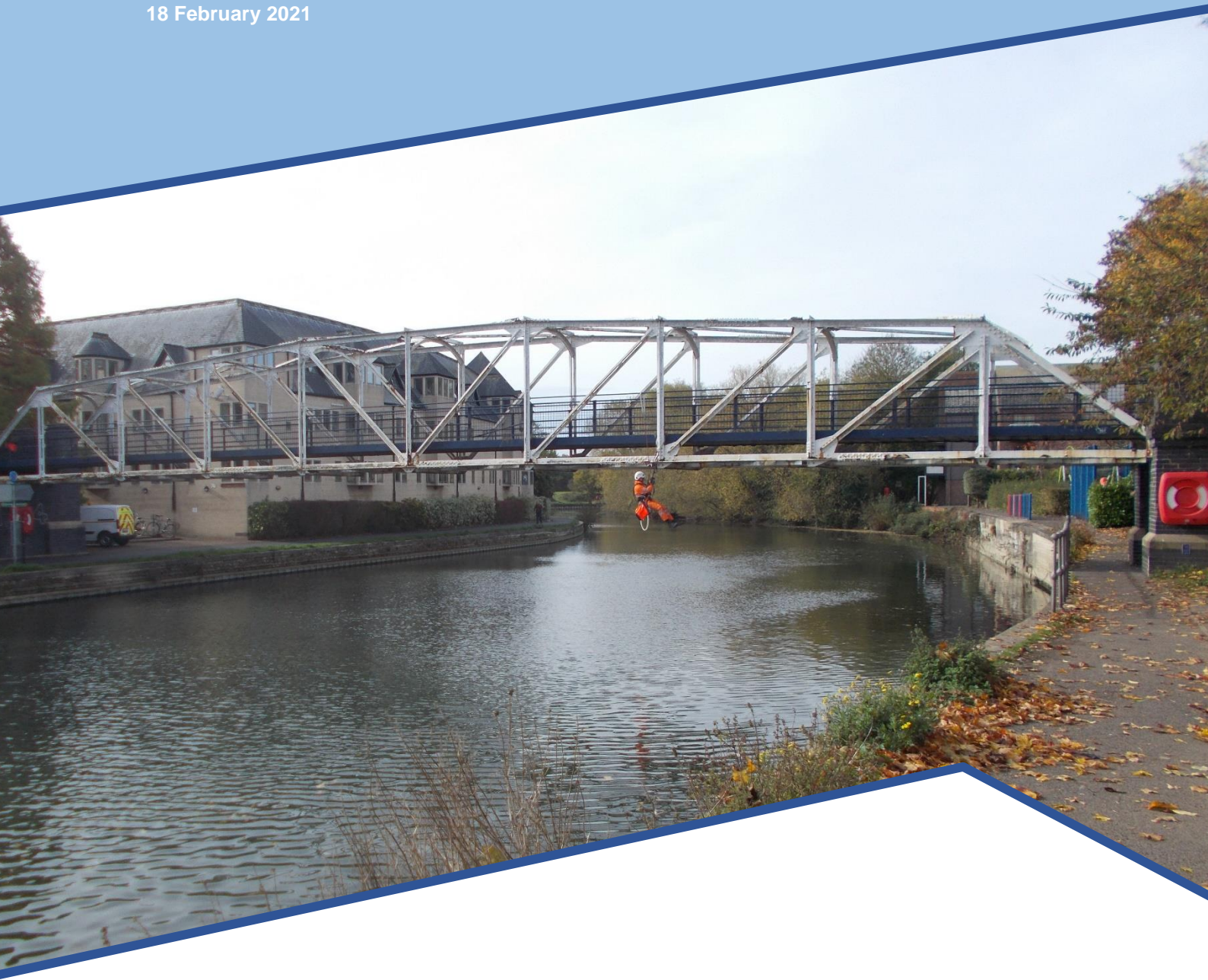


Isis Gasworks Footbridge Assessment Report


Doc. Ref. 8019146-8486-AR-001


18 February 2021

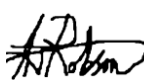


Isis Gasworks Footbridge Assessment Report

Doc. Ref. 8019146-8486-AR-001

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Document History

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0	Draft report	G Jegorovs	B Dodds		
1	Amended as per review comments	G Jegorovs	B Dodds	A Robson	18/02/2021

Summary

8486 Isis Gasworks Footbridge was assessed by Skanska UK Oxfordshire Highways office and the assessment check was completed by Skanska UK TechServ office in accordance with the Approval in Principle document from 06 January 2019. Both the assessor and the checker found that the steel truss to have inadequate capacity for pedestrian crowd loading, while as the Bison type precast concrete decking planks have sufficient capacity only in 'as new' condition, where as in reality they are showing signs of failing. The ramps to the structure were found to have adequate capacity, but do not comply with the current minimum width and maximum gradient standards.

It is concluded that the maintenance and strengthening works on the structure are not likely to be cost-efficient due to the significant number of constraints associated with the works required and the benefits this would bring. Instead, it is recommended to completely replace Isis Gasworks Footbridge including the ramps, as the cost of this is likely to be comparable to the cost of maintenance works.

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1 Introduction

1.1 General

In 2018, Skanska UK was commissioned by Oxfordshire City and Oxfordshire County Councils to complete an assessment of Isis Gasworks Bridge (OCC No. 8486), which carries a footpath between Friars and Baltic Wharfs over river Thames in Grandpont area of Oxford.

In the same year, Skanska have commissioned HRS Services Ltd to complete the Inspection for Assessment on Isis Gasworks Bridge since there were very limited previous records available on the dimensions, the details and the condition of the bridge. As a result, an Inspection for Assessment Report 2018 was prepared, which is attached as Appendix A of this assessment report.

Additional testing on the structure was completed in 2020 by James Fischer Testing Services to determine the approach ramps reinforcement details and concrete strength. This forms Appendix B of this assessment report.

1.2 Structure Description

Isis Gasworks Bridge (OCC No. 8486) is a zero skew single span (approx. 37.3m long) truss bridge with 8no. intermediate bays (approx. 3.66m each) and 1no. bay either side of intermediate bay (approx. 4.03m). The overall width between the centreline of the trusses is approximately 3.04m. The detailed dimensions of the structure are available in the Appendix E of Inspection for Assessment Report from 2018, document ref. 8018122-8486-PIR-001.

The deck of the structure comprises RC hollow planks ("Bison" type), which are supported by 8" x 6" x 5/8" steel angles that are bolted to the plate, which is connected to the vertical members of the trusses. The details of this connection are shown on the Bison Planks Bearing Detail drawing, refer to Appendix C.

The steel trusses are likely to be resting on the old brickwork abutments, although the details are unknown as the brickwork landing might be decorative only and was re-built in 1970s, when the landing slabs were also added to the existing abutments.

There are RC ramps leading to both landings and a staircase at the North end of the bridge leading to the same landing, all constructed in 1970s when Oxford City Council re-purposed the bridge. The bridge had previously supported various utility crossings over the Thames in the current location (Curl, J.S., 1977, 'The Erosion of Oxford').

2 Scope

This report assesses the structural capacity of the steel truss (including connections), RC "Bison" type planks and the RC ramps (excluding the supporting columns) using the DMRB assessment standards. Analysis methods and key assumptions are in accordance with the Approval in Principle Document, refer to Appendix I.

The effects of the condition of the structure on its capacity will be calculated by adopting the condition factors. The condition factors are to be calculated for individual truss elements based on the worst case observed section loss as outlined in the 2018 Inspection for Assessment Report. The derivation of the condition factors will be shown in the calculation sheets, which form part of this report.

The structural capacity of the bridge abutments, bearings (details unknown) and other elements that are not mentioned in the AIP have not been assessed.

3 Assessment Details

3.1 Geometry and Modelling

3.1.1 General

As stated in the AIP, the truss was modelled as a 3d frame in LUSAS FEM software to obtain the member assessment loads. This decision was made to allow application of all asymmetrical actions that might produce the worst-case effects on the transverse members of the bridge i.e. full pedestrian loading applied to half of the deck only and the wind loads. Additionally, a simple 2D truss analysis would not allow to assess the effects of thermal loading on the structure; hence, a decision was made to develop a full 3D model of the bridge.

3.1.2 Truss Members

The members of the truss were modelled as beam elements that can experience axial forces and bending. The sections shown in Appendix E of IfA Report 2018 (pages 118-119) were then assigned to individual beam elements. The sections shown in Appendix E were matched with the corresponding sections from BS 4 “Specification for Structural Steel Sections”, which was current at the time when the bridge was constructed. The full summary of sections that were assigned to different line elements of the truss is shown below in Table 3.1. The local axes of sections was set up to replicate the original orientation of the members in the truss.

All section properties were automatically calculated in LUSAS.

Element	Section	Comment
Truss bottom chord	Double 6" x 6" x 1/2"	
Truss top chord	Double 6" x 6" x 1/2"	
Truss vertical ties	Double 3 1/2" x 3 1/2" x 5/16"	
Truss inclined struts	Double 3" x 3" x 5/16"	

Element	Section	Comment
Bottom horizontal bracings	Single 6" x 5" x 25 lb/ft	
Bottom diagonal bracings	Single 4" x 4" x 5/16"	
Top horizontal bracings		
Arched bracings		
Arched bracings, inclined		
Top diagonal bracings	Single 3½" x 3½" x 5/16"	
Decking angle	Single 8" x 6" x 5/8"	

Table 3.1 – Truss member sections

3.1.3 Truss Connections

Individual connections were not modelled in LUSAS FEM software. Instead, all connections were modelled as pinned i.e. such that the connections would not transfer any moments. Additional nodes were introduced on the individual elements of the truss at the locations, where two truss elements join together and where the loads have to be applied. Some of the examples include the horizontal arched bracing elements between the vertical ties of the trusses that split the truss vertical ties elements into two elements. In such cases, the split of the element was modelled such that it does not result in additional effects to the split element. This is demonstrated in Figure 3.1.

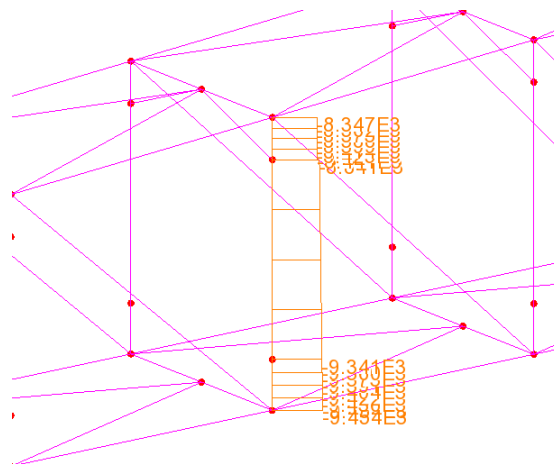


Figure 3.1 – Dummy node example

The connection checks were completed separately, and these are available in the 'Connection Checks' spreadsheet, which forms part of Appendix H of this report. The geometry of the connections was adopted from the IfA 2018 report. The dimensions that are not directly shown on pages 120-129 of the IfA report were measured off the CAD file, which formed part of the IfA report. Where the dimensions of individual rivets/bolts are not specified, it was assumed that these are the same size as the known bolts/rivets. In accordance with the AIP document, where the plate thicknesses are unknown, these are assumed to be the same thickness as the thinnest element that the plate is attached to. Similarly, all bolts/rivets on each connection are assumed to participate.

3.1.4 Bison Planks Decking

The Bison planks and the supporting decking angle were not modelled individually for the purposes of obtaining the loads. Instead, the Bison planks with their hollow core geometry were transformed into a solid slab with a thickness of 80mm that would accurately replicate the effects of the self-weight of the Bison planks. This was done by assuming the geometry of an individual plank as shown in Figure 3.2, finding the area of the concrete and dividing it by the width of the plank.

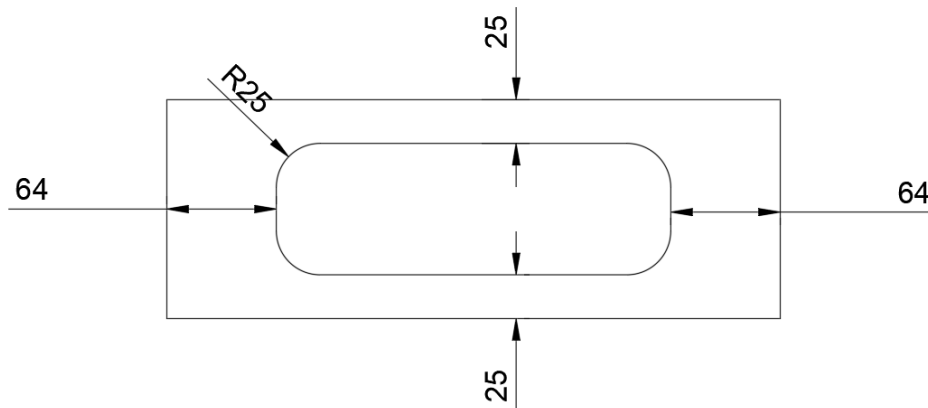


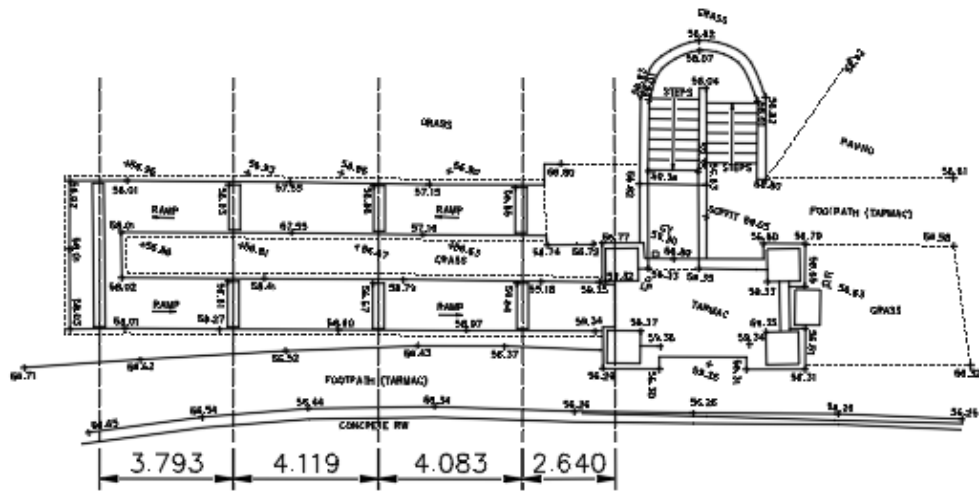
Figure 3.2 – Bison plank cross-section geometry

3.1.5 Ramps

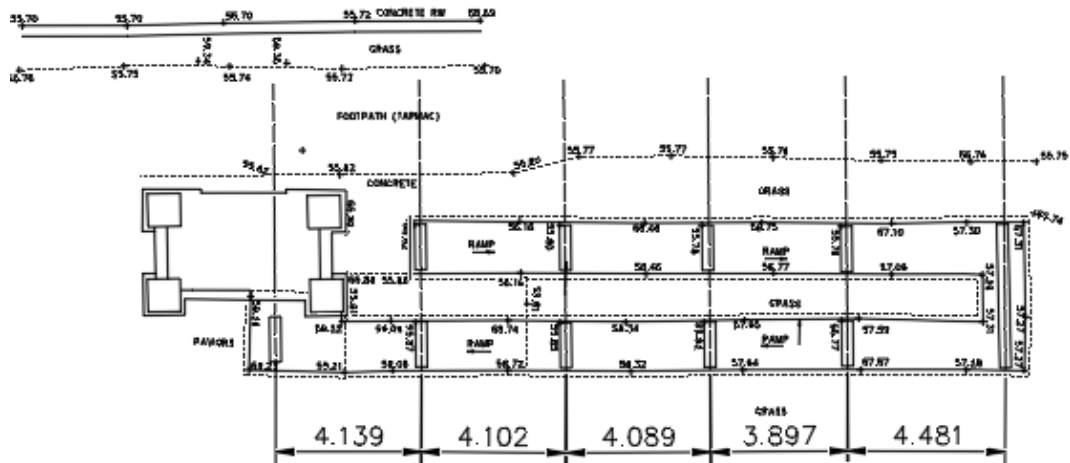
The ramps to the structure were analysed as 1m wide 2D continuously spanning elements over multiple supports in LUSAS. The geometry of the ramps was obtained from the topographical survey of the bridge, which was completed as part of the 2018 IfA. This was combined together with James Fisher Testing Services dimensional survey of the ramp piers and cross-section. The result of this is shown on Figure 3.3.

Since both ramps decking cross-sections are the same, only the section of the ramp with the largest span between the ramp piers will be considered and it is assumed that the analysis of this section will represent the results for both ramps. Based on Figure 3.3, a decision was made to proceed with the analysis of the lower south ramp with a largest span of 4.48m.

James Fisher Testing Services ferroskan results were reviewed and it was established that the longitudinal top and bottom reinforcement bars are 16mm dia. bars at approx. 160mm spacings for the South Ramp. Although the North Ramp was reported to have larger longitudinal bars at similar spacings, the section with 16mm bars was considered for the purpose of this assessment.



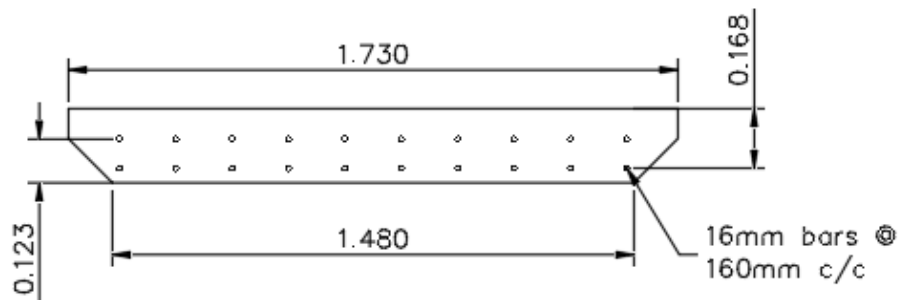
North Ramp
1:200



South Ramp
1:200



South Lower Ramp Elevation
1:200



Ramp Decking Section
for Assessment
1:20

Figure 3.3 – Ramp Geometry

3.2 Loads

3.2.1 Truss Members

The characteristic values of actions considered for assessment of individual members of the truss are shown in Table 3.2. All actions were calculated and applied in accordance with CS454. All action calculations are shown in Appendix D.

Action	Value	Comment
Self-weight (steel)	$\rho = 7850 \text{ kg/m}^3$ CS454 Table 4.1.1a	Automatically calculated in LUSAS based on individual section geometry.
Self-weight (concrete)	$\rho = 2400 \text{ kg/m}^3$ CS454 Table 4.1.1a	Bison plank deck approximated as 80mm thick concrete slab, converted into point loads based on the individual truss bays and truss width.
Pedestrian (whole deck)	4.834 kN/m ²	Reduced due to the length of the bridge and the loaded width.
Pedestrian (applied half of the deck only)	4.834 kN/m ²	Same as above. Applied to half of the deck only as it is likely that during the events on the river (rowing, etc.) a crowd might be spectating and standing on one side of the bridge, which might cause the worst-case effects on the transverse bracing.
Surfacing	0.235 kN/m ²	Assumed self-weight of 2400 kg/m ³ (CS 454, Table 4.1.1a) with 10mm thickness (IfA 2018 report).
Wind	Varies	Transverse and vertical wind loads only applied in accordance with CS 454. Longitudinal loads not applied.
Thermal	Minimum temperature: 40°C Maximum temperature: -20°C Construction temperature: 20°C	Applied as thermal load in LUSAS with the assumed initial temperature of 20°C.
Snow	0.426 kN/m ²	Not required by CS 454, but may contribute to the worst case load effects due to snowing in winter.

Table 3.2 – Loads considered for truss analysis

3.2.2 Bison Planks

The characteristic values of actions considered for assessment of the decking planks are shown in Table 3.3. All loads were calculated and applied in accordance with CS454. All loads calculations are shown in Appendix D.

Load	Value	Comment
Self-weight	$\rho = 2400 \text{ kg/m}^3$ CS454 Table 4.1.1a	Material self-weight in accordance with CS454.
Pedestrian	5.0 kN/m ²	Full value for local effects.
Surfacing	0.235 kN/m ²	Assumed self-weight of 2400 kg/m ³ (CS 454, Table 4.1.1a) with 10mm thickness (IfA 2018).
Snow	0.426 kN/m ²	Although not required by CS 454, this contributes to the worst-case load effects as the bridge is not cleaned during the winter.

Table 3.3 – Loads considered for Bison planks analysis

3.2.3 Ramps

The loads that were considered while assessing the load effects on the ramps and supporting columns are shown in Table 3.4. All loads were calculated and applied in accordance with CS454.

Load	Value	Comment
Self-weight	$\rho = 2400 \text{ kg/m}^3$ CS454 Table 4.1.1a	-
Pedestrian (whole ramp deck)	5.0 kN/m ²	Original 5 kN/m ² as the width of the ramp deck and its length is significantly smaller than the one of the truss bridge.

Table 3.4 – Loads considered for ramps analysis

3.3 Partial Factors for Loads and Combinations

3.3.1 Truss Members

The partial factors for actions at the ultimate limit state were adopted from CS454, Table 3.4. The same assessment standard does not provide guidance on the partial factor for wind, thermal or snow loads. Moreover, CS454 does not provide guidance on the ULS combinations that should be adopted for the purpose of assessment. Hence, the additional partial factors for actions and the combinations were adopted from BD37/01 Loads for Highway Bridges, Table 1. The combinations used for the assessment are summarised in Table 3.5.

CS454 also suggests that the partial factors for all dead and superimposed loads shall be taken as 1.0 where this gives a more severe total effect. This was achieved by using Smart Combinations in LUSAS software, which allows to carry out the analysis using a set of partial factors to find the worst total effects.

Load	Combination			
	1	2.1	2.2	3
Self-weight (steel)	1.05	1.05	1.05	1.05
Self-weight (concrete)	1.15	1.15	1.15	1.15
Pedestrian (whole deck)	1.5	1.25	-	1.25
Pedestrian (applied half of the deck only)	-	-	1.25	-
Surfacing	1.75	1.75	1.75	1.75
Wind	-	1.4	1.4	-
Thermal	-	-	-	1.3
Snow	1.2	1.2	1.2	1.2

Table 3.5 – Partial factors for loads and combinations considered for analysis

3.3.2 Bison Planks

The wind loads are acting vertically on the deck of the structure, however combination 2.1 is aimed at checking the effects of wind loads on the truss to cause the load worst case effects on the transverse members and therefore it is unlikely that combination 2.1 will result in load worst case effects to the individual bison planks. The same is applicable to combination 2.2. Due to the short length of planks, the thermal effects are unlikely to have a significant effect on the stresses in the planks and therefore, combination 3 was not considered for the purpose of the assessment. Hence, the planks were assessed to combination 1 only (see Table 3.5).

3.3.3 Ramps

Due to the small elevation and plan area of the ramp, it was assumed that the wind loads are unlikely to contribute to the worst-case load action. No asymmetric loading has been considered whilst checking the ramps. The side wind loads are also insignificant in comparison to the pedestrian loads and therefore combinations 2.1 and 2.2 were not considered for the purpose of the assessment. The ramp deck is also significantly shorter than the bridge and therefore it was assumed that the thermal load actions will not contribute significantly to the worst-case load effects. Hence, the ramps were assessed to combination 1 only (see Table 3.5).

3.4 Loads Application

3.4.1 Truss Members

The loads from Table 3.5 were applied to individual truss members in LUSAS. The self-weight of the steel truss was applied by the software automatically, based on the section geometry shown in Table 3.1. The wind loads were applied directly to the corresponding members as linear loads. The rest of the loads such as the bison plank self-weight, surfacing self-weight, etc. were applied to the bottom nodes of the bridge as point loads. These loads were calculated based on the loads UDLs, the transverse width of the bridge and the width of individual bays. The bending moment that arises as a result of the loads being applied to the vertical members eccentrically (as shown on Bison Plank connection detail drawing) was applied at the location where the deck is attached to the vertical members. These calculations are shown on sheet 8 of Appendix D.

3.4.2 Bison Planks

With regards to Bison plank checks, the loads from Table 3.3 were transformed into a linear UDL and applied to a single plank to find the worst-case load effects. Based on Section 3.3.2 explanation, it becomes apparent that the worst-case effects will be caused when the linear UDL is applied to the whole length of the plank.

3.4.3 Ramps

Since the length of each individual ramp deck span is significantly exceeding the width of the ramp, to which the loads were applied, the ramp deck is considered to be spanning in one direction only. Hence, a simple metre-strip beam analysis was conducted. The loads from Table 3.3 were transformed into a linear UDL, which was then applied to individual spans of the ramp deck in all possible combinations as shown on sheet 5 of Appendix F.

3.5 Load Effects

3.5.1 Truss Members

The load effects on the individual truss members were obtained by running the LUSAS model with the defined load combinations (refer to section 3.4 for load combinations). Each load combination produced a set of forces/moments (F_x , F_y , F_z , M_x , M_y , M_z) in individual truss members. Hence, for each combination there is at least once combination of forces/moments, where the combined effect of these is the highest. This unique set of highest forces/moments with corresponding forces/moments from that set were then adopted for the assessment of combined load effects for checks in Section 3.6, resulting in a maximum of 6no. sets of forces/moments per element under consideration.

3.5.2 Bison Planks

The planks are analysed as individual beams with UDL applied to the whole length of the plank. There are no loads causing axial compression of the planks or torsion. Since all loads applied to the deck are UDLs and the width of the plank being significantly less than its length, it was assumed that the plank will be subject to bending in longitudinal direction only and the shear force at the supports.

3.5.3 Ramps

Since only a metre-strip analysis is carried out for the ramps and since the ramps are continuous over the piers, the only load effects that are to be considered for checking are the sagging and hogging bending moments as well as the shear force in the ramp deck.

3.6 Checks

3.6.1 Truss Members

The checks on the individual members of the truss were carried out in accordance with CS454, which dictates using BD56/10 that refers to BS5400-3:2000. Since the load combinations take a number of loads into account, all members of the truss were found to be subject to axial forces and moments about the axes of all of the members. Hence, combined axial load and bending checks were completed in accordance with section 9.9.4 of BS5400-3:2000.

The deteriorated condition of individual beams was considered by introducing the condition factor, F_c in the following way:

$$\frac{P}{A_E} + \frac{M_x}{Z_x} + \frac{M_y}{Z_y} \leq \frac{F_c \sigma_y}{\gamma_m \gamma_{f3}} \quad (9.9.4.1) \text{ Yielding of beam}$$

$$\frac{P_{max}}{P_D} + \frac{M_{x,max}}{M_{Dx}} + \frac{M_{y,max}}{M_{Dy}} \leq F_c \quad (9.9.4.2) \text{ Buckling of a beam (compression members only)}$$

Where it could be determined by inspection that the members of the truss are not subject to significant loads and members' section resistance is comparable to the other members, the check was not completed.

The majority of truss sections are compound sections that consist of 2no. angles as shown in Table 3.1. Whilst determining section capacities for the checks above, an assumption was made that the capacity of the whole member will be equal to the sum of capacities of individual angles making up the compound section (whether this is axial compression, tension or bending resistance).

The assumptions and details of checks on the individual members of the truss such as effective sections, lengths, etc. are directly stated on the calculations sheet, refer to Appendix D. The same document is showing the choices and comments with regards to individual member condition factors.

3.6.2 Truss Connections

Similarly to the truss members, the connections of the truss were checked to BS5400-3:2000. In accordance with the AIP, only simple connection check i.e. fastener shear check and the bearing capacity check of the plate was carried out (refer to sections 14.5.3.4 and 14.5.3.6 of BS5400-3).

For the purpose of this assessment, it was assumed that all fasteners (bolts and rivets) would equally contribute to the force transfer. Since the sizes of all fasteners are unknown, the same size for all fasteners participating in the connections was assumed based on the known size of a single fastener.

The deteriorated condition of connection was taken into account by introducing the condition factor, F_c in the following way:

$$\frac{V}{nA_{eq}} \leq \frac{F_c \sigma_q}{\gamma_m \gamma_{f3} \sqrt{2}} \quad (14.5.3.4) \text{ Shear check}$$

$$\frac{V}{A_{eb}} \leq \frac{F_c k_1 k_2 k_3 k_4 \sigma_y}{\gamma_m \gamma_{f3}} \quad (14.5.3.6) \text{ Bearing check}$$

The number of fasteners participating in the connection was determined based on the number of the fasteners in the individual participating truss member. The maximum worst case force (either tensile or compressive) in individual truss member was then adopted to calculate the stress in individual fasteners.

3.6.3 Bison Planks

For the purpose of the assessment, it was assumed that Bison planks are acting as individual beams i.e. there is no force distribution from one plank to another. CS454 refers to BD44/15 for the assessment of reinforced concrete structures and section 5.3.2.3 of BD44/15 was used to find the ultimate limit state bending capacity of individual planks. Since there is no shear reinforcement present in Bison planks, the check to clause 5.3.3.2 of BD44/15 for unreinforced sections was carried out.

Although a number of planks were found to be cracked, the condition factor F_c was not introduced for the purpose of the assessment as the defects could not be accurately represented by this factor.

3.6.4 Ramps

Similarly to Bison planks, the ramp decks are reinforced concrete and due to their aspect ratio in plan there will be no transverse force distribution in them. Hence, the same check for bending to clause 5.3.2.3 of BD44/15 was carried out.

If A 2018 suggests that there are no defects to concrete in ramps and therefore, the ultimate limit state capacity of the decks was not reduced, since the condition factor F_c is assumed to be 1.0.

4 Results

4.1 Truss Members

The truss members were checked in accordance with section 3.5.1 of this assessment report and the results are shown in Table 4.1. The full calculations showing the forces/moments in individual members for different combination are shown in Appendix D. The truss member names are in accordance with Appendix E of IfA 2018 report or Figure 4.1.

Truss Member	Yielding check (BS5400-3, 9.9.4.1)					Buckling check (BS5400-3, 9.9.4.2)					Yielding Utilisation		Buckling Utilisation	
	Components			Total	Stress limit	Components			Total	Util. limit	No Section Loss	Section Loss	No Section Loss	Section Loss
	P/A _e	M _x /Z _x	M _y /Z _y			P/P _D	M _x /M _{dx}	M _y /M _{dy}						
Truss Top Chord														
TU	88.7	0.00	0.00	88.7	199.1	1.08	0.11	0.04	1.23	1.00	0.39	0.45	1.23	1.23
ST	98.1	0.00	0.00	98.1	183.5	1.20	0.11	0.08	1.39	0.92	0.43	0.53	1.39	1.50
QS	92.6	0.01	0.00	92.6	167.8	1.13	0.16	0.02	1.31	0.84	0.40	0.55	1.31	1.56
RQ	77.1	0.01	0.00	77.1	152.1	0.94	0.16	0.06	1.17	0.76	0.34	0.51	1.17	1.53
HR	61.0	0.00	0.00	61.0	167.8	0.68	0.19	2.20	3.07	0.84	0.27	0.36	3.07	3.64
Truss Bottom Chord														
NP	125.19	0.00	0.00	125.20	199.13	--	--	--	--	--	0.54	0.63	--	--
LN	138.38	0.01	0.00	138.38	183.45	--	--	--	--	--	0.60	0.75	--	--
JL	115.21	0.01	0.00	115.22	152.09	--	--	--	--	--	0.50	0.76	--	--
IJ	73.48	0.00	0.00	73.48	136.41	--	--	--	--	--	0.32	0.54	--	--
HI	73.46	0.00	0.00	73.46	183.45	--	--	--	--	--	0.32	0.40	--	--
Truss Vertical Members														
QJ	62.24	0.00	0.16	62.40	199.13	0.31	0.01	2.02	2.35	1.00	0.27	0.31	2.35	2.35
Truss Inclined Members														
RJ	125.88	0.01	0.00	125.89	174.05	--	--	--	--	--	0.55	0.72	--	--

Table 4.1 – Individual truss members' assessment results

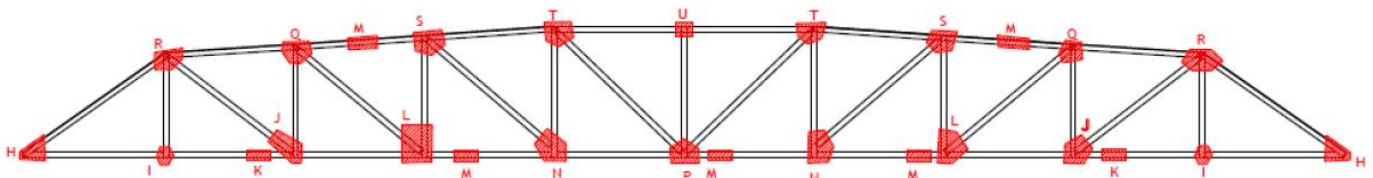


Figure 4.1 – Truss member naming arrangement

Further checks on truss bracings were not completed as the majority of the main truss members (top chords, vertical ties and inclined struts) were found to be failing either the yielding check or the buckling check as shown in Table 4.1.

4.2 Truss Connections

Truss connections were checked in accordance with section 3.5.2 of this assessment report and the results are summarised in Table 4.2. The full calculations showing the connections details and forces they transfer along with the connection naming arrangements are shown in Appendix H.

Connection	Utilisation (F _c = 1)		Condition Factor F _c	Reduced Utilisation	
	Fasteners	Bearing		Fasteners	Bearing
A	23.0%	31.6%	0.60	38.3%	52.7%
B	130.8%	176.8%	0.60	218.0%	294.7%
C	15.2%	21.0%	0.70	21.8%	29.9%
Ca	85.6%	117.7%	0.95	90.1%	123.9%
D	30.0%	25.2%	0.75	39.9%	33.6%
E	75.6%	69.4%	0.60	126.1%	115.6%
F	95.1%	130.8%	0.50	190.2%	261.6%
G	44.5%	61.2%	0.50	89.0%	122.4%
H	47.0%	67.1%	0.90	52.2%	74.6%
I	27.1%	49.7%	1.00	27.1%	49.7%
J	33.8%	71.2%	0.65	52.0%	109.5%
K	8.4%	18.8%	0.60	14.1%	31.3%
L	68.4%	153.2%	0.75	91.2%	204.3%
M	52.6%	113.2%	0.60	87.6%	188.7%
N	170.3%	286.0%	0.80	212.9%	357.5%
P	96.3%	161.7%	0.70	137.6%	231.0%
Q	95.0%	38.8%	0.80	118.8%	48.5%
R	49.1%	135.7%	0.85	57.7%	159.6%
S	181.2%	258.7%	0.85	213.1%	304.3%
T	17.1%	23.6%	1.00	17.1%	23.6%
U	182.1%	229.5%	0.70	260.2%	327.8%
X	142.6%	196.2%	0.50	285.3%	392.4%

Table 4.2 – Individual truss connections assessment results

4.3 Bison Planks

The individual bison plank section was checked in accordance with section 3.5.3 and section 5.2 of the AIP. The results are summarised in Table 4.3.

Mode	Capacity	Applied Load	Utilisation	Comment
Bending	3.48 kNm	3.48 kNm	100.0%	No condition factor applied.
Shear	5.68 kN	5.38 kN	85.4 %	No condition factor applied.

Table 4.3 – Bison plank assessment results

4.4 Ramps

As per section 3.1.5, only the south lower ramp was checked. The results are summarised in Table 4.4.

Mode	Capacity	Applied Load	Utilisation	Comment
Bending (hogging)	26.46 kNm	22.11 kNm	83.6 %	Over the support between bays 2-3 when live load applied to bays 2-3.
Bending (sagging)	37.78 kNm	14.47 kNm	38.3%	At bay 2 midspan when live load applied to bay 2 only.
Shear	91.05 kNm	38.83 kNm	42.6%	Over the support between bays 1-2 when live load applied to bay 1 only.

Table 4.4 – Ramp deck assessment results

5 Assessment Check Results

The assessment check was carried out by independent checkers, Ibrahim Tageldin and Adam Coatman of Skanska TechServ Team. The results of this check are shown below. The full assessment check results document is attached to this assessment report in Appendix G.

5.1 Truss Members

B55400-3, 9.9.4.1 checks

$$\text{Utilisation} = \frac{P}{A_e} \pm \frac{M_x}{Z_x} \pm \frac{M_y}{Z_y} / \frac{\sigma_y}{\gamma_m \gamma_{f3}}$$

Truss member	STL Utilisation	
	Without Section Loss	With Section Loss
Truss Top Chord		
ST	0.50	0.55
QS	0.48	0.58
RQ	0.41	0.53
HR	0.32	0.38
Truss Bottom Chord		
NP	0.49	0.49
LN	0.54	0.59
JL	0.45	0.59
IJ	0.29	0.38
Truss Verticals		
QJ	0.48	0.63
Truss Diagonals		
RJ	0.60	0.67

B55400-3, 9.9.4.2

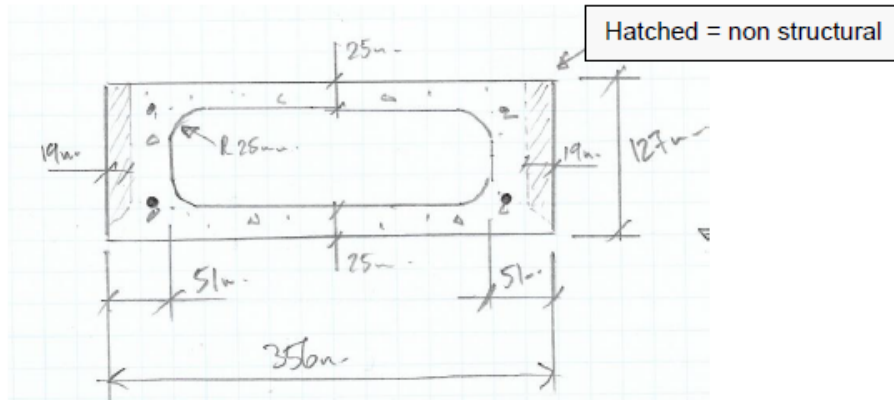
Truss member	STL Utilisation	
	Without Section Loss	With Section Loss
Truss Top Chord		
ST	1.27	1.42
QS	1.21	1.48
RQ	1.01	1.37
HR	0.84	1.00
Truss Verticals		
QJ	0.82	1.07

5.2 Bison Planks

Deck Planks

Assumptions:

- As per AIP, idealised section geometry:



- Assumed Rebar = 2no. 12mm dia bars per unit with 25mm cover
- Concrete strength assumed, $f_{cu} = 20\text{N/mm}^2$
- Rebar, yield strength = 230N/mm^2 mild steel
- Condition factor = 1.0 (as new)

Assessed Utilisation:

Bending	Shear
0.86	0.71

Other comments:

- Note capacities are based on several assumptions on section geometry and reinforcement.
- If deck waterproofing is not present and voids in planks are not sealed there may be water within the voids. If the voids are full of water the utilisations would change to approximately the following:

Bending	Shear
0.90	0.76

5.3 Ramps

Assessed Utilisations:

Concrete strength assumed, $f_{cu} = 20\text{N/mm}^2$

	Hogging	Sagging	Shear
North Upper	0.79	0.46	0.36
North Lower	0.85	0.60	0.28
South Upper	0.90	0.54	0.39
South Lower	0.99	0.63	0.42

Concrete strength assumed, $f_{cu} = 40\text{N/mm}^2$

	Hogging	Sagging	Shear
North Upper	0.71	0.43	0.29
North Lower	0.75	0.56	0.28
South Upper	0.80	0.50	0.31
South Lower	0.88	0.60	0.33

6 Results Comparison

6.1 Truss Members

The percentage difference between the Skanska IS Oxfordshire assessment results and Skanska TechServ assessment check results is shown in Table 6.1 below. The difference was calculated based on the following formula:

$$\% \text{ Difference} = \frac{|\text{Assessment Result} - \text{Check Result}|}{\min(\text{Assessment Result}, \text{Check Result})} \times 100$$

Skanska IS Oxfordshire				Skanska TechServ				Comparison (% Difference)			
Yielding Utilisation		Buckling Utilisation		Yielding Utilisation		Buckling Utilisation		Yielding Utilisation		Buckling Utilisation	
No Section Loss	Section Loss	No Section Loss	Section Loss	No Section Loss	Section Loss	No Section Loss	Section Loss	No Section Loss	Section Loss	No Section Loss	Section Loss
Truss Top Chord											
0.39	0.45	1.23	1.23	-	-	-	-	-	-	-	-
0.43	0.53	1.39	1.50	0.50	0.55	1.27	1.42	17%	3%	9%	6%
0.40	0.55	1.31	1.56	0.48	0.58	1.21	1.48	19%	5%	9%	5%
0.34	0.51	1.17	1.53	0.41	0.53	1.01	1.37	22%	5%	15%	11%
0.27	0.36	3.07	3.64	0.32	0.38	0.84	1.00	21%	4%	265%	264%
Truss Bottom Chord											
0.54	0.63	--	--	0.49	0.49	--	--	11%	28%	--	--
0.60	0.75	--	--	0.54	0.59	--	--	11%	28%	--	--
0.50	0.76	--	--	0.45	0.59	--	--	11%	28%	--	--
0.32	0.54	--	--	0.29	0.38	--	--	10%	42%	--	--
0.32	0.40	--	--	-	-	--	--	-	-	--	--
Truss Vertical Members											
0.27	0.31	2.35	2.35	0.48	0.63	0.82	1.07	77%	101%	186%	119%
Truss Inclined Members											
0.55	0.72	--	--	0.60	0.67	--	--	10%	8%	--	--

Table 6.1 – Truss Members Assessment Results Comparison

6.1.1 Yielding Utilisation

From Table 6.1, it is seen that all of the analysed bridge members are passing the BS5400-3:2000 clause 9.9.4.1 yielding check. Although the results without considering the section losses are approx. 20% different between the assessment and the assessment check, the members would still pass the assessment.

This is valid for all members, with the exception of the critical truss vertical member, QJ, where the difference between the assessment and the assessment check was 100% when considering the section loss. It is unclear why the difference is so significant; however, this also does not affect the end result, which suggests that the member is passing the yielding check.

It is worth noticing that both the original assessment and the check arrived to the same conclusion that out of all vertical members, member QJ (refer to Figure 4.1) is the critical member for checking.

6.1.2 Buckling Utilisation

Both the assessment and the check confirmed that only the truss top chord and the vertical members will be subject to compression loads, which also validates the truss models set up in LUSAS. Unlike with the yielding utilisation check, the BS5400-3:2000 clause 9.9.4.2 buckling check suggest that all of the considered truss members in compression are failing the check, which was confirmed by both the assessor and the checker.

The results were found to be approximately 10% different for the assessor and the checker for the majority of compression members under consideration. However, for one of the truss top chord members, member HR, this difference was found to be significant such that the original assessment fails the check with a utilisation of 364%, while as the check fails the check with a utilisation of 100%, both considering section losses. A similar pattern is observed for the check of the truss vertical members of the truss.

At the same time, the overall result of the assessor and the checker is the same i.e. the truss main elements fail the assessment whilst carrying out the BS5400-3:2000 clause 9.9.4.2 buckling check.

6.2 Bison Planks

Mode	Utilisation		Comment
	Skanska IS Oxfordshire	Skanska TechServ	
Bending	100.0%	90.0%	No condition factor applied.
Shear	85.4 %	76.0%	No condition factor applied.

Table 6.2 – Bison Planks Assessment and Check Results Comparison

From Table 6.2, it is seen that the bison planks pass the assessment if their condition is not considered. Both the assessor and the checker obtained a very similar utilisation for both bending and shear within the individual plank. The condition factor was not taken into account and in reality, the planks were found to be in poor state and are unlikely to be fit for purpose, which is to be discussed in next section.

6.3 Ramps

Mode	Utilisation		Comment	
	Skanska IS Oxfordshire	Skanska TechServ	Skanska IS Oxfordshire	Skanska TechServ
Bending (hogging)	83.6 %	99.0 %	South Lower Ramp. Span over the support between bays 2-3 when live load applied to bays 2-3. $f_c = 20$ MPa	South Lower Ramp. $f_c = 20$ MPa
Bending (sagging)	38.3%	63.0%	South Lower Ramp. Span at bay 2 midspan when live load applied to bay 2 only. $f_c = 20$ MPa	South Lower Ramp. $f_c = 20$ MPa
Shear	42.6%	33.0%	Over the support between bays 1-2 when live load applied to bay 1 only. $f_c = 20$ MPa	South Lower Ramp. $f_c = 20$ MPa

Table 6.3 – Ramp Deck Assessment and Check Results Comparison (South Lower Ramp)

Based on Table 6.3, the ramp deck is passing the assessment. Both the assessor and the checker concluded that the critical section for assessment is south lower ramp, which is discussed by the assessor in section 3.1.5 and follows from the checker's results in section 5.3. Both the assessor and the checker agree that the ramp deck is passing the assessment, although, the checker suggests only a marginal pass with a 99% utilisation for the hogging bending moment. Both the assessment and the check consider a condition factor of 1 since there were no defects identified to the ramp deck.

The capacities stated above do not consider the measured strength of concrete, which was obtained as part of the investigation works on the ramp; however, based on the checker's results from section 5.3, it is evident that the improved concrete strength will increase the capacity by 10%.

7 Discussion

7.1 Truss Members

All of the considered truss members were found to be passing the yielding check by both the assessor and the checker. Despite some differences in the results, the results are similar and lead to the conclusion that the truss members pass the yielding check. The difference in yielding check results is likely to be due to the one of the reasons below, or, most likely, due to the combination of these reasons.

- Section properties
The choice of sections for the assessment is justified in section 3.1.2; however, it is not necessary for the sections to correspond to BS4 sections. Instead, the measurements from the IfA 2018 report could have been adopted for the purpose of assessment, which would influence both the section capacity and the applied self-weight. Additionally, sections properties could be calculated with or without the fillets, which also makes a small impact on the final values.
- Self-weight (due to different section properties)
As discussed above, multiple ways of representing the sections making up the truss members may result in changes to the self-weight of the structure, which may have a small impact on the applied loads values.
- Consideration of the condition factor
The condition factor can have been applied to the section's capacity or considered whilst working out section properties such as plastic section modulus, etc. The difference in applying the condition factor would also affect the utilisation.

With regards to the buckling check, all of the truss members under consideration were found to be failing the assessment for both the assessor and the checker. From Table 4.1 and the BS5400-3:2000 buckling check equation 9.9.4.2 it is seen that the just the axial force component alone exceeds unity. Additional M_x and M_y components of this equation are contributing to approximately 15% of the result.

It may be seen that there is a large difference between the assessor's and the checker's results for truss members HR and truss vertical members and it is seen from Table 4.1 that the majority of this utilisation is likely to come from the M_y component (M_z in LUSAS model).

The utilisation of over 300% for members HR occurs at the load combination 2.2 (refer to section 3.3) and whilst the deck of the structure is loaded asymmetrically, and the side winds loads are applied. The structure behaves in such was that even relatively small wind loads with asymmetrical vertical loads are inducing some significant axial forces in the arched bracings at the end bays of the truss, which push the truss top chord sideways, which results in moment M_y (M_z in LUSAS) appearing. In absolute values, the total moment M_y is approx. 26.7 kNm, when all loads are factored, see Figure 7.1.

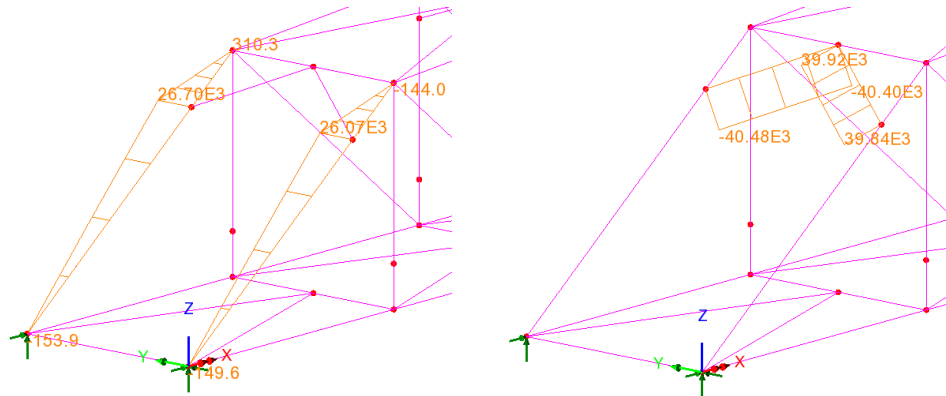


Figure 7.1 – Moment M_y (M_z in LUSAS) in member HR (left) and axial forces in arched (modelled as straight) bracings (right)

The utilisation of over 200% for the truss vertical members is due to the eccentricity of the applied vertical loads, which are causing the moment to appear in the vertical members, refer to section 3.4.1. This result is consistent for all of the vertical members (see Figure and appears at the load combination 2.1, when the vertical wind load acting on the deck is applied, which, in combination with other vertical loads that cause moment due to eccentricity such as bison planks self-weight, surfacing, etc. cause the worst-case moment M_y (M_z in LUSAS). The absolute value of this moment is approx. 12.9 kNm.

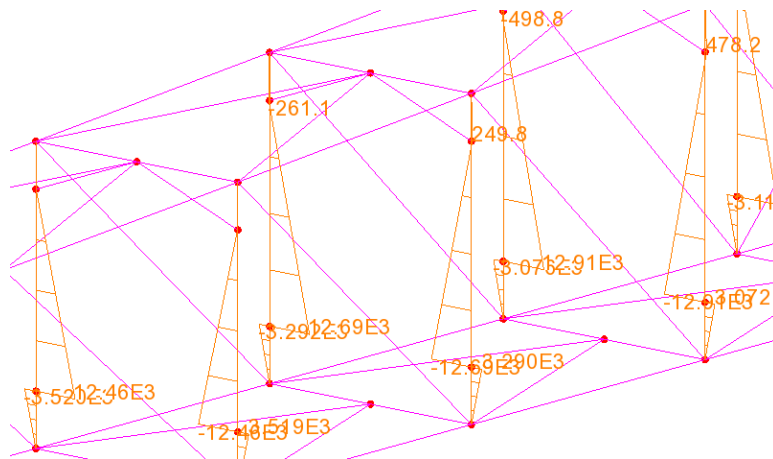


Figure 7.2 – Moment M_y (M_z in LUSAS) in truss vertical members

These absolute moment values appear to be low whilst thinking about modern structures; however, Isis Gasworks bridge was constructed in 1928 and all of the truss members are compound sections made of individual angles as discussed in section 3.6.1. Whilst calculating section capacities, it was assumed that the effective length of the individual members is between the truss joints and it was assumed that the angles, which are making up individual sections, will be acting independently between the joints since there is no connection between them. Whilst calculating the moment capacity considering lateral torsional buckling, angle sections have negligible resistance, so the utilisations of 200% even with relatively low applied moments are observed. In reality, moment resistance taking LTB into consideration may not have such a significant effect on any of the truss members.

At the same time, both members HR and the vertical truss members have a utilisation of over 100% at load combination 1, where axial forces contribute the most, unlike in the more sophisticated combinations, which demonstrates that the members are unsafe and compares well with the checker's results.

7.2 Connections

From Table 4.2 it is evident that the majority of connections are failing.

7.3 Bison Planks

Both the assessment and the check suggest that Bison planks have sufficient capacity, assuming the condition factor F_c of 1.0; however, the planks were found to be in poor condition during the 2018 IfA, but a condition factor could not be easily applied to any of the planks. Some planks, as shown in Figure 7.3, have cracked along the longitudinal axis of the plank (bridge transverse axis). It is unclear how the plank is still intact, but it is possible that this is due to the arching action of the top flange of the hollow core plank and potentially due to the bond between the concrete infill and the planks, which act as an additional support for the planks in along their longitudinal axes not allowing the planks to fall through. Other defects, such as cracking in the plank transverse direction (bridge longitudinal direction) should not significantly affect the plank's capacity, but may have an adverse effect on durability which could lead to a reduction in capacity in the future.

It is also important to note that the size of the reinforcement and section details were assumed based on the 2018 IfA results and the background information on the planks geometry, which, however, could not be verified.



Figure 7.3 – Cracking along the plank longitudinal axis along the reinforcement bar with corrosion staining (left) and plank cracking along its transverse axis showing some deflection of concrete (right)

7.4 Ramps

The decks of the ramps to the structure were found to be passing the assessment and the check. The ramps were found to be in very good condition and no reduced condition factor was applied establish the ramp deck capacity. An additional check was made by the checked to demonstrate that the 40 N/mm² concrete strength, which was established after the concrete testing in 2020, would result in the reduction of the utilisation by approx. 10% in comparison to the assumed concrete strength of 20 N/mm².

Since the ramps form part of the bridge and the bridge is carrying National Cycle Route 5, the ramps should ideally comply with the current minimum bridge widths as per Table 11 of CD 353 document, which suggests that the unsegregated footway and cycle path should have a minimum width of 3.5m. The ramps are only 1.5m wide.

Additionally, the ramps gradient exceeds the maximum allowable gradient of 1 in 12 as advised by the same CD353 document. Ideally, the ramps should have a maximum gradient of 1 in 20 for the comfortable use by the cyclists and pedestrians. The ramp gradient is approximately 1 in 11.

Finally, the safety of both ramps should be reviewed as the south ramp terminates very close to the abutment and cyclists may end up colliding with the abutment wall, if the ramp deck is slippery (either as a result of frost or dead leaves from the trees next to the ramp), or if the cyclist loses control otherwise. The north ramp ends just in front of the stairs from the bridge, which could result in an accident between the cyclist leaving the ramp and the pedestrians using the stairs.

8 Conclusions and Recommendations

The assessment and the check have concluded that the truss, including its connections, has inadequate capacity to carry crowd loading in line with the current CS454 standard. The truss members consist of angle sections, which have poor buckling capacity. Many elements have reduced capacities due to their condition, but the bridge would fail the assessment even if its condition was assumed to be 'as new'. The majority of the connections between the truss members were also found to be fail.

When the truss was originally constructed, it is unlikely that it was designed for pedestrian crowd loading. Initially, the truss bridge was constructed to carry the utilities from Oxford Gasworks (hence, the name) on the river Thames south bank to Oxford City Centre and it is unknown if at the time it was designed to any standard at all. The 1970s repurposing added a significant weight onto the bridge and introduced new types of loads to the truss members. Assessment of this old-type truss bridge to modern standards with increased loads would almost inevitably result in a failure.

The Bison planks, which provide the walking surface on the bridge, were found to have sufficient capacity, but only assuming the condition factor of 1.0. In reality, the planks are showing signs of significant deterioration and based on the defects such as significant deflection at the plank midspan and cracking along the plank length, the planks are deemed to be unsafe. It is also worth noting that the plank geometry was adopted from a textbook and the exact details of the planks are unknown.

The ramps to the bridge were found to have adequate capacity; however, it is deemed that they are not fit for purpose considering the current accessibility and geometric standards. The ramps are currently used by both the pedestrians and the cyclists and are only approx. 1.5m wide between the parapets, whilst the minimum required width to CD 353 should be at least 3.5m. The same is applicable to the gradient of the ramps, which significantly exceeds the CD 353 absolute maximum gradient of 1 in 12. There are also concerns associated with the termination of both ramps, which may result in an accident between the cyclists and the pedestrians.

Overall, nearly all of the elements of the bridge are either failing the assessment or are deemed not fit for purpose.

The capacity of the steel truss could be improved by introducing additional stiffeners or connections to the truss members, which would improve the individual angles' buckling capacity, but this would have to be combined with re-painting of the structure. The re-painting works are likely to require significant temporary works to prevent paint polluting the river underneath since there was lead found in paint (refer to 2018 IfA report), which is challenging due to the river navigation. Individual truss members strengthening and re-painting is likely not to be cost effective and the costs might be comparable to replacing the whole structure.

The Bison type planks are likely to be difficult or impossible to repair and uncertainties with regard to the details of the planks mean that replacement of the deck planks would be highly favourable. Again, the replacement would require significant temporary works to prevent debris polluting the watercourse below.

The ramps to the structure could be potentially reused with the truss alone being replaced, but a safety audit would likely conclude that the ramps should be renewed in line with modern standards.

It is recommended that the bridge be demolished and replaced in its entirety. The cost of replacement may be comparable to the cost of capital maintenance, but would offer a much greater certainty on the life of the structure, would bring the structure up to modern standards and would almost certainly offer lower whole life costs.

Appendices

Refer to files in the 'Appendices' folder.

Appendix A – Inspection for Assessment 2018

Appendix B – Investigation Works by JFT 2020

Appendix C – Bison Planks Bearing Detail

Appendix D – Truss Calculations

Appendix E – Bison Planks Calculations

Appendix F – Ramps Calculations

Appendix G – Assessment Check Results

Appendix H – Truss Connections Calculations

Appendix I – Approval in Principle