph 19: Typical weathering, surface rust and failure of coating system to parapet rails and upstands.

Photograph 20: Typical weathering, surface rust and failure of coating system to parapet rails and upstands.

Photograph 21: Typical cracks propagating from the junction between the steel parapet rails and mass concrete approach wing walls.

Photograph 22: Typical widespread hairline cracking to approach wing walls.

Photograph 23: Typical widespread hairline cracking to approach wing walls.

Photograph 24: Evidence of vehicle collision to north-east approach wing wall with spalled concrete and widespread cracking to wall and cope.

Photograph 25: Evidence of vehicle collision to north-east approach wing wall with spalled concrete and widespread cracking to wall and cope.

Photograph 26: Typical band of vegetation growth within drainage channel to carriageway edge.

Photograph 27: Typical band of vegetation growth within drainage channel to carriageway edge.

Photograph 28: Typical drainage outlet situated to north and south of bridge at mid-point of each span. Typically blocked by silt and vegetation.

Photograph 29: Widespread corrosion, delamination and layered rust build-up to lower section of parapet brace member. Paint coating visible to upper section stops and is not visible to lower section.

Photograph 30: Widespread corrosion, delamination and layered rust build-up to lower section of parapet brace member.

Photograph 31: Widespread corrosion, delamination and layered rust build-up to lower section of parapet brace member.

Photograph 32: Vegetation due to nesting birds resting on parapet brace to east and west of Span 2.

Photograph 33: Visible cut in concrete invert below Span 2 with fast flowing water. Extent of scour below invert unclear. Accumulation of vegetation to upstream cutwater of Pier 2.

Photograph 34: Accumulation of vegetation to upstream cutwater of Pier 2.

Photograph 35: Vertical crack emanating from the support position of PB5 to approximately 950mm above invert level to west abutment. Crack width measured to be approximately 1mm. Historic crack monitor in state of disrepair.

Photograph 36: Typical scour to render at base of west abutment.

Photograph 37: Damp and algae staining to face of west abutment below support position of PB1 and PB7. Two 100mm diameter cores noted to face of abutment extending 700mm back into structure.

Photograph 38: Significant calcite and algae staining to face of east abutment below support positions of steel beams.

Photograph 39: Significant calcite and algae staining to face of east abutment below support positions of steel beams.

Photograph 40: Significant concrete spalling along top of east abutment between beams in each bay. Aggregate exposed and loose when disturbed.

Photograph 41: Significant concrete spalling to south-east corner of east abutment leaving approach wing wall partially unsupported.

Photograph 42: Significant concrete spalled to south-east corner of east abutment leaving approach wing wall partially unsupported.

Photograph 43: Typical scour to base at base of east abutment with exposed aggregate.

Photograph 44: Typical mature vegetation growth to east and west abutment.

Photograph 45: Typical mature vegetation growth to east and west abutment.

Photograph 46: Horizontal crack propagating full width of western face of Pier 1. Height of crack varies between 1.3m and 1.6m above invert level. Width of crack varies along length but is less than 1mm at all times.

Photograph 47: Horizontal crack propagating full width of western face of Pier 1. Height of crack varies between 1.3m and 1.6m above invert level. Width of crack varies along length but is less than 1mm at all times.

Photograph 48: Horizontal crack propagating full width of eastern face of Pier 1. Height of crack varies between 1.3m and 1.6m above invert level. Width of crack varies along length but is less than 1mm at all times. Typical calcite staining to face of Pier 1 and Pier 2.

Photograph 49: Horizontal crack propagating full width of western face of Pier 2. Height of crack varies between 1.3m and 1.6m above invert level. Width of crack varies along length but is less than 1mm at all times.

Photograph 50: Horizontal crack propagating full width of western face of Pier 2. Height of crack varies between 1.3m and 1.6m above invert level. Width of crack varies along length but is less than 1mm at all times.

Photograph 51: Horizontal crack propagating full width of eastern face of Pier 2. Height of crack varies between 1.3m and 1.6m above invert level. Width of crack varies along length but is less than 1mm at all times. Typical calcite staining to face of Pier 1 and Pier 2.

Photograph 52: Typical concrete breakout and cracking with vegetation growth to decorative cutwater capping.

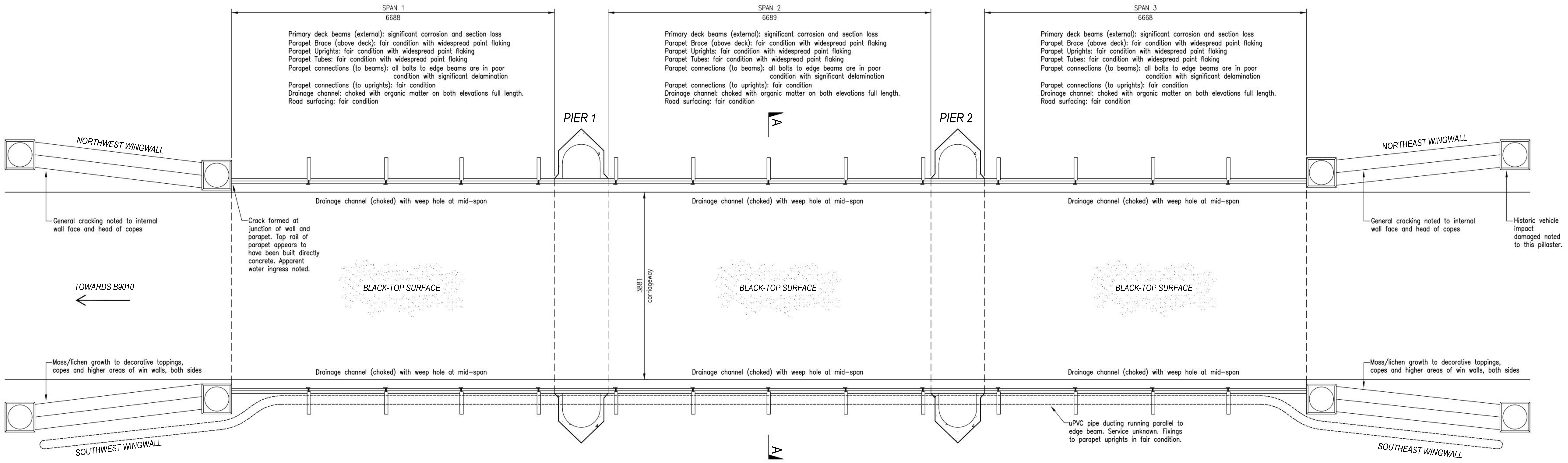
Photograph 53: Typical concrete breakout and cracking with vegetation growth to decorative cutwater capping.

Photograph 54: Upstream elevation.

Photograph 55: Downstream elevation.

Appendix B
Drawings

Drawing No	Title
140163F-01	Cloddach Bridge Defects Schedule 1 of 2
140163F-02	Cloddach Bridge Defects Schedule 2 of 2



PLAN LAYOUT OF BRIDGE DECK SURFACING (1:50)

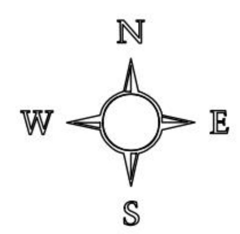
KEY FOR DEFECTS SCHEDULING :

Hatch Type	Defect observed	Localised/Widespread
(red line)	Significant section corrosion and section loss to members. Delaminating and layered rusting.	Widespread
(yellow line)	Mild corrosion to members with loss of protective paint and minimal pitting to exposed steel	Widespread

KEY FOR CONCRETE CRACKING :

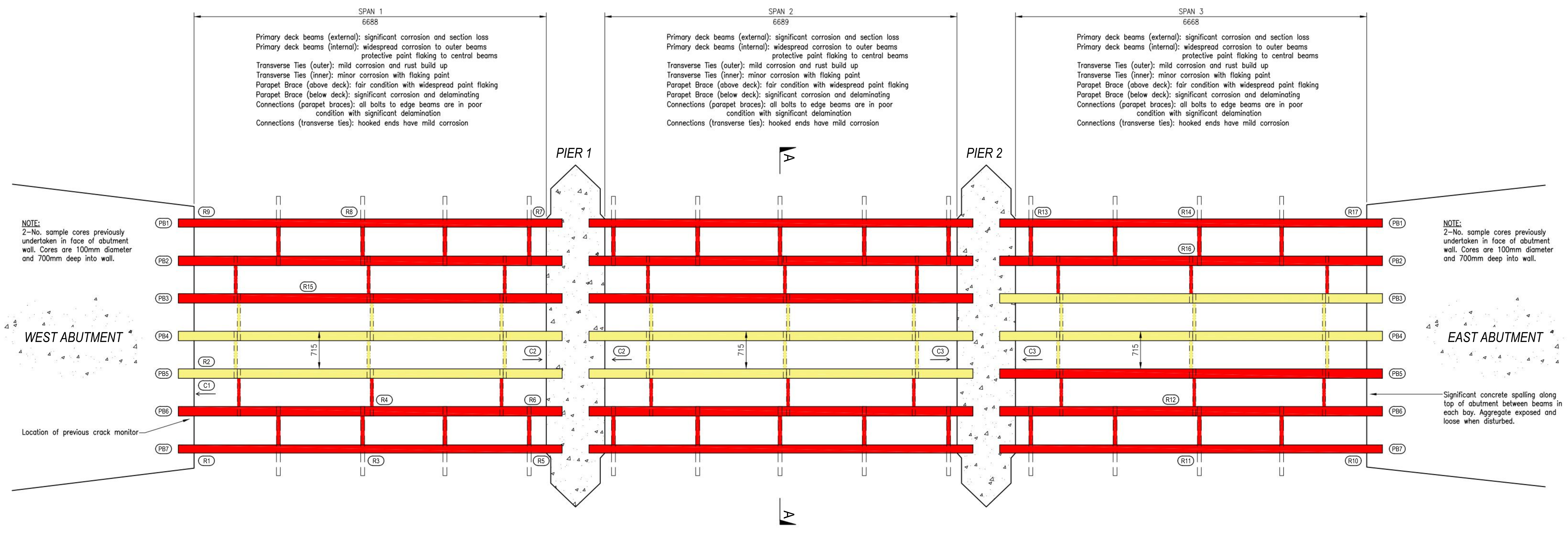
Location	Description	Thickness (approx.)
C1	Vertical crack propagating from soffit of beam (PB5) down to approximately 950mm above ground.	1mm
C2	Horizontal crack running full width of pier between 1.3m & 1.6m above ground.	Varies <1mm
C3	Horizontal crack running full width both sides of pier between 1.3m & 1.6m above ground.	Varies <1mm

FLOW OF RIVER LOSSIE



STEELWORK MEASUREMENTS FROM MAIN DECK MEMBERS :

Location	Reading from bottom flange (mm) taken at quarter points (u.n.o)	Reading from web (mm)
R1	12.7	9.9
R2	11.4	Not Accessible
R3	11.5	11.5
R4	11.1	Not Accessible
R5	18.1 (mid)	5.8
R6	15.4 - 16.5 (mid)	Not Accessible
R7	12.7	13.0
R8	9.9	12.5
R9	10.6	8.9
R10	12.4	8.8 - 10.1
R11	13.0	10.9
R12	12.9	Not Accessible
R13	11.5	13.5
R14	12.0	11.4
R15	16.6	Not Accessible
R16	18.5 (mid)	Not Accessible
R17	12.1	7.1



PLAN LAYOUT OF BRIDGE DECK STRUCTURE (1:50)

USE GIVEN DIMENSIONS SHOWN ON THIS DRAWING.

SAFETY HEALTH AND ENVIRONMENTAL INFORMATION:

IN ADDITION TO THE HAZARD/RISKS NORMALLY ASSOCIATED WITH THE TYPES OF WORK DETAILED ON THIS DRAWING, NOTE THE FOLLOWING RISKS AND INFORMATION.

RISKS LISTED HERE ARE NOT EXHAUSTIVE.
REFER TO DESIGN ASSESSMENT FORM NUMBER: 140163F–DAF–01

CONSTRUCTION:
THERE ARE NO SIGNIFICANT HAZARDS THAT WILL NOT BE APPARENT TO A COMPETENT CONTRACTOR.

DEMOLITION:
NOT APPLICABLE

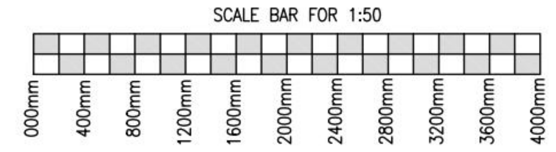
FOR INFORMATION RELATING TO USE, CLEANING AND MAINTENANCE SEE THE HEALTH AND SAFETY FILE

IT IS ASSUMED THAT ALL WORKS WILL BE CARRIED OUT BY A COMPETENT CONTRACTOR WORKING, WHERE APPROPRIATE, TO AN APPROVED METHOD STATEMENT.

GENERAL NOTES :

1. ALL DIMENSIONS SHOWN ARE IN MILLIMETERS.
2. BRIDGE SURVEY/INSPECTION WAS CARRIED OUT ON MONDAY 14TH FEBRUARY 2022.

SCALE BAR FOR 1:50



REV.	DATE	DESCRIPTION	DRWN.	CHKD.	APPD.

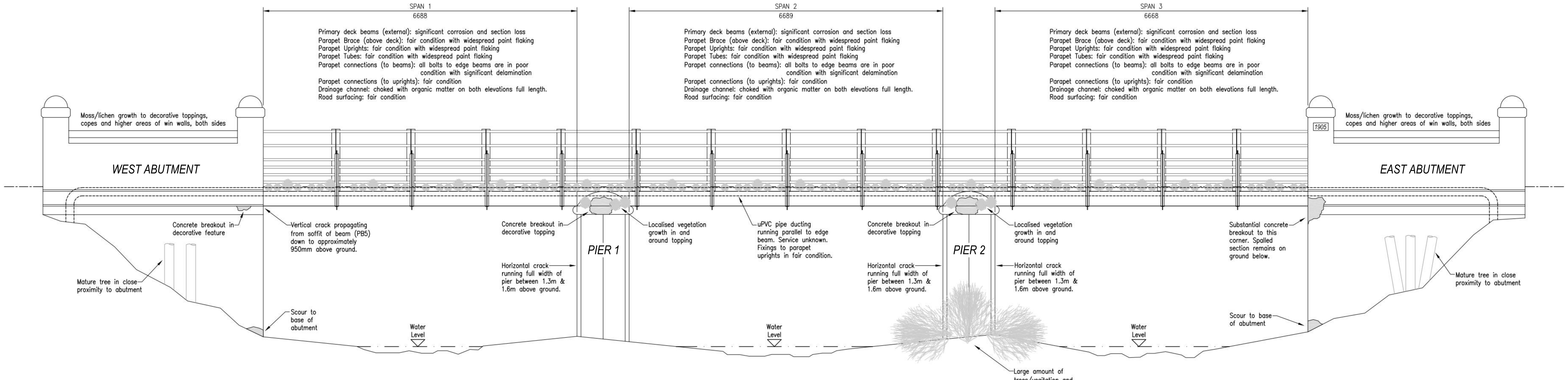
FAIRHURST
25a HIGH STREET, ELGIN, IV30 1EE
TELEPHONE: 01343 543876
FAX: 0844 381 4412

CLIENT :
THE MORAY COUNCIL
ECONOMY, ENVIRONMENT & FINANCE
PO BOX 6760
ELGIN IV30 1BX

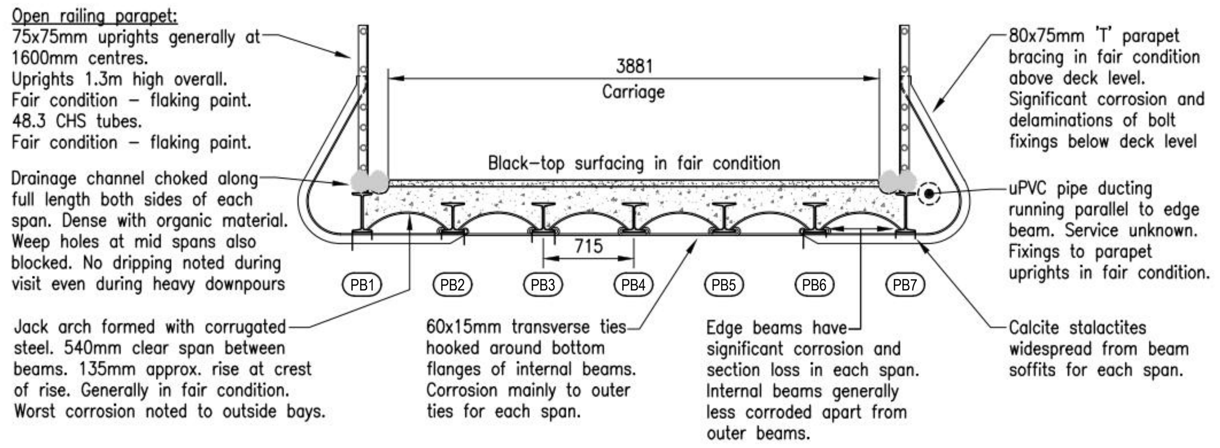
PROJECT TITLE :
ASSESSMENT OF
CLODDACH BRIDGE
BIRNIE ELGIN

DRAWING TITLE :
DEFECTS SCHEDULE
(SHEET 1 OF 2)

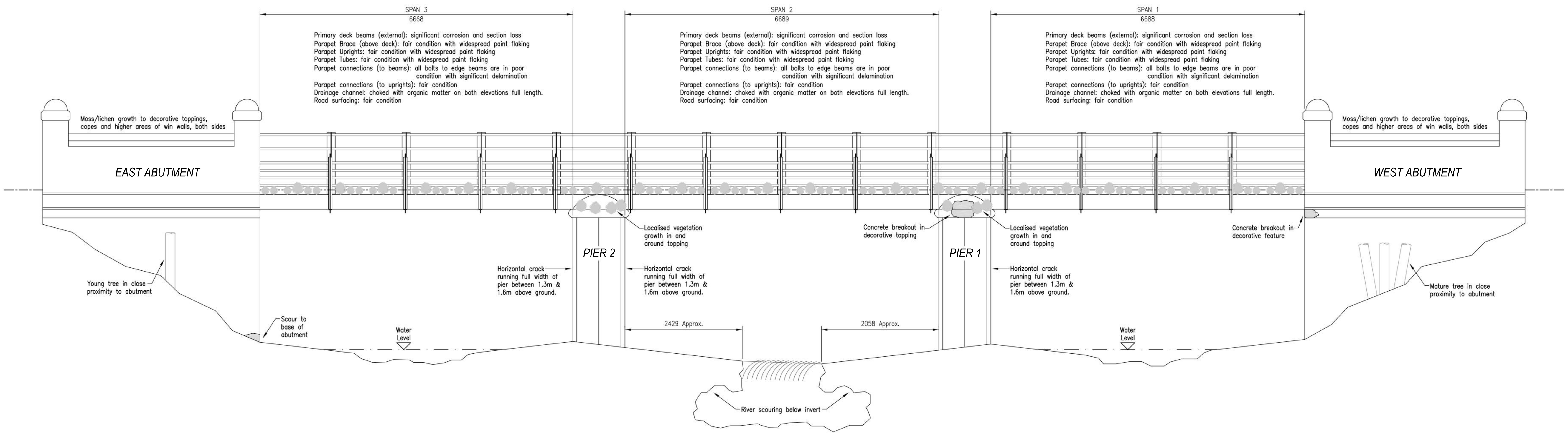
SCALE AT A1 / A3 : 1:50 / 1:100	DRAWING STATUS : INFORMATION		
DESIGNED :	DRAWN : CC	CHECKED : MW	APPROVED : GS
DATE :	DATE : 17/02/2022	DATE : 18/02/2022	DATE : 18/02/2022
DRAWING NUMBER : 140163F/01			REVISION : –



UPSTREAM ELEVATION (1:50)



SECTION 'A-A' :
TYPICAL SECTION THROUGH BRIDGE DECK (1:50)



DOWNSTREAM ELEVATION (1:50)

USE GIVEN DIMENSIONS SHOWN ON THIS DRAWING.

SAFETY HEALTH AND ENVIRONMENTAL INFORMATION:

IN ADDITION TO THE HAZARD/RISKS NORMALLY ASSOCIATED WITH THE TYPES OF WORK DETAILED ON THIS DRAWING, NOTE THE FOLLOWING RISKS AND INFORMATION.

RISKS LISTED HERE ARE NOT EXHAUSTIVE.
REFER TO DESIGN ASSESSMENT FORM NUMBER: 140163F–DAF–01

CONSTRUCTION:
THERE ARE NO SIGNIFICANT HAZARDS THAT WILL NOT BE APPARENT TO A COMPETENT CONTRACTOR.

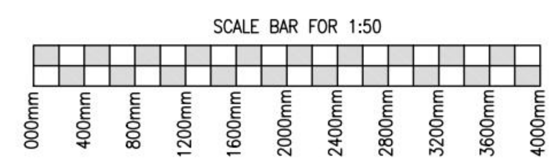
DEMOLITION:
NOT APPLICABLE

FOR INFORMATION RELATING TO USE, CLEANING AND MAINTENANCE SEE THE HEALTH AND SAFETY FILE

IT IS ASSUMED THAT ALL WORKS WILL BE CARRIED OUT BY A COMPETENT CONTRACTOR WORKING, WHERE APPROPRIATE, TO AN APPROVED METHOD STATEMENT.

GENERAL NOTES :

1. ALL DIMENSIONS SHOWN ARE IN MILLIMETERS.
2. BRIDGE SURVEY/INSPECTION WAS CARRIED OUT ON MONDAY 14TH FEBRUARY 2022.



REV.	DATE	DESCRIPTION	DRWN.	CHKD.	APPD.

FAIRHURST
25a HIGH STREET, ELGIN, IV30 1EE
TELEPHONE: 01343 543876
FAX: 0844 381 4412

CLIENT :
THE MORAY COUNCIL
ECONOMY, ENVIRONMENT & FINANCE
PO BOX 6760
ELGIN IV30 1BX

PROJECT TITLE :
ASSESSMENT OF
CLODDACH BRIDGE
BIRNIE ELGIN

DRAWING TITLE :
DEFECTS SCHEDULE
(SHEET 2 OF 2)

SCALE AT A1 / A3 : 1:50 / 1:100	DRAWING STATUS : INFORMATION		
DESIGNED :	DRAWN : CC	CHECKED : MW	APPROVED : GS
DATE :	DATE : 17/02/2022	DATE : 18/02/2022	DATE : 18/02/2022
DRAWING NUMBER : 140163F/02			REVISION : –

Appendix C
Approval in Principle (AIP)

Approval in Principle

Moray Council

Cloddach Bridge Assessment

February 2022

CONTROL SHEET

CLIENT: The Moray Council

PROJECT TITLE: C2E/20 Cloddach Bridge Assessment

REPORT TITLE: Approval in Principle

PROJECT REFERENCE: 140163

DOCUMENT NUMBER: 140163/AIP/01

STATUS: Final

Issue & Approval Schedule	ISSUE 0	Name	Signature	Date
	Prepared by	Richard de Groot		01-03-2022
	Checked by	Stewart McLaren		1st March 2022
	Approved by	Ellen Halkon		1st March 2022

Revision Record	Rev.	Date	Status	Description	Signature	
	1				By	
					Checked	
					Approved	
	2				By	
					Checked	
					Approved	

This document has been prepared in accordance with procedure OP/P02 of the *Fairhurst Quality and Environmental Management System*

This document has been prepared in accordance with the instructions of the client, City of Edinburgh Council, for the client's sole and specific use. Any other persons who use any information contained herein do so at their own risk.

CONTENTS

Appendices

Project Details

Name of Project:	**Cloddach Bridge Assessment**
Name of Bridge:	**Cloddach Bridge**
Structure Reference Number:	**C2E/20**
Summary: Fairhurst have been appointed by Moray Council to undertake the inspection and assessment of C2E/20 Cloddach Bridge.	

1 HIGHWAY DETAILS

1.1 Type of Highway

Single carriageway.

1.2 Permitted Traffic Speed

60mph (96kph)

1.3 Existing Restrictions

3.0Tonne weight limit. 2.0m height restriction with gantries.

Bridge is temporarily closed to traffic.

2 SITE DETAILS

2.1 Obstacles Crossed

The bridge spans over the River Lossie.

3 STRUCTURE

3.1 Description of Structure and Design Working Life

Cloddach Bridge is a three span bridge over the River Lossie. The bridge is constructed of steel beams acting compositely with jack arched insitu concrete deck slab. The bridge was is currently closed to traffic.

The structure is generally in poor condition with significant corrosion to the steel beams. The invert has been altered and a large gorge formed which is creating scour close to the bridge piers.

The carriageway is approx. 3.9m wide and there is no verge or kerb to either side of the carriageway. The substructure is formed of mass concrete with a finishing render to give the appearance of masonry.

The intermediate supports have cutwaters upstream and downstream with a metal plate fixed over the upstream point. Foundations are unknown but are assumed to be spread footings onto the shallow bedrock.

3.2 Structural Type

The deck comprises of seven steel beams, acting compositely with jack arched insitu concrete deck slab. There are 7 No. I-beams at a distance of 700mm c/c.

It is assumed that each span is simply supported.

3.3 Foundation Type

Foundations are unknown but are assumed to be spread footings onto the shallow bedrock.

3.4 Span Arrangements

The bridge is formed of three simply supported clear spans of 6688mm, 6689mm and 6668mm. It is assumed that the beams bear onto the masonry piers with a bearing length of 300mm. The assumed span for assessment purposes will be taken as 7088mm

3.5 Articulation Arrangement

The bridge is assumed to be simply supported.

3.6 Road Restraint Systems Requirements

The bridge has Post and Rail steel parapets with bracing which are not to current standards. The approaches to the bridge deck have solid mass concrete parapet with a render to give the appearance of masonry on all sides. The north-west approach has a timber fence that is not connected to the bridge structure.

3.7 Proposals for Water Management

There is currently no evidence of waterproofing to the bridge deck.

3.8 Proposed Arrangements for Future Maintenance and Inspection for Assessment

3.8.1 Traffic management

No traffic management is required to access the structure.

3.8.2 Arrangements for future maintenance and inspection of structure. Access arrangements to structure

No traffic management is required to access the structure. The deck soffit to the side spans can be accessed via a ladder or access scaffold. Access to the central span is not currently planned. The central span will be inspected via binoculars from the side spans.

3.8.3 Intrusive or further investigations proposed

No further investigations are currently planned.

3.9 Environment and Sustainability

N/A

3.10 Durability: Materials and Finishes/Material Strengths Assumed and Basis of Assumption

The capacity of the steel sections will be determined in accordance with CS 454 Section 8, assuming end fixing factor 1.0 (both ends pin jointed).

The unit weight of steel will be assumed to be 7850kg/m^3.

Section thicknesses are based on estimates of remaining section thickness from site inspection report (to be provided)

A condition factor of 0.45 will be applied to the concrete jack arches and deck slab.

3.11 Risk and Hazards Considered for Design, Execution, Maintenance and Demolition

Not applicable.

3.12 Resilience and Security

Not applicable.

3.13 Year of Construction

The year of construction is unknown.

3.14 Reason for Assessment

The condition of the structure appears to have deteriorated since the last assessment in 2019.

3.15 Part of Structure to be Assessed

The main deck superstructure including steel beams, concrete jack arches and concrete deck will be assessed quantitatively. The substructure and parapets will be assessed qualitatively.

4 ASSESSMENT CRITERIA

4.1 Actions

4.1.1 Permanent Actions

Permanent loads acting on the structure shall be determined in accordance with CS 454, Assessment of Highway Bridges and Structures (DMRB 3.4.3).

4.1.2 Snow, Wind and Thermal Actions

Snow loads will be ignored for assessment purposes.

Wind loads will be ignored for assessment purposes.

4.1.3 Actions Relating to Normal Traffic Under AW Regulations and C&U Regulations

Assessment Live Loads (ALL) will be considered in accordance with CS 454. If the structure is found to have insufficient capacity for 40 tonnes ALL, the structure will be checked for reduced loading until the vehicle capacity of the structure can be confirmed.

4.1.4 Actions Relating to General Order Traffic Under STGO Regulations

SV and STGO loadings from CS 458 will not be included in this assessment.

4.1.5 Footway and Footbridge Variable Actions

Footway and crowd loading will be considered in accordance with CS 454.

4.1.6 Actions relating to Special Order traffic, provision for exceptional abnormal indivisible loads including location of vehicle track on deck cross-section

HB traffic loads will not to be considered as part of this assessment.

4.1.7 Accidental Actions

Accidental actions will not be considered during the analysis.

4.1.8 Actions during construction

Not applicable.

4.1.9 Any special actions not covered above

Not applicable.

4.2 Permanent Actions Heavy or high load route requirements and arrangements being made to preserve the route, including any provision for future heavier loads or future widening

Not applicable.

4.3 Minimum Headroom Provided

Not applicable.

4.4 Authorities consulted and any special conditions required

Not applicable.

4.5 Standards and documents listed in the Technical Approval Schedule

A full list of standards and documents is given in the Technical Approval Schedule in Appendix B.

4.6 Departures relating to departures from standards given in 4.5

Not applicable.

4.7 Proposed Departures relating to methods for dealing with aspects not covered by standards in 4.5

Not applicable.

4.8 Proposals for assessment of safety critical fixings

Not applicable.

5 STRUCTURAL ANALYSIS

5.1 Methods of analysis proposed for superstructure, substructure and foundations

Superstructure

The bridge deck will be analysed using a 2D Line and Finite Element plate model in analysis software assuming load is transferred through the bridge deck via the concrete jack arches to the steel longitudinal girders. The Finite Element plates will be assumed as concrete only, with minimum thickness used.

The longitudinal girders will be assessed as simply supported.

Substructure

The abutments and wingwalls will be assessed qualitatively in accordance with Section 2 of CS 459.

A drawing showing defects recorded can be found in Appendix C.

5.2 Description and diagram of idealised structure to be used for analysis

An idealised structure diagram is given in Appendix D.

5.3 Assumptions intended for calculation of structural element stiffness

Gross section properties shall be used throughout, using measured values where possible.

5.4 Proposed range of soil parameters to be used in the design/assessment of earth retaining elements

Not applicable.

6 GEOTECHNICAL CONDITIONS

6.1 Acceptance of recommendations of the Geotechnical Design Report to be used in the design/assessment1 and reasons for any proposed changes

Not applicable.

6.2 Summary of design for highway structure in Geotechnical Design Report

Not applicable.

6.3 Differential settlement to be allowed for in the design/assessment1 of the structure

Not applicable.

6.4 If the Geotechnical Design Report is not yet available, state when the results are expected and list the sources of information used to justify the preliminary choice of foundations

Not applicable.

7 CHECK

7.1 Proposed Category

Category 2

7.2 If Category 3, name of proposed independent Checker

Not applicable.

8 DRAWINGS AND DOCUMENTS

8.1 List of assessment and record drawings (including numbers) to be used in the assessment

All drawings are included in Appendix B.

Table 1: List of Assessment Drawings

Title	Drawing Number	Date
Inspection of Cloddach Bridge - Defect Schedule Sheet 1 of 2	140163F/01	February 2022
Inspection of Cloddach Bridge - Defect Schedule Sheet 2 of 2	140163F/02	February 2022

8.2 If Category 3, name of proposed independent Checker

Not applicable.

8.3 List of pile driving or other construction records

Not applicable.

8.4 List of Previous Inspection and Assessment Reports

1. Cloddach Bridge Inspection Report - 27 Sep 1995
2. Cloddach Bridge Assessment Calculations - 06 Feb 1996
3. Cloddach Bridge Inspection Report - 17 July 1997
4. Cloddach Bridge Assessment Calculations - 26 July 2000
5. Cloddach Bridge Principal Inspection Report - 26 Sep 2019
6. Cloddach Bridge PI Defect Sketch (.dwg) - 26 Sep 2019
7. Cloddach Bridge Load Review Calculations - 17 Oct 2019 (Check 11 Mar 2020)
8. Cloddach Bridge Special Inspection Report - 28 Jan 2022

THE ABOVE IS SUBMITTED FOR ACCEPTANCE

SIGNED	
NAME	Ellen Halkon
POSITION HELD	ASSESSMENT TEAM LEADER
ENGINEERING QUALIFICATIONS	MEng CEng MICE
NAME OF ORGANISATION	FAIRHURST
DATE	1st March 2022

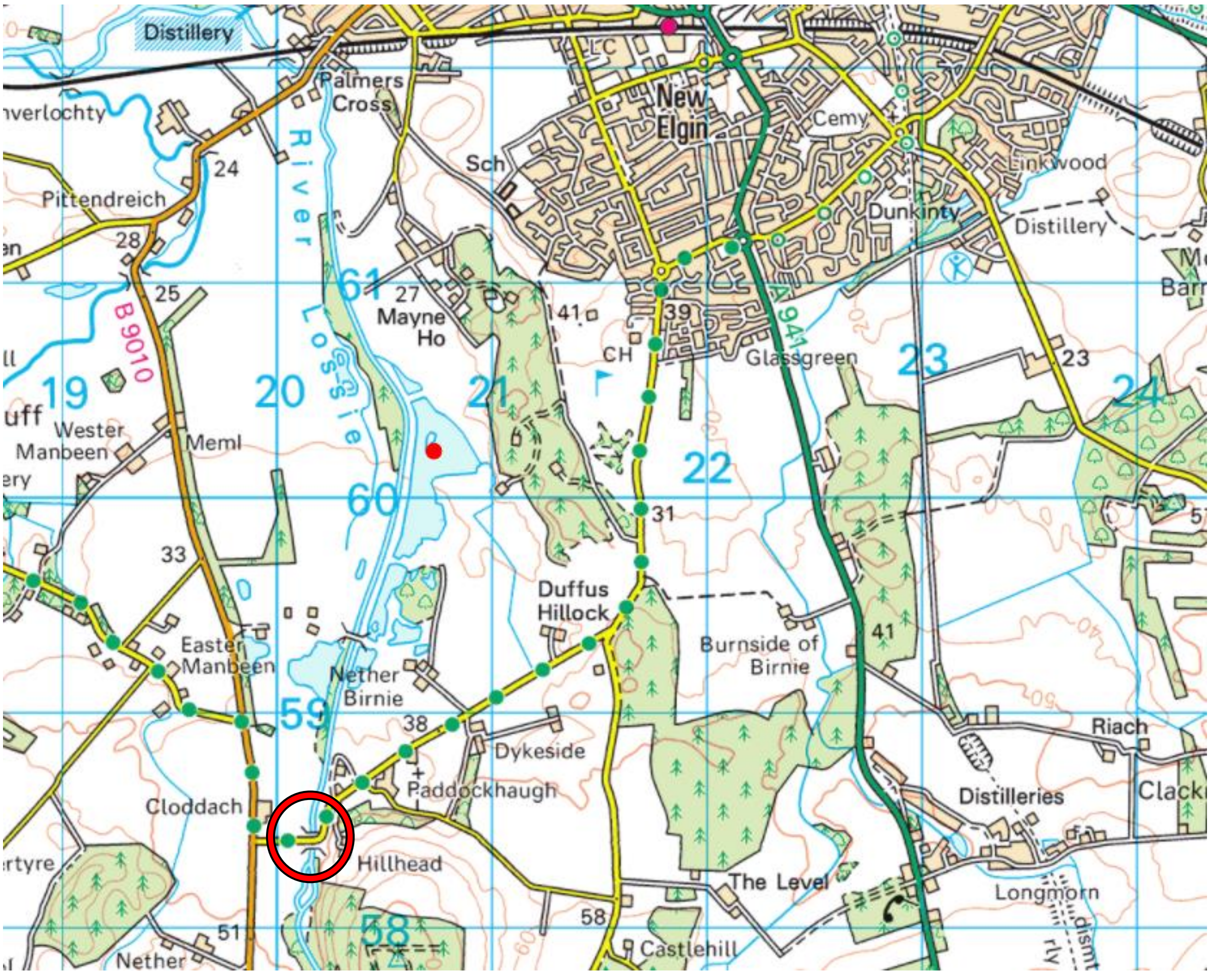
SIGNED	
NAME	John Campbell
POSITION HELD	CHECK TEAM LEADER
ENGINEERING QUALIFICATIONS	MEng CEng FICE
NAME OF ORGANISATION	FAIRHURST
DATE	1st March 2022

9 THE ABOVE IS REJECTED/AGREED SUBJECT TO THE AMENDMENTS AND CONDITIONS SHOWN BELOW

SIGNED	
NAME	Daniel Preston
POSITION HELD	Senior Engineer (Structures)
ENGINEERING QUALIFICATIONS	MEng CEng MICE
TAA	Moray Council
DATE	01 March 2022

APPENDIX A

Location Plan



Structure Location Plan

APPENDIX B

Technical Approval Schedule (TAS)

Schedule of Documents Relating to Design of Highways Bridges and Structures

(All documents are taken to include revisions 1st December 2021)

Eurocodes and associated UK National Annexes

Eurocode part	Title	Amendment / Corrigenda	Notes	Used
Eurocode 0	**Basis of structural design**			
BS EN 1990:2002 +A1:2005	Eurocode 0: Basis of structural design	+A1:2005 Incorporating corrigenda December 2008 and April 2010	See CD 350 section 7 for additional guidance.	NA
NA to BS EN 1990:2002 +A1:2005	UK National Annex to Eurocode 0 Basis of structural design	National Amendment No.1	See CD 350 section 7 for additional guidance.	NA
Eurocode 1				
BS EN 1991-1-1:2002	Eurocode 1: Actions on structures. General Actions. Densities, self-weight, imposed load for buildings	Corrigenda December 2004 and March 2009		NA
NA to BS EN 1991-1-1:2002	UK National Annex to Eurocode 1: Actions on structures. General Actions. Densities, self-weight, imposed load for buildings	Corrigenda July 2019		NA
BS EN 1991-1-3:2003+A1:2015	Eurocode 1: Actions on structures. General Actions. Snow loads	+A1:2015 Incorporating corrigenda December 2004 and March 2009		NA
NA + A2:18 to BS EN 1991-1-3:2003+A1:2015	UK National Annex to Eurocode 1: Actions on structures. General Actions. Snow loads	+A2:2018 Incorporating corrigenda June 2007, December 2015 and October 2018		NA
BS EN 1991-1 4:2005 +A1:2010	Eurocode 1: Actions on structures. General Actions. Wind actions	+A1:2010 Corrigenda July 2009 and January 2010		NA
NA to BS EN 1991-1-4:2005 + A1:2010	UK National Annex to Eurocode 1: Actions on structures. General Actions. Wind actions	National Amendment No.1		NA

Eurocode part	Title	Amendment / Corrigenda	Notes	Used
BS EN 1991-1-5:2003	Eurocode 1: Actions on structures. General Actions. Thermal actions	Corrigenda December 2004 and March 2009		NA
NA to BS EN 1991-1-5:2003	UK National Annex to Eurocode 1: Actions on structures. General Actions. Thermal actions	-		NA
BS EN 1991-1-6:2005	Eurocode 1: Actions on structures. General Actions. Actions during execution	Corrigenda July 2008, November 2012 and February 2013		NA
NA to BS EN 1991-1-6:2005	UK National Annex to Eurocode 1: Actions on structures. General Actions. Actions during execution	-		NA
BS EN 1991-1-7:2006 +A1:2014	Eurocode 1: Actions on structures. General Actions. Accidental actions	+A1: 2014 Corrigendum February 2010		NA
NA+A1 to BS EN 1991-1-7:2006+A1:2014	UK National Annex to Eurocode 1: Actions on structures. Part 1-7: Accidental actions	+A1:2014 Incorporating corrigenda August 2014 and November 2015	See CD 350 for additional guidance.	NA
BS EN 1991-2:2003	Eurocode 1: Actions on structures. Traffic loads on bridges	Corrigenda December 2004 and February 2010	See CD 350 section 7 for additional guidance.	NA
NA +A1:2020 to BS EN 1991-2:2003	UK National Annex to Eurocode 1: Actions on structures. Traffic loads on bridges	Corrigendum No.1 Amendment June 2020	See CD 350 section 7 for additional guidance.	NA
Eurocode 2	**Design of concrete structures**			
BS EN 1992-1-1:2004 + A1:2014	Eurocode 2: Design of concrete structures– Part 1-1: General rules and rules for buildings	Incorporating corrigendum January 2008, November 2010 and January 2014		NA
NA + A2:2014 to BS EN 1992-1-1:2004 + A1:2014	UK National Annex to Eurocode 2: Design of concrete structures – Part 1-1: General rules and rules for buildings			NA
BS EN 1992-2:2005	Eurocode 2: Design of concrete structures – Part 2: Concrete bridges – Design and detailing rules	Corrigendum July 2008		NA
NA to BS EN 1992-2:2005	UK National Annex to Eurocode 2: Design of concrete structure – Part 2: Concrete bridges – Design and detailing rules	-		NA

Eurocode part	Title	Amendment / Corrigenda	Notes	Used
BS EN 1992-3:2006	Eurocode 2: Design of concrete structures – Part 3: Liquid retaining and containment structures	-		NA
NA to BS EN 1992-3:2006	UK National Annex to Eurocode 2: Design of concrete structures – Part 3: Liquid retaining and containment structures	-		NA
BS EN 1992-4:2018	Eurocode 2: Design of concrete structures – Part 4: Design of fastenings for use in concrete			NA
NA to BS EN 1992-4:2018	UK National Annex to Eurocode 2: Design of concrete structures – Part 4: Design of fastenings for use in concrete			NA
Eurocode 3	**Design of steel structures**			
BS EN 1993-1-1:2005 + A1:2014	Eurocode 3: Design of steel structures – Part 1-1 General rules and rules for buildings	Corrigenda February 2006 and April 2009		NA
NA + A1:2014 to BS EN 1993-1-1:2005 + A1:2014	UK National Annex to Eurocode 3: Design of steel structures – Part 1-1 General rules and rules for buildings	-		NA
BS EN 1993-1-3:2006	Eurocode 3: Design of steel structures – Part 1-3 General rules – Supplementary rules for cold-formed members and sheeting	Corrigendum November 2009		Na
NA to BS EN 1993-1-3:2006	UK National Annex to Eurocode 3: Design of steel structures – Part 1-3 Supplementary rules for cold-formed members and sheeting	-		Na
BS EN 1993-1-4:2006 + A2:2020	Eurocode 3: Design of steel structures – Part 1-4 General rules – Supplementary rules for stainless steels	+ A1:2015 Amendment No. 1 + A2:2020 Amendment No. 2	Supersedes BS EN 1993-1-4:2006 + A1:2015	NA
NA+A1:15 to BS EN 1993-1-4:2006+A1:2015	UK National Annex to Eurocode 3: Design of steel structures – Part 1-4 Supplementary rules for stainless steels	+ A1:2015 Amendment No. 1		NA

Eurocode part	Title	Amendment / Corrigenda	Notes	Used
BS EN 1993-1-5:2006+A2:2019	Eurocode 3: Design of steel structures – Part 1-5 Plated structural elements	Corrigendum April 2009, +A1:2017 Amendment No. 2, +A2:2019		NA
NA+A1:2016 to BS EN 1993-1-5:2006	UK National Annex to Eurocode 3: Design of steel structures – Part 1-5 Plated structural elements	+ A1:2016 Amendment No. 1		NA
BS EN 1993-1-6:2007+ A1:2017	Eurocode 3: Design of steel structures – Part 1-6 Strength and stability of shell structures	~~+ A1:2017~~ ~~Amendment No. 1~~		Na
BS EN 1993-1-7:2007	Eurocode 3: Design of steel structures – Part 1-7 Plated structures subject to out of plane loading	~~Corrigendum April 2009~~		Na
BS EN 1993-1-8:2005	Eurocode 3: Design of steel structures – Part 1-8 Design of joints	Corrigenda December 2005, September 2006, July 2009 and August 2010		NA
NA to BS EN 1993-1-8:2005	UK National Annex to Eurocode 3: Design of steel structures – Part 1-8 Design of joints	-		NA
BS EN 1993-1-9:2005	Eurocode 3: Design of steel structures – Part 1-9 Fatigue	Corrigenda December 2005, September 2006 and April 2009		NA
NA to BS EN 1993-1-9:2005	UK National Annex to Eurocode 3: Design of steel structures – Part 1-9 Fatigue	-		NA
BS EN 1993-1-10:2005	Eurocode 3: Design of steel structures – Part 1-10 Material toughness and through-thickness properties	Corrigenda December 2005, September 2006 and March 2009		Na
NA to BS EN 1993-1-10:2005	UK National Annex to Eurocode 3: Design of steel structures – Part 1-10 Material toughness and through thickness properties	-		Na
BS EN 1993-1-11:2006	Eurocode 3: Design of steel structures – Part 1-11 Design of structures with tension components	Corrigendum April 2009		Na
NA to BS EN 1993-1-11:2006	UK National Annex to Eurocode 3: Design of steel structures – Part 1-11 Design of structures with tension components	-		Na

Eurocode part	Title	Amendment / Corrigenda	Notes	Used
BS EN 1993-1-12:2007	Eurocode 3: Design of steel structures – Part 1-12 Additional rules for the extension of EN 1993 up to steel grades S 700			Na
NA to BS EN 1993-1-12:2007	UK National Annex to Eurocode 3: Design of steel structures – Part 1-12 Additional rules for the extension of EN 1993 up to steel grades S 700	-		Na
BS EN 1993-2:2006	Eurocode 3: Design of steel structures – Part 2 Steel bridges	Corrigendum July 2009		Na
NA + A1:2012 to BS EN 1993-2:2006	UK National Annex to Eurocode 3: Design of steel structures – Part 2 Steel bridges	+ A1:2012		Na
BS EN 1993-5:2007	Eurocode 3: Design of steel structures – Part 5 Piling	Corrigendum May 2009		Na
NA + A1:2012 to BS EN 1993-5:2007	UK National Annex to Eurocode 3: Design of steel structures – Part 5 Piling	+ A1:2012		Na
Eurocode 4	Design of composite steel and concrete structures			
BS EN 1994-1-1:2004	Eurocode 4: Design of composite steel and concrete structures – Part 1-1 General rules and rules for buildings	Corrigendum April 2009		Na
NA to BS EN 1994-1-1:2004	UK National Annex to Eurocode 4: Design of composite steel and concrete structures – Part 1-1 General rules and rules for buildings	-		Na
BS EN 1994-2:2005	Eurocode 4: Design of composite steel and concrete structures – Part 2 General rules and rules for bridges	Corrigendum July 2008		Na
NA to BS EN 1994-2:2005	UK National Annex to Eurocode 4: Design of composite steel and concrete structures – Part 2 General rules and rules for bridges	-		Na
Eurocode 5	**Design of timber structures**			

Eurocode part	Title	Amendment / Corrigenda	Notes	Used
BS EN 1995-1-1:2004 + A2:2014	Eurocode 5: Design of timber structures – Part 1-1 General – common rules and rules for buildings	+ A2:2014 Incorporating corrigendum June 2006		Na
NA to BS EN 1995-1-1:2004 + A2:2014	UK National Annex to Eurocode 5: Design of timber structures – Part 1-1 General – common rules and rules for buildings	+ A2:2014		Na
BS EN 1995-2:2004	Eurocode 5: Design of timber structures – Part 2 Bridges	-		Na
NA to BS EN 1995-2:2004	UK National Annex to Eurocode 5: Design of timber structures – Part 2 Bridges	-		Na
Eurocode 6	**Design of masonry structures**			
BS EN 1996-1-1:2005+A1:2012	Eurocode 6: Design of masonry structures – Part 1-1 General rules for reinforced and unreinforced masonry structures	+A1:2012 Corrigenda February 2006 and July 2009		Na
NA to BS EN 1996-1-1:2005 +A1:2012	UK National Annex to Eurocode 6: Design of masonry structures – Part 1-1 General rules for reinforced and unreinforced masonry structures	+A1:2012		Na
BS EN 1996-2:2006	Eurocode 6: Design of masonry structures – Part 2 Design considerations, selection of materials and execution of masonry	Corrigendum September 2009		Na
NA to BS EN 1996-2:2006	UK National Annex to Eurocode 6: Design of masonry structures – Part 2 Design considerations, selection of materials and execution of masonry	Corrigendum No.1		Na
BS EN 1996-3:2006	Eurocode 6: Design of masonry structures – Part 3 Simplified calculation methods for unreinforced masonry structures	Corrigendum October 2009		Na
NA +A1:2014 to BS EN 1996-3:2006	UK National Annex to Eurocode 6: Design of masonry structures – Part 3 Simplified calculation methods for unreinforced masonry structures	+A1:2014		Na
Eurocode 7	**Geotechnical design**			

Eurocode part	Title	Amendment / Corrigenda	Notes	Used
BS EN 1997-1:2004+A1:2013	Eurocode 7: Geotechnical design – Part 1 General rules	+A1:2013 Corrigendum February 2009		Na
NA+A1:2014 to BS EN 1997-1:2004+A1:2013	UK National Annex to Eurocode 7: Geotechnical design – Part 1 General rules	+A1:2013 Incorporating Corrigendum No.1		Na
BS EN 1997-2:2007	Eurocode 7: Geotechnical design – Part 2 Ground investigation and testing	Corrigendum June 2010		Na
NA to BS EN 1997-2:2007	UK National Annex to Eurocode 7: Geotechnical design – Part 2 Ground investigation and testing			Na
Eurocode 8	**Design of structures for earthquake resistance**			
BS EN 1998-1:2004 + A1:2013	Eurocode 8: Design of structures for earthquake resistance – Part 1 General rules, seismic actions and rules for buildings	Corrigendum June 2009, January 2011 and March 2013		na
NA to BS EN 1998-1:2004	UK National Annex to Eurocode 8: Design of structures for earthquake resistance – Part 1 General rules, seismic actions and rules for buildings	-		na
BS EN 1998-2:2005+A2:2011	Eurocode 8: Design of structures for earthquake resistance – Part 2 Bridges	Corrigenda February 2010 and February 2012		na
NA to BS EN 1998-2:2005	UK National Annex to Eurocode 8: Design of structures for earthquake resistance – Part 2 Bridges	-		na
BS EN 1998-5:2004	Eurocode 8: Design of structures for earthquake resistance – Part 5 Foundations, retaining structures and geotechnical aspects	-		na
NA to BS EN 1998-5:2004	UK National Annex to Eurocode 8: Design of structures for earthquake resistance – Part 5 Foundations, retaining structures and geotechnical aspects	-		na
Eurocode 9	**Design of aluminium structures**			

Eurocode part	Title	Amendment / Corrigenda	Notes	Used
BS EN 1999-1-1:2007 + A2:2013	Eurocode 9: Design of aluminium structures– Part 1-1 General structural rules	+ A2:2013 Incorporating corrigendum March 2014		na
NA to BS EN 1999-1-1:2007 + A1:2009	UK National Annex to Eurocode 9: Design of aluminium structures – Part 1-1 General structural rules	National Amendment No.1 Corrigendum No.1		na
BS EN 1999-1-3:2007 + A1:2011	Eurocode 9: Design of aluminium structures – Part 1-3 Structures susceptible to fatigue	+ A1:2011		na
NA to BS EN 1999-1-3:2007 + A1:2011	UK National Annex to Eurocode 9: Design of aluminium structures – Part 1-3 Structures susceptible to fatigue	+ A1:2011		na
BS EN 1999-1-4:2007 +A1:2011	Eurocode 9: Design of aluminium structures – Part 1-4 Cold formed structural sheeting	+ A1:2011 Corrigendum November 2009		na
NA to BS EN 1999-1-4:2007	UK National Annex to Eurocode 9: Design of aluminium structures – Part 1-4 Cold formed structural sheeting	-		na

Bsi Published Documents

For guidance only unless clauses are otherwise specified in CD 350 Appendix A.

Document reference	Title	Notes	Used
PD 6687-1:2020	Background paper to the UK National Annexes to BS EN 1992-1 and BS EN 1992-3	Supersedes PD 6687-1:2010 See CD 350 clauses 3.6, 4.1, 4.2 and Appendix A for additional guidance. Clause 3.6 in CD 350 refers to clause 2.5 in PD 6687-1, this is now clause 4.5 in PD 6687-1 Clause 4.2 in CD 350 refers to clause 2.22 in PD 6687-1, this is now clause 4.21.4 in PD 6687-1	na
PD 6687-2:2008	Recommendations for the design of structures to BS EN 1992-2:2005	See CD 350 clauses 4.1, 4.2 and Appendix A for additional guidance.	Na

Document reference	Title	Notes	Used
PD 6688-1-1:2011	Recommendations for the design of structures to BS EN 1991-1-1	See CD 350 Appendix A for additional guidance.	na
PD 6688-1-4:2015	Background paper to the UK National Annex to BS EN 1991-1-4	See CD 350 Appendix A for additional guidance.	Na
PD 6688-1-7:2009 +A1:2014	Recommendations for the design of structures to BS EN 1991-1-7	See CD350 clause 3.7 and Appendix B for additional guidance.	Na
PD 6688-2:2011	Recommendations for the design of structures to BS EN 1991-2	See CD 350 Appendix A for additional guidance.	Na
~~PD 6694-1:2011 + A1:2020~~	~~Recommendations for the design of structures subject to traffic loading to BS EN 1997-1~~	~~See CD 350 Appendix A for additional guidance.~~ ~~Amended 27 May 2020~~ ~~(Temporarily withdrawn due to technical errors)~~	Na
PD 6695-1-9:2008	Recommendations for the design of structures to BS EN 1993-1-9	See CD 350 Appendix A for additional guidance.	Na
PD 6695-1-10:2009	Recommendations for the design of structures to BS EN 1993-1-10	See CD 350 Appendix A for additional guidance.	Na
PD 6695-2:2008 + A1:2012 Incorporating Corrigendum No.1	Recommendation for the design of bridges to BS EN 1993	See CD 350 Appendix A for additional guidance.	Na
PD 6696-2:2007 + A1:2012	Background paper to BS EN 1994-2 and the UK National Annex to BS EN 1994-2	See CD 350 Appendix A for additional guidance.	Na
PD 6698:2009	Recommendations for the design of structures for earthquake resistance to BS EN 1998	See CD 350 section 7 for additional guidance.	Na
PD 6702-1:2009+A1:2019	Structural use of aluminium. Recommendations for the design of aluminium structures to BS EN 1999	Amended 31 May 2019	Na
PD 6703:2009	Structural bearings – Guidance on the use of structural bearings		Na
PD 6705-2:2020	Structural use of steel and aluminium. Execution of steel bridges conforming to BS EN 1090-2. Guide	Replaces PD 6705-2:2010 + A1:2013	Na
PD 6705-3:2009	Recommendations on the execution of aluminium structures to BS EN 1090-3		Na

Execution Standards referenced in British Standards or Eurocodes

Document reference	Title	Notes	Used

Document reference	Title	Notes	Used
BS EN 1090-1:2009+A1:2011	Execution of steel structures and aluminium structures - Part 1: Requirements for conformity assessment of structural components		Na
BS EN 1090-2:2018	Execution of steel structures and aluminium structures. Technical requirements for the execution of steel structures	Supersedes BS EN 1090-2:2008+A1:2011	Na
BS EN 1090-3:2019	Execution of steel structures and aluminium structures – Part 3: Technical requirements for aluminium structures	Supersedes BS EN 1090-3:2008	Na
BS EN 13670:2009 Incorporating corrigenda October 2015 and November 2015	Execution of concrete structures		Na

Product Standards referenced in British Standards or Eurocodes

Document reference	Title	Notes	Used
BS EN 206:2013+A2:2021	Concrete – Specification, performance, production and conformity	Supersedes BS EN 206:2013+A1:2016	Na
BS EN 1317-1:2010	Road Restraint Systems – Part 1 – Terminology and general criteria for test methods		Na
BS EN 1317-2:2010	Road Restraint Systems – Part 2 – Performance classes, impact test acceptance criteria and test methods for safety barriers.		Na
BS EN 1317-3:2010	Road Restraint Systems – Part 3 – Performance classes, impact test acceptance criteria and test methods for crash cushions.		Na
DD ENV 1317-4:2002	Road Restraint Systems – Part 4 – Performance classes, impact test acceptance criteria and test methods for terminals and transitions of safety barriers.	Draft BS EN 1317-4 for public comment published in June 2012	Na
BS EN 1317-5:2007+A2:2012	Road Restraint Systems – Part 5 - Product requirements and evaluation of conformity for vehicle restraint systems	Incorporating corrigendum August 2012 Draft prEN 1317-5 for public comment published in December 2013	Na
PD CEN/TR 16949:2016	Road Restraint System – Pedestrian restraint system - Pedestrian parapets	Bsi Published Document / CEN Technical Report published in July 2016	na

Document reference	Title	Notes	Used
		(This document should not be used. The requirements of BS 7818:1995 apply.)	
Draft prEN 1317-7	Road restraint systems - Part 7: Performance classes, impact test acceptance criteria and test methods for terminals of safety barriers	Draft prEN 1317-7 for public comment published in June 2012 (This document should not be used. All terminals should continue to be in accordance with ENV1317-4.)	Na
PD CEN/TS 17342:2019	Road restraint systems - Motorcycle road restraint systems which reduce the impact severity of motorcyclist collisions with safety barriers	Replaces PD CEN/TS 1317-8:2012 (This document should not be used.)	Na
PD CEN/TR 17081:2018	Design of fastenings for use in concrete – Plastic design of fastenings with headed and post-installed fasteners		Na
BS EN 1337-1:2000	Structural bearings – Part 1: General Design Rules		Na
BS EN 1337-2:2004	Structural bearings – Part 2: Sliding elements		Na
BS EN 1337-3:2005	Structural bearings – Part 3: Elastomeric bearings		Na
BS EN 1337-4:2004	Structural bearings – Part 4: Roller bearings	Corrigendum No.1 March 2007	Na
BS EN 1337-5:2005	Structural bearings – Part 5: Pot bearings		Na
BS EN 1337-6:2004	Structural bearings – Part 6: Rocker bearings		Na
BS EN 1337-7:2004	Structural bearings – Part 7: Spherical and cylindrical PTFE bearings		Na
BS EN 1337-8:2007	Structural bearings – Part 8: Guide bearings and restraint bearings		Na
BS EN 1337-9:1998	Structural bearings – Part 9: Protection		Na
BS EN 1337-10:2003	Structural bearings – Part 10: Inspection and maintenance	Corrigendum No.1 November 2003	Na
BS EN 1337-11:1998	Structural bearings – Part 11: Transport, Storage and Installation.		Na
BS EN 10025-1:2004	Hot rolled products of structural steels Part 1: General technical delivery conditions.		Na
BS EN 10025-2:2019	Hot rolled products of structural steels Part 2: Technical delivery conditions for non-alloy structural steels.	Supersedes BS EN 10025-1:2004	Na

Document reference	Title	Notes	Used
BS EN 10025-3:2019	Hot rolled products of structural steels Part 3: Technical delivery conditions for normalized/normalized rolled weldable fine grain structural steels.	Supersedes BS EN 10025-3:2004	Na
BS EN 10025-4:2019	Hot rolled products of structural steels Part 4: Technical delivery conditions for thermomechanical rolled weldable fine grain structural steels.	Supersedes BS EN 10025-4:2004	Na
BS EN 10025-5:2019	Hot rolled products of structural steels – Part 5: Technical delivery conditions for structural steels with improved atmospheric corrosion resistance	Supersedes BS EN 10025-5:2004	Na
BS EN 10025-6:2019	Hot rolled products of structural steels – Part 6: Technical delivery conditions for flat products of high yield strength structural steels in the quenched and tempered condition.	Supersedes BS EN 10025-6:2004+A1:2009	Na
BS EN 10025-1:2004	Hot rolled products of structural steels Part 1: General technical delivery conditions.		Na
BS EN 10025-2:2019	Hot rolled products of structural steels Part 2: Technical delivery conditions for non-alloy structural steels.	Supersedes BS EN 10025-1:2004	Na
BS EN 10025-3:2019	Hot rolled products of structural steels Part 3: Technical delivery conditions for normalized/normalized rolled weldable fine grain structural steels.	Supersedes BS EN 10025-3:2004	Na
BS EN 10025-4:2019	Hot rolled products of structural steels Part 4: Technical delivery conditions for thermomechanical rolled weldable fine grain structural steels.	Supersedes BS EN 10025-4:2004	Na
BS EN 10025-5:2019	Hot rolled products of structural steels – Part 5: Technical delivery conditions for structural steels with improved atmospheric corrosion resistance	Supersedes BS EN 10025-5:2004	Na
BS EN 10025-6:2019	Hot rolled products of structural steels – Part 6: Technical delivery conditions for flat products of high yield strength structural steels in the quenched and tempered condition.	Supersedes BS EN 10025-6:2004+A1:2009	Na
BS EN 10080:2005	Steel for the reinforcement of concrete – Weldable reinforcing steel - General		Na
BS EN 10210-1:2006	Hot finished structural hollow sections of non-alloy and fine grain steels – Part 1: Technical delivery conditions		Na
BS EN 10210-2:2019	Hot finished structural hollow sections – Part 2: Tolerances, dimensions and sectional properties	Supersedes BS EN 10210-2:2006	Na
BS EN 10248-1:1996	Hot rolled sheet piling of non alloy steels.		Na
BS EN 10248-	Hot rolled sheet piling of non alloy steels.		na

Document reference	Title	Notes	Used
2:1996			
BS EN 12063:1999	Execution of special geotechnical work. Sheet pile walls.		Na
BS EN 14388:2005	Road traffic noise reducing devices	There is a 2015 version, however the 2015 version is not harmonised.	Na
BS EN 15050:2007 + A1:2012	Precast concrete products – Bridge elements	See CD 350 clause 3.8.1 for additional guidance.	Na
BS EN 15258:2008	Precast concrete products - Retaining wall elements		Na

British Standards

Document reference	Title	Notes	Used
BS 4449:2005 +A3:2016	Steel for the reinforcement of concrete	No longer covers plain round bar. (See BS4482 up to 12mm dia, see BS EN 10025-1 for larger sizes and dowels. See BS EN 13877-3 for dowel bars in concrete pavements.)	Na
BS 5896:2012	Specification for high tensile steel wire and strand for the prestressing of concrete		Na
BS 7818:1995	Specification for pedestrian restraint systems in metal	Incorporating Corrigendum No.1 May 2004 and Corrigendum No.2 September 2006 Currently the requirements of BS 7818:1995 are to be used instead of PD CEN/TR 16949:2016	Na
BS 8002:2015	Code of practice for earth retaining structures		Na
BS 8004:2015 +A1 2020	Code of practice for foundations	Amendment +A1:2020	Na
BS 8006-1:2010+A1:2016	Code of practice for strengthened/reinforced soils and other fills		Na
BS 8500-1:2015+A2:2019	Concrete – Complementary British Standard to BS EN 206: Method of specifying and guidance for the specifier.	Incorporating Corrigendum No.1 and Corrigendum No.2 June 2020 Amendment +A2:2019	Na
BS 8500-2:2015+A2:2019	Concrete – Complementary British Standard to BS EN 206: Specification for constituent materials and concrete.	Amendment +A2:2019	Na

Document reference	Title	Notes	Used
BS 8666:2020	Scheduling, dimensioning, bending and cutting of steel reinforcement for concrete	Supersedes BS 8666:2005	Na

The Manual Contract Document for Highway Works (MCHW)

Document reference	Title	Notes	Used
MCHW Volume 1: November 2021	Specification for Highway Works	Specification compliant with the execution standards must be used. A Departure is necessary for the parts where a compliant revision has not been published. Amendments November 2021	Na
MCHW Volume 2: November 2021	Notes for guidance on the Specification for Highway Works	Notes for guidance compliant with the execution standards must be used. A Departure is necessary for the parts where a compliant revision has not been published. Amendments November 2021	Na
MCHW Volume 3: February 2017	Highway Construction Details		Na

The Design Manual for Roads and Bridges (DMRB)

Document reference	Title	Notes	Used
GG 101 Revision 0.1.0	Introduction to the Design Manual for Roads and Bridges	Replaces GG 101 Revision 0	√
GG 102 Revision 0	Quality Management Systems for Highway Design	Supersedes GD 02/16	Na
GG 103 Revision 0	Introduction and general requirements for sustainable development and design		Na
GG 104 Revision 0	Requirements for Safety Risk Assessment	Replaces GD04/12 and IAN 191/16	Na
GG 184 Revision 0	Specification for the use of Computer Aided Design	Replaces IAN 184/16	Na
CG 300 Revision 0.1.0	Technical approval of highway structures	Supersedes BD 2/12	√
CG 302 Revision 0	As-built, operational and maintenance records for highway structures	Supersedes BD 62/07	√

Document reference	Title	Notes	Used
CG 303 Revision 0	Quality assurance scheme for paints and similar protective coatings	Supersedes BD 35/14	na
CG 304 Revision 0	Conservation of highway structures	Supersedes BD 89/03	na
CG 305 Revision 0	Identification marking of highway structures	Supersedes BD 45/93	na
CG 501 Revision 2	Design of highway drainage systems	Supersedes HD 33/16, TA 80/99	na
CD 127 Revision 1.0.1	Cross-sections and headrooms	Replaces TD 27/05 and TD 70/08	na
CD 350 Revision 0	The design of highway structures	Supersedes BD 100/16, BA 57/01, BD 57/01 and IAN 124/11	Na
CD 351 Revision 0	The design and appearance of highway structures	Supersedes BA 41/98	Na
CD 352 Revision 0	Design of road tunnels	Supersedes BD 78/99	Na
CD 353 Revision 0	Design criteria for footbridges	Supersedes BD 29/17	Na
CD 354 Revision 1	Design of minor structures	Supersedes BD 94/17	Na
CD 355 Revision 0	Application of whole-life costs for design and maintenance of highway structures	Supersedes BD 36/92 and BA 28/92	Na
CD 356 Revision 1	Design of highway structures for hydraulic action	Supersedes BA 59/94	Na
CD 357 Revision 1	Bridge expansion joints	Supersedes BD 33/94, BA 26/94, IAN 168/12 and IAN 169/12	Na
CD 358 Revision 2	Waterproofing and surfacing of concrete bridge decks	Replaces BD 47/99, BA 47/99 and IAN 96/07	Na
CD 359 Revision 0	Design requirements for permanent soffit formwork	Supersedes BA 36/90 and IAN 131/11	Na
CD 360 Revision 2	Use of compressive membrane action in bridge decks	Supersedes BD 81/02	Na
CD 361 Revision 0	Weathering steel for highway structures	Supersedes BD 7/01	Na
CD 362 Revision 1	Enclosure of bridges	Supersedes BD 67/96 and BA 67/96	Na
CD 363	Design rules for aerodynamic effects	Supersedes BD 49/01	na

Document reference	Title	Notes	Used
Revision 0	on bridges		
CD 364 Revision 0	Formation of continuity joints in bridge decks	Supersedes BA 82/00	Na
CD 365 Revision 1	Portal and cantilever signs/signals gantries	Supersedes BD 51/14, IAN 193/16, BE 7/04	Na
CD 366 Revision 0	Design criteria for collision protection beams	Supersedes BD 65/14	Na
CD 367 Revision 0	Treatment of existing structures on highways widening schemes	Supersedes BD 95/07	Na
CD 368 Revision 0	Design of fibre reinforced polymer bridges and highway structures	Supersedes BD 90/05	Na
CD 369 Revision 0	Surface protection for concrete highway structures	Supersedes BA 85/04	Na
CD 370 Revision 2	Cathodic protection for use in reinforced concrete highway structures	Supersedes BA 83/02	Na
CD 371 Revision 0	Strengthening highway structures using fibre-reinforced polymers and externally bonded steel plates	Supersedes BA 85/08	Na
CD 372 Revision 0	Design of post-installed anchors and reinforcing bar connections in concrete	Supersedes IAN 104/15	Na
CD 373 Revision 0	Impregnation of reinforced and prestressed concrete highway structures using hydrophobic pore-lining impregnants	Supersedes BD 43/03	Na
CD 374 Revision 0	The use of recycled aggregates in structural concrete	Supersedes BA 92/07	Na
CD 375 Revision 1	Design of corrugated steel buried structures	Supersedes BD 12/01	Na
CD 376 Revision 0	Unreinforced masonry arch bridges	Supersedes BD 91/04	Na
CD 377 Revision 2	Requirements for road restraint systems	Supersedes TD 19/06	Na
CD 378 Revision 0	Impact test and assessment criteria for truck mounted attenuators	Supersedes TD 49/07	Na
CD 622 Revision 1	Managing geotechnical risk	Supersedes HD 22/08, BD 10/97 and HA 120/08	Na
CS 450 Revision 0	Inspection of highway structures	Supersedes BD 63/17	√
CS 451 Revision 0	Structural review and assessment of highway structures	Supersedes BD 101/11	√

Document reference	Title	Notes	Used
CS 452 Revision 0	Inspection and records for road tunnel systems	Supersedes BD 53/95	Na
CS 453 Revision 0	The assessment of highway bridge supports	Supersedes BD 60/04 & IAN 091/07	Na
CS 454 Revision 1	Assessment of highway bridges and structures	Supersedes BD 21/01, BA 16/97 and BD 37/01	√
CS 455 Revision 1	The assessment of concrete highway bridges and structures	Supersedes BD 44/01, BA 51/96, BA 52/94, BA 40/93 & BA 38/93	√
CS 456 Revision 0	The assessment of steel highway bridges and structures	Supersedes BD 56/10, BD 13/06, BA 19/85, BA 09/81 & BD 09/81	√
CS 457 Revision 1	The assessment of composite highway bridges and structures	Supersedes BD 61/10	√
CS 458 Revision 0	The assessment of highway bridges and structures for the effects of special type general order (STGO) and special order (SO) vehicles	Supersedes BD 86/11	Na
CS 459 Revision 1	The assessment of bridge substructures, retaining structures and buried structures (formerly BA 55/06)	Supersedes BD 21/01 and BA 16/97,	√
CS 460 Revision 1	Management of corrugated steel buried structures	Supersedes BA 87/04	na
CS 461 Revision 0	Assessment and upgrading of in-service parapets	Supersedes BA 37/92 and IAN 97/07	na
CS 462 Revision 0	Repair and management of deteriorated concrete highway structures	Supersedes BA 52/94 & BA 35/90	√
CS 463 Revision 0	Load testing for bridge assessment	Supersedes BA 54/94	na
CS 464 Revision 1	Non-destructive testing of highways structures	Supersedes BA 86/06	na
CS 465 Revision 0	Management of post-tensioned concrete bridges	Supersedes BD 54/15	na
CS 466 Revision 0	Risk management and structural assessment of concrete half-joint deck structures	Supersedes BA 39/93 & IAN 053/04	na
CS 467 Revision 1	Risk management and structural assessment of concrete deck hinge structures	Supersedes BA 93/09	na
CS 468 Revision 1	Assessment of Freyssinet concrete hinges in highway structures	Supersedes BE 5/75	na
CS 470 Revision 0	Management of sub-standard highway structures	Supersedes BD 79/13	√

Document reference	Title	Notes	Used
GD 304 Revision 2	Designing health and safety into maintenance	Supersedes IAN 69/15	√

Transport Scotland Interim Amendments

Document reference	Title	Notes	Used
TSIA 22	BS 4449:2005, BS 4482:2005, BS 4483:2005 AND BS 8666:2005		na
TSIA 23	Implementation of BS 8500-1 2006 Concrete Complimentary British Standard to BS EN 206-1		na
TSIA 24	Guidance on implementing results of research on bridge deck waterproofing		na
TSIA 26	The Anchorage of Reinforcement and Fixings into Hardened Concrete		na
TSIA 30	The Use of Foamed Concrete		na
TSIA 39	Use of Eurocodes for the Design of Bridges and Road Related Structures		Na

Miscellaneous

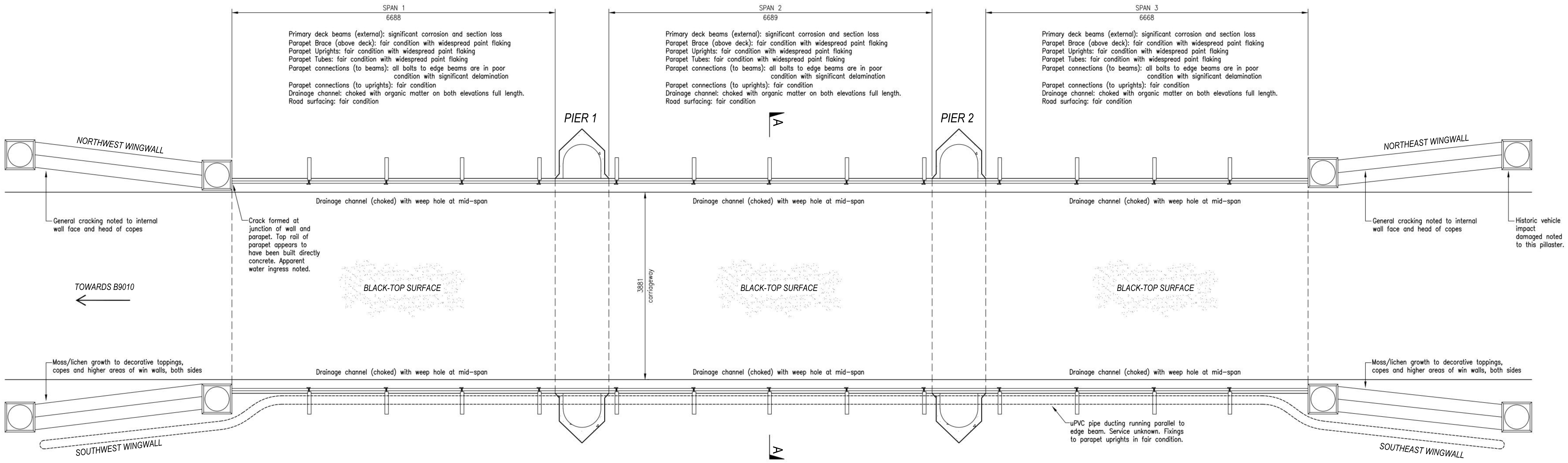
Document reference	Title	Notes	Used
CHE Memorandum 227/08	The Impregnation of Reinforced and Prestressed Concrete Highway Structures using Hydrophobic Pore Lining Impregnants	CHE memoranda are internal Highways England documents and not available to external organisations. This CHE memorandum is included as a useful reference for the Technical Approval Authority.	Na
CIRIA C543	Bridge Detailing Guide		Na
CIRIA C766	Control of cracking caused by restrained deformation in concrete	Supersedes C660	Na
CIRIA C686	Safe Access for Maintenance and Repair		Na
~~CIRIA C760~~	~~Guidance on embedded retaining wall design~~		Na
Sustrans	National Cycle Network Design Principles		Na
Sustrans	Sustrans traffic-free routes and greenways design guide		Na

FAIRHURST

Document reference	Title	Notes	Used
Transport Scotland	Cycling by Design		Na

APPENDIX C

Record and Inspection Drawings



PLAN LAYOUT OF BRIDGE DECK SURFACING (1:50)

KEY FOR DEFECTS SCHEDULING :

Hatch Type	Defect observed	Localised/Widespread
(red line)	Significant section corrosion and section loss to members. Delaminating and layered rusting.	Widespread
(yellow line)	Mild corrosion to members with loss of protective paint and minimal pitting to exposed steel	Widespread

FLOW OF RIVER LOSSIE

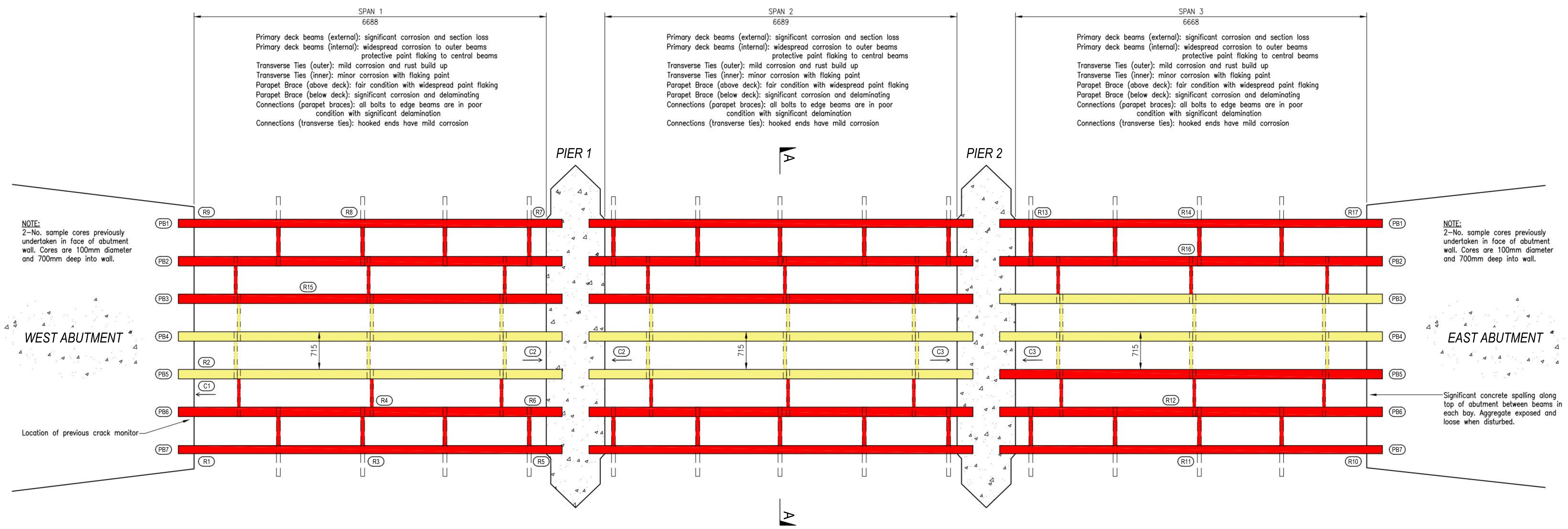
STEELWORK MEASUREMENTS FROM MAIN DECK MEMBERS :

Location	Reading from bottom flange (mm) taken at quarter points (u.n.o)	Reading from web (mm)
R1	12.7	9.9
R2	11.4	Not Accessible
R3	11.5	11.5
R4	11.1	Not Accessible
R5	18.1 (mid)	5.8
R6	15.4 - 16.5 (mid)	Not Accessible
R7	12.7	13.0
R8	9.9	12.5
R9	10.6	8.9
R10	12.4	8.8 - 10.1
R11	13.0	10.9
R12	12.9	Not Accessible
R13	11.5	13.5
R14	12.0	11.4
R15	16.6	Not Accessible
R16	18.5 (mid)	Not Accessible
R17	12.1	7.1

KEY FOR CONCRETE CRACKING :

Location	Description	Thickness (approx.)
C1	Vertical crack propagating from soffit of beam (PB5) down to approximately 950mm above ground.	1mm
C2	Horizontal crack running full width of pier between 1.3m & 1.6m above ground.	Varies <1mm
C3	Horizontal crack running full width both sides of pier between 1.3m & 1.6m above ground.	Varies <1mm

N
W E
S



PLAN LAYOUT OF BRIDGE DECK STRUCTURE (1:50)

USE GIVEN DIMENSIONS SHOWN ON THIS DRAWING.

SAFETY HEALTH AND ENVIRONMENTAL INFORMATION:

IN ADDITION TO THE HAZARD/RISKS NORMALLY ASSOCIATED WITH THE TYPES OF WORK DETAILED ON THIS DRAWING, NOTE THE FOLLOWING RISKS AND INFORMATION.

RISKS LISTED HERE ARE NOT EXHAUSTIVE.
REFER TO DESIGN ASSESSMENT FORM NUMBER: 140163F-DAF-01

CONSTRUCTION:
THERE ARE NO SIGNIFICANT HAZARDS THAT WILL NOT BE APPARENT TO A COMPETENT CONTRACTOR.

DEMOLITION:
NOT APPLICABLE

FOR INFORMATION RELATING TO USE, CLEANING AND MAINTENANCE SEE THE HEALTH AND SAFETY FILE

IT IS ASSUMED THAT ALL WORKS WILL BE CARRIED OUT BY A COMPETENT CONTRACTOR WORKING, WHERE APPROPRIATE, TO AN APPROVED METHOD STATEMENT.

GENERAL NOTES :

1. ALL DIMENSIONS SHOWN ARE IN MILLIMETERS.
2. BRIDGE SURVEY/INSPECTION WAS CARRIED OUT ON MONDAY 14TH FEBRUARY 2022.

SCALE BAR FOR 1:50

000mm 400mm 800mm 1200mm 1600mm 2000mm 2400mm 2800mm 3200mm 3600mm 4000mm

REV.	DATE	DESCRIPTION	DRWN.	CHKD.	APPD.

FAIRHURST

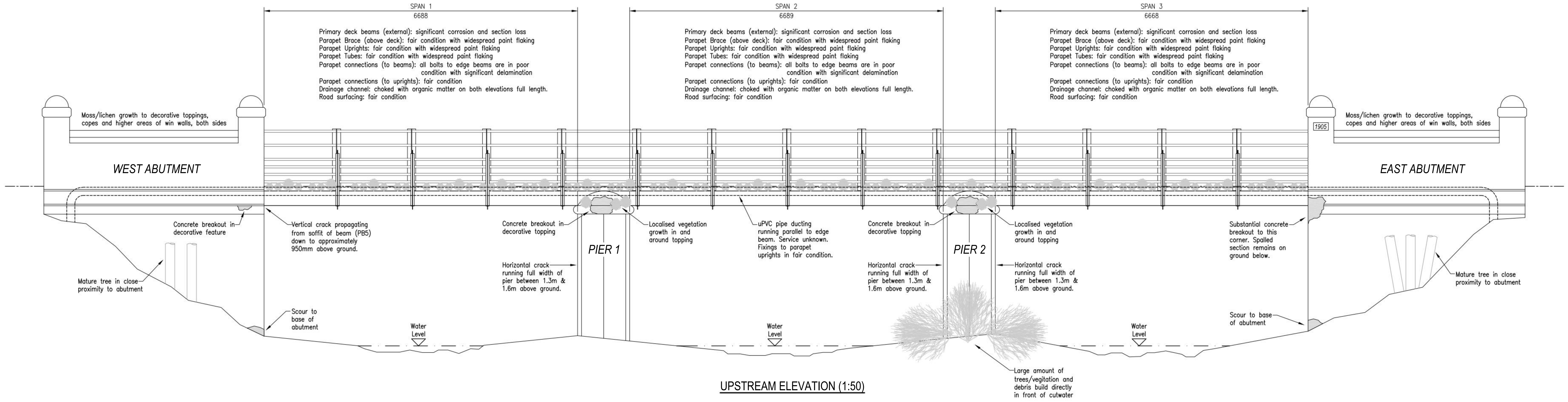
25a HIGH STREET, ELGIN, IV30 1EE
TELEPHONE: 01343 543876
FAX: 0844 381 4412

CLIENT :
THE MORAY COUNCIL
ECONOMY, ENVIRONMENT & FINANCE
PO BOX 6760
ELGIN IV30 1BX

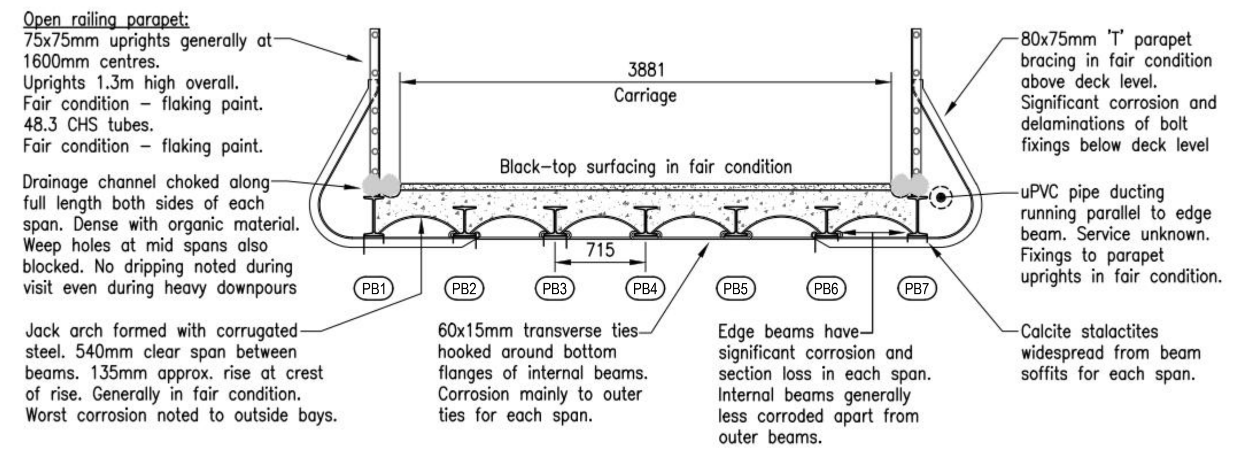
PROJECT TITLE :
ASSESSMENT OF
CLODDACH BRIDGE
BIRNIE ELGIN

DRAWING TITLE :
DEFECTS SCHEDULE
(SHEET 1 OF 2)

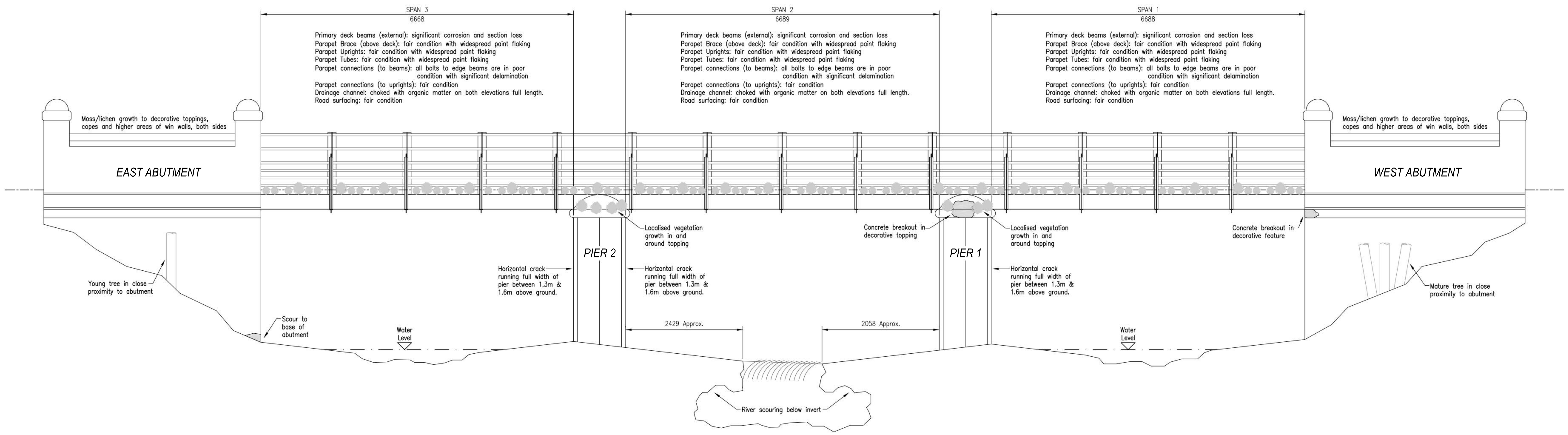
SCALE AT A1 / A3 : 1:50 / 1:100	DRAWING STATUS : INFORMATION		
DESIGNED :	DRAWN : CC	CHECKED : MW	APPROVED : GS
DATE :	DATE : 17/02/2022	DATE : 18/02/2022	DATE : 18/02/2022
DRAWING NUMBER : 140163F/01			REVISION : -



UPSTREAM ELEVATION (1:50)



SECTION 'A-A' :
TYPICAL SECTION THROUGH BRIDGE DECK (1:50)



DOWNSTREAM ELEVATION (1:50)

USE GIVEN DIMENSIONS SHOWN ON THIS DRAWING.

SAFETY HEALTH AND ENVIRONMENTAL INFORMATION:

IN ADDITION TO THE HAZARD/RISKS NORMALLY ASSOCIATED WITH THE TYPES OF WORK DETAILED ON THIS DRAWING, NOTE THE FOLLOWING RISKS AND INFORMATION.

RISKS LISTED HERE ARE NOT EXHAUSTIVE.
REFER TO DESIGN ASSESSMENT FORM NUMBER: 140163F-DAF-01

CONSTRUCTION:
THERE ARE NO SIGNIFICANT HAZARDS THAT WILL NOT BE APPARENT TO A COMPETENT CONTRACTOR.

DEMOLITION:
NOT APPLICABLE

FOR INFORMATION RELATING TO USE, CLEANING AND MAINTENANCE SEE THE HEALTH AND SAFETY FILE

IT IS ASSUMED THAT ALL WORKS WILL BE CARRIED OUT BY A COMPETENT CONTRACTOR WORKING, WHERE APPROPRIATE, TO AN APPROVED METHOD STATEMENT.

GENERAL NOTES :

1. ALL DIMENSIONS SHOWN ARE IN MILLIMETERS.
2. BRIDGE SURVEY/INSPECTION WAS CARRIED OUT ON MONDAY 14TH FEBRUARY 2022.

SCALE BAR FOR 1:50

000mm 400mm 800mm 1200mm 1600mm 2000mm 2400mm 2800mm 3200mm 3600mm 4000mm

REV.	DATE	DESCRIPTION	DRWN.	CHKD.	APPD.

FAIRHURST

25a HIGH STREET, ELGIN, IV30 1EE
TELEPHONE: 01343 543876
FAX: 0844 381 4412

CLIENT :
THE MORAY COUNCIL
ECONOMY, ENVIRONMENT & FINANCE
PO BOX 6760
ELGIN IV30 1BX

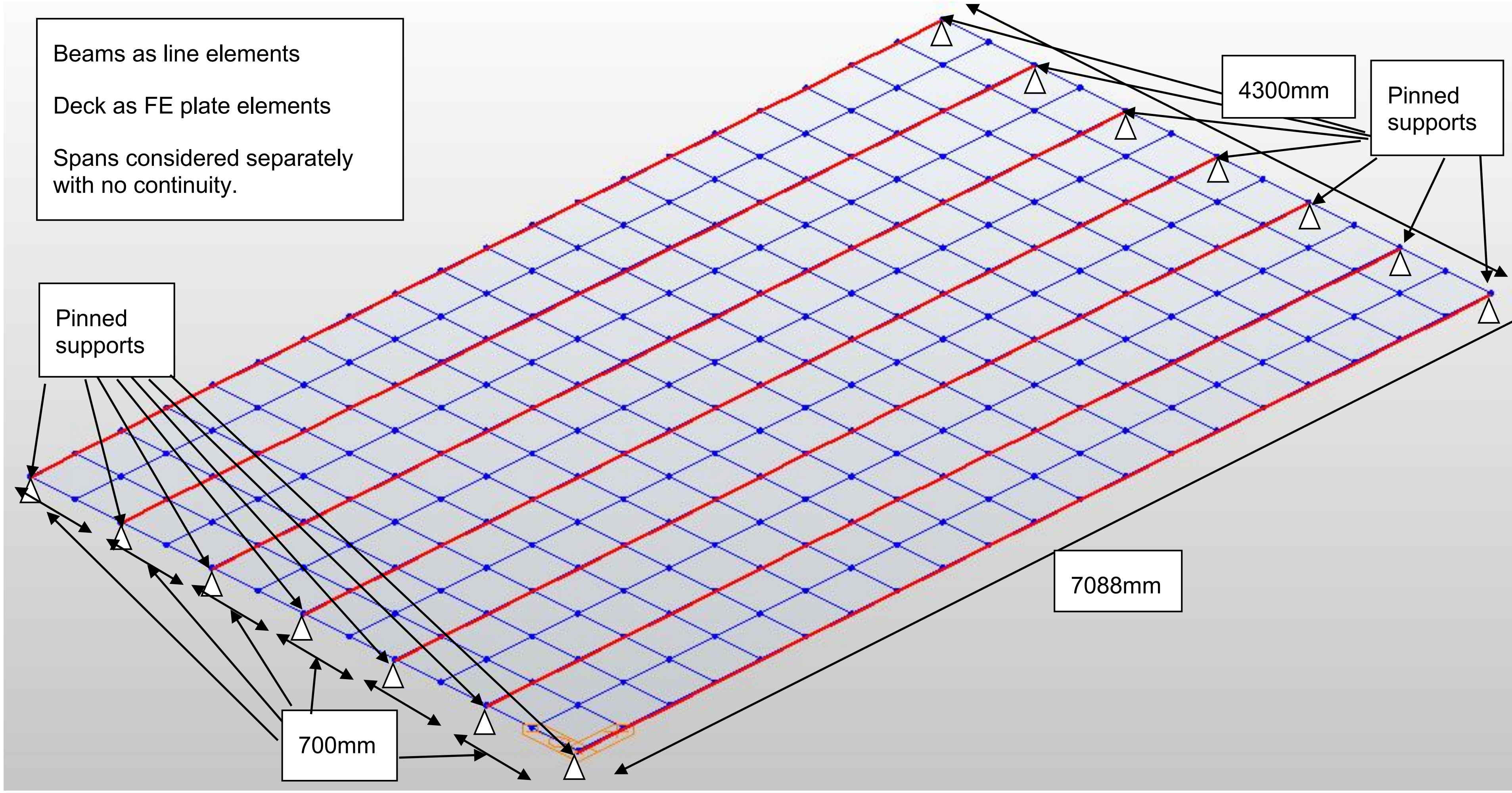
PROJECT TITLE :
ASSESSMENT OF
CLODDACH BRIDGE
BIRNIE ELGIN

DRAWING TITLE :
DEFECTS SCHEDULE
(SHEET 2 OF 2)

SCALE AT A1 / A3 : 1:50 / 1:100	DRAWING STATUS : INFORMATION		
DESIGNED :	DRAWN : CC	CHECKED : MW	APPROVED : GS
DATE :	DATE : 17/02/2022	DATE : 18/02/2022	DATE : 18/02/2022
DRAWING NUMBER : 140163F/02			REVISION : –

APPENDIX D

Idealised Structure Diagrams


Beams as line elements
Deck as FE plate elements
Spans considered separately with no continuity.
Pinned supports
4300mm
Pinned supports
7088mm
700mm

www.fairhurst.co.uk

Aberdeen
Aberdeen Westhill
Birmingham
Bristol
Dundee
Edinburgh
Elgin
Glasgow
Huddersfield
Inverness
Leeds
London
Newcastle upon Tyne
Sevenoaks
Taunton
Thurso
Watford

FAIRHURST

CIVIL ENGINEERING • STRUCTURAL ENGINEERING • TRANSPORTATION • ROADS & BRIDGES PORTS & HARBOURS • GEOTECHNICAL & ENVIRONMENTAL ENGINEERING • PLANNING & DEVELOPMENT • WATER SERVICES • HEALTH & SAFETY / CDM SERVICES

Appendix D

Assessment Calculations

Summary and Assumptions

- Fairhurst was appointed by Moray Council to carry out an inspection and structural assessment of the Cloddach Bridge near Elgin. The Inspection for Assessment was undertaken on the 14th of February 2022. The inspection of the structure found it to be in a poor condition. Significant deterioration of the steelwork was noted and large areas of scour near the structure supports. Cloddach Bridge is a three span bridge over the River Lossie. The bridge is constructed of steel beams acting compositely with jack arched insitu concrete deck slab. The bridge is currently closed to traffic.
- The structure is generally in poor condition with significant corrosion to the steel beams. The invert has been altered and a large gorge formed which is creating scour close to the bridge piers.
- The carriageway is approx. 3.9m wide and there is no verge or kerb to either side of the carriageway. The substructure is formed of mass concrete with a finishing render to give the appearance of masonry.
- Foundations are unknown but are assumed to be spread footings onto the shallow bedrock.
- The bridge deck will be analysed using a 2D Line and Finite Element plate model in analysis software assuming load is transferred through the bridge deck via the concrete jack arches to the steel longitudinal girders. The Finite Element plates will be assumed as concrete only, with minimum thickness used.
- The longitudinal girders will be assessed as simply supported. Permanent loads acting on the structure shall be determined in accordance with CS 454, Assessment of Highway Bridges and Structures (DMRB 3.4.3).

Assessment Live Loads (ALL) will be considered in accordance with CS 454. If the structure is found to have insufficient capacity for 40 tonnes ALL, the structure will be checked for reduced loading until the vehicle capacity of the structure can be confirmed.

- The capacity of the steel sections will be determined in accordance with CS 454 Section 8, assuming end fixing factor 1.0 (both ends pin jointed).
- The unit weight of steel will be assumed to be 7850kg/m3.
- Section thicknesses are based on estimates of remaining section thickness from site inspection report
- A condition factor of 0.45 will be applied to the concrete jack arches and deck slab.

Load generation for model application

Inner Beam

Density of Steel; D	=	78.5	kN/m^3	CS454 Table 4.1.1a
Area of steel based on design section; A	=	10031.96	mm^2	
UDL load imposed by Self-weight load case; Q	=	0.788	kN/m	

Outer Beam

Density of Steel; D	=	78.5	kN/m^3	CS454 Table 4.1.1a
Area of steel based on design section; A	=	10100.77	mm^2	
UDL load imposed by Self-weight load case; Q	=	0.793	kN/m	

Concrete

Density of Concrete; D	=	24	kN/m^3	CS454 Table 4.1.1a
Thickness of equivalent concrete section; t	=	332	mm^2	Noted in Assessment calc
UDL load imposed by Self-weight load case; Q	=	7.968	kN/m^2	26/07/2000

Surfacing

Density of Surfacing material; D	=	23	kN/m^3	CS454 Table 4.1.1a
Thickness of surfacing section; t	=	50	mm^2	
UDL load imposed by Self-weight load case; Q	=	1.15	kN/m^2	

Parapets

Estimated self weight parapet per unit length; Q	=	0.6	kN/m

Vehicles

Vehicle load based on wheel and axle layout as noted in CS 454 Table B.1 — CS454 Table B.1

Placing particular focus on reference O, N and M.

Additional vehicles were checked for Fire Engine Group 1 and 2 as noted in CD 454 Table B.2 — CS454 Table B.2

Assessment Live Load 2 loads are based on CS454 5.17 to 5.19

The calculation was based on low traffic with poor surface

Partial factor for normal traffic action	=	1.30	
Loaded length	=	7.08	m
Assessment Live Load 2 UDL (3T)	=	16.24	kN/m
Assessment Live Load 2 KEL (3T)	=	21.32	kN
Assessment Live Load 2 UDL (7.5T)	=	29.38	kN/m
Assessment Live Load 2 KEL (7.5T)	=	38.58	kN
Assessment Live Load 2 UDL (18T)	=	53.10	kN/m
Assessment Live load 2 KEL (18T)	=	69.72	kN
Assessment Live Load 2 UDL (G1FE)	=	40.34	kN/m
Assessment Live Load 2 KEL (G1FE)	=	52.98	kN
Assessment Live Load 2 UDL (G2FE)	=	20.30	kN/m
Assessment Live Load 2 KEL (G2FE)	=	26.65	kN

Structure assumptions

For shear calculation, webs in inner beams are considered as corroded using a similar level of corrosion to the beams. This conservatively reduces the web to 6mm thick.
For shear calculation, webs in outer beams are consdered affected by corrosion, and are assumed to be 7.1mm thick, as noted in the worst measured section near supports

For bending calculation, inner beams are checked for bending assuming torsion and buckling are not required. The presence of concrete around the beam and the concrete deck means that lateral movement in the compression flanges are unlikely.

Structure assumptions (continued)

Tension flange thickness for inner beams are taken as 12.9mm, as measured in the worst measured section near midspan, see photo 26, midspan of east span, 2nd beam from the south.
Thinner sections of flange are recorded, but these are noted near the hogging points where tension in the exposed flange is negligible.
Outer beam moment calculations shall assume a flange thickness of 9.9mm, as measured at midspan of the northernmost beam on the East span. This is assumed equal for both tension and compression flange.

Presence of shear links is not known, so composite effects are not considered. Bending effects will be assumed to solely be taken by the steel beam
Shear effects at support points are assumed taken by the beams, with an allowance for potential concrete assistance if it would otherwise be a leading cause for failure.
EG, if the beams would fail in bending at 7.5 tonne and shear suggests 18 tonne capacity, concrete assistance would not be considered as the structure would have failed before reaching shear capacity.

Analysis processes

Confirm correct load application

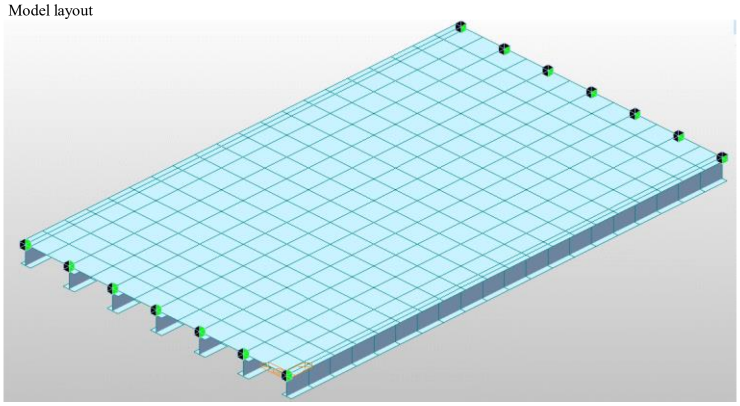
Load application is checked through the use of manual comparisons with expected results from reactions. These load checks were completed for all dead loads
Live load conparisons were checked on a basis of expected results rather than direct comparison due to the variable nature of vehicle load application in the software.

Checking for material and section effects

Sensitivity checks were performed on variable sections or where details of materials were inconclusive or in doubt. Specifically this refers to the deck, where a variety of thicknesses were modelled in the software, and the worst effect chosen to create the more onerous load scenario.

Model assumptions

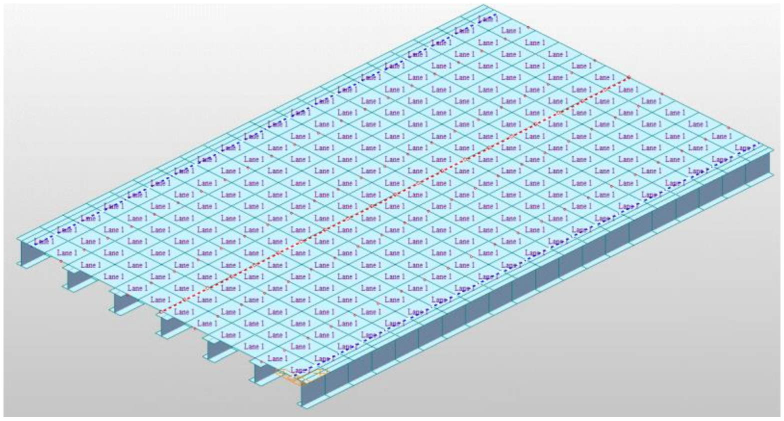
Model layout



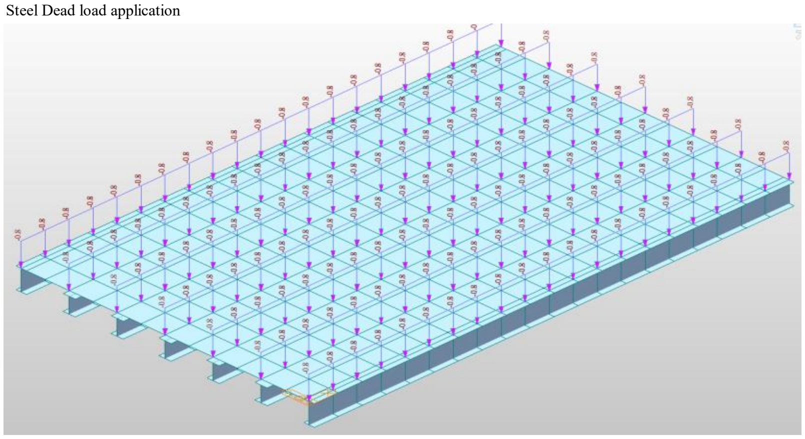
Span	7.08m
CTC between beams	700mm
Deck thickness range	1mm to 332mm

Model assumptions

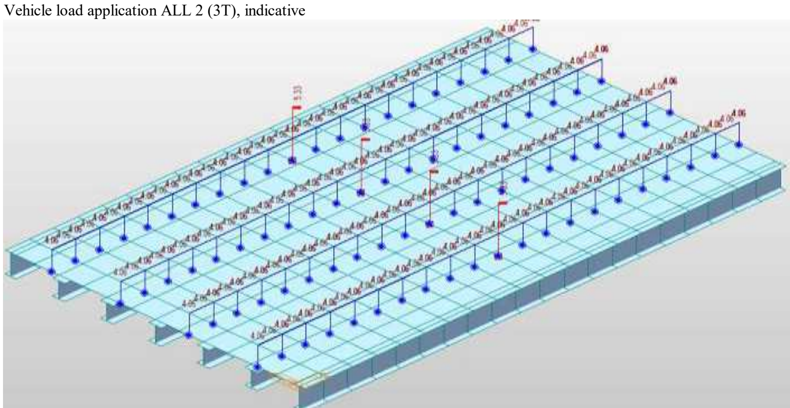
Vehicle lane load location



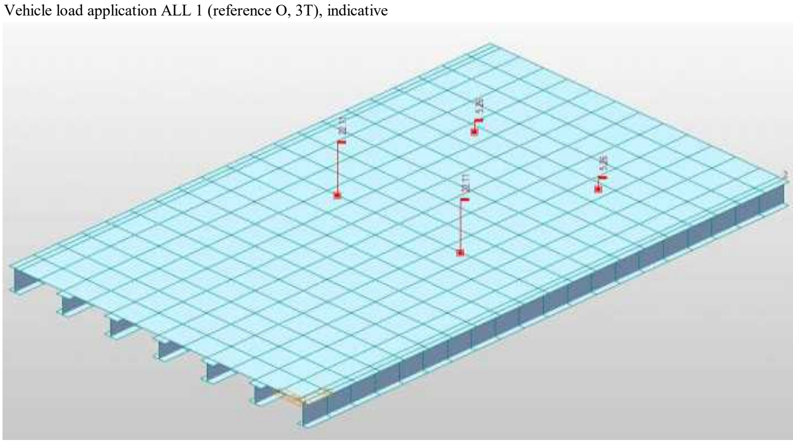
Steel Dead load application



Vehicle load application ALL 2 (3T), indicative



Vehicle load application ALL 1 (reference O, 3T), indicative



Live Load check

Live load positions were determined using a Line Influence Diagram analysis to determine the worst positioning for the applied vehicle. This method was completed automatically through the software. As expected, at maximum bending moment the calculation places vehicle axles at or near the midspan of the bridge. In shear, the calculation places vehicle axles at or near the supports, where higher shear loads are expected.

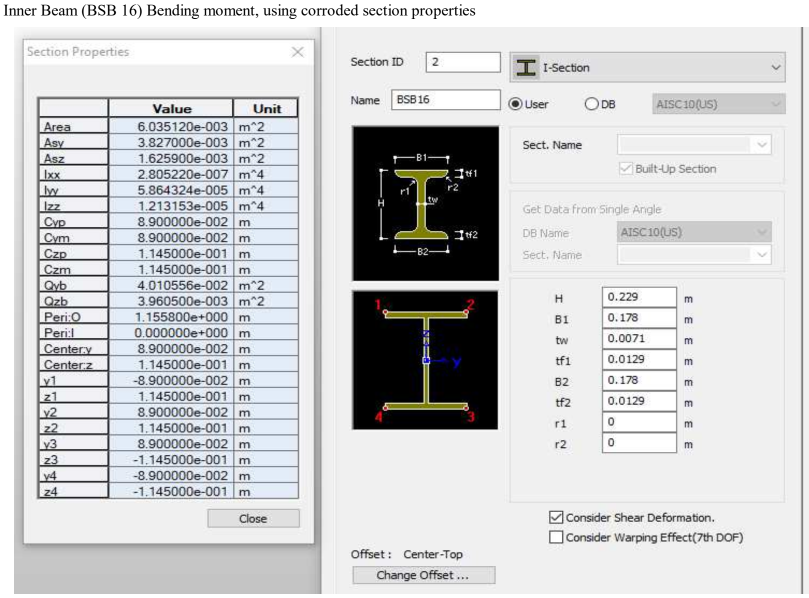
Images above are noted as indicative, as in the case of ALL2 the only notable changes are load values where position remains identical. For ALL1 live loads, the vehicle layouts are taken as required for table B.1 and B.2 in CS454.

Load factors

Condition Factor	=	0.45	From inspection
Steel Self weight	=	1.05	CS454 Table A1
Concrete Self Weight	=	1.20	CS454 Table A1
Surfacing	=	1.75	CS454 Table A1
Parapets	=	1.20	CS454 Table A1
Footway Loading	=	1.50	CS454 Table A1
Live Loads	=	1.50	CS454 Table A1
Impact factor	=	1.80	CS454 Table 5.9a
Traffic flow factor	=	0.90	CS454 Table 5.9b
Lane factor	=	1.00	CS454 Table 5.9c
ULS load factor γ_{f3} ULS	=	1.10	ULS load factor
SLS load factor γ_{f3} ULS	=	1.00	SLS load factor
Material factor γm	=	1.05	Material factor

Section Capacities

Inner Beam (BSB 16) Bending moment, using corroded section properties



	Value	Unit
Area	6.035120e-003	m^2
Asy	3.827000e-003	m^2
Asz	1.625900e-003	m^2
Ixx	2.805220e-007	m^4
Iyy	5.864324e-005	m^4
Izz	1.213153e-005	m^4
Cyp	8.900000e-002	m
Cym	8.900000e-002	m
Czp	1.145000e-001	m
Czm	1.145000e-001	m
Qyb	4.010556e-002	m^2
Qzb	3.960500e-003	m^2
Peri:O	1.155800e+000	m
Peri:I	0.000000e+000	m
Center:y	8.900000e-002	m
Center:z	1.145000e-001	m
y1	-8.900000e-002	m
z1	1.145000e-001	m
y2	8.900000e-002	m
z2	1.145000e-001	m
y3	8.900000e-002	m
z3	-1.145000e-001	m
y4	-8.900000e-002	m
z4	-1.145000e-001	m

Section properties
Calculated using
MIDAS Civil software

Section properties

Ultimate bending moment capacity: M_{ult}	=	117.80	kNm	BS 5400-3 2000 Cl 9.9.1
Section modulus compression flange: Z_{xc}	=	512168.03	mm^3	As modified by CS 456
Steel Yield capacity compression flange: σyc	=	230	N/mm^2	
Section modulus tension flange: Z_{xt}	=	512168.03	mm^3	
Steel Yield capacity tension flange: σyt	=	230	N/mm^2	
Section modulus web: Z_{xw}	=	626048.77	mm^3	
Steel Yield capacity web: σyw	=	230	N/mm^2	
Second moment of Inertia: I	=	5.86E+07	mm^4	
Extreme fiber distance flanges: y_{flange}	=	114.5	mm	
Extreme fiber disutance web: yweb	=	93.672	mm	
Design bending moment capacity: M_D	=	**91.79**	kNm	

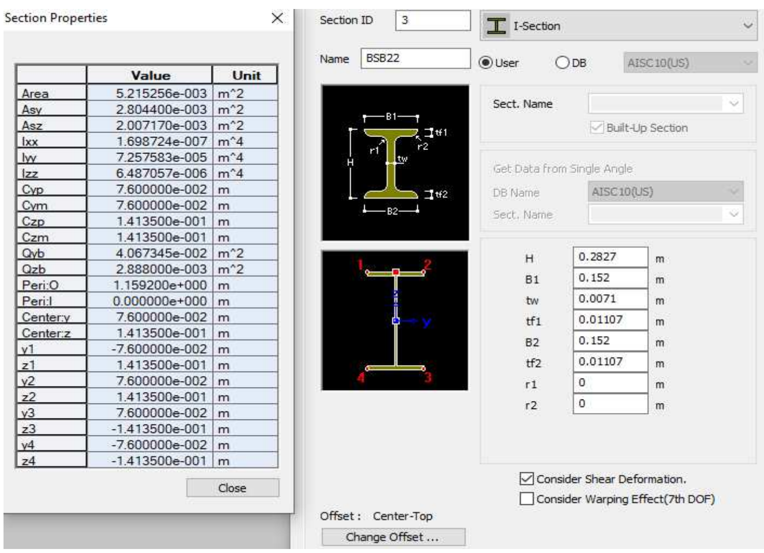
As the capacity is calculated through the corroded section properties, no additional allowance needs to be made for the inclusion of the condition factor of the structure.

Section Capacities (continued)

Inner Beam (BSB 16) Shear, using corroded section properties

Design Shear resistance: V_D	219.70	kN	BS 5400-3 2000 Cl 9.9.2
Thickness web: tw	6	mm	As modified by CS 456
Depth of web between flanges: dw	187.34	mm	
Height of largest hole or cut-out: hh	0	mm	
Limiting shear strength of the web panel: τl	1.7		
Shear strength of web panel: τy	132.79	N/mm^2	
Slenderness ratio for determination of τl: λ	25.13	mm	
Depth of web clear between flange plates: dwe	187.34	mm	
Value for determining limiting shear strength of web: mfw	0.0916		
Distance from mid-plane of web to nearer edge flange: bfe	88.9	mm	
Flange plate thickness: tf	20.828	mm	
Aspect ration of web panel: φ	3.736	mm	
clear length of panel between transverse stiffeners: a	700	mm	
Value for VD when mfw = 0: VR	129.23	kN	

Outer Beam (BSB 22) Bending moment, using corroded section properties


Section Properties

	Value	Unit
Area	5.215256e-003	m^2
Asy	2.804400e-003	m^2
Asz	2.007170e-003	m^2
Ixx	1.698724e-007	m^4
Iyy	7.257583e-005	m^4
Izz	6.487057e-006	m^4
Cyp	7.600000e-002	m
Cym	7.600000e-002	m
Czp	1.413500e-001	m
Czm	1.413500e-001	m
Qyb	4.067345e-002	m^2
Qzb	2.888000e-003	m^2
Peri:O	1.159200e+000	m
Peri:I	0.000000e+000	m
Center:y	7.600000e-002	m
Center:z	1.413500e-001	m
y1	-7.600000e-002	m
z1	1.413500e-001	m
y2	7.600000e-002	m
z2	1.413500e-001	m
y3	7.600000e-002	m
z3	-1.413500e-001	m
y4	-7.600000e-002	m
z4	-1.413500e-001	m

Ultimate bending moment capacity: M_{ult}	=	109.53	kNm	BS 5400-3 2000 Cl 9.9.1
Section modulus compression flange: Z_{xc}	=	476219.36	mm^3	As modified by CS 456
Steel Yield capacity compression flange: σyc	=	230	N/mm^2	
Section modulus tension flange: Z_{xt}	=	476219.36	mm^3	
Steel Yield capacity tension flange: σyt	=	230	N/mm^2	
Section modulus web: Z_{xw}	=	509304.07	mm^3	
Steel Yield capacity web: σyw	=	230	N/mm^2	
Second moment of Inertia: I	=	7.26E+07	mm^4	
Extreme fiber distance flanges: y_{flange}	=	152.4	mm	
Extreme fiber disutance web: yweb	=	142.5	mm	

Design bending moment capacity: M_D = **85.35** kNm

As the capacity is calculated through the corroded section properties, no additional allowance needs to be made for the inclusion of the condition factor of the structure.

Section Capacities (continued)

Outer Beam (BSB 22) Shear, using corroded section properties

Design Shear resistance: V_D	248.93	kN	BS 5400-3 2000 Cl 9.9.2
Thickness web: tw	7.1	mm	As modified by CS 456
Depth of web between flanges: dw	285.00	mm	
Height of largest hole or cut-out: hh	0	mm	
Limiting shear strength of the web panel: τl	1.07		
Shear strength of web panel: τy	132.79	N/mm^2	
Slenderness ratio for determination of τl: λ	32.31	mm	
Depth of web clear between flange plates: dwe	285.00	mm	
Value for determining limiting shear strength of web: mfw	0.0076		
Distance from mid-plane of web to nearer edge flange: bfe	88.9	mm	
Flange plate thickness: tf	9.9	mm	
Aspect ration of web panel: φ	2.456	mm	
clear length of panel between transverse stiffeners: a	700	mm	
Value for VD when mfw = 0: VR	232.64	kN	

Load results

Internal Beams

	Mmax	Vcoex	Vmax	Mcoex	MD	VD	Ratio M	Ratio V
Dead Load (factored)	52.99	1.90	29.60	10.30	91.791	103.95	58%	28%
Live 3t	27.80	12.65	24.47	8.34	91.791	103.95	88%	52%
Live 7.5t	75.62	33.26	65.96	22.50	91.791	103.95	140%	92%
Live 18t	144.60	62.93	130.55	44.54	91.791	103.95	215%	154%
Live Type O (3t)	37.04	14.45	30.45	10.46	91.791	103.95	98%	58%
Live Footway	27.75	4.61	15.15	5.43	91.791	103.95	88%	43%
Live ALL2 3T	35.45	9.93	19.95	6.96	91.791	103.95	96%	48%
Live ALL2 7.5T	64.14	16.92	36.41	12.59	91.791	103.95	128%	63%
Live ALL2 18T	115.91	30.62	65.79	22.74	91.791	103.95	184%	92%
Live ALL2 G1FE	88.08	26.24	50.00	17.30	91.791	103.95	154%	77%
Live ALL2 G2FE	44.31	12.42	25.16	8.70	91.791	103.95	106%	53%

External Beams

	Mmax	Vcoex	Vmax	Mcoex	MD	VD	Ratio M	Ratio V
Dead Load (factored)	36.68	-	18.32	-	85.347	232.64	43%	8%
Live 3t	10.73	-	0.83	-	85.347	232.64	56%	8%
Live 7.5t	28.61	-	2.13	-	85.347	232.64	76%	9%
Live 18t	54.80	-	4.68	-	85.347	232.64	107%	10%
Live Type O (3t)	16.11	-	1.88	-	85.347	232.64	62%	9%
Live Footway	16.08	-	2.64	-	85.347	232.64	62%	9%
Live ALL2 3T	19.19	-	3.59	-	85.347	232.64	65%	9%
Live ALL2 7.5T	34.73	-	6.36	-	85.347	232.64	84%	11%
Live ALL2 18T	62.75	-	11.51	-	85.347	232.64	116%	13%
Live ALL2 G1FE	47.67	-	9.02	-	85.347	232.64	99%	12%
Live ALL2 G2FE	23.99	-	4.49	-	85.347	232.64	71%	10%

Note, loads in report are different. Those values are based on Checker values, which are based on a more conservative check

Appendix E
Assessment Certificates

DMRB Assessment Certificate

Name of Project: **The Moray Council Inspections and Assessments**
Name of Structure: **Cloddach Bridge Assessment**

1. We certify that reasonable professional skill and care has been used in the preparation of the assessment of Cloddach Bridge with a view to securing that:

   It has been assessed and checked in accordance with **the Approval in Principle dated 1st March 2022**

   The assessed capacity of the structure is as follows:

   - Pedestrian ALL in accordance with CS454

Signed ___

Name ___ Ellen Halkon ___

Assessment Team Leader

Engineering Qualifications __MEng CEng MICE___

Signed ___

Name ___Ross Gray___

Position held ___ Partner ___

Name of Organisation ___ Fairhurst___

Date ___22nd March 2022___

2. ~~The Departures and additional criteria given in paragraph 1 are agreed~~ 3. The certificate is accepted by the TAA

Signed ___

Name ___Daniel Preston ___

Position held ___Senior Engineer (Structures)___

Engineering Qualifications __MEng CEng MICE___

TAA ___Moray Council___

Date ___28 March 2022___

DMRB Check Certificate

Name of Project: **The Moray Council Inspections and Assessments**
Name of Structure: **Cloddach Bridge Assessment**

1. We certify that reasonable professional skill and care has been used in the preparation of the assessment check of Cloddach Bridge with a view to securing that:

It has been checked in accordance with **the Approval in Principle dated 1st March 2022**
The assessed capacity of the structure is as follows:

- Pedestrian ALL in accordance with CS454

Signed _ _

Name __________________ Chris Butler ___________________

Check Team Leader

Engineering Qualifications __MEng CEng MICE_________________

Signed ____

Name __________________Ross Gray_____________________

Position held ____________ Partner ______________________

Name of Organisation ____ Fairhurst______________________

Date ____________22nd March 2022__________________________

2. ~~The Departures and additional criteria given in paragraph 1 are agreed~~ 3. The certificate is accepted by the TAA

Signed ____

Name _________Daniel Preston ___________________________

Position held ___Senior Engineer (Structures)_______________

Engineering Qualifications __MEng CEng MICE______________

TAA ____________Moray Council___________________________

Date ____________28 March 2022___________________________

Appendix F
Cost Estimate Breakdown

Option 1

Ongoing Inspections - Assumptions

1 visit per month by local inspector cost £1000/month

1 visit every three months by inspector with ladder access to access and measure flange thickness at reference points

Install bollards and signage - Assumed £15,000

Option 2

Demolition of the superstructure:

	QTY	UNIT	Assumed Rate (SPONS2020)	Cost
Demolition of reinforced concrete superstructure	44.00	m^3	£67.31	£2,961.64
Take down and removal of steel girders	11.65	tonne	£216.30	£2,519.90
Temporary Works				£20,000
Preliminaries				£10,000
Risk				£5000
TOTAL (Rounded) SUPERSTRUCTURE DEMOLITION				**£40,000**

The inclusion of the substructure would increase the likely budget costs to approximately £120,000.

	QTY	UNIT	Assumed Rate (SPONS 2020)	Cost
Demolition of reinforced concrete superstructure	44.00	m^3	£67.31	£2,961.64
Take down and removal of steel girders	11.65	tonne	£216.30	£2,519.90
Substructure Demolition	200	m^3	£183.56	£36,712.00
Temporary Works				£40,000

Preliminaries				£20,000
Risk				£15,000
TOTAL (Rounded) DEMOLITION				**£120,000**

Option 3

Steelwork and Concrete Repairs

Estimated at £250,000 based on tendered process of similar recent projects

Option 4

Full refurbishment to allow vehicle use with no restriction

Superstructure Repairs - Substantial steelwork repairs, repainting, upgrade of edge beam and parapets - estimated cost £1,000,000

Substructure Repairs: Underpinning of concrete abutments, strengthening of abutments – estimated cost £750,000

No allowance for design fees or risk has been included

Total estimated cost £1,750,000

Option 5

Demolition and Replacement

Site Clearance	**£ 80,000**
Temporary Works	**£ 50,000**
Substructure	**£ 860,000**
Superstructure	**£ 370,000**
Preliminaries	**£ 400,000**
Total	**£ 1,760,000**

The costs above have been based on SPONS 2020 rates assuming a single span new bridge width 7m and span 24m. **No allowance for design fees or risk has been included**