HRE Assessment Programme

MKT/461, A530, Baddington Road Bridge, Cheshire East

CS 454 Assessment and Inspection Report

0451398 | Form BA

October 2022

Highways England - Historical Railways Estate

MKT/461

HRE Assessment Programme

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Contents

Executive Summary

Structure Type: Skewed single span overbridge

Superstructure Form: Propped longitudinal cast iron girders with brick jack arches

Substructure Form: Brick abutments and wingwalls

Span: Clear Skew Span: 9.14m (30' – 0") Square Span: 7.87m (25' – 10")

Assessment Code: CS 454

Live load capacity: 7.5 tonnes GVW

Capacity Critical Element: Internal girders in bending between prop and east abutment

Critical Span: 5.60m

Critical Load Case: Vehicle ref H (44 tonnes)

Capacity Factor: 0.8
(taken as the capacity available for live loads/the live load capacity required for case considered)

Restriction: 7.5 tonnes GVW

Condition: Superstructure: Fair Propping: Poor Substructure: Poor

Local Authority: Cheshire East Council

OS Reference: SJ 646 505

This report presents the load carrying capacity for the bridge and identifies the data used to derive the assessment. The load carrying capacity of the bridge is based only on the assessment of visible spans that were amenable to inspection. It has been prepared by Jacobs for the exclusive use by HRE and should not be relied on by third parties. It has been based on site measurements and investigation by Jacobs or historical information provided by HRE, as appropriate.

The description of condition does not represent a principal inspection; nor should it be relied on for the development of maintenance works. Close inspection of members was limited by the constraints of safe access possible within a single site visit.

Identification of defects is principally based on ground level observation of visible members. The structural arrangement of the bridge meant that the following elements were not examined as part of the inspection for assessment:

- Internal girders – Due to the jack arch construction, only the underside of the bottom flanges of the internal girders were visible. The webs and top flanges were considered as built-in parts protected by the deck construction and not amenable for inspection.
- Edge Girders – The parapet wall supported by the edge girders restricted the inspection of the top flanges. The internal face of the web panel and internal outstands of both flanges were considered as built-in parts and therefore not amenable for inspection. Unexposed surfaces were assumed to be competent owing to their protection by the deck construction or wall.
- Propping – where the girders have been continuously propped prevented inspection of the underside of the cast iron girder flanges and the top of the propping girder flanges at these locations.

1. General Description and Structural Details

1.1 Introduction

Jacobs was appointed by Highways England – Historical Railways Estate (HRE) to undertake a CS454 assessment of structure MKT/461.

Structural Soils Ltd provided a scaffold tower for high level access to the superstructure soffit.

No temporary traffic management was in place for the survey, therefore minimal survey and inspection works were undertaken on the road over the structure due to the busy road. Existing information has been utilised supplemented by site observations.

1.2 Location and General Description

Structure MKT/461, carries the A530 over the trackbed of the former Market Drayton to Wellington railway line approximately 7.8km to the south west of Crewe.

The structure carries a single lane carriageway controlled by permanent traffic lights located approximately 60m and 35m to the east and west respectively from the structure. The road is surfaced the full width between the parapets, 6.13m. The carriageway width over the structure is reduced using road markings only to 3.83m Concrete blocks restrict the width of the carriageway way on the east approach preventing vehicle loading on a 1.80m width of carriageway at the south east pilaster only. Refer to the plan in Appendix E for carriageway layout and dimensions.

Traffic flow across the bridge is moderate with frequent HGV use observed during the survey.

The OS grid reference is SJ 646 505.

The Market Drayton to Wellington railway was completed in 1867 and it is likely that the bridge was constructed at this time. The propping was installed circa 1969 with modifications made to the columns supporting the edge girders circa 2001.

1.3 Construction Type

The structure is a skewed, single span overbridge which has been propped under one half of the span. The original clear skew span for the internal girders is 9.14m (30′ – 0″) and the clear square span is 7.87m (25′ – 10″) giving an angle of skew of 31°. Returns on the abutments increase the clear span of the edge girders to 9.24m (30′ – 3 ⅝″).

The superstructure comprises of six longitudinal cast iron girders with brick jack arches spanning between the bottom flanges. The internal girders are placed at 1.34m (4′ – 4 ¾″) centre to centre. The edge girders are placed with a gap of 0.84m (2′ – 9″) from the bottom flange of the adjacent internal girder. Each girder has been propped, supported for a length of 3.05m (10′ – 0″) starting 1.76m (5′ – 9″) from the west abutment.

Edge Girders

The edge girders have an overall depth of 610mm (24″). The bottom flanges are 305mm (12″) wide and 51mm (2″) thick. The top flanges are 178mm (7″) wide and 38mm (1 ½″) thick. The webs are 51mm (2″) thick with a depth of 521mm (20 ½″) between the flanges. Refer to Figure 1 below.

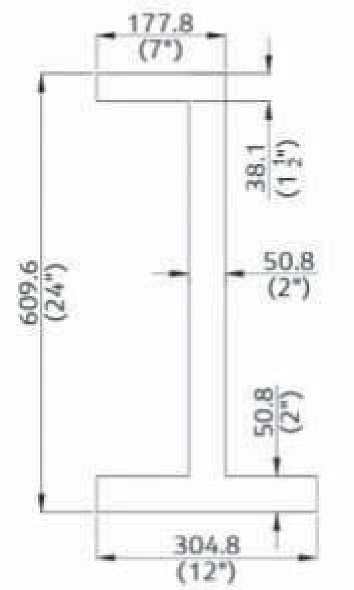


Figure 1. Edge girder cross section

<u>Internal Girders</u>

The internal girders have an overall depth of 559mm (22″) at midspan reducing to 352mm (13 ⅞″) at support. The bottom flange is 508mm (20″) wide and 44mm (1 ¾″) thick. The top flange is 95mm (3 ¾″) wide and 51mm (2″) thick. The web is 44mm (1 ¾″) with a depth between the flanges of 464mm (18 ¼″) at midspan and 257mm (10 ⅛″) at support.

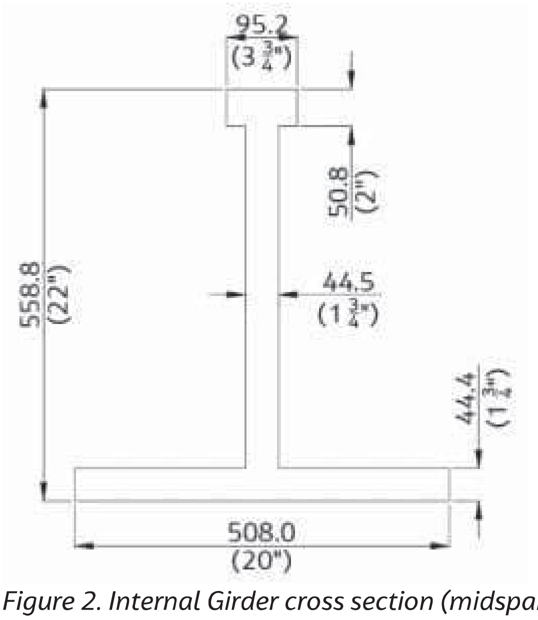


Figure 2. Internal Girder cross section (midspan)

<u>Jack Arches</u>

The jack arches between all girders have a span of 832mm (2′ – 8 ¾″) and a rise of 0.24m (9 ¼″). Existing information indicates that the jack arches are 4 ½″ (114mm) thick backed with rubble fill.

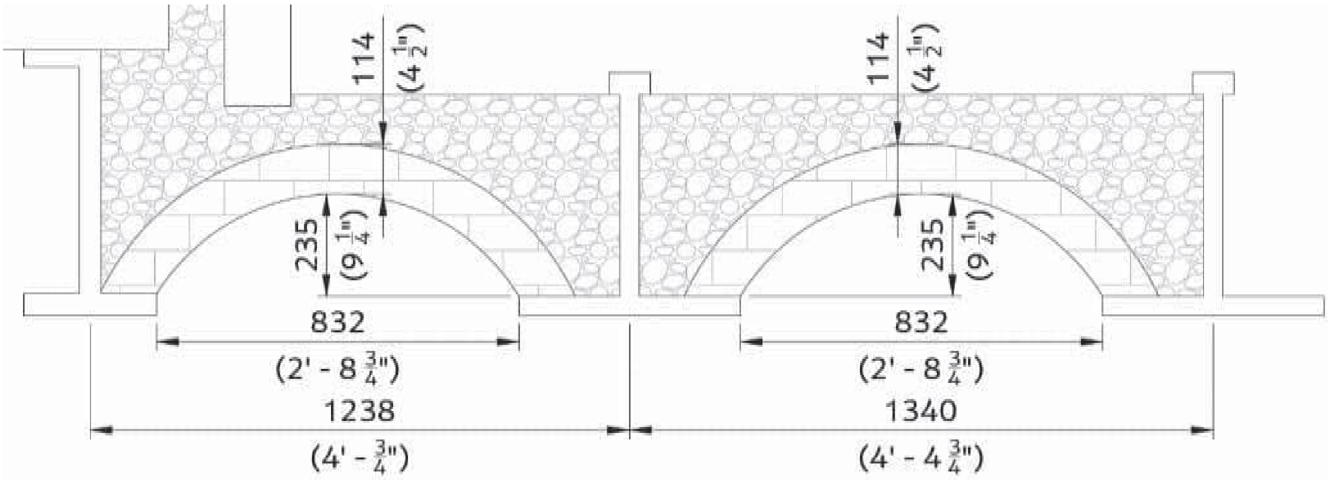


Figure 3. Jack Arch Arrangement

There are three original tie bars in each jack arch bay apart from the central one. The ties are spaced at 2.00m (6' – 6") centres with a diameter of 25mm (1"). There are retro fitted tie bars fixed to the bottom flanges of the same bays located 2.1m from the east abutment with a residual diameter measured as 25mm.

<u>Propping</u>

Existing information for the installation of the original propping indicates that the columns and cross beams are 10" x 10" x 49lbs universal column (UC) sections. The flange and web thicknesses were measured using an ultra-sonic thickness gauge as 14.5mm and 9.4mm respectively, commensurate with the 0.56" (14.22mm) and 0.34" (8.64mm) for the flange and web thicknesses from historical steelwork table for 10" x 10" x 49lbs UC. Each cross beam is 3.05m (10 – 0") in length supported on two columns. There are seven rubber pads between each propping UC and the cast iron girders.

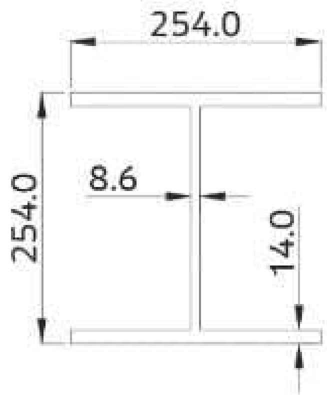


Figure 4. Original Propping Columns / Beams Section

The columns supporting the edge girders were replaced circa 2001 and are formed from two channel sections placed back-to-back. There is a gap of 90mm between the channel webs so they form a square section and are fixed together by 200 x 230 x 8mm plates welded to the sections. The channels are formed from a 6mm plate rolled into a C shape with a full depth of 280mm, a full width of 95mm and 40mm returns on the flanges, refer to figure 5 below. Each replacement column has a screw jack at the top with a short section of the channels between the jack and cross beam. Each column is formed of three sections below the jack, 2280mm, 240mm and 415mm from concrete base to jack respectively. Each section is separated by two 15mm thick plates, see figure 6 below.

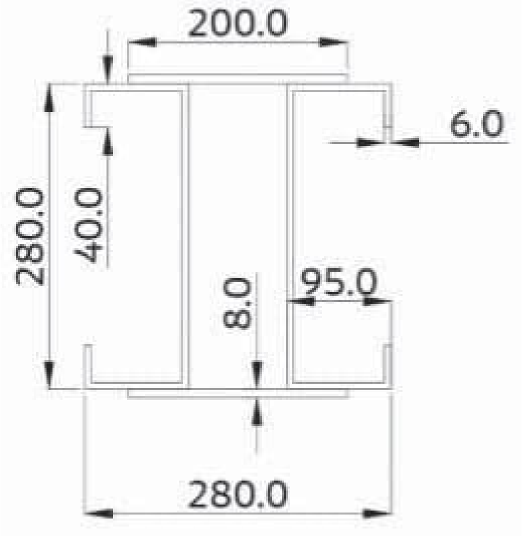


Figure 5. Replacement Propping Columns Cross Section

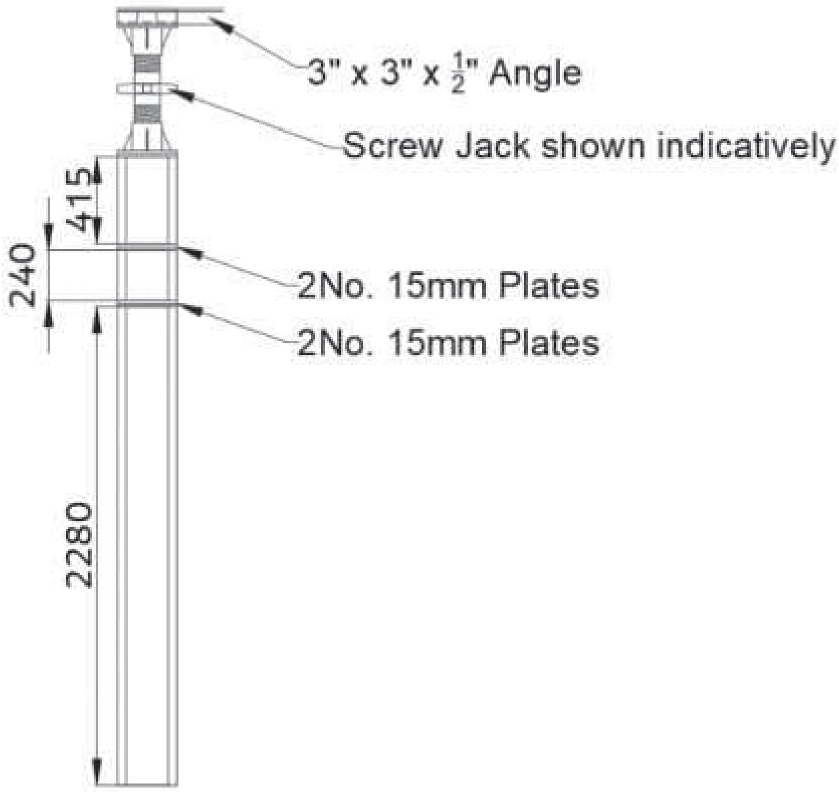


Figure 6. Replacement Propping Column

The columns are placed so that the centres of the columns are 1.78m from the west abutment along the skew span and 1.96m centre to centre. The cross UC is positioned such that it supports the cast iron girders from 1.59m from the west abutment, 1.68m for the north edge girder due to the return on the abutment wall. Refer to figure 7 below.

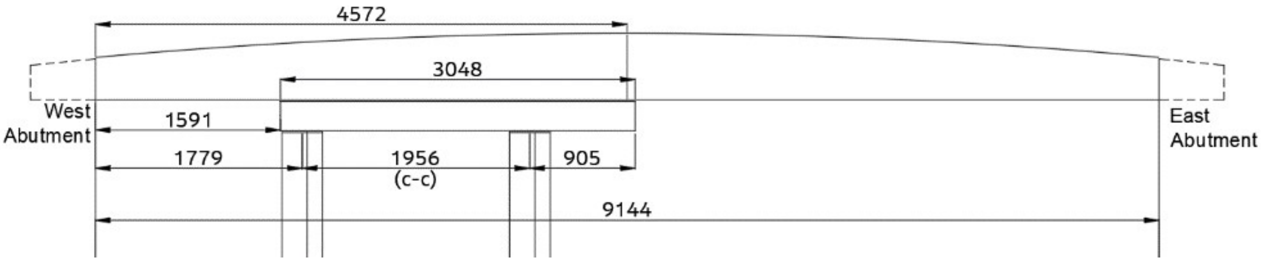


Figure 7. Propping arrangement at internal girders

Each transverse row of columns is connected at the top via 3" x 3" x ½" angles. The edge columns and the adjacent columns have bracing between them formed from 3" x 3" angles with residual thickness measured on site of 9.5mm.

The propping columns were originally anchored into concrete strip foundations, one for each row of columns. When the end columns were replaced, raised plinths were cast to encase the bases of the original and to anchor the new columns into. Refer to the existing drawing information in Appendix D.

<u>Substructure</u>

The substructure comprises gravity type abutments and wingwalls constructed from common red bricks. The parapets are constructed from the same material.

Sketches of the plan at road level, section of the deck and elevation are included in Appendix E.

2. Information Search

2.1 Services Search

Intrusive investigations were not within the scope of this survey therefore a services search was not undertaken.

2.2 Site Investigation Description and Results

No intrusive investigations were undertaken as part of the works. An ultra-sonic thickness gauge was used to determine the thickness of the web and flanges of the propping elements.

2.3 Existing Drawings

A drawing from British Railways London Midland Region contains a cross section of the deck, plan, elevation of the original arrangement and details of the original proposed propping arrangement.

A Cheshire County Council Engineering Service drawing contains sections with outline information on the parts to be replaced and the modifications to be made to the propping arrangement.

3. Structure Condition

3.1 General

Jacobs undertook a survey and inspection for CS 454 assessment on Wednesday 27th October 2021. Weather conditions were overcast with a temperature of 17°C. Parking was agreed with the landowner of Baddington Park to the south east of the structure.

3.2 Structure Condition

3.2.1 Edge Girders

Both edge girders are in fair condition (Photographs 20 & 22). The paint system is intact apart from on the underside of the bottom flanges and in isolated patches on the exposed web. The sections of exposed web have only surface corrosion. The undersides of the bottom flanges are stained with rust throughout. The north edge girder has isolated patches of deeper section loss, up to 5mm deep.

At the supports and midspan, the girders are in fair condition with only up to 1mm section loss to the underside and inside edge of the bottom flange. At the exposed former blast line i.e. unpropped section, the corrosion is up to 2mm on the underside and inside vertical face of the flange and 1mm on the outside vertical face.

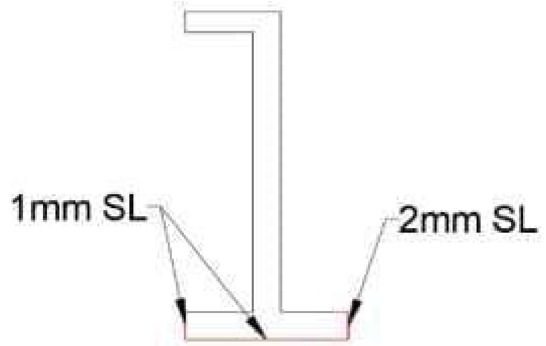


Figure 8. Edge girders section loss

The condition of the underside of the flange where it has been propped could not be determined. The exposed sections are in similar condition to that of the exposed blast line section.

3.2.2 Internal Girders

All internal girders are in fair condition. There is rust staining throughout and where the paint has failed and the flanges are exposed there is up to 2mm section loss to all areas. (Photographs 5, 6 & 24)

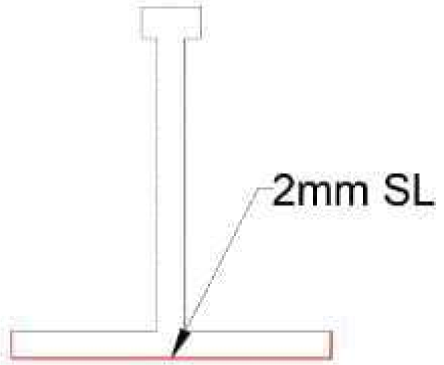


Figure 9. Internal girders section loss

3.2.3 Jack Arches

All of the jack arches have damp staining, open joints and isolated displaced bricks throughout. There are also hollow sounding patches of brickwork. There are isolated patches of spalling up to 30mm in deep. (Photographs 28 & 29)

Both the original and retro fitted tie bars have widespread corrosion. The original tie bars have a residual thickness of 24mm. The retro fitted tie bars have a residual thickness of 25mm. The connections for the retro fitted tie bars to the girders display extensive lamination corrosion. (Photograph 30)

3.2.4 Propping

All columns in the propping are in fair condition (Photographs 8 & 9). The paint system is generally intact but there is up to 1mm section loss from all exposed areas where the paint has failed. The corrosion is present on the top 1.0m of the column sections, below this the corrosion is present in isolated patches. (Photographs 25 & 26)

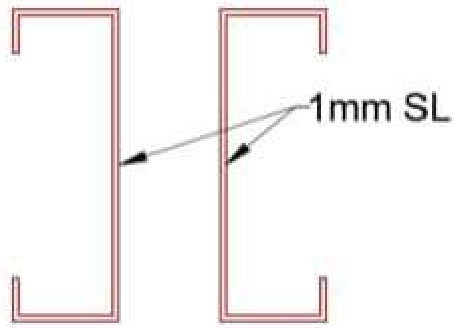


Figure10. End propping columns section loss

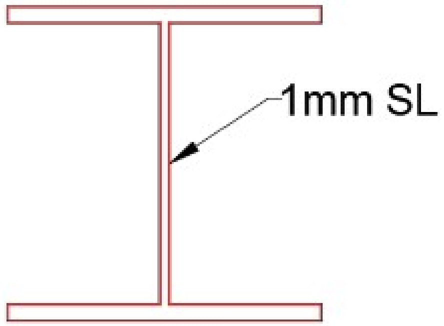


Figure 11. Internal propping columns section loss

The cross beams are in poor condition with extensive severe corrosion. Confirmation of the original section size for the cross beams was not possible due to the severe lamination. Residual section sizes were measured as: 10.2mm thick top flange, 7.9mm web thickness and 9.7mm bottom flange with 2mm section loss on the ends of the flanges. (Photographs 21 – 24)

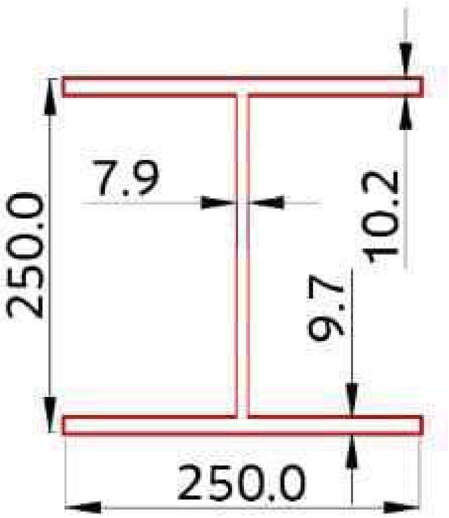


Figure 12. Propping cross beams residual section

The angles used for bracing generally have a residual thickness of 10mm though there are sections with more severe section loss leaving up to 1mm at the ends of the angles. (Photographs 26 & 27)

3.2.5 Abutments

Both abutments are in fair to poor condition. Both have some weathering to the bricks and isolated open joints.

The east abutment has two vertical fractures. One at the inside edge of the north edge girder open 4mm the full height of the abutment through bricks for the top two thirds and joints only for the remainder. At the top of the fracture is an area of re-pointing 0.5m^2 below the bearing shelf. The second fracture is located between the second and third internal girders from the south faces. It starts at the south edge of the third internal girder travels diagonally through the joints for five courses and then becomes a vertical fracture. It is open 10mm at the bearing shelf reducing to 3mm at the base of the abutment. (Photograph 10)

The west abutment has a fracture that has previously been repointed and has re-opened to hairline located at the inside edge of the south edge girder. There is a second fracture on the south face of the abutment though joints open 2mm up to 1.2m below the bearing shelf. (Photograph 11)

3.2.6 Wingwalls and Pilasters

The wingwalls are in poor condition. All have diagonal fractures running parallel with the copers which propagate into the pilasters and into the parapet at three of the pilasters.

3.2.6.1 South East

The diagonal fracture in the south east wingwall is approximately 250mm below the copers. A change in the brickwork above the fracture suggests that an attempt to repair the fracture was made and it has re-opened. The fracture is open to 10mm along the length between the pilaster and where it meets the vertical fracture described below. It reduces from 10mm at the vertical fracture to hairline at the newel. There is an area of mortar loss to the joints at the end of the fracture where it meets the pilaster.

There is a vertical fracture extending from the diagonal fracture 2.5m from the newel. It is open up to 10mm where it meets the diagonal fracture and hairline at the base of the wingwall. (Photograph 12)

There is a vertical fracture between the pilaster and the abutment starting from the underside of the bearing shelf and terminating at the top of the edge girder. It is open 5mm at the base of the bearing shelf reducing to hairline at the top. The fracture then propagates into the parapet (refer to section 3.2.8.1). The fracture in the pilaster has previously been re-mortared and the section of the pilaster from the base of the fracture up has previously been re-built. (Photograph 19)

3.2.6.2 South West

The diagonal fracture in the south west wingwall is 600mm below the copers at the pilaster and propagates at a 45 angle down towards ground level increasing the distance from the copers. Inspection of the wingwall near to the base of the wingwall was prevented by dense vegetation growth. The fracture is open up to 10mm where it is visible. There is also displacement of the wingwall above the fracture up to 10mm. There is a vertical fracture a third of the way along the wingwall from the newel. The fracture is hairline for the majority of the length but opens to 2mm at the copers. (Photographs 13 & 16)

There is a vertical fracture between the pilaster and abutment at the bearing shelf open 3mm. The section of pilaster above the bearing shelf has been rebuilt previously. There is a fracture system within the pilaster which propagates into the parapet (refer to section 3.2.8.2) and wingwall. The fractures are open up to 5mm with displace at some locations up to 10mm. See figure X below for fracture locations. (Photograph 17)

3.2.6.3 North East

The diagonal fracture in the north east wingwall is 700mm below the copers and open 1-2mm throughout. The copers and end two thirds of the wingwall above the fracture have previously been re-built. There are open joints in the wingwall adjacent to the pilaster over 0.5m^2 at the top of the wingwall where the fracture meets the pilaster. There are also isolated open joints throughout the rest of the wingwall. (Photograph 14)

The top section of this pilaster from the level of the bearing shelf has also been re-built. There is a diagonal fracture open to 2mm from the top of the edge girder to the centreline of the pilaster where it becomes a horizontal hairline fracture across the remaining pilaster width.

3.2.6.4 North West

Inspection of the base of this wingwall was prevented by dense bramble and nettle growth.

The diagonal fracture in this wingwall is 300mm below the copers at the pilaster increasing towards the newel. There is a second diagonal fracture another 300mm below the first. Both fractures are generally open up to 5mm along the length increasing to 10mm at the pilaster with open joints around the fractures at the pilaster as well. The wingwall has also been re-built above the first fracture. (Photograph 15)

There is also a fracture between the pilaster and the abutment for the height of the bearing shelf. Plants growing in joints with mortar loss prevented examination to determine if the pilaster had further similar fractures to those observed on the other three pilasters. (Photograph 18)

3.2.7 Formation

The formation is now a well-used farm access track for the adjacent fields. There appears to be no use by the public. (Photographs 36 – 38)

3.2.8 Parapets

3.2.8.1 North Parapet

There is some minor weathering to both faces of the parapet. There is a diagonal fracture through the mortar joints at the west end which starts at the pilaster where it meets the top of the edge girder. (Photograph 34)

3.2.8.2 South Parapet

There are two diagonal fractures, one at each end of the parapet which start at the pilasters where they meet the top of the edge girder. The fracture at the west end is through the mortar joints, has some minor displacement, up to 5mm, and minor plant growth along its length. The fracture at the east end is through the bricks on the outside face and through the mortar joints on the road face. It is open up to 2mm along its length. The east half of the parapet has previously been re-constructed. (Photographs 31 & 35)

3.2.9 Road Surface and Traffic Flow

The road surface over the structure is in fair condition with no signs of settlement or tracking (Photograph 5).

On the approaches to the structure there are fractures in the road surface on the south side of the carriageway. The fracture to the west has a large depression in the road (Photographs 5 & 32). The fracture on the east side is not trafficked by vehicles due to the concrete barriers. The east fracture is open up to 4cm at the manhole covers for BT cables (Photographs 3 & 33).

Regular HGV use was observed during the survey. AAHGVF will be taken as ‘medium’. National speed limit of 60mph applies over the structure but due to the traffic lights controlling flow over the structure actual speeds are likely to be lower.

4. Assessment to CS454

4.1 Methodology

Capacities of the elements were calculated using estimates of reduced section sizes where corrosion is present. A general condition factor was not applied.

Resistance capacities of the cast iron girders was determined in accordance with CS 454 Clause 8. The capacity of the internal girders was not enhanced using the section modulus factor F_I, as the embedment condition of the girders is not homogenous.

The propping elements are constructed from steel, therefore a yield strength of 230N/mm^2 was used as the default value from BD21/01 as agreed with the TAA (NH HRE) for circumstances where testing for characteristic strength has not been undertaken.

The cross beams were checked for bearing. The columns were checked for axial compression with the effective length taken as 1.5L.

Given the poor condition of some elements of the propping, the cast iron girders were checked in both propped and unpropped condition.

The barriers on the eastern approach to the bridge do not physically prevent vehicle loading on the south side of the structure though the traffic management in place generally restricts the traffic to a single lane.

In an unpropped condition, the cast iron girders were checked for normal traffic using ALL model 2 (UDL + KEL). The simplified method as outlined in BA 16/97 Section 2 was used to derive the load effects on a single girder as a method agreed with the TAA (NH HRE) for complying bridges.

In a propped condition the cast iron girders were checked using ALL model 1 (Appendix B vehicle loads), initially assuming no distribution and therefore a single line of wheels applied to a girder. As the girders were found to be deficient, a grillage model was used to determine the load effects on a single girder. The ratio between longitudinal and transverse stiffness was taken as EI_T/EI_L=0.0305 as suggested in Annex A of BA 16/97 Clause A1 as a fair correspondence for jack arch structures.

Determination of the adequacy of the jack arches was initially based upon the empirical method as described in Bridgeguard 3 Current Information Sheet No. 22 (Pro-forma for the empirical assessment of brick, masonry and concrete jack arches and associated ties). As the structural backing was found to be deficient, a check using LimitState RING software was undertaken.

The substructure was assessed qualitatively.

4.2 Results

Element: **Internal Girders**

Propped Condition

Action	Location	DL & SDL Effect	ALL loading	Total Load Effect	Assessed Resistance	Live Load Capacity*
Bending Single lane traffic loads	Mid span (Max dead load)	252 kNm	24 kNm ALL model 1, propped	292 kNm	370 kNm	**Normal Traffic**
Bending Single lane traffic loads	Between prop and abutment (Max live load)	159 kNm	101 kNm ALL model 1, propped	260 kNm	240 kNm	Fail 40/44 tonne
Bending Single lane traffic loads	Between prop and abutment	159 kNm	87 kNm ALL model 1, propped	246 kNm	240 kNm	Fail 18 tonne
Bending Single lane traffic loads	Between prop and abutment	159 kNm	33 kNm ALL model 1, propped	192 kNm	240 kNm	**Pass 7.5 tonne**
Bending Single lane traffic loads	Prop	245 kNm	-106 kNm** ALL model 1	139 kNm	390 kNm	**Normal Traffic**

*Assessment live loading as defined in CS454 Note following Clause 5.7
**Hogging moment

Unpropped Condition

Action	Location	DL & SDL Effect	ALL loading	Total Load Effect	Assessed Resistance	Live Load Capacity*
Bending	Mid span	252 kNm	254 kNm ALL model 2, unpropped, single lane	506 kNm	370 kNm	Fail 40/44 tonne in unpropped state
Shear	Support Abutment	106 kN	139 kN ALL model 2, unpropped, 2 lanes	245 kN	520 kN	**Normal Traffic**

Element: **Edge Girders**

Propped Condition

Action	Location	DL & SDL Effect	ALL loading	Total Load Effect	Assessed Resistance	Live Load Capacity*
Bending Single lane traffic loads	Mid span	227 kNm	30 kNm ALL model 1, propped	257 kNm	342 kNm	**Normal Traffic**
Bending Single lane traffic loads	Between prop and abutment	158 kNm	71 kNm ALL model 1, propped	229 kNm	303 kNm	**Normal Traffic**
Bending Single lane traffic loads	Prop	219 kNm	-64 kNm** ALL model 1, propped	155 kNm	337 kNm	**Normal Traffic**

*Assessment live loading as defined in CS454 Note following Clause 5.7
**Hogging moment

Unpropped Condition

Action	Location	DL & SDL Effect	ALL loading	Total Load Effect	Assessed Resistance	Live Load Capacity*
Bending	Mid span	227 kNm	337 kNm ALL model 2, unpropped, single lane	564 kNm	342 kNm	Fail 40/44 tonne
Shear	Support Abutment	93 kN	131 kN ALL model 2, unpropped, 2 lanes	224 kN	825 kN	**Normal Traffic**

<u>Element:</u> **Propping Beams**

Action	Location	DL & SDL Effect	ALL loading	Total Load Effect	Assessed Resistance	Live Load Capacity**
Bearing Single lane traffic loads	Support	Self weight negligible	146.8 kN	146.8 kN	137.7 kN	Fail 40/44 tonne
Bearing Single lane traffic laods	Support	Self weight negligible	113.7 kN	113.7 kN	137.7 kN	**Pass 26 tonne GVW**

Element: **Propping Columns**

Action	Location	DL & SDL Effect	ALL loading	Total Load Effect	Assessed Resistance	Live Load Capacity**
Axial Compression Single lane traffic loads	End Columns	Self weight negligible	58.8 kN	58.8 kN	911.0 kN	**Normal Traffic**
Axial Compression Single lane traffic loads	Internal Columns	Self weight negligible	146.8 kN	146.8 kN	992.7 kN	**Normal Traffic**

Element: **Jack Arches and Ties**

The jack arches are non-compliant with Bridgegaurd 3 Current Information Sheet No. 22 due to the absence of structural backing and deficient ties. The specific area of the ties is 217 mm^2/m, less than the required 260mm^2/m. A RING analysis was undertaken and determined that the jack arches could be shown to have capacity for wheel loads associated with Normal Traffic loading subject to additional tie bars being installed.

Ring Analysis – CS 454 Authorised Axle Loads for Normal Traffic

Case	Adequacy Factor RING Result	CS 454 Assessment Result
Single Axle (11.5t) – Single Wheel Load	3.53	Normal Traffic

Element: **Substructure**

The abutments and wingwalls are all in poor condition with fractures present throughout. A review of available recent visual examination reports suggests that there is no significant ongoing movement in the fractures in the substructure. By qualitative assessment the abutments and wingwalls are therefore considered satisfactory for the current imposed loading as restricted by the traffic control.

5. Conclusions and Recommendations

A weight restriction of 7.5 tonnes GVW and maintenance of the current traffic control restriction to single lane loading is required as determined by this assessment.

The internal girders marginally fail the 18 tonne GVW loading. A reduction of the surfacing partial factor from 1.5 to 1.0 (as permitted in defined circumstances by CS 454 Clause 3.4.1 Note 2) may result in a pass for 18 tonne GVW. This would require an assurance from the local highway authority that the thickness of the surfacing would not be increased for the remaining lifetime of the structure.

Monitoring of the fractures in the abutments and wingwalls is recommended to check for movement. It is also suggested that the fractures in the road surface on the approaches to the bridge are monitored as part of the same programme of works.

Infilling is a proven long term solution to permanently support the structure and road over, but installation of a corrugated steel arch may be a preferable option due to the need to maintain access on the well-used farm track beneath the structure. Embankments formed on both sides may also provide some support to the approach embankments. Further investigations are recommended into the cause of the fractures in the road on both approaches.

Appendix A. Photographs

1. South elevation

2. North elevation

3. Carriageway approach looking east

4. Carriageway approach looking west

5. Carriageway over structure looking west

6. General soffit view looking north

7. General soffit view looking east

8. General view of soffit along propping

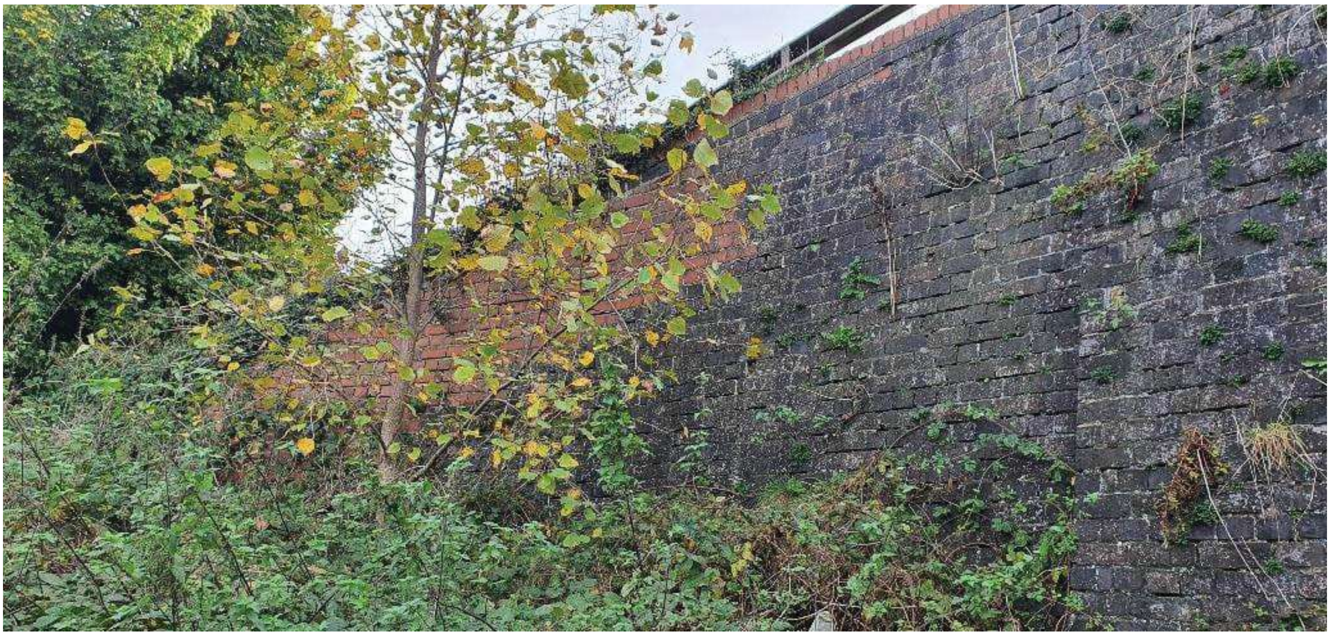
9. General view of propping

10. East abutment

11. West abutment

12. South east wingwall

13. South west wingwall

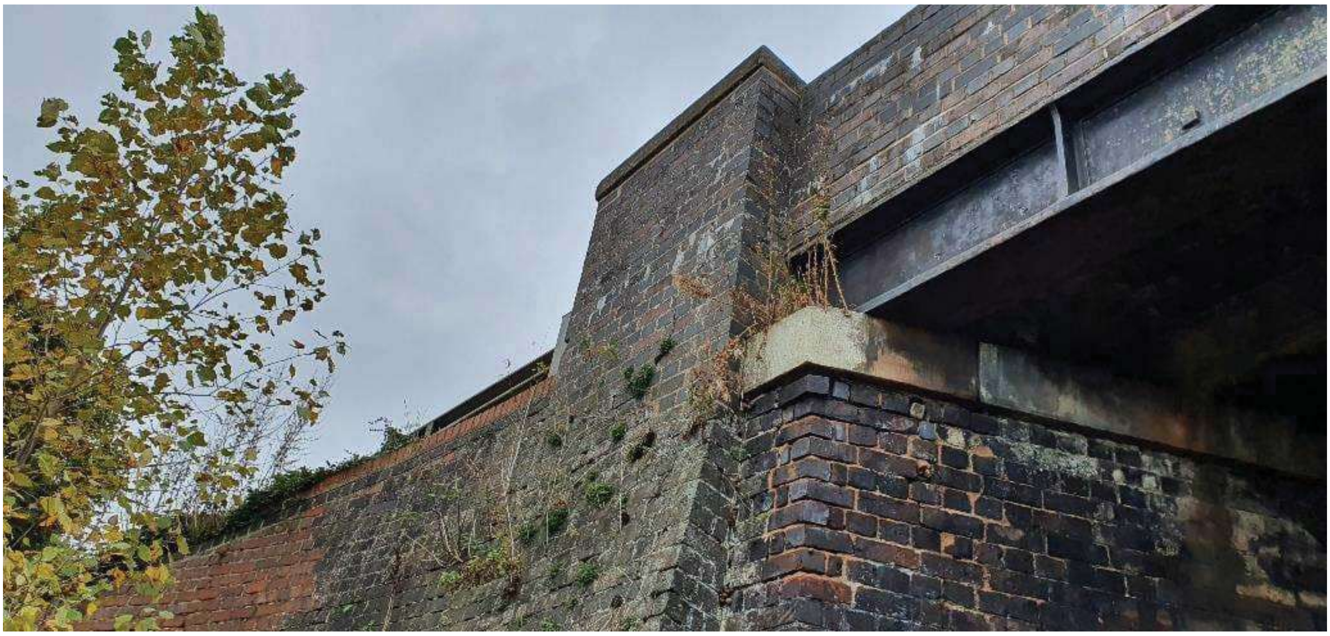


14. North east wingwall

15. North west wingwall

16. South west wingwall showing displacement to fracture

17. South west pilaster



18. North west pilaster

19. South east pilaster – showing fracture between pilaster and abutment

20. Typical condition of edge girders at support

21. Typical condition of edge propping beams

22. Typical condition of edge girders and propping beams

23. Typical condition of internal propping beams

24. Typical condition of internal girders and propping beams

25. Condition of edge girder propping columns at top

26. Condition of propping at interface between columns and beams

27. Typical bracing condition

28. Typical edge jack arch bay condition

29. Typical internal jack arch bay condition

30. Typical condition of retro fitted ties

31. Fracture at east end of south parapet on inside face

32. Settlement / crack in road surface to west

33. Crack in road surface to east

34. North parapet

35. South parapet

36. Formation looking north

37. Formation looking south

38. Formation under looking north

Appendix B. Form AA

APPROVAL IN PRINCIPLE FOR ASSESSMENT

Bridge/Line Name: **Baddington Road Bridge / Market Drayton to Wellington**

ELR/Bridge No. **MKT/461**

Brief Description of Existing Bridge:

(a) Span Arrangement

The structure is a skewed single span overbridge which has been propped under one half of the span. The original clear skew span for the internal girders is 9.14m (30' – 0") and the clear square span is 7.87m (25' – 10") giving an angle of skew of 31°. Returns on the abutments increase the clear span of the edge girders to 9.24m (30' – 3 ⅝").

(b) Superstructure Type

The superstructure comprises of six longitudinal cast iron girders with brick jack arches spanning between the bottom flanges The internal girders are placed at 1.34m (4' – 4 ¾") centre to centre. The edge girders are placed with a gap of 0.84m (2' – 9") from the bottom flange of the adjacent internal girder. Each girder has been propped, supported for a length of 3.05m (10' – 0") starting 1.76m (5' – 9") from the west abutment.

<u>Edge Girders</u>

The edge girders have an overall depth of 610mm (24"). The bottom flanges are 305mm (12") wide and 51mm (2") thick. The top flanges are 178mm (7") wide and 38mm (1 ½") thick. The webs are 51mm (2") thick with a depth of 521mm (20 ½") between the flanges.

<u>Internal Girders</u>

The internal girders have an overall depth of 559mm (22") at midspan reducing to 352mm (13 ⅞") at support. The bottom flange is 508mm (20") wide and 44mm (1 ¾") thick. The top flange is 95mm (3 ¾") wide and 51mm (2") thick. The web is 44mm (1 ¾") with a depth between the flanges of 464mm (18 ¼") at midspan and 257mm (10 ⅛") at support.

<u>Jack Arches</u>

The jack arches between all girders have a span of 832mm (2' – 8 ¾") and a rise of 0.24m (9 ¼"). Existing information indicates that the jack arches are 4 ½" (114mm) thick backed with rubbles fill.

There are three original tie bars in each jack arch bay apart from the central one. The ties are spaced at 2.00m (6' – 6") centres with a diameter of 25mm (1"). There are retro-fitted tie bars fixed to the bottom flanges of the same bays located 2.1m from the east abutment with a residual diameter measured as 25mm.

Propping

Existing information for the installation of the original propping indicates that the columns and cross beams are 10" x 10" x 49lbs universal column (UC) sections. The flange and web thicknesses were measured using an ultra-sonic thickness gauge as 14.5mm and 9.4mm respectively, commensurate with the 0.56" (14.22mm) and 0.34" (8.64mm) for the flange and web thickness from historical steelwork table for 10" x 10" x 49lbs UC. Each cross beam is 3.05m (10 – 0") in length supported on two columns. There are seven rubber pads between each propping UC and the cast iron girders.

The columns supporting the edge girders were replaced circa 2001 and are formed from two channel sections placed back-to-back. There is a gap of 90mm between the channel webs so they form a square section and re fixed together by 200 x 230 x 8mm plates welded to the sections. The channels are formed from a 6mm plate rolled into a C shape with a full depth of 280mm, a full width of 95mm and 40mm returns on the flanges. Each replacement column has a screw jack at the top with a short section of the channels between the jack and cross beam. Each column is formed from three sections below the jack, 2280mm, 240mm and 415mm from concrete base to jack respectively. Each section is separated by two 15mm thick plates.

The columns are placed so that the centres of the columns are 1.78m from the west abutment along the skew span and 1.96m centre to centre. The cross UC is positioned such that it supports the cast iron girders from 1.59m from the west abutment, 1.68m from the north edge girders due to the return on the abutment wall. Each transverse row of columns is connected at the top via 3" x 3" x ½" angles. The edge columns and the adjacent columns have bracing between them formed from 3" x 3" angles with residual thickness measured on site of 9.5mm.

The propping columns were originally anchored into concrete strip foundations, one for each row of columns. When the end columns were replaced, raised plinths were cast to encase the bases of the original and to anchor the new columns into.

(c) Substructure Type

The substructure comprises gravity type abutments and wingwalls constructed from common red bricks. The parapets are constructed from the same material.

(d) Planned highway works/modifications at this site

None.

(e) Road designation class and whether classed as a heavy load route

The structure carries a single lane carriageway, the A530, controlled by permanent traffic lights approximately 60m and 35m to the east and west respectively from the structure. The road is surfaced the full width between the parapets, 6.13m. The carriageway width over the structure is reduced using road markings only to 3.83m. Concrete blocks restrict the width of the carriageway on the east approach preventing vehicle loading on a 1.8m width of carriageway at the south east pilaster only.

Usage by all vehicles of all types is frequent. HGV use was observed during the survey. AAHHGV will be taken as medium. National speed limit of 60mph applies over the structure but due to the traffic lights controlling flow over the structure actual speeds are likely to be lower.

Assessment Criteria

(a) Loadings and Speed

Dimensions and condition are obtained from site measurements and inspection. (See Jacobs report "VAR9/6429 Assessment Programme – Assessment and Inspection Report – Bridge Ref.: "MKT/461" – December 2021).
The bridge is to be assessed for normal traffic, with reduced loading being determined where this capacity is not reached.

(b) Codes to be used

CS 454 – The assessment of highway bridges and structures

CS 456 – The assessment of steel highway bridges and structures

(c) Proposed Method of Structural Analysis

Capacities of the elements will be calculated using estimates of reduced section sizes where corrosion is present. A general condition factor will not be applied.

Resistance capacities of the cast iron girders will be determined in accordance with CS 454 Clause 8. The capacity of the internal girders will not be enhanced using the section modulus factor, F_I, as the embedment condition of the girders is not known.

The propping elements are constructed from steel therefore a yield strength of 230N/mm^2 will be used being the default value from BD21/01 as agreed with the TAA (NH-HRE) for circumstances where testing for characteristic strength has not been undertaken.

The cross beams are not restrained against lateral torsional buckling therefore k_1 will be taken as 1.

The columns will be checked for axial compression with the effective length taken as 1.5L.

The cast iron girders will be checked in both propped and unpropped conditions.

The barriers do not physically prevent vehicle loading on the south side of the structure though the traffic management in place generally restricts the traffic to a single lane. Therefore, checks for both single and double lane loading will be undertaken.

The girders will be checked for normal traffic using ALL model 2 initially (UDL + KEL). The simplified method as outlined in BA 16/97 Section 2 will be used to derive the load effects on a single girder as a method agreed with the TAA (NH-HRE) for complying bridges. Should the girders be found to below full capacity utilising this loading model, a check using ALL model 1 shall be undertaken.

Determination of the adequacy of the jack arches will be based upon the empirical method as described in Bridgeguard 3 Current Information Sheet No. 22 (Pro-forma for the empirical assessment of brick, masonry and concrete jack arches and associated ties).

The substructure will be assessed qualitatively.

FORM 'AA' (BRIDGES) **GC/TP0356**

Appendix: 4
Issue: 1
Revision: B (Nov 2000)

ELR/ Bridge No MKT/461

APPROVAL IN PRINCIPLE FOR ASSESSMENT

Senior Civil Engineer's Comments

None

Proposed Category for Independent Check 1

Superstructure 1

Substructure 1

Name of Checker suggested if Cat 2 or 3 Not applicable

Category 1

The above assessment, with amendments shown, is approved in principle:

Signed [redacted]
Title Senior Civil Engineer
Date 13/01/2022

Category 2 and 3

The above assessment, with amendments shown, is approved in principle:

Signed ....
Title ....
Date ....

Signed ....
Title ....
Date ....

v

Appendix C. Form BA

FORM 'BA' (BRIDGES) **GC/TP0356**
Appendix: 4
ELR/ Bridge No MKT/461 Issue: 1
Revision: A (Dec 2005)

CERTIFICATION FOR ASSESSMENT CHECK

Assessment Group: **Jacobs UK Ltd**

Bridge/Line Name: **Baddington Road Bridge / Market Drayton to Wellington**

Category of Check: **1**

ELR/ Bridge No: **MKT/461**

We certify that reasonable professional skill and care have been used in the assessment of the above structure with a view to securing that:

(1) It has been assessed in accordance with the Approval in Principle as recorded on the Form AA approved on 13th January 2022.

(2) It has been checked for compliance with the following principal British Standards, Codes of Practice, BRB (Residuary) Limited technical notes and Assessment standards:

- CS 454 – The Assessment of Highway Bridges and Structures

List any departures from the above and additional methods or criteria adopted, with reference and justification for their acceptance.

None

Category 1

Name Signature Date

[redacted]
.... Assessor

[redacted]
.... Assessment Checker

[redacted]
.... Authorised signatory of the firm of Consulting Engineers to whom Assessor/Checker is responsible.

FORM 'BA' (BRIDGES) **GC/TP0356**

Appendix: 4

ELR/ Bridge No MKT/461 Issue: 1

Revision: A (Dec 2005)

CERTIFICATION FOR ASSESSMENT CHECK

Category 2 and 3 (Note: Category 1 check must also be signed)

(a) Assessment

Name	Signature	Date	
…………	…………	…………	Assessor
…………	…………	…………	Assessment Checker
…………	…………	…………	Authorised signatory of the firm of Consulting Engineers to whom Assessor/Checker is responsible.

(b) Check

Name	Signature	Date	
…………	…………	…………	Assessor
…………	…………	…………	Assessment Checker
…………	…………	…………	Authorised signatory of the firm of Consulting Engineers to whom Assessor/Checker is responsible.

This Certificate is accepted by. [redacted]…….

FORM 'BAA' (BRIDGES) **GC/TP0356**

Appendix: 4

ELR/ Bridge No MKT/461 Issue: 1

Revision: A (Dec 2005)

CERTIFICATION FOR ASSESSMENT CHECK

Notification of Assessment Check

Assessment Group	Jacobs UK Ltd
Bridge Name/Road No.	Baddington Road Bridge / A530
Line Name	Market Drayton to Wellington
ELR Code/Structure No.	MKT/461

The above structure has been assessed and checked in accordance with Standards which are listed on the appended Form BA. A summary of the results of the assessment in terms of capacity and restrictions are as follows:-

STATEMENT OF CAPACITY

Edge Girders	Normal Traffic (with current lane restrictions)
Internal Girders	7.5 tonne GVW
Jack Arches	40 tonne GVW
Masonry Substructure	Normal Traffic (by qualitative assessment)

Recommended Loading Restrictions

7.5 tonnes and maintenance of the current traffic control restriction to single lane loading over the structure.

Description of Structural Deficiencies and Recommended Strengthening

The 40 tonne capacity of the jack arches is subject to the installation of additional ties otherwise the structure is suitable for dead loads only. Installation of a corrugated steel arch (or similar) is recommended to permanently support the structure and road over.

Name Signature Date

[redacted]

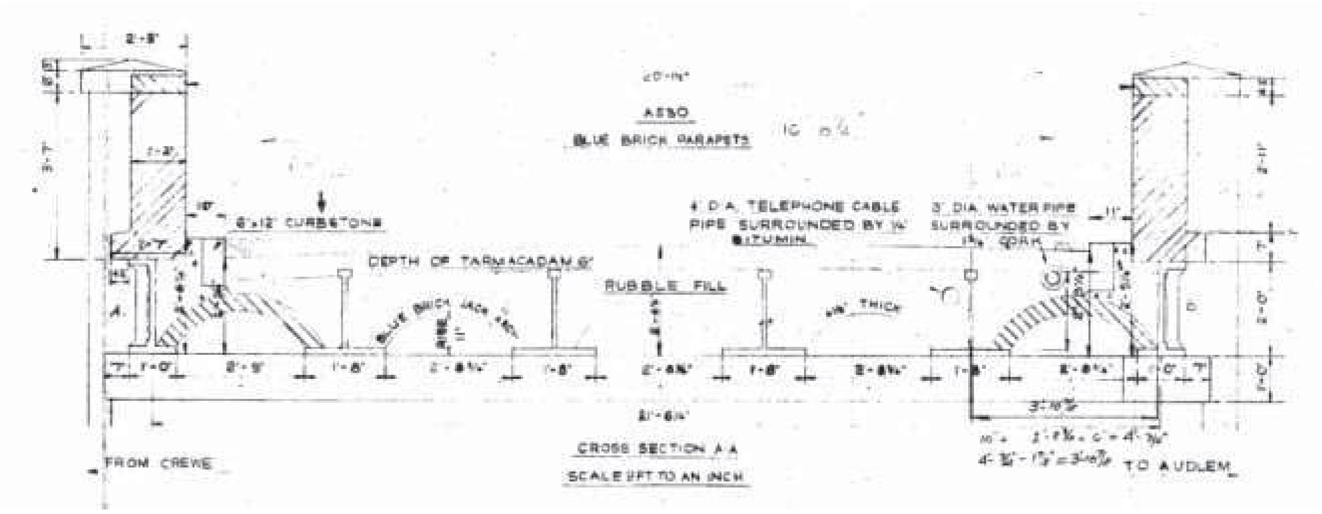
……… ……………… Assessor

……… ……………… Assessment Checker

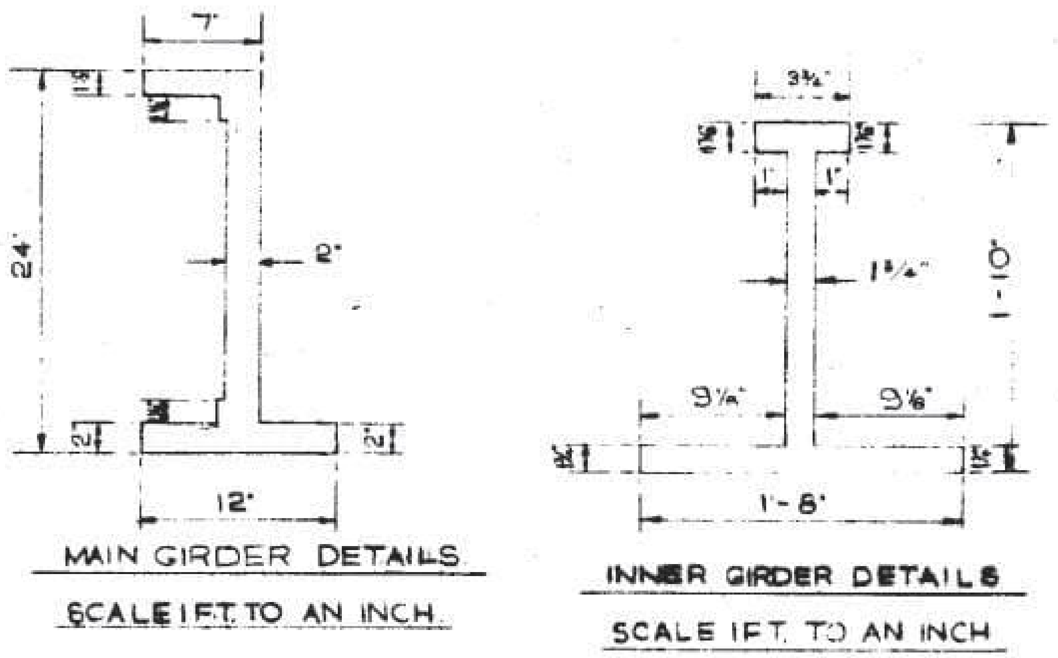
……… ……………… Authorised signatory of the firm of Consulting Engineers to whom Assessor/Checker is responsible.

This Certificate is accepted by. [redacted] ………………..……………..

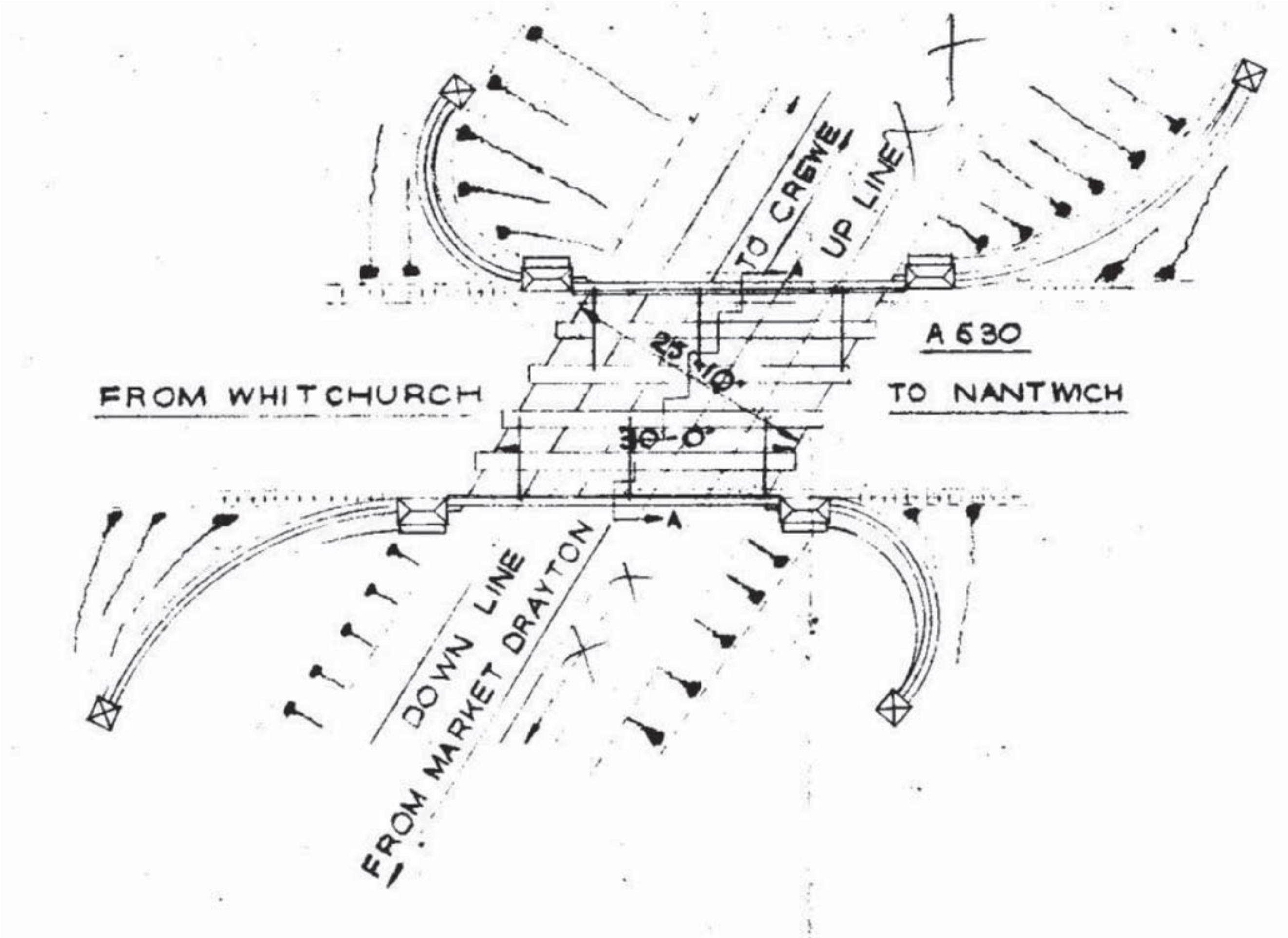
Appendix D. Historical Information



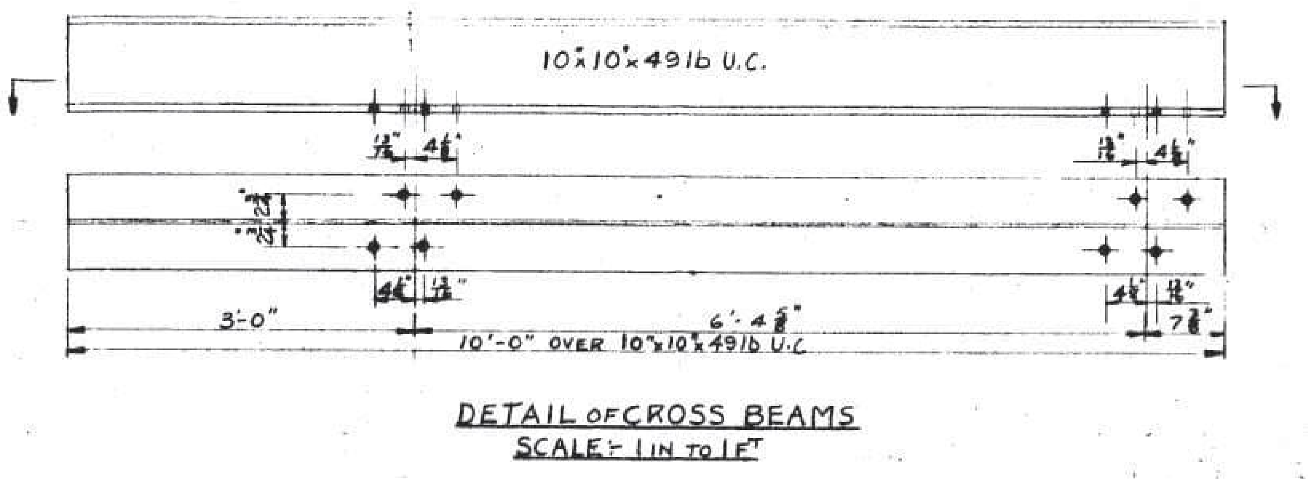
BR LMR Fabrication drawing – Cross Section



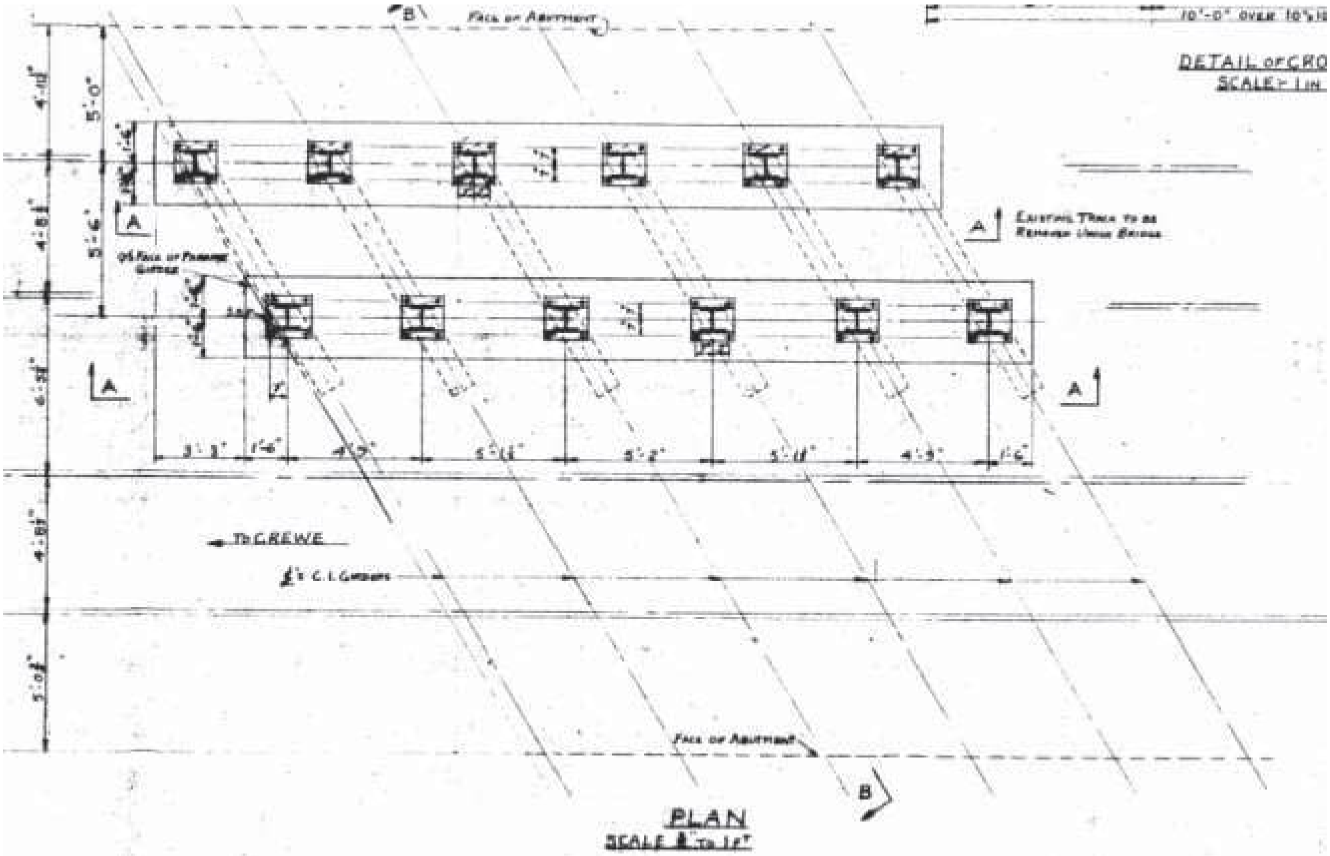
BR LMR Fabrication drawing – Girder Sections



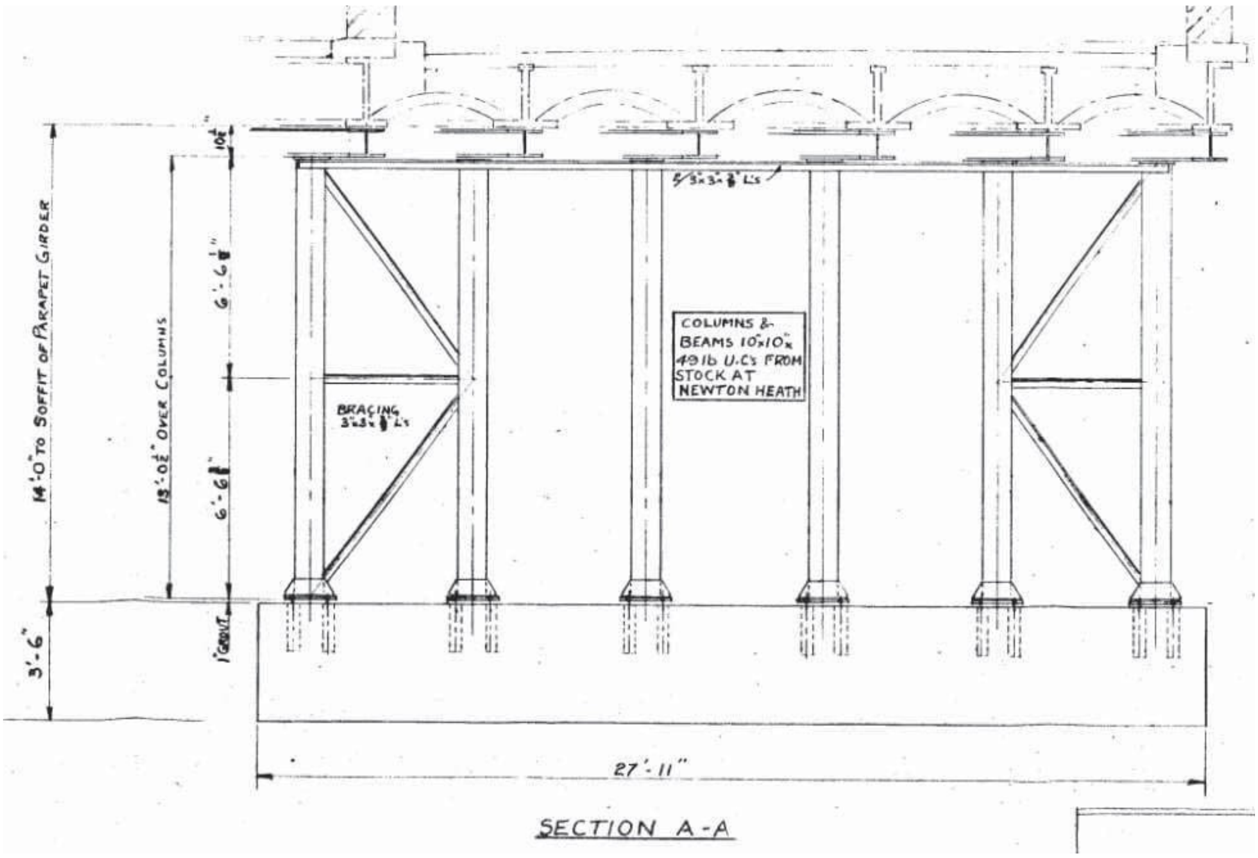
BR LMR Fabrication drawing – Plan



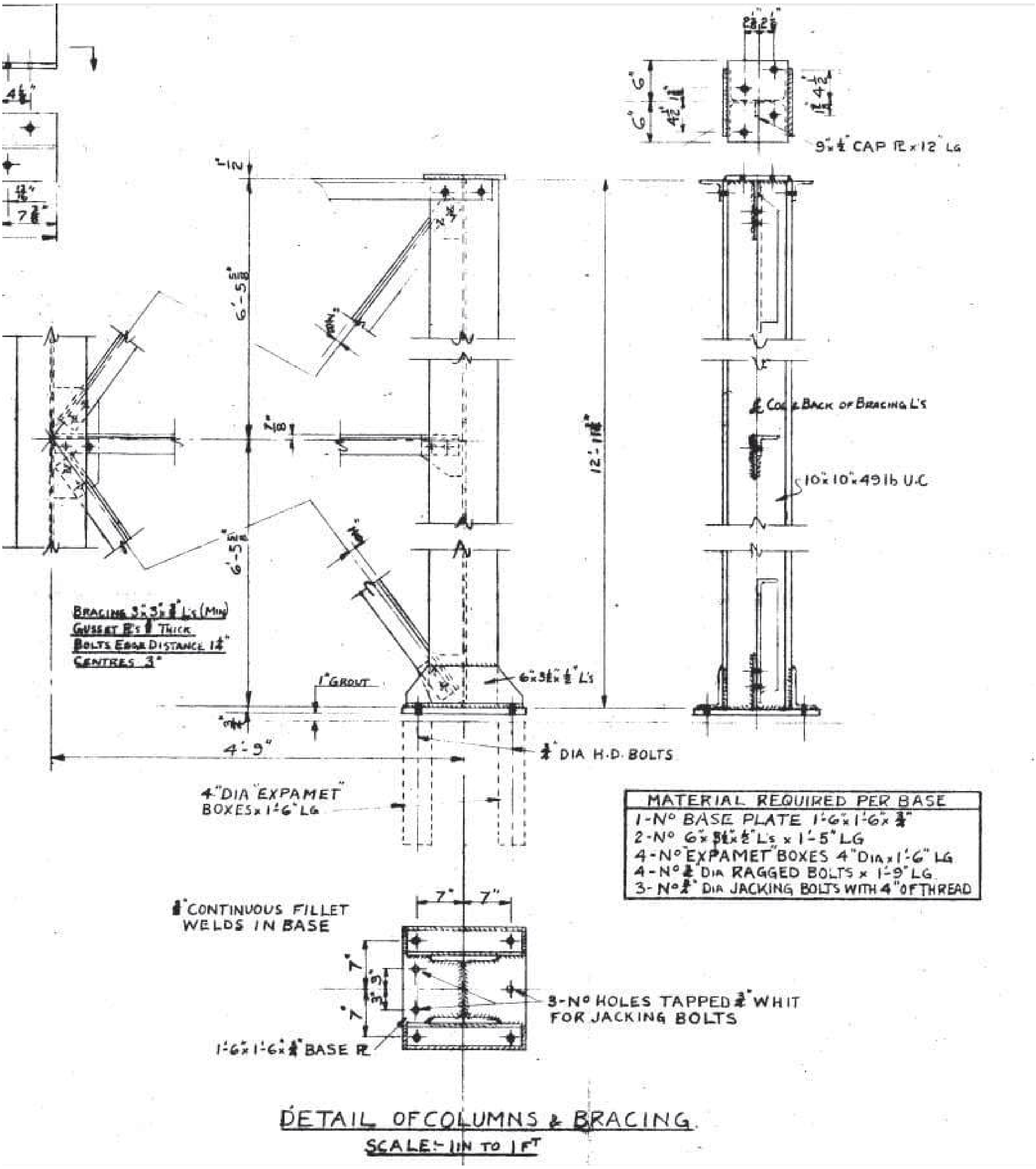
BR LMR Fabrication drawing – Propping Beam Elevation / Plan



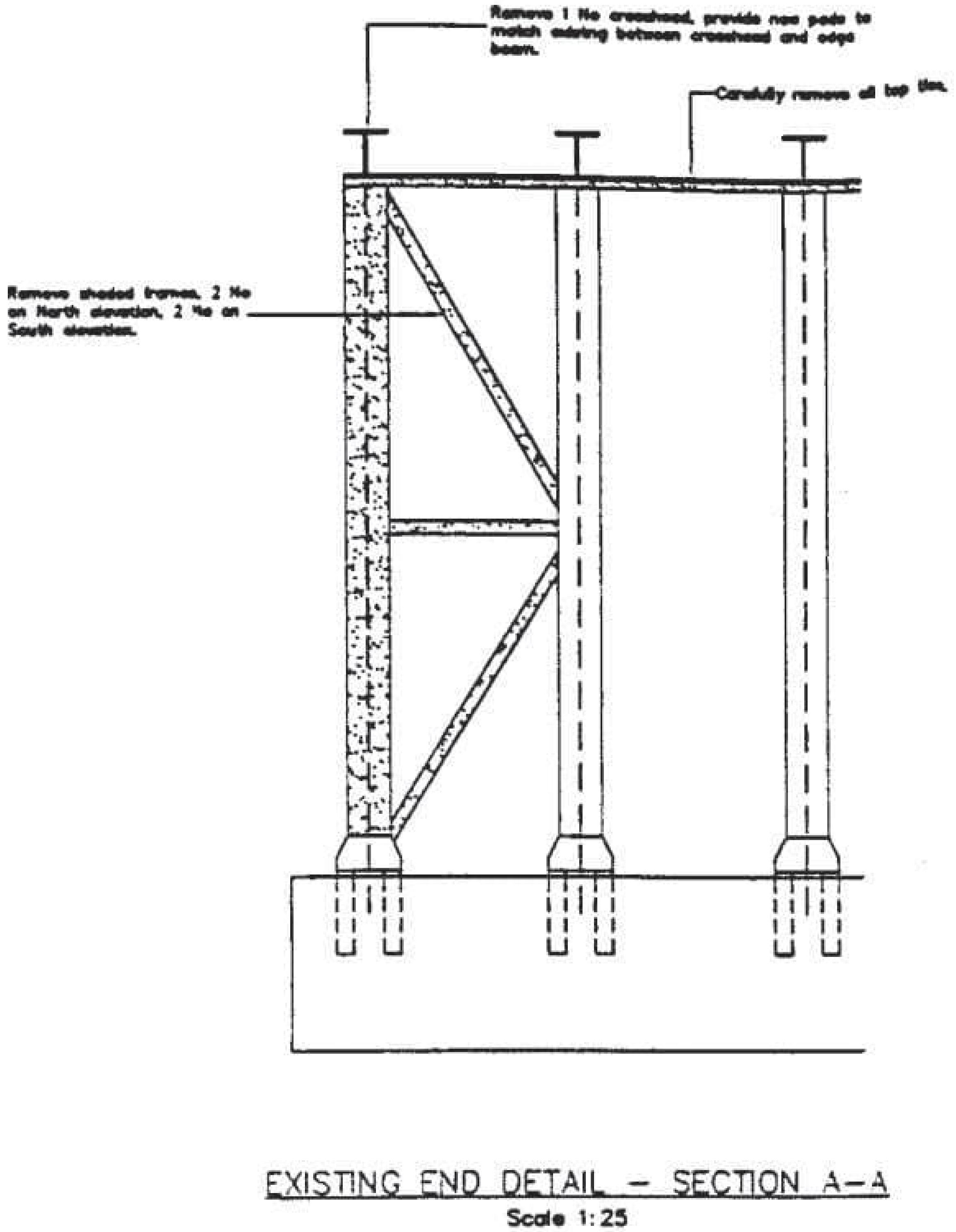
BR LMR Fabrication drawing – Propping Plan



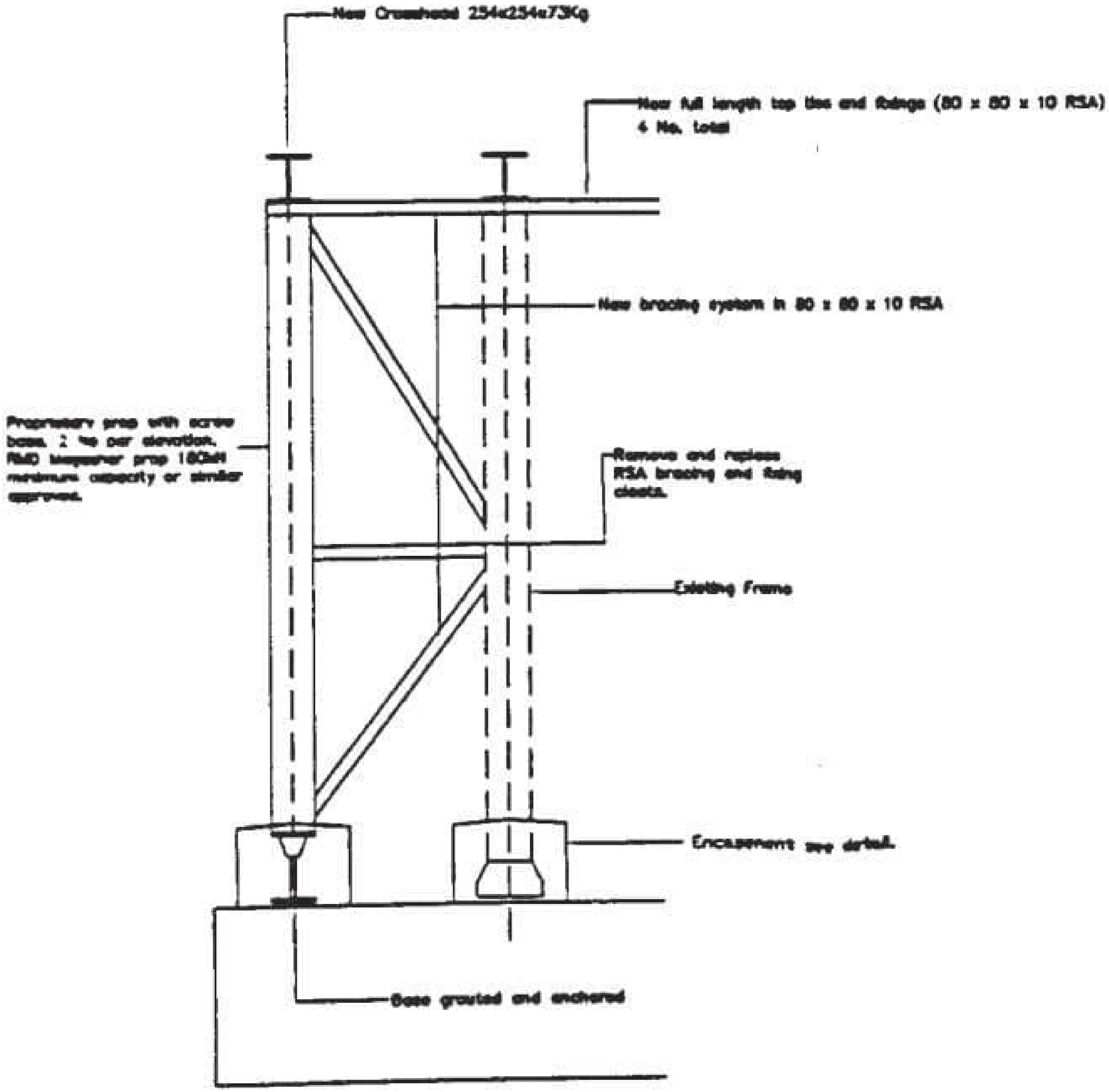
BR LMR Fabrication drawing – Propping Section



BR LMR Fabrication drawing – Column Details

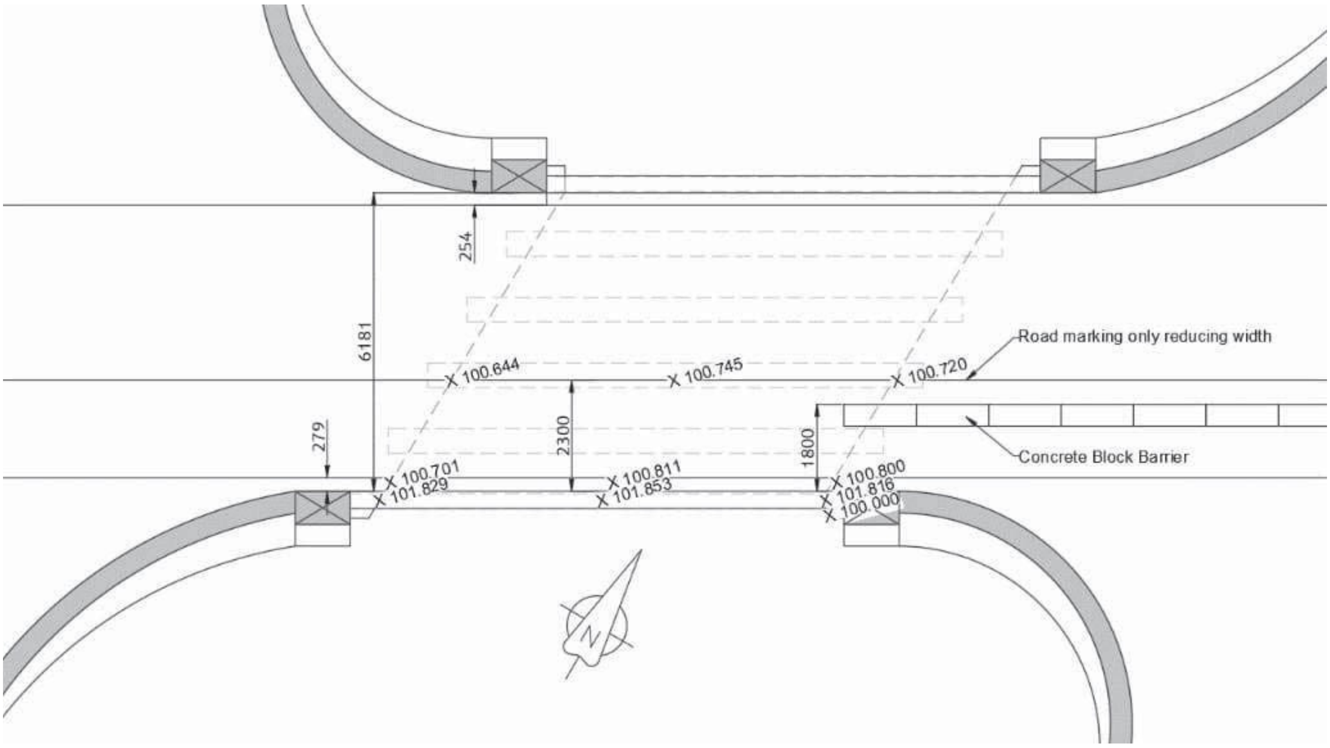


Cheshire CC Prop Replacement drawing – Existing Propping Arrangement

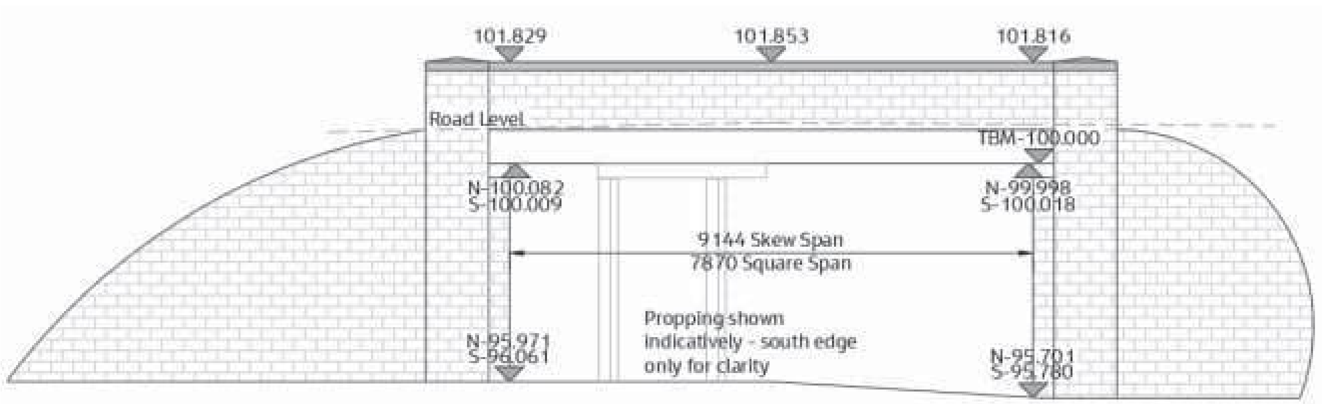


Cheshire CC Prop Replacement drawing – Replacement Prop Arrangement

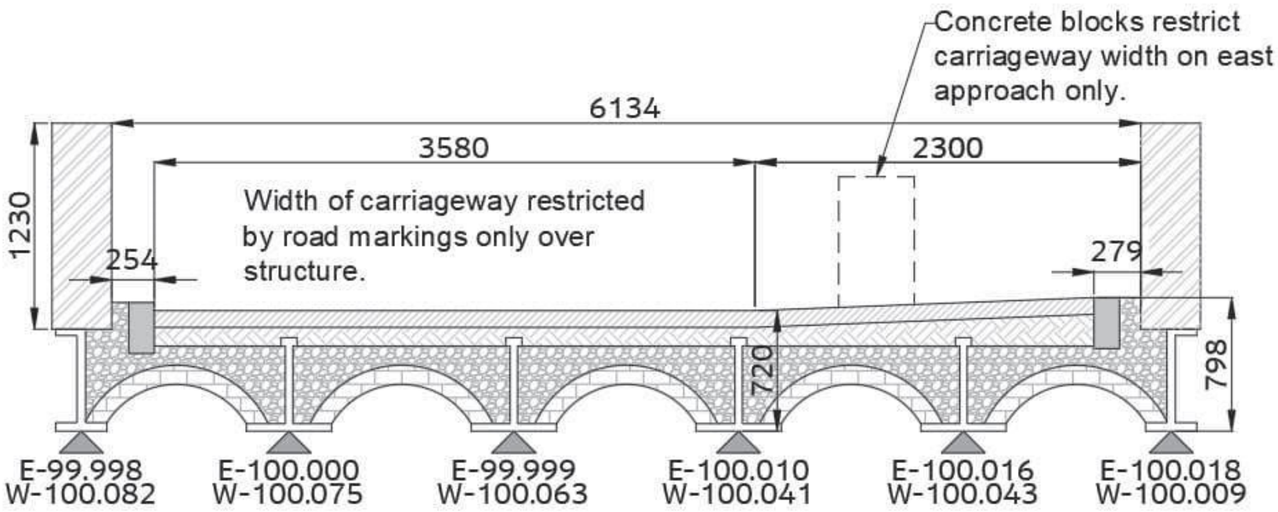
Appendix E. Survey Sketches



Plan at road level



South elevation



Section at midspan

Appendix F. Calculations

CALCULATION COVER SHEET

Jacobs
York

Project Title: HRE Assessment Programme			Calc. No.: 0451398
Job No:	B38380BA		File: VAR9/6429
Project Manager	[redacted]	Subject: **MKT/461**	
Assessor		Baddington Road Bridge, Cheshire East	
Project Group	31200	CS 454 Assessment	

	Total Sheets	Made by	Date	Checked by	Date	Reviewed by	Date		
Original	69	[redacted]	Oct-22	[redacted]	Oct-22	[redacted]	Oct-22		
Rev									
Rev									
Rev									
Rev									
Rev									

Superseded by Calculation No. Date

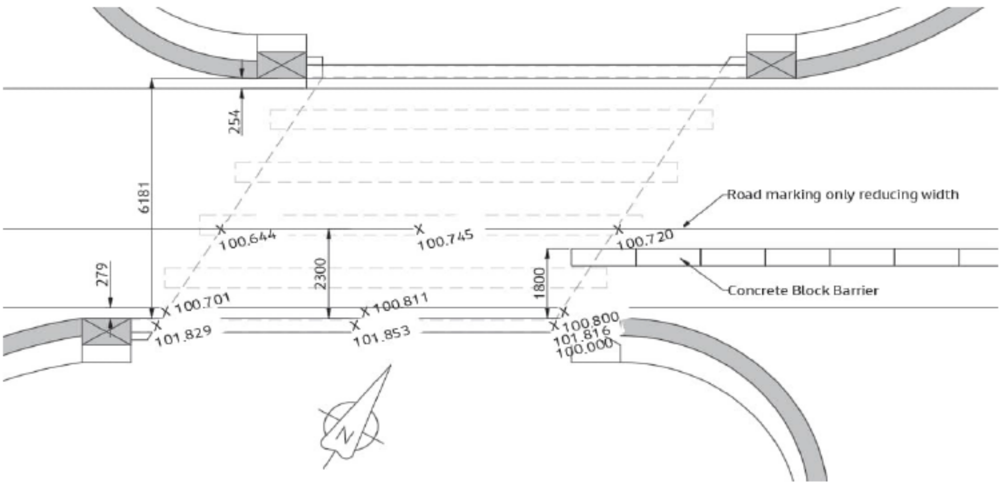
For assessment criteria, refer to Approval in Principle (Form AA) document

Jacobs

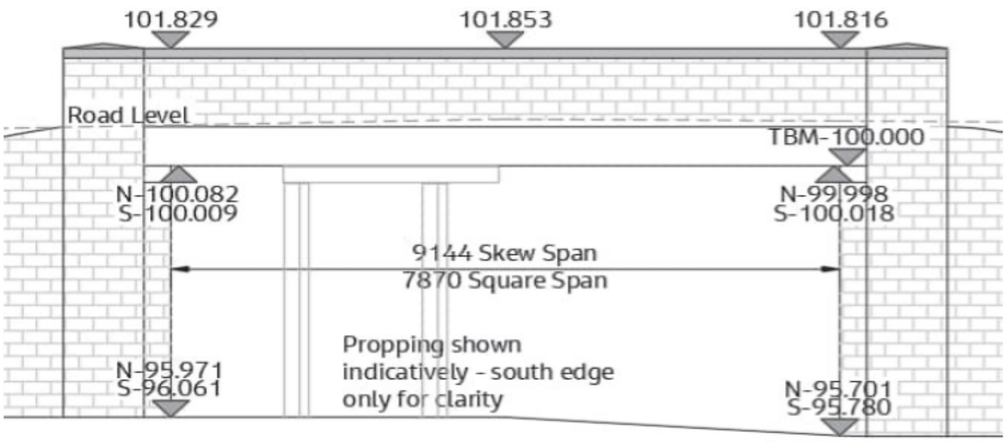
CALCULATION SHEET

Office	York Office	Page No.	1	Calc No.	0451398
Job No. & Title	VAR9/6429 HRE Assessment Programme MKT/461 CS 454 Assessment	Calcs by	[redacted]	Date	Jan-22
Section	Sketches	Checker		Date	

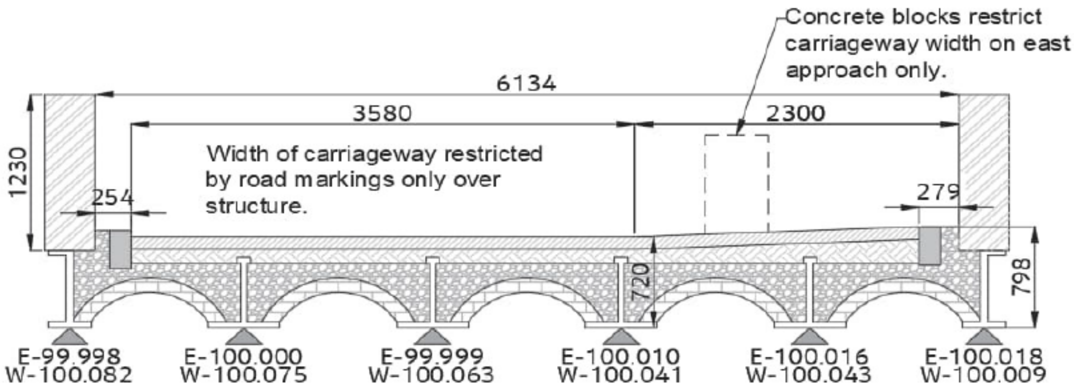
Plan at Road Level



South Elevation



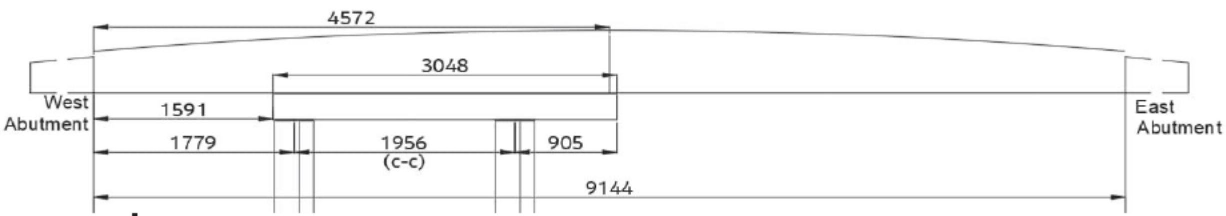
Section at Midspan



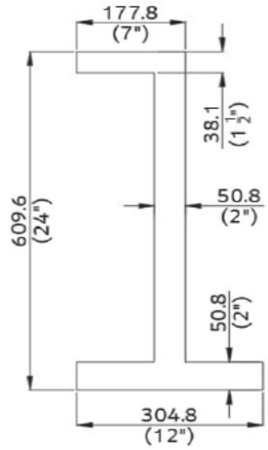
Jacobs **CALCULATION SHEET**

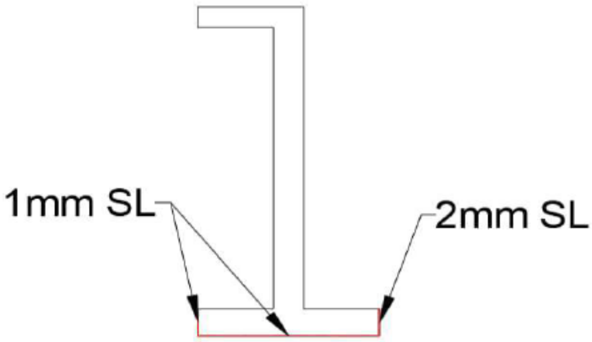
Office	York Office	Page No.	2	Calc No.	0451398
Job No. & Title	VAR9/6429 HRE Assessment Programme MKT/461 CS 454 Assessment	Calcs by	[redacted]	Date	Jan-22
Section	Sketches	Checker		Date	

<u>Sketch of Propping</u>



Cast Iron Edge Girders





Gross elastic section properties about x-x

No	Section	b	d	A	y	A·y	A(y-yt)²	Ixx
		mm	mm	mm²	mm	mm³	mm³	mm⁴
1	Top Flange	177.8	38.1	6774.18	19.05	1.29E+05	7.44E+08	8.19E+05
2	Web	50.8	520.7	26451.56	298.45	7.89E+06	7.15E+07	5.98E+08
3	Bottom Flange	304.8	50.8	15483.84	584.20	9.05E+06	8.46E+08	3.33E+06
			Area	48709.58		1.71E+07	1.66E+09	6.02E+08

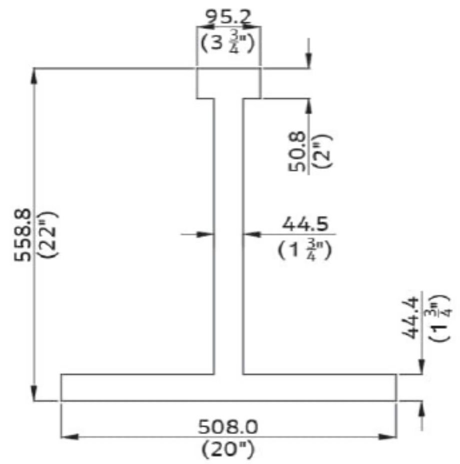
Depth of full section	D	=	609.60	mm
Depth of web	dw	=	520.70	mm
Distance to NA from top of section	yt	=	350.43	mm
Distance to NA from bottom of section	yb	=	259.17	mm
Second moment of area of section	Ixx	=	2.26E+09	mm⁴
Elastic section modulus (compression flange)	Zxc	=	6.46E+06	mm³
Elastic section modulus (tension flange)	Zxt	=	8.73E+06	mm³

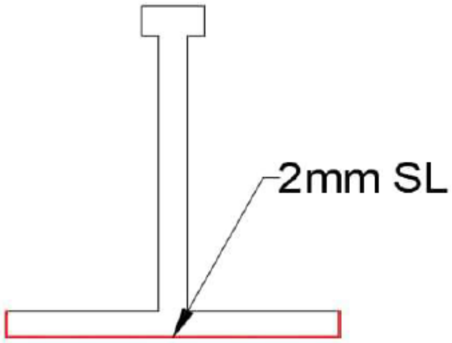
Corroded elastic section properties about x-x

No	Section	b	d	A	y	A·y	A(y-yt)²	Ixx
		mm	mm	mm²	mm	mm³	mm³	mm⁴
0	Beam	-	-	48709.58	350.43	1.71E+07	2.70E+05	2.26E+09
1	SL Flange inner	-2.0	50.8	-101.60	584.20	-5.94E+04	-5.66E+06	-2.18E+04
2	SL Flange outer	-1.0	50.8	-50.80	584.20	-2.97E+04	-2.83E+06	-1.09E+04
3	SL Flange soffit	-301.8	1.0	-301.80	609.10	-1.84E+05	-2.06E+07	-2.52E+01
			Net Area	48255.38		1.68E+07	-2.88E+07	2.26E+09

Depth of full section	D	=	608.60	mm
Depth of web panel	dw	=	520.70	mm
Distance to NA from top of section	yt	=	348.07	mm
Distance to NA from bottom of section	yb	=	260.53	mm
Second moment of area of section	Ixx	=	2.23E+09	mm⁴
Elastic section modulus (compression flange)	Zxc	=	6.42E+06	mm³
Elastic section modulus (tension flange)	Zxt	=	8.58E+06	mm³

Cast Iron Internal Girders
Section at Midspan





Section loss constant thoughout

Gross elastic section properties about x-x

No	Section	b	d	A	y	A·y	A(y-yt)²	Ixx
		mm	mm	mm²	mm	mm³	mm³	mm⁴
1	Top Flange	95.3	50.8	4838.70	25.40	1.23E+05	5.95E+08	1.04E+06
2	Web	44.5	463.6	20604.80	282.58	5.82E+06	1.80E+08	3.69E+08
3	Bottom Flange	508.0	44.5	22580.60	536.58	1.21E+07	5.82E+08	3.72E+06
			Area	48024.10		1.81E+07	1.36E+09	3.74E+08

Depth of full section	D	=	558.80	mm
Depth of web	dw	=	463.6	mm
Distance to NA from top of section	yt	=	376.09	mm
Distance to NA from bottom of section	yb	=	182.71	mm
Second moment of area of section	Ixx	=	1.73E+09	mm⁴
Elastic section modulus (compression flange)	Zxc	=	4.60E+06	mm³
Elastic section modulus (tension flange)	Zxt	=	9.47E+06	mm³

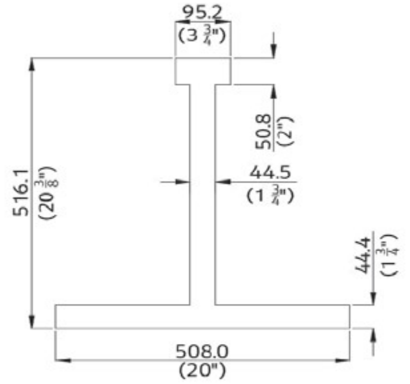
Corroded elastic section properties about x-x

No	Section	b	d	A	y	A·y	A(y-yt)²	Ixx
		mm	mm	mm²	mm	mm³	mm³	mm⁴
0	Beam	-	-	48024.10	376.09	1.81E+07	9.81E+05	1.73E+09
1	SL Flange ends	-4.0	44.5	-177.80	536.58	-9.54E+04	-4.84E+06	-2.93E+04
2	SL Flange soffit	-504.0	2.0	-1008.00	557.80	-5.62E+05	-3.50E+07	-3.36E+02
			Net Area	46838.30		1.74E+07	-3.88E+07	1.73E+09

Depth of full section	D	=	556.80	mm
Depth of web	dw	=	463.55	mm
Distance to NA from top of section	yt	=	371.57	mm
Distance to NA from bottom of section	yb	=	185.23	mm
Second moment of area of section	Ixx	=	1.69E+09	mm⁴
Elastic section modulus (compression flange)	Zxc	=	4.55E+06	mm³
Elastic section modulus (tension flange)	Zxt	=	9.13E+06	mm³

Sections at prop column locations

The depth of the girders has been estimated using linear interpolation from the depth at midspan to the depth at the ends of the girder.



Section at east prop column location

Gross elastic section properties about x-x

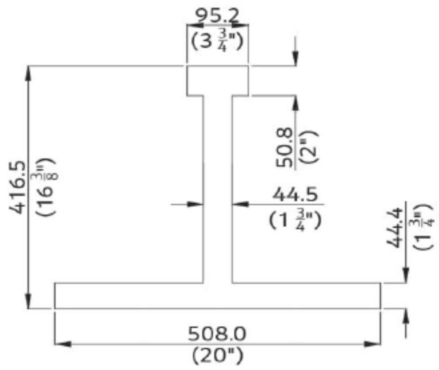
No	Section	b	d	A	y	A·y	A(y-yt)²	Ixx
		mm	mm	mm²	mm	mm³	mm³	mm⁴
1	Top Flange	95.3	50.8	4838.70	25.40	1.23E+05	5.11E+08	1.04E+06
2	Web	44.5	420.9	18706.78	261.23	4.89E+06	1.49E+08	2.76E+08
3	Bottom Flange	508.0	44.5	22580.60	493.88	1.12E+07	4.65E+08	3.72E+06
			Area	46126.08		1.62E+07	1.12E+09	2.81E+08

Depth of full section	D	=	516.10	mm
Depth of web	dw	=	420.9	mm
Distance to NA from top of section	yt	=	350.38	mm
Distance to NA from bottom of section	yb	=	165.72	mm
Second moment of area of section	Ixx	=	1.41E+09	mm⁴
Elastic section modulus (compression flange)	Zxc	=	4.01E+06	mm³
Elastic section modulus (tension flange)	Zxt	=	8.48E+06	mm³

Corroded elastic section properties about x-x

No	Section	b	d	A	y	A·y	A(y-yt)²	Ixx
		mm	mm	mm²	mm	mm³	mm³	mm⁴
0	Beam	-	-	46126.08	350.38	1.62E+07	8.38E+05	1.41E+09
1	SL Flange ends	-4.0	44.5	-177.80	493.88	-8.78E+04	-3.88E+06	-2.93E+04
2	SL Flange soffit	-504.0	2.0	-1008.00	515.10	-5.19E+05	-2.88E+07	-3.36E+02
			Net Area	44940.28		1.56E+07	-3.18E+07	1.41E+09

Depth of full section	D	=	514.10	mm
Depth of web	dw	=	420.85	mm
Distance to NA from top of section	yt	=	346.12	mm
Distance to NA from bottom of section	yb	=	167.98	mm
Second moment of area of section	Ixx	=	1.37E+09	mm⁴
Elastic section modulus (compression flange)	Zxc	=	3.97E+06	mm³
Elastic section modulus (tension flange)	Zxt	=	8.18E+06	mm³



Section at west prop column location

Section for location of maximum bending between the props and abutments will be similar.

Gross elastic section properties about x-x

No	Section	b	d	A	y	A·y	A(y-yt)²	Ixx
		mm	mm	mm²	mm	mm³	mm³	mm⁴
1	Top Flange	95.3	50.8	4838.70	25.40	1.23E+05	3.36E+08	1.04E+06
2	Web	44.5	321.3	14279.56	211.43	3.02E+06	8.56E+07	1.23E+08
3	Bottom Flange	508.0	44.5	22580.60	394.28	8.90E+06	2.51E+08	3.72E+06
			Area	41698.86		1.20E+07	6.72E+08	1.28E+08

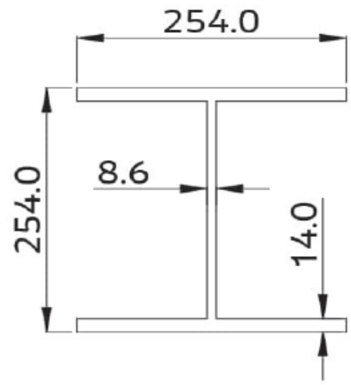
Depth of full section	D	=	416.50	mm
Depth of web	dw	=	321.3	mm
Distance to NA from top of section	yt	=	288.86	mm
Distance to NA from bottom of section	yb	=	127.64	mm
Second moment of area of section	Ixx	=	8.00E+08	mm⁴
Elastic section modulus (compression flange)	Zxc	=	2.77E+06	mm³
Elastic section modulus (tension flange)	Zxt	=	6.27E+06	mm³

Corroded elastic section properties about x-x

No	Section	b	d	A	y	A·y	A(y-yt)²	Ixx
		mm	mm	mm²	mm	mm³	mm³	mm⁴
0	Beam	-	-	41698.86	288.86	1.20E+07	5.45E+05	8.00E+08
1	SL Flange ends	-4.0	44.5	-177.80	394.28	-7.01E+04	-2.11E+06	-2.93E+04
2	SL Flange soffit	-504.0	2.0	-1008.00	415.50	-4.19E+05	-1.71E+07	-3.36E+02
			Net Area	40513.06		1.16E+07	-1.87E+07	8.00E+08

Depth of full section	D	=	414.50	mm
Depth of web	dw	=	321.25	mm
Distance to NA from top of section	yt	=	285.24	mm
Distance to NA from bottom of section	yb	=	129.26	mm
Second moment of area of section	Ixx	=	7.81E+08	mm⁴
Elastic section modulus (compression flange)	Zxc	=	2.74E+06	mm³
Elastic section modulus (tension flange)	Zxt	=	6.04E+06	mm³

Propping - Universal Columns



Gross elastic section properties about x-x

No	Section	b	d	A	y	A·y	A(y-yt)²	Ixx
		mm	mm	mm²	mm	mm³	mm³	mm⁴
1	Flange	254.0	14.0	3556.00	7.00	2.49E+04	5.12E+07	5.81E+04
2	Web	8.6	226.0	1943.60	127.00	2.47E+05	3.93E-25	8.27E+06
3	Flange	254.0	14.0	3556.00	247.00	8.78E+05	5.12E+07	5.81E+04
			Net Area	9055.60		1.15E+06	1.02E+08	8.39E+06

Depth of full section	D	=	254.00	mm
Depth of web between flanges	dw	=	226.00	mm
Distance to NA from top of section	yt	=	127.00	mm
Distance to NA from bottom of section	yb	=	127.00	mm
Second moment of area of section	Ixx	=	1.11E+08	mm⁴
Elastic section modulus (compression flange)	Zxc	=	8.72E+05	mm³
Elastic section modulus (tension flange)	Zxt	=	8.72E+05	mm³
Radius of Gyration	Rx	=	110.62	mm

Gross elastic section properties about y-y

No	Section	b	d	A	x	A·x	A(x-xt)²	Iyy
		mm	mm	mm²	mm	mm³	mm³	mm⁴
1	Flange	14.0	254.0	3556.00	0.00	0.00E+00	0.00E+00	1.91E+07
2	Web	226.0	8.6	1943.60	0.00	0.00E+00	0.00E+00	1.20E+04
3	Flange	14.0	254.0	3556.00	0.00	0.00E+00	0.00E+00	1.91E+07
			Net Area	9055.60		0.00E+00	0.00E+00	3.82E+07

Width of full section	B	=	254.00	mm
Distance to NA	xt	=	0.00	mm
Second moment of area of section	Iyy	=	3.82E+07	mm⁴
Radius of Gyration	Ry	=	64.99	mm

Net plastic section properties

No	Section	b	d	A	y	A(yp-y)
		mm	mm	mm²	mm	mm³
1	Flange	254.0	14.0	3556.00	7.00	4.27E+05
2	Web	8.6	226.0	1943.60		
3	Flange	254.0	14.0	3556.00	247.00	4.27E+05
				9055.60		8.53E+05

Area above web = 3556.00 mm²

Area below web = 3556.00 mm²

Section	b	d	A	y	A(yp-y)
Upper Web	8.6	113.0	971.80	310.50	-1.78E+05
Lower Web	8.6	113.0	971.80	423.50	2.88E+05
			1943.60		1.10E+05

Distance from top of web to top of girder = 14.00 mm

Distance to PNA from top of girder y_p = 127.00 mm

$Z_{pe}= \Sigma Ay$ = 9.63E+05 mm³

Compactness Check

BS5400:3-2000 Cl. 9.3.7.3.1

The projection of the compression flange outstand (bfo) should not exceed: $7t_{fo}\sqrt{355/\sigma_{yf}}$

Where:

b_{fo} = the width of the outstand measured from the edge of the flange to the toe of a root fillet of a rolled section

t_{fo} = the mean thickness of the outstand

σ_{yf} = the minimum yield strength of the flange material

b_{fo} = 120.00 mm

t_{fo} = 14.00 mm

Form AA

σ_{yf} = 230 N/mm²

$7t_{fo}\sqrt{355/\sigma_{yf}}$ = 121.75 mm

CS 456 Cl. 9.3.8.2

The depth between the plastic neutral axis of the beam and the compressive edge of the web, d1, shall not exceed the following limits:

For $d_1 \le 0.5d_w$ $\quad 28t_w\sqrt{\dfrac{355}{\sigma_{yw}}}$

For $d_1 > 0.5d_w$ the greater of

$$\begin{cases} \left(32 - \frac{8d_1}{d_w}\right) t_w\sqrt{\frac{355}{\sigma_{yw}}} \\ 24t_w\sqrt{\frac{355}{\sigma_{yw}}} \end{cases}$$

Where:

t_w = the thickness of the web plate = 8.6 mm

d_w = the depth of the web plate = 226.0 mm

Form AA

σ_{yw} = Minimum yeild strength of web material = 192 N/mm²

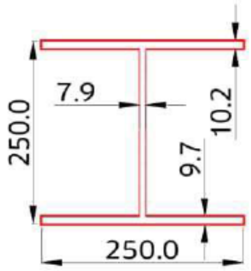
d_1 = depth between plastic neutral axis and top of web = 113.00 mm

$0.5d_w$ = 113.00 mm

Therefore the depth of the web should not exceed = 327.43 mm

Girder is considered compact

Propping Beams



Corroded elastic section properties about x-x

No	Section	b	d	A	y	A·y	A(y-yt)²	Ixx
		mm	mm	mm²	mm	mm³	mm³	mm⁴
1	Flange	250.0	10.2	2550.00	5.10	1.30E+04	3.60E+07	2.21E+04
2	Web	7.9	231.9	1832.01	126.15	2.31E+05	9.79E+03	8.21E+06
3	Flange	250.0	9.7	2425.00	246.95	5.99E+05	3.68E+07	1.90E+04
			Net Area	6807.01		8.43E+05	7.27E+07	8.25E+06

Depth of full section	D	=	251.8	mm
Depth of web panel	d_w	=	231.9	mm
Distance to NA from top of section	y_t	=	123.84	mm
Distance to NA from bottom of section	y_b	=	127.96	mm
Second moment of area of section	I_{xx}	=	8.10E+07	mm⁴
Elastic section modulus (compression flange)	Z_{xc}	=	6.54E+05	mm³
Elastic section modulus (tension flange)	Z_{xt}	=	6.33E+05	mm³

Corroded elastic section properties about y-y

No	Section	b	d	A	x	A·x	A(x-xt)²	Iyy
		mm	mm	mm²	mm	mm³	mm³	mm⁴
1	Flange	10.2	250.0	2550.00	0.00	0.00E+00	0.00E+00	1.33E+07
2	Web	231.9	7.9	1832.01	0.00	0.00E+00	0.00E+00	9.53E+03
3	Flange	9.7	250.0	2425.00	0.00	0.00E+00	0.00E+00	1.26E+07
			Net Area	6807.01		0.00E+00	0.00E+00	2.59E+07

Width of full section	B	=	250.0	mm
Distance to NA	x_t	=	0.00	mm
Second moment of area of section	I_{yy}	=	2.59E+07	mm⁴
Radius of Gyration	R_y	=	61.71	mm

Jacobs **CALCULATION SHEET**

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Job No. & Title	VAR9/6429 HRE Assessment Programme MKT/461 CS 454 Assessment	**Calcs by**	[redacted]	**Date**	Jan-22
Section	Section Properties	**Checker**	[redacted]	**Date**	Jan-22

Corroded plastic section properties

No	Section	b	d	A	y	A(yp-y)
		mm	mm	mm²	mm	mm³
1	Flange	250.0	10.2	2550.00	5.10	2.89E+05
2	Web	7.9	231.9	1832.01		
3	Flange	250.0	9.7	2425.00	246.95	3.12E+05
				6807.01		6.01E+05

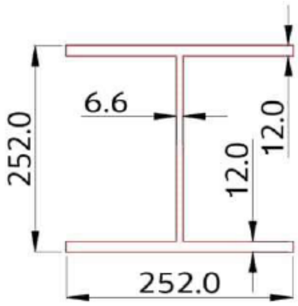
Area above web = 2550.00 mm²

Area below web = 2425.00 mm²

	b	d	A	y	A(yp-y)
Upper Web	7.9	108.0	853.51	305.82	-1.60E+05
Lower Web	7.9	123.9	978.51	421.77	2.97E+05
			1832.01		1.37E+05

Distance from top of web to top of girder		=	10.2	mm
Distance to PNA from top of girder	y_p	=	118.24	mm
	$Z_{pe} = \Sigma Ay$	=	7.38E+05	mm³

Propping Columns



Corroded elastic section properties about x-x

No	Section	b	d	A	y	A·y	A(y-yt)²	Ixx
		mm	mm	mm²	mm	mm³	mm³	mm⁴
1	Flange	252.0	12.0	3024.00	6.00	1.81E+04	4.35E+07	3.63E+04
2	Web	6.6	228.0	1504.80	126.00	1.90E+05	0.00E+00	6.52E+06
3	Flange	252.0	12.0	3024.00	246.00	7.44E+05	4.35E+07	3.63E+04
			Net Area	7552.80		9.52E+05	8.71E+07	6.59E+06

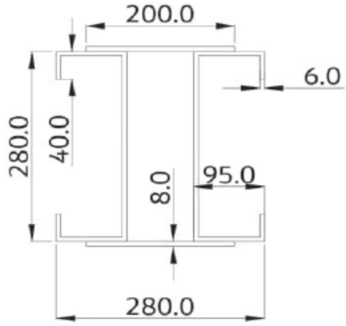
Depth of full section	D	=	252.0	mm
Depth of web panel	d_w	=	228.0	mm
Distance to NA from top of section	y_t	=	126.00	mm
Distance to NA from bottom of section	y_b	=	126.00	mm
Second moment of area of section	I_{xx}	=	9.37E+07	mm⁴
Elastic section modulus (compression flange)	Z_{xc}	=	7.44E+05	mm³
Elastic section modulus (tension flange)	Z_{xt}	=	7.44E+05	mm³
Radius of Gyration	R_x	=	111.37	mm

Corroded elastic section properties about y-y

No	Section	b	d	A	x	A·x	A(x-xt)²	Iyy
		mm	mm	mm²	mm	mm³	mm³	mm⁴
1	Flange	12.0	252.0	3024.00	0.00	0.00E+00	0.00E+00	1.60E+07
2	Web	228.0	6.6	1504.80	0.00	0.00E+00	0.00E+00	5.46E+03
3	Flange	12.0	252.0	3024.00	0.00	0.00E+00	0.00E+00	1.60E+07
			Net Area	7552.80		0.00E+00	0.00E+00	3.20E+07

Width of full section	B	=	252.0	mm
Distance to NA	xt	=	0.00	mm
Second moment of area of section	Iyy	=	3.20E+07	mm⁴
Radius of Gyration	Ry	=	65.10	mm

Edge Beam Propping Columns



Gross elastic section properties about x-x

No	Section	b	d	A	y	A·y	A(y-yt)²	Ixx
		mm	mm	mm²	mm	mm³	mm³	mm⁴
1	Flange	190.0	6.0	1140.00	3.00	3.42E+03	2.14E+07	3.42E+03
2	Web / Retuns	24.0	34.0	816.00	23.00	1.88E+04	1.12E+07	7.86E+04
3	Web	12.0	200.0	2400.00	140.00	3.36E+05	0.00E+00	8.00E+06
4	Web / Retuns	24.0	34.0	816.00	257.00	2.10E+05	1.12E+07	7.86E+04
5	Flange	190.0	6.0	1140.00	277.00	3.16E+05	2.14E+07	3.42E+03
			Net Area	6312.00		8.84E+05	6.51E+07	8.16E+06

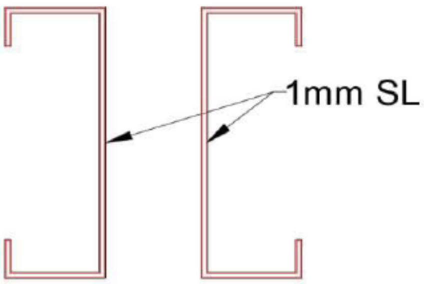
Depth of full section	D	=	280.0	mm
Depth of web panel	dw	=	268.0	mm
Distance to NA from top of section	yt	=	140.00	mm
Distance to NA from bottom of section	yb	=	140.00	mm
Second moment of area of section	Ixx	=	7.33E+07	mm⁴
Elastic section modulus (compression flange)	Zxc	=	5.24E+05	mm³
Elastic section modulus (tension flange)	Zxt	=	5.24E+05	mm³
Radius of Gyration	Rx	=	107.76	mm

Gross elastic section properties about y-y

No	Section	b	d	A	x	A·x	A(x-xt)²	Iyy
		mm	mm	mm²	mm	mm³	mm³	mm⁴
1	Return L	34.0	6.0	204.00	-137.00	-2.79E+04	3.83E+06	6.12E+02
2	Flange L	6.0	95.0	570.00	-92.50	-5.27E+04	4.88E+06	4.29E+05
3	Web L	268.0	6.0	1608.00	-48.00	-7.72E+04	3.70E+06	4.82E+03
4	Flange L	6.0	95.0	570.00	-92.50	-5.27E+04	4.88E+06	4.29E+05
5	Return L	34.0	6.0	204.00	-137.00	-2.79E+04	3.83E+06	6.12E+02
6	Return R	34.0	6.0	204.00	137.00	2.79E+04	3.83E+06	6.12E+02
7	Flange R	6.0	95.0	570.00	92.50	5.27E+04	4.88E+06	4.29E+05
8	Web R	268.0	6.0	1608.00	48.00	7.72E+04	3.70E+06	4.82E+03
9	Flange R	6.0	95.0	570.00	92.50	5.27E+04	4.88E+06	4.29E+05
10	Return R	34.0	6.0	204.00	137.00	2.79E+04	3.83E+06	6.12E+02
			Net Area	2382.00		0.00E+00	4.22E+07	1.73E+06

Width of full section	B	=	280.0	mm
Distance to NA	xt	=	0.00	mm
Second moment of area of section	Iyy	=	4.40E+07	mm⁴
Radius of Gyration	Ry	=	135.85	mm

Corroded Section



Corroded elastic section properties about x-x

No	Section	b	d	A	y	A·y	A(y-yt)²	Ixx
		mm	mm	mm²	mm	mm³	mm³	mm⁴
1	Flange	186.0	4.0	744.00	2.00	1.49E+03	1.40E+07	9.92E+02
2	Web / Retuns	16.0	34.0	544.00	21.00	1.14E+04	7.57E+06	5.24E+04
3	Web	12.0	202.0	2424.00	139.00	3.37E+05	0.00E+00	8.24E+06
4	Web / Retuns	16.0	34.0	544.00	257.00	1.40E+05	7.57E+06	5.24E+04
5	Flange	186.0	4.0	744.00	276.00	2.05E+05	1.40E+07	9.92E+02
			Net Area	5000.00		6.95E+05	4.31E+07	8.35E+06

Depth of full section	D	=	278.0	mm
Depth of web panel	dw	=	270.0	mm
Distance to NA from top of section	yt	=	139.00	mm
Distance to NA from bottom of section	yb	=	139.00	mm
Second moment of area of section	Ixx	=	5.14E+07	mm⁴
Elastic section modulus (compression flange)	Zxc	=	3.70E+05	mm³
Elastic section modulus (tension flange)	Zxt	=	3.70E+05	mm³
Radius of Gyration	Rx	=	101.42	mm

Corroded elastic section properties about y-y

No	Section	b	d	A	x	A·x	A(x-xt)²	Iyy
		mm	mm	mm²	mm	mm³	mm³	mm⁴
1	Return L	34.0	4.0	136.00	-137.00	-1.86E+04	2.55E+06	1.81E+02
2	Flange L	4.0	93.0	372.00	-92.50	-3.44E+04	3.18E+06	2.68E+05
3	Web L	270.0	4.0	1080.00	-48.00	-5.18E+04	2.49E+06	1.44E+03
4	Flange L	4.0	93.0	372.00	-92.50	-3.44E+04	3.18E+06	2.68E+05
5	Return L	34.0	4.0	136.00	-137.00	-1.86E+04	2.55E+06	1.81E+02
6	Return R	34.0	4.0	136.00	137.00	1.86E+04	2.55E+06	1.81E+02
7	Flange R	4.0	93.0	372.00	92.50	3.44E+04	3.18E+06	2.68E+05
8	Web R	270.0	4.0	1080.00	48.00	5.18E+04	2.49E+06	1.44E+03
9	Flange R	4.0	93.0	372.00	92.50	3.44E+04	3.18E+06	2.68E+05
10	Return R	34.0	4.0	136.00	137.00	1.86E+04	2.55E+06	1.81E+02
			Net Area	1588.00		0.00E+00	2.79E+07	1.08E+06

Width of full section	B	=	278.0	mm
Distance to NA	xt	=	0.00	mm
Second moment of area of section	Iyy	=	2.90E+07	mm⁴
Radius of Gyration	Ry	=	135.12	mm

Jacobs					CALCULATION SHEET
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Job No. & Title	VAR9/6429 HRE Assessment Programme MKT/461 CS 454 Assessment	**Calcs by**	[redacted]	**Date**	Jan-22
Section	Effective Spans	**Checker**	[redacted]	**Date**	Jan-22

Effective Span

The following effective spansare for the cast iron girders in an un-propped state.

CS 454 Cl. 6.6

The effective span shall be calculated based on an assessment of the position of the centroids of the support reactions.

CS 454 Cl. 6.6.1

Where there are no bearing stiffeners and the member rests directly on brick, the effective span should be taken as the distance between the centroids of bearing pressure diagrams assuming the reaction is linearly distributed from a maximum at the front edge of the support to zero at the back of the bearing area.

CS 454 Cl. 6.6.2

Where the support is brick or masonry, the length of the bearing area should be taken as no greater than the depth of the member.

As the length of the bearing area is not known, assume the depth of the girder.

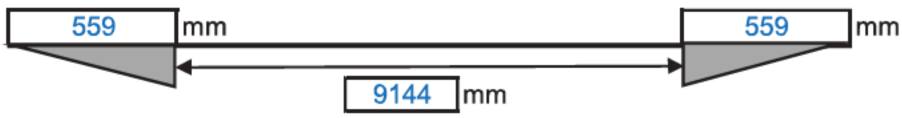
Edge Girders

Depth of girder = 609.60 mm

Effective Span = Span + 2/3(Bearing)
= 9733 mm

Internal Girders

Depth of girder = 558.80 mm (midspan used for conservative assessment)



Effective Span = Span + 2/3(Bearing)
= 9517 mm

Dead Loads and Superimposed Dead Loads

Unit Weights of Materials

CS 454
Table 4.1.1a

Material	kg/m³	kN/m³
Cast Iron	7200	70.63
Steel	7850	77.01
Bricks	2100	20.60
Miscellaneous Fill	2200	21.58
Macadam (tar)	2400	23.54
Concrete (plain)	2300	22.56

Partial Factors for Actions

CS 454
Appendix A

Dead	Cast Iron	1.00
	Steel	1.05
	Brick	1.00
Superimposed Dead	Miscellaneous Fill	1.00
	Surfacing	1.50
	Concrete	1.00

Self Weight

Gross midspan areas used for a conservative assessment

Edge Girder

Area of girder = 0.049 m²

Girder Self Weight UDL 0.049 x 70.63 x 1.00 = 3.44 kN/m

Internal Girder

Area of girder = 0.048 m²

Girder Self Weight UDL 0.048 x 70.63 x 1.00 = 3.39 kN/m

Universal Columns

Area of Column = 0.009 m²

Column Self Weight UDL 0.009 x 77.01 x 1.05 = 0.73 kN/m

Channels Columns

Area of Column = 0.006 m²

Column Self Weight UDL 0.006 x 77.01 x 1.05 = 0.51 kN/m

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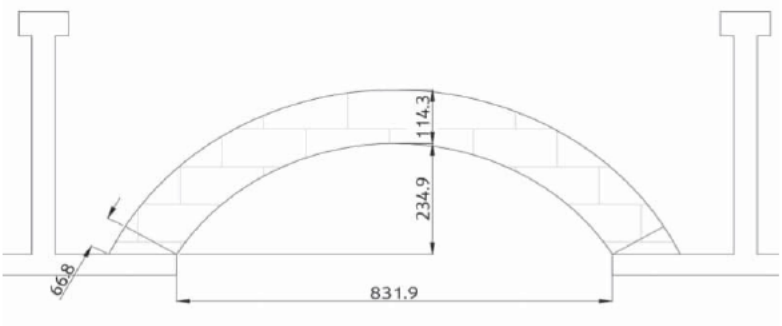
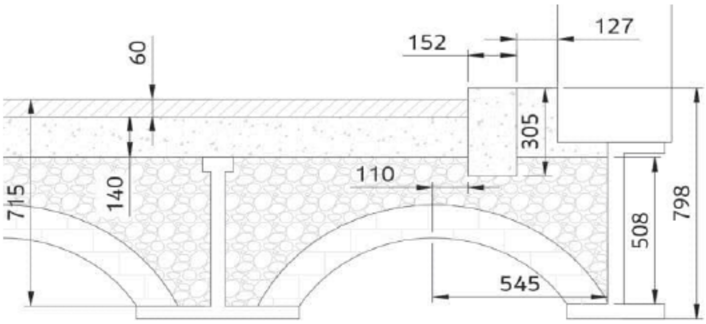
Jack Arch



Arch Barrel Thickness			darch=	0.114	m
Arch Barrel Rise			a=	0.235	m
Jack Arch Span			2b=	0.832	m
			b=	0.416	m
Jack Arch Radius	R(Intrados)	$(a^2+b^2)/2a$	=	0.486	m
	R(Extrados)	R(Intrados)+darch	=	0.600	m
Sinθ=(b/R(Intrados))			θ=	58.9	°
Area of sector		$(2\theta/360) \times \pi \times R(\text{Intrados})^2$	=	0.243	m^2
Area of triangle		b(R(Intrados)-a)	=	0.104	m^2
Area of segment		Area of sector - Area of triangle	Aseg =	0.138	m^2
Area of jack arch		$(2\theta/360) \times \pi(R(\text{Extrados})^2 - R(\text{Intrados})^2)$	Aarch =	0.128	m^2
Areas between JA and girder		0.114 x 0.067 =		0.008	m^2
Area of Jack arch		0.128 + 0.008 =		0.135	m^2
Jack arch UDL		0.135 x 20.60 x 1.00 =		2.79	kN/m

Trial pits undertaken in 2018 indicate that the jack arches are backed with rubble fill with concrete over the top of the girders and 60mm of surfacing. The following arrangement shall be used for the dead loads imposed on the girders from the deck makeup as a conservative assessment using the greater fill levels to the south.



Miscellaneous Fill

Internal Girders

Depth of fill = 0.515 m
Width of fill (girder spacing) = 1.340 m

Area of fill (0.515 x 1.340) - 0.135 - 0.138 - 0.021 - 0.005 = 0.391 m^2
(Total area - area JA - area seg - area web - area top flange)
Internal Girder Fill UDL 0.391 x 21.58 x 1.00 = 8.44 kN/m

Edge Girders

Depth of fill = 0.508 m
Width of fill = 0.545 m

Area of fill (0.508 x 0.545)-0.5(0.135 + 0.138)-(0.066 x 0.152) = 0.130 m^2
(Total area - 0.5(area JA + area seg) - part area kerb)
Edge Girder Fill UDL 0.130 x 21.58 x 1.00 = 2.81 kN/m

Concrete

Internal Girders

CS 454 Cl. 3.4.1

Top 100mm will be considered as "surfacing"

Depth of concrete = 0.140 m
Width of concrete = 1.340 m

Concrete UDL 22.56((0.04 x 1.340 x 1.5)+(0.10 x 1.340 x 1.0)) = 4.84 kN/m

Edge Girders

Area to be considered as 'surfacing' 0.110 x 0.040 = 0.004 m^2

Remaining carriageway area 0.110 x 0.100 = 0.011 m^2

Area of kerb 0.305 x 0.152 = 0.046 m^2

Area adjacent to parapet (0.127 x 0.239) + (0.156 x 0.051) = 0.038 m^2

Concrete UDL 22.56((1.5 x 0.004)+1.0(0.011+0.046+0.038)) = 2.31 kN/m

Surfacing

Depth of surfacing		=	0.060	m
Width of surfacing	Internal girder spacing	=	1.340	m
Area of surfacing		=	0.080	m²

Surfacing UDL 0.080 x 23.54 x 1.50 = 2.84 kN/m

The edge girders carry 8% of the surfacing load

Surfacing UDL 2.84 x 0.08 = 0.23 kN/m

Parapets

Width of parapets	=	0.356	m
Height of parapets	=	1.230	m
Area of parapets	=	0.438	m²

Parapet UDL 0.438 x 20.60 x 1.00 = 9.02 kN/m

Internal Girder UDL Self weight, JA, misc. fill, concrete, surfacing

3.39 + 2.79 + 8.44 + 4.84 + 2.84 = 22.30 kN/m

Edge Girder UDL Self weight, 0.5JA, misc. fill, concrete, surfacing, parapet

3.44 + 1.39 + 2.81 + 2.31 + 0.23 + 9.02 = 19.20 kN/m

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Dead Load Effects

It will be assumed that the dead loads are carried by the cast iron girders and no dead load forces will be transferred to the propping.

Cast Iron Girders

The dead loads on the cast iron girders to determine the stress due to dead loads will be applied with the girders in an unpropped state.

The maximum bending moment and shear force in a beam with a UDL are given by:

Maximum Bending Moment $$M_{max} = \frac{wl^2}{8}$$

Maximum Shear Force $$V_{max} = \frac{wl}{2}$$

Internal Girder

DL + SDL UDL = 22.30 kN/m

M_{DL} = 252.41 kNm

V_{DL} = 106.09 kN

L = 9.517 m

Edge Girder

DL + SDL UDL = 19.20 kN/m

M_{DL} = 227.40 kNm

V_{DL} = 93.45 kN

L = 9.733 m

The dead load effects will also need to be calculated for the following locations:

At the east prop support closest to midspan
At the location of the maximum sagging moment between the east prop and east abt.

At prop

The location of the prop is 3.921 m from the west abutment

Internal Girder

Imposed shear = 106.09 - (22.30 x 3.921)
= 18.67 kN

Imposed moment = (106.09 + 18.67)/2 x 3.921
= 244.60 kNm

Edge Girder

Imposed shear = 93.45 - (19.20 x 3.921)
= 18.16 kN

Imposed moment = (93.45 + 18.16)/2 x 3.921
= 218.81 kNm

Between prop and abutment

Internal Girder

Location of maximum sagging is 2.102 m from the east abutment

Imposed shear = 106.09 - (22.30 x 2.102)
= 59.24 kN

Imposed moment = (106.09 + 59.24)/2 x 2.102
= 173.72 kNm

Edge Girder

Location of maximum sagging is 2.183 m from the east abutment

Imposed shear = 93.45 - (19.20 x 2.183)
= 51.53 kN

Imposed moment = (93.45 + 51.53)/2 x 2.183
= 158.27 kNm

Propping

The propping dead loads are self weight alone. Tedds software was used to determine the imposed moments and shear forces in the cross beams.

ANALYSIS

Tedds calculation version 1.0.37

Geometry

Geometry (m) - Steel (BS5950) - UC 254x254x73

Loading

Permanent - Loading (kN/m)

Results

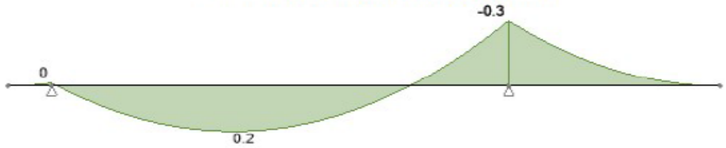
Reactions

Load case: Permanent

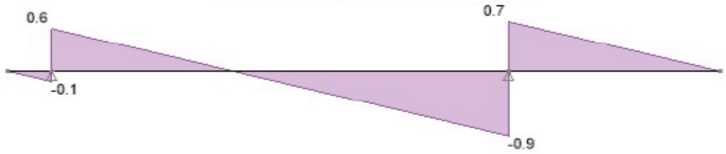
Node	Force Fx (kN)	Force Fz (kN)	Moment My (kNm)
2	0	0.7	0
3	0	1.5	0

Forces

All load cases - Moment envelope (kNm)



All load cases - Shear envelope (kN)



Element	Position (m)	Shear Force (kN)	Moment (kNm)
1	0	0	0
	0.187	-0.1	0
2	0	0.6	0
	0.978	0	0.2
	1.956	-0.9	-0.3
3	0	0.7	-0.3
	0.905	0	0

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Live Loads

Form AA

The barriers do not physically prevent vehicle loading on the south side of the structure though the traffic management in place generally restricts the traffic to a single lane. Therefore, checks for both single and double lane loading will be undertaken. The cast iron girders will be checked in both propped and unpropped conditions.

The girders will be checked for ALL model 2 loading.

The UDL and KEL applied in each lane will be as follows:

For L ≤ 20m $UDL = \frac{230}{L^{0.67}}$ KEL = 82

Internal Girders			Edge Girders		
L =	9.517	m	L =	9.733	m
UDL =	50.83	kN/m	UDL =	50.07	kN/m

The UDL and KEL are to be modified by the following factors:

CS 454
Table 5.19b
Figure 5.19c
Form AA
Form AA

Lane Factor = 1.0 for both lanes 1 and 2
K factor for 40 tonne loading = 0.9
Surface condition taken as poor
Traffic flow taken as medium

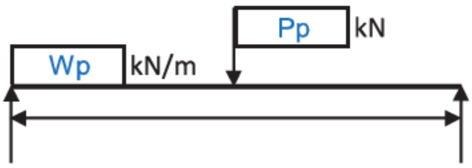
Form AA

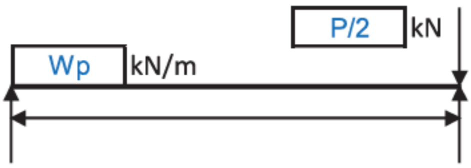
In agreement with HRE, the simple distribution method outlined in BA16/97 shall be used to determine the proportion of UDL and KEL applied onto a single girder.

For the girders in an un-propped condition:

$$M_{max} = \frac{W_p L^2}{8} + \frac{P_p L}{4}$$

$$V_{max} = \frac{W_p L}{2} + \frac{P}{2}$$





Internal Girders

Girder spacing = 1.340 m

From BA 16/97 Figure 2/2 - Single lane loading
Proportion factor = 0.292

Applied UDL (40 tonne loading) (50.83 x 1.0 x 0.90 x 0.292) = 13.36 kN/m
Applied KEL (40 tonnes loading) (82.0 x 1.0 x 0.90 x 0.292) = 21.55 kN

Mmax = 253.77 kNm
Vmax = 100.47 kNm

From BA 16/97 Figure 2/2 - Multiple lane loading
Proportion factor = 0.468

Applied UDL (40 tonne loading) (50.83 x 1.0 x 0.90 x 0.468) = 21.41 kN/m
Applied KEL (40 tonnes loading) (82.0 x 1.0 x 0.90 x 0.468) = 34.54 kN

Mmax = 406.73 kNm
Vmax = 138.78 kNm

Edge Girders

Girder spacing = 1.238 m

From BA 16/97 Figure 2/3 - Single lane loading
Proportion factor = 0.378

Applied UDL (40 tonne loading) (50.07 x 1.0 x 0.90 x 0.378) = 17.03 kN/m
Applied KEL (40 tonnes loading) (82.0 x 1.0 x 0.90 x 0.378) = 27.90 kN

Mmax = 337.49 kNm
Vmax = 119.80 kNm

From BA 16/97 Figure 2/3 - Multiple lane loading
Proportion factor = 0.428

Applied UDL (40 tonne loading) (50.07 x 1.0 x 0.90 x 0.428) = 19.29 kN/m
Applied KEL (40 tonnes loading) (82.0 x 1.0 x 0.90 x 0.428) = 31.59 kN

Mmax = 382.13 kNm
Vmax = 130.77 kNm

See following pages for extracted figures from BA 16/97

Propped Condition

The girders will be checked using ALL model 1 in a propped condition. An initial assessment assuming no distribution, i.e. a single line of wheel loads on an internal girder and applied using simple statics on the edge girders, will be undertaken as a conservative assesment as given the short spans there will be little distribution. Should the girders be found to be inadequate, a grillage model will be completed to determine the actual distribution.

The following arrangement shall be used for the propping.

1.965 m 1.956 m 5.596 m For the internal girders
5.812 m For the south edge girder

CS 454 Cl. 5.27.4

The characteristic accidental vehicle loading arrangement for non-cantilevered members should consist of a single vehicle, applied in accordance with the provisions of ALL model 1, including the impact factor and assuming medium traffic flow.

From CS 454 Table 5.9a
Impact factor = $\boxed{1.8}$ *applied to the most critical axle
The road surface condition shall be taken as poor.

From CS 454 Table 5.9b
Traffic flow factor = $\boxed{0.95}$

CS 454 Table A.1

Partial factor for traffic loading on cast iron structures = $\boxed{1.0}$

From Appendix B

Axle Ref	W1	W2	W3	W4	W5
Nominal Axle Load (kN)	69	113	74	88	88
Wheel Load (x γfL x traffic flow)	32.78	53.68	35.15	41.80	41.80
Wheel Load (incl impact)	32.78	96.62	35.15	41.80	41.80

*W2 and W3 can be swapped for worst case loading.

Using a moving loads effects spreadsheet to determine the worst cases for sagging, hogging and shear.

Results are for full vehicle loads with impact factor only applied.

For Internal girders:

Maximum sagging occurs at 7.415 m when W1 is positioned 10.215 m from the west support.

Sagging moment = 251.73 kNm
Factored moment = 251.73 x 0.5 x 0.95 x 1.0 = 119.57 kNm

Maximum hogging occurs at 3.921 m when W1 is positioned 9.105 m from the west support.

Hogging moment = -222.70 kNm
Factored moment = -222.70 x 0.5 x 0.95 x 1.0 = -105.78 kNm

Maximum shear at the abutment supports occurs at the east abutment when W1 is positioned 12.316 m from the west support

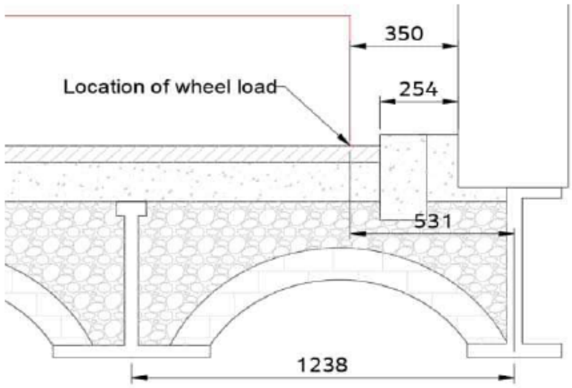
Shear = 253.59 kN
Factored shear = 253.59 x 0.5 x 0.95 x 1.0 = 120.45 kN

Maximum shear at a prop support occurs at the east prop when W1 is positioned 8.022 m and axles W3 and W2 have been swapped.

Shear = 293.28 kN
Factored shear = 293.28 x 0.5 x 0.95 x 1.0 = 139.31 kN

The imposed load on the edge girders will be determined through simple statics. For the edge girders, the positioning of the loads for accidental vehicle loading will be more onerous and the wheel loads remain the same as for carriageway loading.

Therefore consider the arrangement below:



Edge girder considered to carry 57% of a single wheel load
29% of an axle load

For Edge girders:

Maximum sagging occurs at 7.55 m when W1 is positioned 12.65 m from the west support.

Sagging moment = 262.09 kNm
Factored moment = 262.09 x 0.29 x 0.95 x 1.0 = 71.10 kNm

Maximum hogging occurs at 3.921 m when W1 is positioned 9.185 m from the west support.

Hogging moment = -237.46 kNm
Factored moment = -237.46 x 0.29 x 0.95 x 1.0 = -64.41 kNm

Maximum shear at the abutment supports occurs at the east abutment when W1 is positioned 12.532 m from the west support

Shear = 254.42 kN
Factored shear = 254.42 x 0.29 x 0.95 x 1.0 = 69.02 kN

Maximum shear at a prop support occurs at the east prop when W1 is positioned 8.022 m and axles W3 and W2 have been swapped.

Shear = 296.28 kN
Factored shear = 296.28 x 0.29 x 0.95 x 1.0 = 80.37 kN

The girders will also be checked at the midspan of the original span where the moment is maximum from imposed dead loads.

Tedds was used to determine the imposed moments. An extract of the results can be found on the following page.

Imposed moment at the original midspan

Internal Girder = 24.4 kNm
Edge Girder = 29.5 kNm

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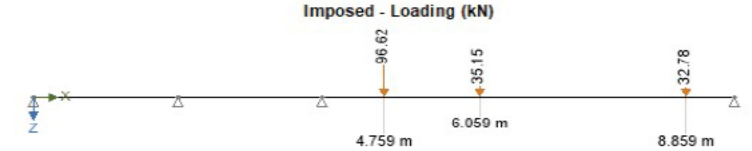
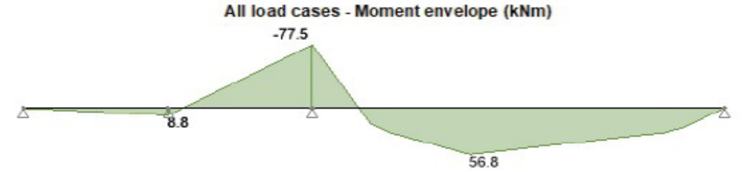
Internal Girder

Loading



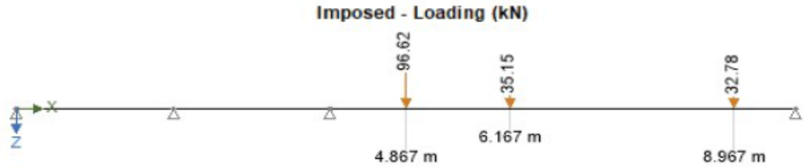
Results

Forces



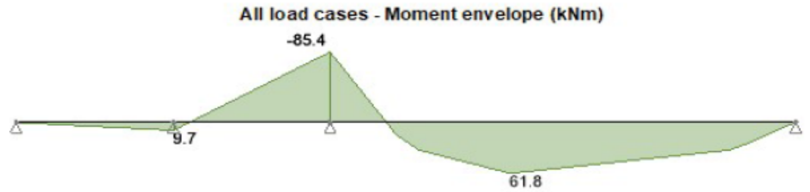
Edge Girder

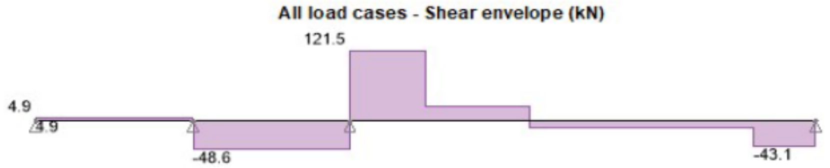
Loading



Results

Forces





Summary of Live Loads

Girder	Location	Loading	Action	Result
Internal	Original midspan	Model 2 multi.	Bending	406.73 kNm
		Model 2 single	Bending	253.77 kNm
		Model 1	Bending	24.40 kNm
	Bet. Prop & E abt.	Model 1	Bending	119.57 kNm
	East prop	Model 1	Bending	-105.78 kNm
		Model 1	Shear	139.31 kN
	Abutment	Model 2 multi.	Shear	138.78 kN
		Model 2 single	Shear	100.47 kN
		Model 1	Shear	120.45 kN
Edge	Original midspan	Model 2 multi.	Bending	382.13 kNm
		Model 2 single	Bending	337.49 kNm
		Model 1	Bending	29.50 kNm
	Bet. Prop & E abt.	Model 1	Bending	71.10 kNm
	East prop	Model 1	Bending	-64.41 kNm
		Model 1	Shear	80.37 kN
	Abutment	Model 2 multi.	Shear	130.77 kN
		Model 2 single	Shear	119.80 kN
		Model 1	Shear	69.02 kN

Permissible Stresses in Cast Iron

CS 454

Cl. 8.3	The total compressive stress shall not exceed	154	MPa
Cl. 8.4	The total tensile stress shall not exceed	46	MPa
Cl. 8.7	The total shear stress in cast iron shall not exceed	46	MPa

Eq. 8.5 Tensile stress limit for traffic loading on cast iron: $\sigma_{tr} \leq f_{p1}$

Where:

$f_{p1} = 25 - 0.44\sigma_g$ when $\sigma_g > -16MPa$

$f_{p1} = 20 - 0.76\sigma_g$ when $\sigma_g \leq -16MPa$

σ_g = stress due to permanent loads, stresses are in Mpa, tensile stress is positive

Eq. 8.6 Compressive stress limit for traffic loading in cast iron: $-\sigma_{tr} \leq -f_{p2}$

Where:

$f_{p2} = -(44 - 0.79\sigma_g)$ when $\sigma_g > 16MPa$

$f_{p2} = -(81 - 3.15\sigma_g)$ when $\sigma_g \leq 16MPa$

σ_g = stress due to permanent loads, stresses are in Mpa, tensile stress is positive

Eq. 8.8 Shear stress limit for traffic loading in cast iron:

$$\tau_{tr} \leq \min \begin{cases} 25 - 0.44\tau_g \\ 44 + 0.79\tau_g \end{cases}$$

Where:

τ_{tr} = the shear stress due to traffic loads only, in Mpa, taken as positive

τ_g = the shear stress due to permanent loads, in Mpa taken as positive where it acts in the same direction as τ_{tr} and negative when it acts in the opposite direction as τ_{tr}

Form AA The section modulus of the cast iron girders will not be increased by the section modulus factor FI

Internal Girder Capacity
Original Midspan

Tension section modulus = 9.13E+06 mm^3
Compression section modulus = 4.55E+06 mm^3

Unpropped, Two Lane Traffic

Moment due to dead load = 252.41 kNm
Moment due to live load = 406.73 kNm
Total imposed moment = 659.14 kNm

Tensile stress due to dead loads = 27.64 N/mm^2
Tensile stress due to live loads = 44.53 N/mm^2
Total imposed tensile stress = 72.17 N/mm^2

Imposed stress exceeds permissible tensile stress

fp1 = 12.84 N/mm^2 Tensile stress limit for traffic loading

Tensile stress due to traffic loads exceeds permissible limit

Compressive stress due to dead loads = -55.44 N/mm^2
Compressive stress due to live loads = -89.33 N/mm^2
Total imposed compressive stress = -144.77 N/mm^2

Total permissible compressive stress acceptable

fp2 = -255.64 N/mm^2 Compressive stress limit for traffic loading

Compressive stress due to traffic loads acceptable

Unpropped, Single Lane Traffic

Moment due to dead load = 252.41 kNm
Moment due to live load = 253.77 kNm
Total imposed moment = 506.18 kNm

Tensile stress due to dead loads = 27.64 N/mm^2
Tensile stress due to live loads = 27.79 N/mm^2
Total imposed tensile stress = 55.42 N/mm^2

Imposed stress exceeds permissible tensile stress

fp1 = 12.84 N/mm^2 Tensile stress limit for traffic loading

Tensile stress due to traffic loads exceeds permissible limit

Compressive stress due to dead loads	=	-55.44	N/mm²
Compressive stress due to live loads	=	-55.74	N/mm²
Total imposed compressive stress	=	-111.18	N/mm²

Total permissible compressive stress acceptable

fp2 = -255.64 N/mm² Compressive stress limit for traffic loading

Compressive stress due to traffic loads acceptable

<u>Propped</u>

Moment due to dead load	=	252.41	kNm
Moment due to live load	=	24.4	kNm
Total imposed moment	=	276.81	kNm

Tensile stress due to dead loads	=	27.64	N/mm²
Tensile stress due to live loads	=	2.67	N/mm²
Total imposed tensile stress	=	30.31	N/mm²

Total permissible tensile stress acceptable

fp1 = 12.84 N/mm² Tensile stress limit for traffic loading

Tensile stress due to traffic loads acceptable

Compressive stress due to dead loads	=	-55.44	N/mm²
Compressive stress due to live loads	=	-5.36	N/mm²
Total imposed compressive stress	=	-60.80	N/mm²

Total permissible compressive stress acceptable

fp2 = -255.64 N/mm² Compressive stress limit for traffic loading

Compressive stress due to traffic loads acceptable

Between Prop and East Abutment - point of maximum sagging due to live loads

Tension section modulus	=	6.04E+06	mm^3
Compression section modulus	=	2.74E+06	mm^3
Moment due to dead load	=	173.72	kNm
Moment due to live load	=	119.57	kNm
Total imposed moment	=	293.30	kNm
Tensile stress due to dead loads	=	28.74	N/mm^2
Tensile stress due to live loads	=	19.78	N/mm^2
Total imposed tensile stress	=	48.52	N/mm^2

Imposed stress exceeds permissible tensile stress

fp1 = 12.35 N/mm^2 Tensile stress limit for traffic loading

Tensile stress due to traffic loads exceeds permissible limit

Compressive stress due to dead loads	=	-63.43	N/mm^2
Compressive stress due to live loads	=	-43.66	N/mm^2
Total imposed compressive stress	=	-107.08	N/mm^2

Total permissible compressive stress acceptable

fp2 = -280.79 N/mm^2 Compressive stress limit for traffic loading

Compressive stress due to traffic loads acceptable

At East Prop - point of maximum hogging due to live loads

Tension section modulus	=	8.18E+06	mm³	(bottom flange)
Compression section modulus	=	3.97E+06	mm³	(top flange)

Moment due to dead load	=	244.60	kNm
Moment due to live load	=	-105.78	kNm
Total imposed moment	=	138.82	kNm

As the dead loads impose a larger effect on the girder than the live loads, the girder does not need to be checked in hogging.

Tensile stress due to dead loads	=	40.47	N/mm²
Tensile stress due to live loads	=	-17.50	N/mm²
Total imposed tensile stress	=	22.97	N/mm²

Total permissible tensile stress acceptable

fp1 = 7.19 N/mm² Tensile stress limit for traffic loading

Tensile stress due to traffic loads acceptable

Compressive stress due to dead loads	=	-89.30	N/mm²
Compressive stress due to live loads	=	38.62	N/mm²
Total imposed compressive stress	=	-50.68	N/mm²

Total permissible compressive stress acceptable

fp2 = -362.3 N/mm² Compressive stress limit for traffic loading

Compressive stress due to traffic loads acceptable

Depth of girder at support	=	514.10	mm	(2mm SL to bottom flange soffit)
Web thickness	=	44.45	mm	
Area of web	=	22852	mm²	

Imposed shear due to dead loads	=	18.67	kN
Imposed shear due to live loads	=	139.31	kN
Total imposed shear	=	157.98	kN

Shear stress due to dead loads	=	0.82	N/mm²
Shear stress due to live loads	=	6.10	N/mm²
Total imposed shear stress	=	6.91	N/mm²

Total permissible shear stress acceptable

$25 - 0.44\tau_g$ = 24.64 N/mm²

$44 + 0.79\tau_g$ = 44.65 N/mm²

Shear stress due to traffic loads acceptable

Abutment

Unpropped Two Lane Loading

Depth of girder at support	=	414.50 mm	(2mm SL to bottom flange soffit)
Web thickness	=	44.45 mm	
Area of web	=	18425 mm²	

Imposed shear due to dead loads	=	106.09 kN
Imposed shear due to live loads	=	138.78 kN
Total imposed shear	=	244.87 kN

Shear stress due to dead loads	=	5.76 N/mm²
Shear stress due to live loads	=	7.53 N/mm²
Total imposed shear stress	=	13.29 N/mm²

Total permissible shear stress acceptable

$25 - 0.44\tau_g$ = 22.47 N/mm²

$44 + 0.79\tau_g$ = 48.55 N/mm²

Shear stress due to traffic loads acceptable

Girder passes 40/44 tonne multiple lane loading in an unpropped state for shear, therefore will be adequate for all other arrangements considered.

Edge Girder

Tension section modulus = 8.58E+06 mm^3 section same throughout
Compression section modulus = 6.42E+06 mm^3

Original Midspan
Unpropped, Two Lane Traffic

Moment due to dead load = 227.40 kNm
Moment due to live load = 382.13 kNm
Total imposed moment = 609.53 kNm

Tensile stress due to dead loads = 26.51 N/mm^2
Tensile stress due to live loads = 44.55 N/mm^2
Total imposed tensile stress = 71.07 N/mm^2

Total permissible tensile stress acceptable

fp1 = 13.33 N/mm^2 Tensile stress limit for traffic loading

Tensile stress due to traffic loads exceeds permissible limit

Compressive stress due to dead loads = -35.42 N/mm^2
Compressive stress due to live loads = -59.53 N/mm^2
Total imposed compressive stress = -94.95 N/mm^2

Total compressive stress exceeds permissible stress

fp2 = -192.58 N/mm^2 Compressive stress limit for traffic loading

Compressive stress due to traffic loads acceptable

Unpropped, Single Lane Traffic

Moment due to dead load = 227.40 kNm
Moment due to live load = 337.49 kNm
Total imposed moment = 564.89 kNm

Tensile stress due to dead loads = 26.51 N/mm^2
Tensile stress due to live loads = 39.35 N/mm^2
Total imposed tensile stress = 65.86 N/mm^2

Total permissible tensile stress acceptable

fp1 = 13.33 N/mm^2 Tensile stress limit for traffic loading

Tensile stress due to traffic loads exceeds permissible limit

Compressive stress due to dead loads = -35.42 N/mm^2
Compressive stress due to live loads = -52.57 N/mm^2
Total imposed compressive stress = -87.99 N/mm^2

Total compressive stress exceeds permissible stress

fp2 = -192.58 N/mm^2 Compressive stress limit for traffic loading

Compressive stress due to traffic loads acceptable

<u>Propped</u>

Moment due to dead load = 227.40 kNm
Moment due to live load = 29.50 kNm
Total imposed moment = 256.90 kNm

Tensile stress due to dead loads = 26.51 N/mm^2
Tensile stress due to live loads = 3.44 N/mm^2
Total imposed tensile stress = 29.95 N/mm^2

Total permissible tensile stress acceptable

fp1 = 13.33 N/mm^2 Tensile stress limit for traffic loading

Tensile stress due to traffic loads acceptable

Compressive stress due to dead loads = -35.42 N/mm^2
Compressive stress due to live loads = -4.60 N/mm^2
Total imposed compressive stress = -40.02 N/mm^2

Total permissible compressive stress acceptable

fp2 = -192.58 N/mm^2 Compressive stress limit for traffic loading

Compressive stress due to traffic loads acceptable

Between Prop and East Abutment - point of maximum sagging due to live loads

Moment due to dead load	=	158.27	kNm
Moment due to live load	=	71.10	kNm
Total imposed moment	=	229.37	kNm

Tensile stress due to dead loads	=	18.45	N/mm²
Tensile stress due to live loads	=	8.29	N/mm²
Total imposed tensile stress	=	26.74	N/mm²

Total permissible tensile stress acceptable

fp1 = 16.88 N/mm² Tensile stress limit for traffic loading

Tensile stress due to traffic loads acceptable

Compressive stress due to dead loads	=	-24.65	N/mm²
Compressive stress due to live loads	=	-11.07	N/mm²
Total imposed compressive stress	=	-35.73	N/mm²

Total permissible compressive stress acceptable

fp2 = -158.66 N/mm² Compressive stress limit for traffic loading

Compressive stress due to traffic loads acceptable

At East Prop - point of maximum hogging due to live loads

Moment due to dead load	=	218.81	kNm
Moment due to live load	=	-64.41	kNm
Total imposed moment	=	154.40	kNm

Tensile stress due to dead loads	=	25.51	N/mm²
Tensile stress due to live loads	=	-7.51	N/mm²
Total imposed tensile stress	=	18.00	N/mm²

Total permissible tensile stress acceptable

fp1 = 13.77 N/mm² Tensile stress limit for traffic loading

Tensile stress due to traffic loads acceptable

Compressive stress due to dead loads	=	-34.08	N/mm²
Compressive stress due to live loads	=	10.03	N/mm²
Total imposed compressive stress	=	-24.05	N/mm²

Total permissible compressive stress acceptable

fp2 = -188.37 N/mm² Compressive stress limit for traffic loading

Compressive stress due to traffic loads acceptable

Depth of girder at support	=	608.60	mm
Web thickness	=	50.80	mm
Area of web	=	30917	mm²

Imposed shear due to dead loads	=	18.16	kN
Imposed shear due to live loads	=	80.37	kN
Total imposed shear	=	98.53	kN

Shear stress due to dead loads	=	0.59	N/mm²
Shear stress due to live loads	=	2.60	N/mm²
Total imposed shear stress	=	3.19	N/mm²

Total permissible shear stress acceptable

$25 - 0.44\tau_g$ = 24.74 N/mm²

$44 + 0.79\tau_g$ = 44.46 N/mm²

Shear stress due to traffic loads acceptable

Abutment

Unpropped Two Lane Loading

Imposed shear due to dead loads	=	93.45	kN
Imposed shear due to live loads	=	130.77	kN
Total imposed shear	=	224.22	kN

Shear stress due to dead loads	=	3.02	N/mm²
Shear stress due to live loads	=	4.23	N/mm²
Total imposed shear stress	=	7.25	N/mm²

Total permissible shear stress acceptable

$25 - 0.44\tau_g$ = 23.67 N/mm²

$44 + 0.79\tau_g$ = 46.39 N/mm²

Shear stress due to traffic loads acceptable

Girder passes 40/44 tonne multiple lane loading in an unpropped state for shear, therefore will be adequate for all other arrangements considered.

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Summary of Results

Internal Girder

At the original span midspan	Dead Load =	252.41	kNm
	Unpropped, 2 lane loading, ALL model 2 =	406.73	kNm
	Unpropped, 1 lane loading, ALL model 2 =	253.77	kNm
	Propped, ALL model 1 =	24.40	kNm
	Capacity =	369.68	kNm

Girder has sufficient capacity for 40/44 tonne GVW at midspan in a propped state

Between prop and east abutment	Dead Load =	173.72	kNm
	Propped, ALL model 1 =	119.57	kNm
	Capacity =	248.39	kNm

Girder does not have sufficient capacity for 40/44 tonne GVW between the prop and the east abutment.

At east prop	Dead Load =	244.60	kNm
	Propped, ALL model 1 =	-105.78	kNm
	Capacity =	389.75	kNm

Girder has sufficient capacity in bending for 40/44 tonne GVW at prop

Dead Load =	18.67	kN
Propped, ALL model 1 =	139.31	kN
Capacity =	581.75	kN

Girder has sufficient capacity in shear for 40/44 tonne GVW at prop

At abutment	Dead Load =	106.09	kN
	Unpropped, 2 lane loading, ALL model 2 =	138.78	kN
	Capacity =	520.03	kN

Girder has sufficient capacity in shear for 40/44 tonne GVW at abutment

Edge Girder

At the original span midspan	Dead Load =	227.40	kNm
	Unpropped, 2 lane loading, ALL model 2 =	382.13	kNm
	Unpropped, 1 lane loading, ALL model 2 =	337.49	kNm
	Propped, ALL model 1 =	29.50	kNm
	Capacity =	341.76	kNm

Girder has sufficient capacity for 40/44 tonne GVW at midspan in a propped state

Between prop and east abutment	Dead Load =	158.27	kNm
	Propped, ALL model 1 =	71.10	kNm
	Capacity =	303.05	kNm

Girder has sufficient capacity in bending for 40/44 tonne GVW between prop and east abutment

At east prop	Dead Load =	218.81	kNm
	Propped, ALL model 1 =	-64.41	kNm
	Capacity =	336.95	kNm

Girder has sufficient capacity in bending for 40/44 tonne GVW at prop

	Dead Load =	18.16	kN
	Propped, ALL model 1 =	80.37	kN
	Capacity =	783.09	kN

Girder has sufficient capacity in shear for 40/44 tonne GVW at prop

At abutment	Dead Load =	93.45	kN
	Unpropped, 2 lane loading, ALL model 2 =	130.77	kN
	Capacity =	825.25	kN

Girder has sufficient capacity in shear for 40/44 tonne GVW at abutment

Grillage

As the internal girders at the point of mximum bending between the prop and east abutment are not sufficient when a single line of wheel loads is assumed, a grillage model will be used to determine the imposed load effects on a single girder.

Advice on the ration between longitudinal and transverse stiffness for masonry jack arches with fill above is no longer available in CS 454 or CS 458, therefore a value of $EI_T / EI_L =$ 0.0305 as suggested in Annex A of BA 16/97 Clause A1 will be used. This was the basis of the simple distribution methods in BA16/97 which have proved to be reliable for complying structures.

$$\frac{E_T}{E_L} = 0.0305$$

I_L = 1.73E+09 mm4 gross midspan girders

E_L = 130.0 kN/mm2 youngs modulus for cast iron

$E_L \cdot I_L$ = 2.25E+11

E_T = 17 kN/mm2 youngs modulus for concrete

I_T = (0.0305 x 224,973,155,976)/17.0 = 4.04E+08 mm4

The following Poisson's ratio shall be used for the material properties:

Material	Poisson's Ratio
Cast Iron	0.24
Concrete	0.2

The width of the sections will be taken as the square distance between the transverse members.

Width of section = 1599 mm

Depth of sections will be taken as = ((403,628,309 x 12) / 1,599) ^ 1/3

= 145 mm

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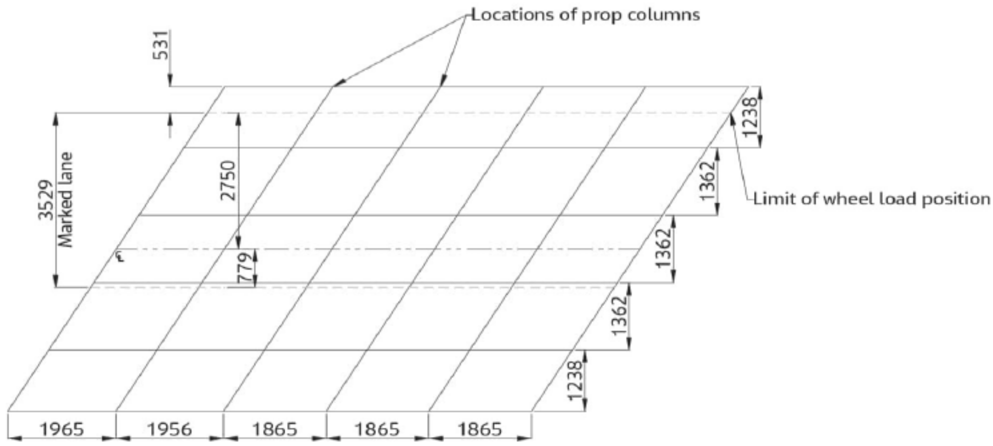
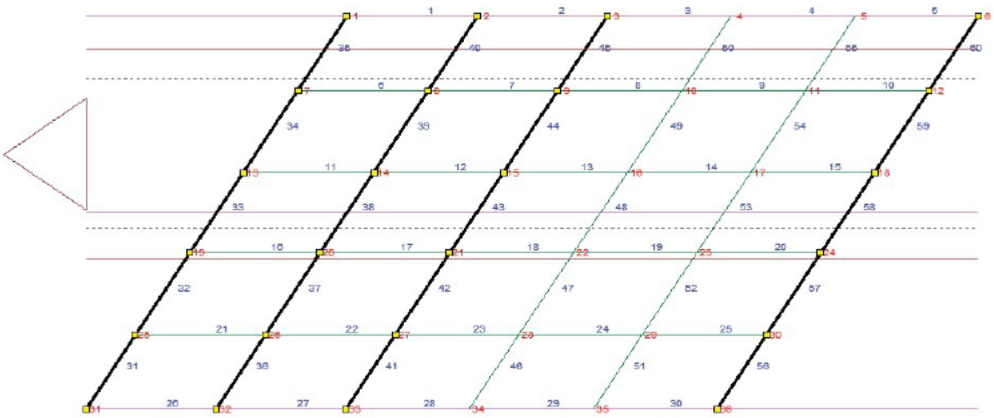
Proposed mesh layout:



Extract from Grillage model showing member layout



Internal Girder dead load UDL = $\boxed{22.30}$ kN/m

Moment due to dead loads at 1.865 m and 3.730 m from the east abutment.

Reaction at East Abutment = $\boxed{106.09}$ kN

Imposed shear at 1.865 m = 106.09 - (22.30 x 1.865)
= $\boxed{64.51}$ kN

Imposed moment at 1.865 m = (106.09 + 64.51)/2 x 1.865
= $\boxed{159.09}$ kNm

Imposed shear at 3.730 m = 106.09 - (22.30 x 3.730)
= $\boxed{22.93}$ kN

Imposed moment at 3.730 m = (106.09 + 22.93)/2 x 3.730
= $\boxed{240.62}$ kNm

The influence surface function in the SBD software was used to determine the worst case loading arrangement for each of the girders.

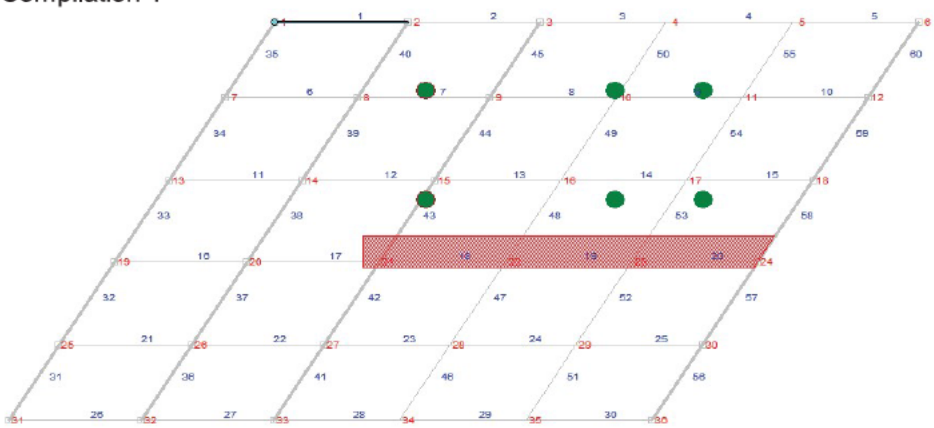
CS 454
Cl. 5.16

Where assessment live loading exceeds 7.5 tonnes, a UDL of 5 kN/m² shall be applied over the remaining area of carriageway, except where this provides a relieving effect

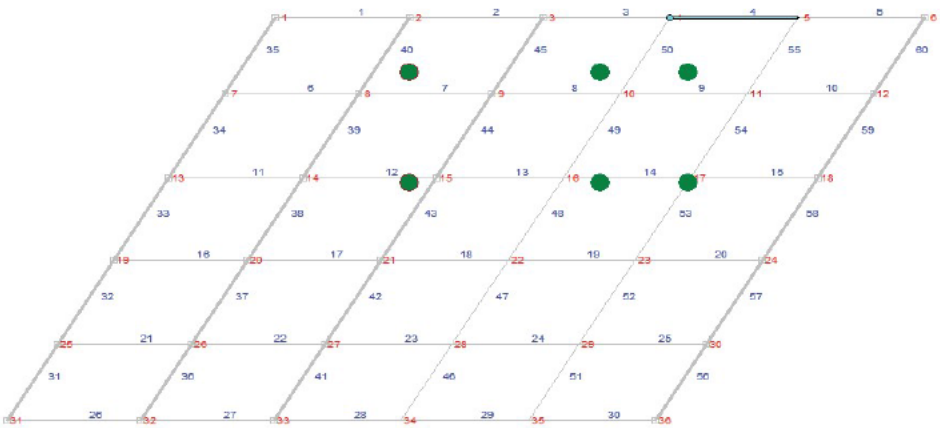
From the grillage analysis, the worst case imposed moments for internal girders:

Joint	Moment (kNm)	Compilation	DL Moment (kNm)	Total (kNm)
10	72.10	C1	240.62	312.72
11	67.16	C6	159.09	226.25
16	53.60	C24	240.62	294.22
17	100.51	C4	159.09	259.60
22	47.34	C27	240.62	287.96
23	71.43	C30	159.09	230.52

Compilation 1



Compilation 4



Capacity at Joint 10

Section is similar to the section at the east prop support

Tension section modulus	=	8.18E+06	mm³	(bottom flange)
Compression section modulus	=	3.97E+06	mm³	(top flange)

Moment due to dead load	=	240.62	kNm
Moment due to live load	=	72.10	kNm
Total imposed moment	=	312.72	kNm

Tensile stress due to dead loads	=	29.43	N/mm²
Tensile stress due to live loads	=	8.82	N/mm²
Total imposed tensile stress	=	38.24	N/mm²

Total permissible tensile stress acceptable

fp1 = 12.05 N/mm² Tensile stress limit for traffic loading

Tensile stress due to traffic loads acceptable

Capacity = 339.19 kNm

Capacity at Joint 17

Section is similar to the section at the west prop support

Tension section modulus	=	6.04E+06	mm³	(bottom flange)
Compression section modulus	=	2.74E+06	mm³	(top flange)

Moment due to dead load	=	159.09	kNm
Moment due to live load	=	100.51	kNm
Total imposed moment	=	259.60	kNm

Tensile stress due to dead loads	=	26.32	N/mm²
Tensile stress due to live loads	=	16.63	N/mm²
Total imposed tensile stress	=	42.95	N/mm²

Total permissible tensile stress acceptable

fp1 = 13.42 N/mm² Tensile stress limit for traffic loading

Tensile stress due to traffic loads exceeds permissible limit

Capacity = 240.20 kNm

Girder fails 40/44 tonne GVW loading

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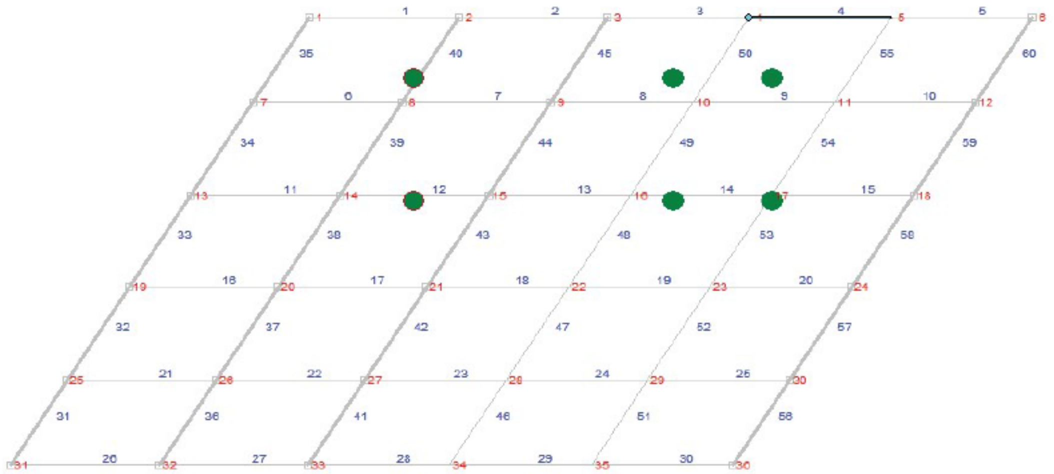
Reduced Live Loads

As the girder fails 40/44 tonne GVW, the girder will be checked for reduced live loads.

Consider 26 tonne GVW

Loading arrangement for 26 tonne loading



Capacity at Joint 17

Tension section modulus	=	6.04E+06	mm³	(bottom flange)
Compression section modulus	=	2.74E+06	mm³	(top flange)

Moment due to dead load	=	159.09	kNm
Moment due to live load	=	99.45	kNm
Total imposed moment	=	258.54	kNm

Tensile stress due to dead loads	=	26.32	N/mm²
Tensile stress due to live loads	=	16.45	N/mm²
Total imposed tensile stress	=	42.77	N/mm²

Total permissible tensile stress acceptable

fp1 = 13.42 N/mm² Tensile stress limit for traffic loading

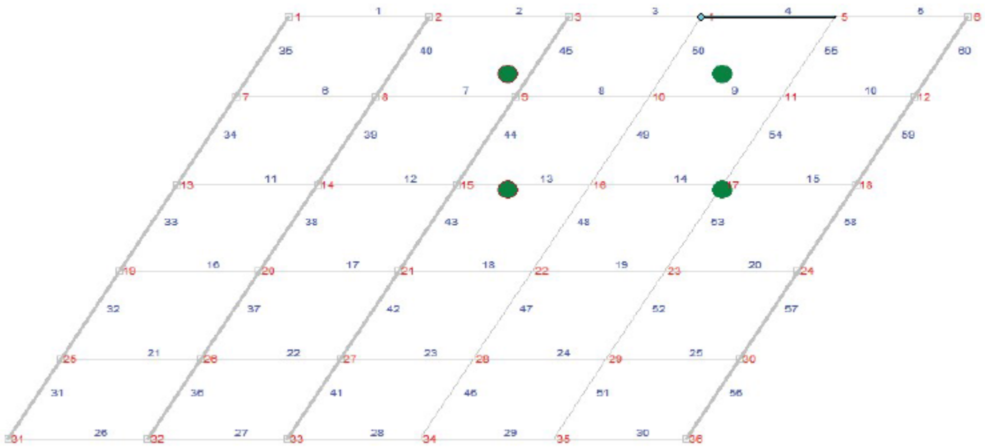
Tensile stress due to traffic loads exceeds permissible limit

Capacity = 240.20 kNm

Girder fails 26 tonne GVW loading

Consider 18 tonne GVW

Loading arrangement for 18 tonne GVW



Capacity at Joint 17

Tension section modulus = 6.04E+06 mm^3 (bottom flange)
Compression section modulus = 2.74E+06 mm^3 (top flange)

Moment due to dead load = 159.09 kNm
Moment due to live load = 86.53 kNm
Total imposed moment = 245.62 kNm

Tensile stress due to dead loads = 26.32 N/mm^2
Tensile stress due to live loads = 14.32 N/mm^2
Total imposed tensile stress = 40.64 N/mm^2

Total permissible tensile stress acceptable

fp1 = 13.42 N/mm^2 Tensile stress limit for traffic loading

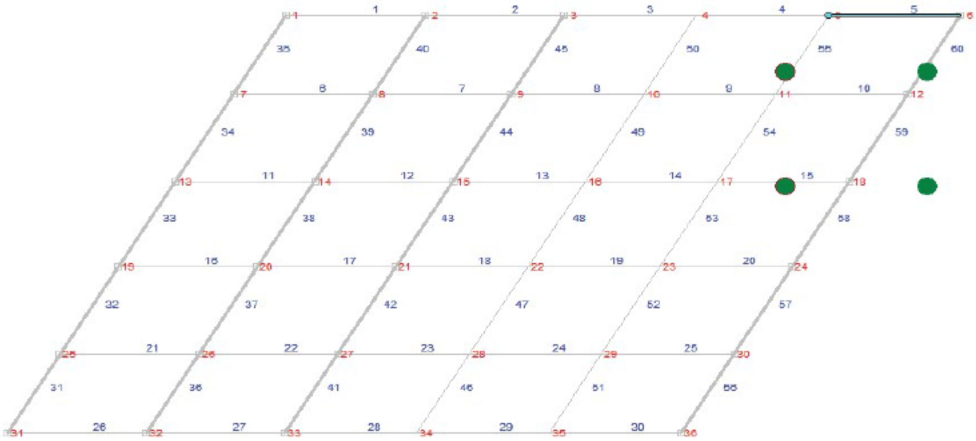
Tensile stress due to traffic loads exceeds permissible limit

Capacity = 240.20 kNm

Girder fails 18 tonne GVW loading

Consider 7.5 tonne GVW

Loading arrangement for 7.5 tonne GVW



Capacity at Joint 11 (worst case for 7.5 tonne loading)

Tension section modulus = 6.04E+06 mm³ (bottom flange)
Compression section modulus = 2.74E+06 mm³ (top flange)

Moment due to dead load = 159.09 kNm
Moment due to live load = 33.38 kNm
Total imposed moment = 192.47 kNm

Tensile stress due to dead loads = 26.32 N/mm²
Tensile stress due to live loads = 5.52 N/mm²
Total imposed tensile stress = 31.84 N/mm²

Total permissible tensile stress acceptable

fp1 = 13.42 N/mm² Tensile stress limit for traffic loading

Tensile stress due to traffic loads acceptable

Capacity = 240.20 kNm

Girder fails 18 tonne GVW loading

Propping Beam Capacity

The propping beams will be checked for bearing.

CS 456 Cl. 9.14.6.2

The buckling resistance of an unstiffened web over a bearing shall be according to:

Equation 9.14.6.2 $$P_D = \frac{\sigma_c b_{eff} t_w}{\gamma_m \gamma_{f3}}$$

Where:

σ_c = ultimate compressive stress about an axis along the centre line of the web obtained from σ_c/σ_y in accordance with curve C of Figure 37. l_e is taken as the effective length for web buckling determined taking into account of the lateral and rotational restraint of the flange.

b_{eff} = effective breadth of web obtained as $b_{eff} = \sqrt{d^2 + s^2}$ but not beyond the extent of the beam.

d = overall depth of the beam.

s = bearing length

γ_m = 1.05 for ultimate limit state

γ_{f3} = 1.1

$\frac{l_e}{r}\sqrt{\frac{\sigma_y}{355}}$ is required for determining σ_c/σ_y from Figure 37

BS 5400:3 Cl. 9.6.2

l_e = effective length taken as 1.5L (restrained at one end, partially restrained at other)

L = 0.232 m depth of web

l_e = 0.348 m

Form AA

σ_y = 230 N/mm²

r = radius of gyration (for the web) = $\sqrt{I/A}$

t = 7.9 mm

I = $(bt^3)/12$ = 41.09b

A = 7.9b mm

r = 2.28 mm

Therefore $\frac{l_e}{r}\sqrt{\frac{\sigma_y}{355}}$ = 122.8 r/y = 0.58

From Figure 37 using Curve B σ_c/σ_y = 0.35

Therefore σ_c = 80.5 N/mm²

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d = 251.8 mm
s = 296 mm (taken as the length of the bearing over the column)

Width of flange = 250 mm

beff = 388.61 mm
Therefore take beff as 250 mm

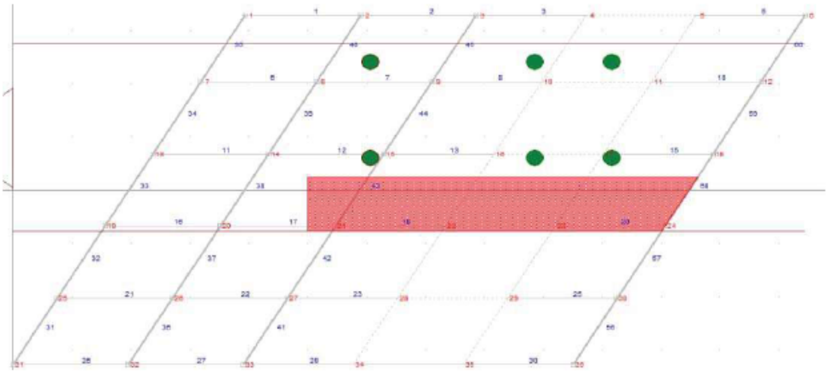
tw = 7.9 mm

P_D = 137.65 kN

Imposed shear at prop support = 146.76 kN (Internal girder at east prop)

Buckling resistance of the web is insufficient for full 40/44 tonne loading

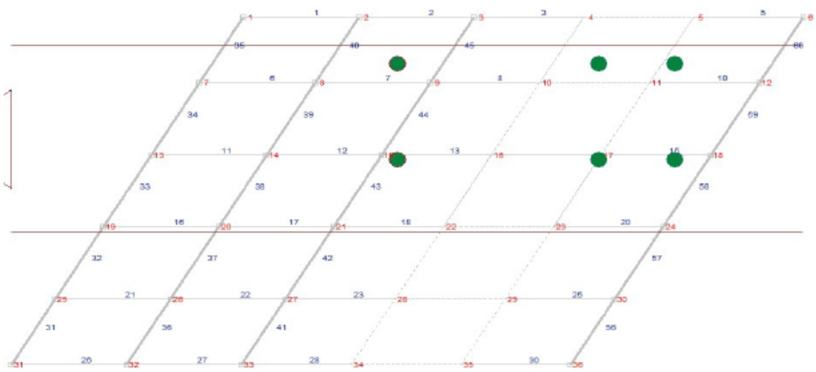
Worst case imposed shear at prop support occurs at joint 15 due to the following load arrangement for 40/44 tonne loading



Imposed shear at prop support = 113.69 kN (Internal girder at east prop

Buckling resistance of the web is sufficient for 26 tonne GVW loading

Worst case imposed shear at prop support occurs at joint 9 due to the following load arrangement for 26 tonne loading



Propping Column Capacity

BS 5400:3

The resistance of a member subject to axial compression is given by

$$P_D = \frac{A_e \sigma_c}{\gamma_m \gamma_{f3}}$$

Ae = effective area of a section
σc = least ultimate compressive stress for buckling about any axis to be obtained from σc/σy in accordance with Figure 37
γm = 1.05
γf3 = 1.1

For Figure 37 the values of $\frac{r}{y}$ and $\frac{l_e}{r}\sqrt{\frac{\sigma_y}{355}}$ are required

r = radius of gyration
y = distance from the same axis to the extreme fibre of the section based on the gross section of the member, but ignoring battening or lacing.

σy = 230 N/mm²

Form AA

le = effective length taken as 1.5L
L = length of the member between end restraints

Universal Columns

Ae = 7553 mm²
r = 65 mm
y = 126 mm
$\frac{r}{y}$ = 0.52 Therefore use Curve B
L = 3631 mm Greatest column length measured on site used for a conservative assessment
le = 5447 mm
$\frac{l_e}{r}\sqrt{\frac{\sigma_y}{355}}$ = 67.3

From Figure 37 using Curve C σc/σy = 0.66
Therefore σc = 151.8 N/mm²

PD = 992.65 kN

Maximum imposed live loads on column = 146.76 kN
Self weight dead loads are negligible
The columns propping the internal girders are sufficient for 40/44 tonne loading

Channel Section Columns

A_e = 5000 mm²
r = 135 mm
y = 139 mm

$\frac{r}{y}$ = 0.97 Therefore use Curve A

L = 3618 mm Average height of edge beam propping columns and screw jack ignored for a conservative assessment

le = 5427 mm

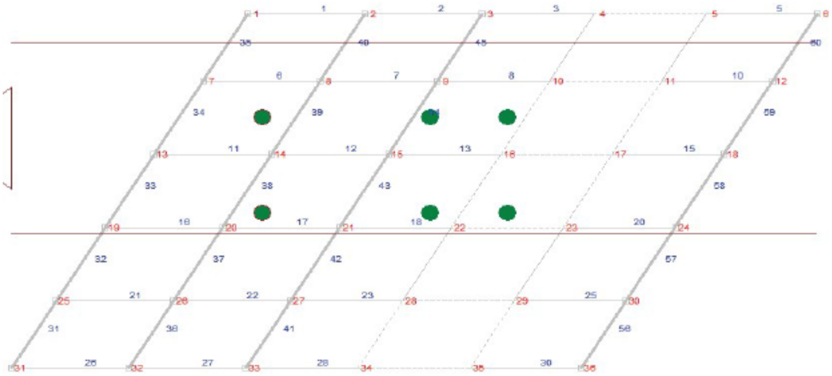
$\frac{l_e}{r}\sqrt{\frac{\sigma_y}{355}}$ = 32.3

From Figure 37 using Curve A σ_c/σ_y = 0.915
Therefore σ_c = 210.45 N/mm²

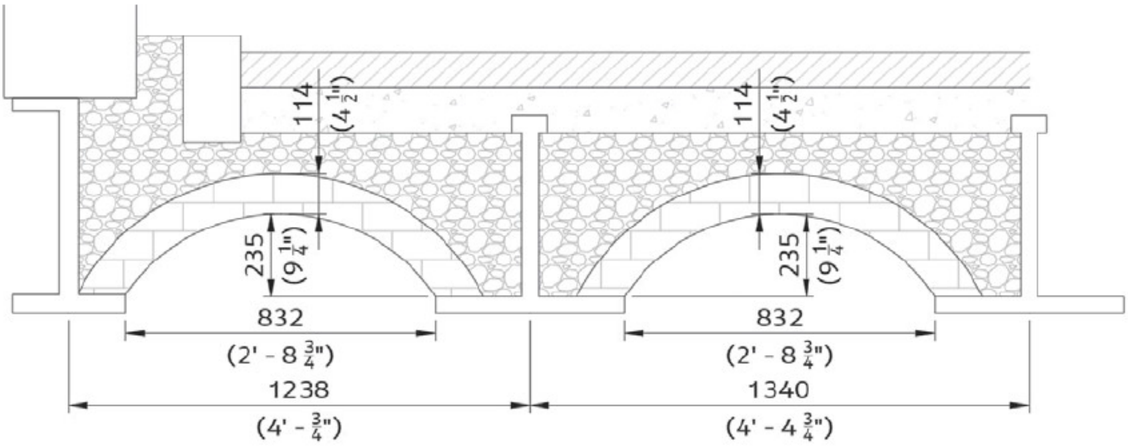
PD = 911.04 kN

Maximum imposed live loads on column = 58.83 kN
Self weight dead loads are negligible
The columns propping the edge girders are sufficient for 40/44 tonne loading

Loading arrangement for worst case on channel section columns



Jack Arch Assessment



**Review of the adequacy of the jack arches and tie-rods will be based upon the empirical method described in Bridgeguard 3 Current Information Sheet No 22 (Pro-forma for the empirical assessment of brick, masonry and concrete jack arches and associated ties.)*

SECTION 1 Check for Compliance with 40T Configuration Requirements

					Compliant Yes/No
What is the maximum span of the arch? **Non-compliant if greater than 2.0m*			0.832	m	**Yes**
Do jack arches spring from bottom flanges of beams? **If not, non-compliant*			Yes		**Yes**
What is the beam spacing?	(web - web) b	=	1.340	m	
What is the rise of the arch?	r_c	=	0.235	m	
Gross aspect ratio?	b/r_c	=	5.70		
**Non-compliant if b/r_c greater than 10*					**Yes**
What is the arch barrel thickness? (include concrete fill above)	d	=	114	mm	
How is thickness derived? From previous assessment calculations					
**Non-compliant if thickness is less than 220mm*					**No**

Jacobs					CALCULATION SHEET		
Office	York	**Page No.**	53	**Calc No.**	0451398		
Job No. & Title	VAR9/6429 HRE Assessment Programme MKT/461 CS 454 Assessment	**Calcs by**	[redacted]	**Date**	Jul-22		
Section	Jack Arches	**Checker**	[redacted]	**Date**	Sep-22		

SECTION 2 Check for Deficency

Type No.	Deficency	Pass/ Fail
1	What is the backing material? Is it structural? » Miscellaneous fill Does the structural backing extend to at least the crown level of the arch extrados? **No** **If not, fail*	Fail
	What is effective shear depth of deck? D_s = 349 mm » *arch rise* 235 mm *+ barrel thickness* 114 mm *+ height of structural fill above crown of extrados* 0 mm Is $D_s \geq$ minimum requirement Fig1. Fig1. Min = 303.2 mm **Fail if D_s < Fig 1* » *349 > 303.2*	Pass
2	Do jack arches span longitudinally or transversely between longitudinal girders? **Transversely** *For Longitunal spanning jack arches, ignore following questions on ties/lateral restraint and state N/A.* Are ties provided in edge bays of transverse spanning jack arches? **Yes** **If yes, go to 3a/3b* **If not, fail, unless edge bay is 'hard' (see 5)*	Pass
3a CI	What is the cross sectional area of one tie? A = 506.71 mm^2 *(Allowing for corrosion losses)* What is the number of ties per beam length? n = 3 What is the clear skew span? L = 9.327 m Specific area of tie (As) = ((n+1) x A) /L A_s = 217.31 mm^2/m **Non-compliant if less than 260mm^2 /m*	Fail
	What is the maximum tie spacing? S = 2.516 m **Non compliant if greater than 3.0m for wrought iron/steel*	Pass
3b WI/ ST	What is the cross sectional area of one tie? A = - mm^2 *(Allowing for coorsion losses)* What is the number of ties per beam length? n = - What is the clear skew span? L = - m Specific area of tie (As) = ((n+1) x A) /L A_s = N/A mm^2/m **Non-compliant if less than 260mm^2 /m* What is the maximum tie spacing? S = - m **Non compliant if greater than 3.0m for wrought iron/steel*	
4	Are ties located within crown of external arch? **No** **If so, then fail CI or possible fail for wrought iron/steel*	Pass
5	Does external bay construction provide alternative lateral restraint? *(ie not soft edge)* **If so, pass* **If not, are ties provided in first Jack Arch bay?* **If yes, treat as 3a (or 3b)* *Otherwise fail.*	N/A

SECTION 3 Checks for Defects

Type No.	Defect	Empirical Assessment		Pass/ Fail
		CI Decks	WI/Steel Decks	
6	Rotation of supporting beam	Fail	Fail	Pass
		No		
7	Horizontal Displacement of supporting beam	Fail	Fail	Pass
		No		
8	Inadequate support to springings eg. Corrosion of bottom flange of supporting beam over a significant length, missing bedding mortar	Possible fail	Possible Fail	Pass
		No		
9	Transversely bowed bottom flange of supporting beam	Fail	Fail	Pass
		No		
10	Cracking at crown of arch owing to spreading of springings (other than 12, 13)	Fail	Fail	Pass
		No		
11	Distortion and any associated cracking of jack arch barrel	Fail	Fail	Pass
		No		
12	Arch crack resulting in substructure crack	Fail	Fail (5)	Pass
		No		
13	Substructure crack or other distress resulting in crack to jack arch	Possible fail (3)	Possible Fail (3) (5)	Pass
		No		

Notes

(3) Substructure renovation' or 'Monitoring' as appropriate; 'Repair of arch' (if appropriate)

(5) Niot applicable in general to longitudinally spanning arches

Jack Arches - RING Assessment

As there is no structural backing over the jack arches, they are considered deficiant using the empirical method described in Bridgeguard 3 Current Information Sheet No 22. An additional capcity check of the jack arch has been undertaken using LimitState RING to assess the effect of normal traffic loading upon the jack arch.

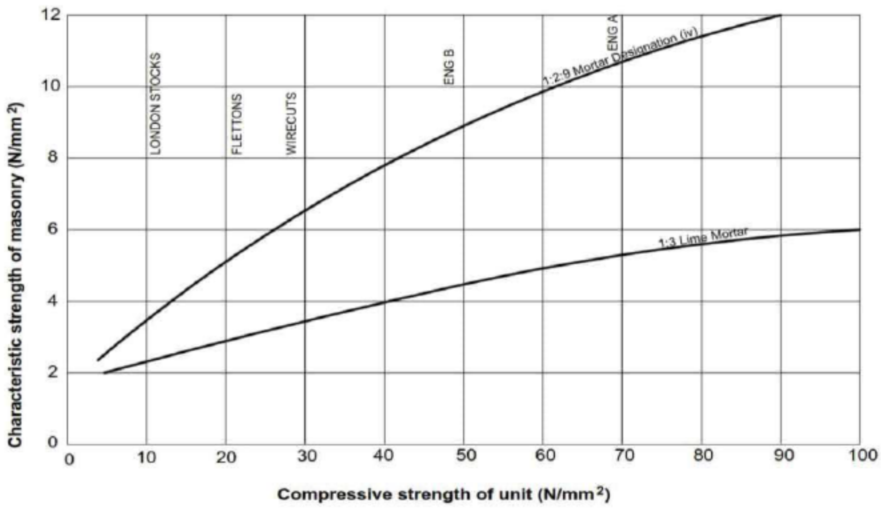
In the RING assessment, the girders have been modelled as abutments and the jack arch has been considered to be a masonry arch, following the guidence in CS 454 section 7.

Material Properties

CS 454 Fig. 4.2.7a

Assuming London Stocks:

Figure 4.2.7a Characteristic strength of brick masonry



Compressive Strength = 2.2 N/mm²

CS 454 Table 4.1.1a

Density = 2200 kg/m³ (taken as the mean value for brick unit weights)
= 21.58 kN/m³

Assume miscellaneous fill above the jack arches

CS 454 Table 4.1.1a

Density = 2200 kg/m³
= 21.58 kN/m³

Angle of Internal Friction (Φ) will be taken as 30° = 0.524 Radians

Earth Pressure Coefficient = 1.0

The earth pressure coefficient has been set as 1.0 as the limitation of the deformation is already accounted for in the capacity factor, C.

Assuming Macadam (tar) for surfacing material

CS 454 Table 4.1.1a

Density = 2400 kg/m³
= 23.54 kN/m³

Effective Width

The wheel load shall be dispersed to the crown of the jack with an angle of dispersal of 45°

The wheel load shall be considered to act over a square area 300mm x 300mm.

Fill Depth = 0.365 m
Dispersal width = 0.3 + 0.365 = 0.665 m

Jack Arch Span = 0.832 m

As the dispersal width is less than the span of the jack arch, take the effective width as the jack arch span for single wheel load.

Effective width = 0.832 m

Consider the following loads:

From CS 454 Table 5.9a The road surface condition shall be taken as poor.
Impact factor = 1.8 *applied to the most critical axle

From CS 454 Table 5.9b Traffic flow will be taken as medium
Traffic flow factor = 0.95

CS 454 Table 3.4 Cl. 3.9

Partial factor for normal traffic = 1.5
γ_{f3} = 1.0

Cl. 7.2

Capacity Factor, C_{min} = 1.2 for normal traffic

Factor for critical axle = 1.8 x 0.95 x 1.5 x 1.0 x 1.2 = 3.08
Factor for other axles = 0.95 x 1.5 x 1.0 x 1.2 = 1.71

The single axle will be the most critical for the jack arches.

CS 454 Table 7.3.1a

Axle load = 11.5 tonnes
= 112.82 kN

Factored axle load = 112.82 x 3.08
= 347.24 kN

Factored Wheel Load = 173.62 kN

This report was generated by LimitState:RING 3.2.b.20773

Summary

Details

Bridge name	Location	Reference No.	Map reference
MKT/416 Jack Arch			

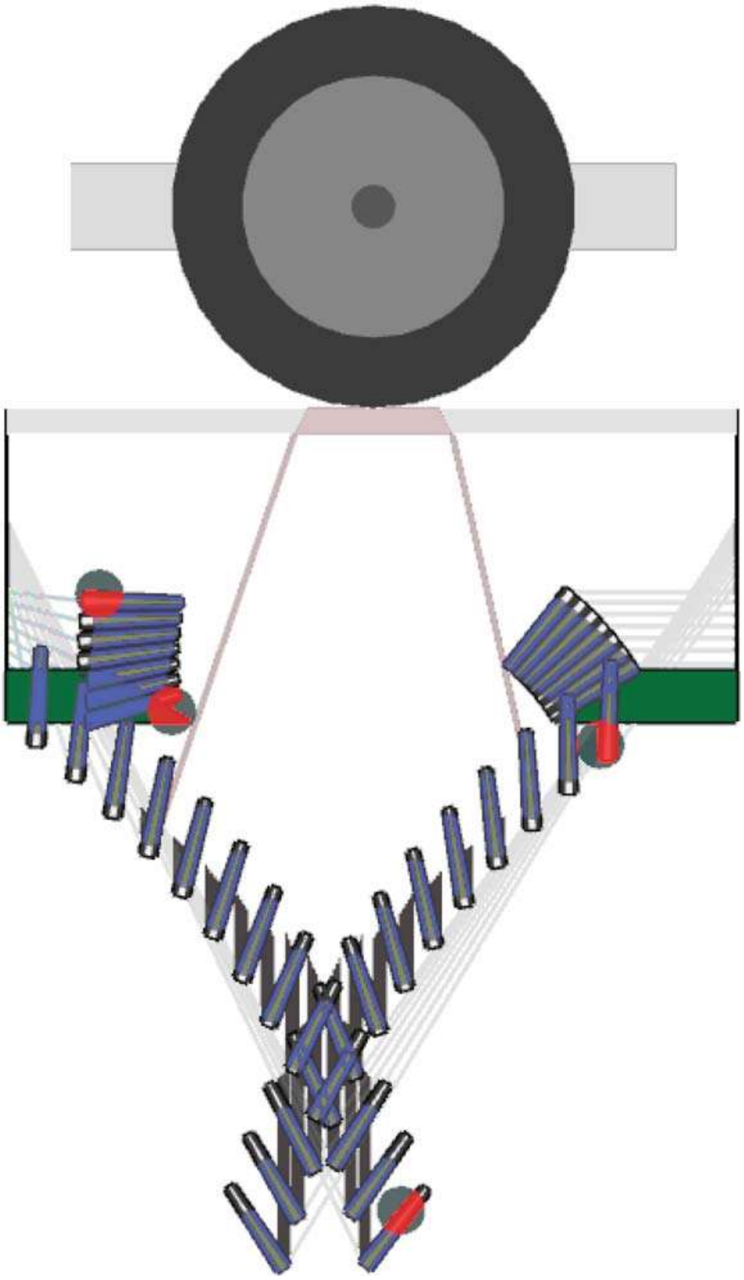
Bridge type	Name of assessor	Assessing organization	Date of assessment
Highway	[redacted]	Jacobs	Thursday, August 25, 2022

Comments

Results

Adequacy factor	Solver used (if not default)
3.63 at load case #8 (this is the critical load case)	CLP solver

Mode of Response for Current Load Case



Units

Unless specified otherwise, the following units are used throughout this report:

Distance	Force*	Moment*	Angle	Unit weight	Material strength
mm	kN	kNmm	Degrees	kN/m3	N/mm2

* = per metre width

Geometry

Global:	No. Spans	Effective bridge width
	1	1245

Span 1:	Type	Shape	No. Rings	Span	Midspan rise	Auto-calc. abutment angles?	LHS angle	RHS angle
	Multi-ring	Segmental	1	832	235	Yes	31	31
	Ring 1:	**No. Blocks**	**Ring thickness**					

		40	225

Fill Profile Properties

Distances measured from left springing point of left span.

Horizontal distance (x)	Height to surface fill (y)	Surface fill depth (d)	Surface level (y+d)
0	655	60	715

Partial Factors

Loads

Masonry unit weight	Fill unit weight	Surface unit weight	Axle load	Dynamic
1.15	1.2	1.75	1	1

Materials

Masonry strength	Masonry friction
1	1

Fill Properties

Backfill

Unit weight	**Angle of friction**	**Cohesion**
21.58	30	0
Model dispersion of live load?	**Model horizontal 'passive' pressures?**	
Yes	Yes	
Dispersion type	**Cutoff angle**	
Boussinesq	30	
Soil arch interface, friction multiplier	**Soil arch interface, cohesion multiplier**	
0.66	0.5	
Mobilisation multiplier on Kp (mp)	**Mobilisation multiplier on cohesion (mpc)**	
0.33	0.05	
Keep mp.Kp > 1?	**Auto identify passive zones?**	
Yes	Yes	

Surface Fill

Unit weight	Load dispersion limiting angle
23.54	26.6

Backing

Position	Backing height	Passive pressures modelled?
Abutment 0	0	Yes
Abutment 1	0	Yes

Vehicles in Project

Name	Axle No.	Load magnitude	Axle position

Default 1kN Single Axle	1	1	0	60
Single Wheel Load	1	173.62	0	

Vehicles in Load Cases

#	Load Case Name	Vehicle(s)	Position	Mirror?	Dynamic Axles
1	Load Case 1	Single Wheel Load	0	No	-
2	Load Case 2	Single Wheel Load	60	No	-
3	Load Case 3	Single Wheel Load	120	No	-
4	Load Case 4	Single Wheel Load	180	No	-
5	Load Case 5	Single Wheel Load	240	No	-
6	Load Case 6	Single Wheel Load	300	No	-
7	Load Case 7	Single Wheel Load	360	No	-
8	Load Case 8	Single Wheel Load	420	No	-
9	Load Case 9	Single Wheel Load	480	No	-
10	Load Case 10	Single Wheel Load	540	No	-
11	Load Case 11	Single Wheel Load	600	No	-
12	Load Case 12	Single Wheel Load	660	No	-
13	Load Case 13	Single Wheel Load	720	No	-
14	Load Case 14	Single Wheel Load	780	No	-
15	Load Case 15	Single Wheel Load	840	No	-

Load Cases

#	Load Case Name	Effective Width	Adequacy Factor
1	Load Case 1	1245	4.01
2	Load Case 2	1245	3.99
3	Load Case 3	1245	3.9
4	Load Case 4	1245	3.91
5	Load Case 5	1245	3.91
6	Load Case 6	1245	3.83
7	Load Case 7	1245	3.66
8	Load Case 8	1245	3.63
9	Load Case 9	1245	3.67
10	Load Case 10	1245	3.86
11	Load Case 11	1245	3.92
12	Load Case 12	1245	3.91
13	Load Case 13	1245	3.9
14	Load Case 14	1245	4.02
15	Load Case 15	1245	4.15

Blocks

Label	Position	Point 1	Point 2	Point 3	Point 4	Area	Unit weight	Support	Support movement X/Y/Rot.	Fill force (V)	Fill force (H)
Block 0	Skewback 0	-416/0	0/0	-192/116	-416/116	37122.77	21.58	X/Y/Rot	0/0/0	3.67	0
Block 1	Span 1, Ring 1	0/0	13/21	-173/147	-192/116	6918.06	21.58	None	0/0/0	0.32	0.49
Block 2	Span 1, Ring 1	13/21	28/41	-151/177	-173/147	6918.06	21.58	None	0/0/0	0.32	0.45
Block 3	Span 1, Ring 1	28/41	43/61	-129/205	-151/177	6918.06	21.58	None	0/0/0	0.33	0.41
Block 4	Span 1, Ring 1	43/61	60/80	-104/233	-129/205	6918.06	21.58	None	0/0/0	0.33	0.38
Block 5	Span 1, Ring 1	60/80	77/98	-79/259	-104/233	6918.06	21.58	None	0/0/0	0.33	0.34
Block 6	Span 1, Ring 1	77/98	96/114	-52/284	-79/259	6918.06	21.58	None	0/0/0	0.33	0.31
Block 7	Span 1, Ring 1	96/114	115/130	-24/307	-52/284	6918.06	21.58	None	0/0/0	0.33	0.28
Block 8	Span 1, Ring 1	115/130	135/145	5/329	-24/307	6918.06	21.58	None	0/0/0	0.33	0.25
Block 9	Span 1, Ring	135/145	156/159	35/349	5/329	6918.06	21.58	None	0/0/0	0.32	0

	1										
Block 10	Span 1, Ring 1	156/159	177/172	66/368	35/349	6918.06	21.58	None	0/0/0	0.32	0
Block 11	Span 1, Ring 1	177/172	199/184	99/385	66/368	6918.06	21.58	None	0/0/0	0.31	0
Block 12	Span 1, Ring 1	199/184	222/194	132/401	99/385	6918.06	21.58	None	0/0/0	0.31	0
Block 13	Span 1, Ring 1	222/194	245/204	166/414	132/401	6918.06	21.58	None	0/0/0	0.30	0
Block 14	Span 1, Ring 1	245/204	269/212	200/426	166/414	6918.06	21.58	None	0/0/0	0.29	0
Block 15	Span 1, Ring 1	269/212	292/219	235/437	200/426	6918.06	21.58	None	0/0/0	0.29	0
Block 16	Span 1, Ring 1	292/219	317/225	271/445	235/437	6918.06	21.58	None	0/0/0	0.29	0
Block 17	Span 1, Ring 1	317/225	341/229	307/452	271/445	6918.06	21.58	None	0/0/0	0.28	0
Block 18	Span 1, Ring 1	341/229	366/232	343/456	307/452	6918.06	21.58	None	0/0/0	0.28	0
Block 19	Span 1, Ring 1	366/232	391/234	379/459	343/456	6918.06	21.58	None	0/0/0	0.28	0
Block 20	Span 1, Ring 1	391/234	416/235	416/460	379/459	6918.06	21.58	None	0/0/0	0.28	0
Block 21	Span 1, Ring 1	416/235	441/234	453/459	416/460	6918.06	21.58	None	0/0/0	0.28	0
Block 22	Span 1, Ring 1	441/234	466/232	489/456	453/459	6918.06	21.58	None	0/0/0	0.28	0
Block 23	Span 1, Ring 1	466/232	491/229	525/452	489/456	6918.06	21.58	None	0/0/0	0.28	0
Block 24	Span 1, Ring 1	491/229	515/225	561/445	525/452	6918.06	21.58	None	0/0/0	0.28	0
Block 25	Span 1, Ring 1	515/225	540/219	597/437	561/445	6918.06	21.58	None	0/0/0	0.29	0
Block 26	Span 1, Ring 1	540/219	563/212	632/426	597/437	6918.06	21.58	None	0/0/0	0.29	0
Block 27	Span 1, Ring 1	563/212	587/204	666/414	632/426	6918.06	21.58	None	0/0/0	0.29	0
Block 28	Span 1, Ring 1	587/204	610/194	700/401	666/414	6918.06	21.58	None	0/0/0	0.30	0
Block 29	Span 1, Ring 1	610/194	633/184	733/385	700/401	6918.06	21.58	None	0/0/0	0.31	0
Block 30	Span 1, Ring 1	633/184	655/172	766/368	733/385	6918.06	21.58	None	0/0/0	0.31	0
Block 31	Span 1, Ring 1	655/172	676/159	797/349	766/368	6918.06	21.58	None	0/0/0	0.32	0
Block 32	Span 1, Ring 1	676/159	697/145	827/329	797/349	6918.06	21.58	None	0/0/0	0.32	0
Block 33	Span 1, Ring 1	697/145	717/130	856/307	827/329	6918.06	21.58	None	0/0/0	0.33	0
Block 34	Span 1, Ring 1	717/130	736/114	885/284	856/307	6918.06	21.58	None	0/0/0	0.33	0
Block 35	Span 1, Ring 1	736/114	755/98	911/259	885/284	6918.06	21.58	None	0/0/0	0.33	0.04
Block 36	Span 1, Ring 1	755/98	772/80	937/233	911/259	6918.06	21.58	None	0/0/0	0.33	0.34
Block 37	Span 1, Ring 1	772/80	789/61	961/205	937/233	6918.06	21.58	None	0/0/0	0.33	0.38
Block 38	Span 1, Ring 1	789/61	804/41	984/177	961/205	6918.06	21.58	None	0/0/0	0.33	0.41
Block 39	Span 1, Ring 1	804/41	819/21	1005/147	984/177	6918.06	21.58	None	0/0/0	0.32	0.45
Block 40	Span 1, Ring 1	819/21	832/0	1025/116	1005/147	6918.06	21.58	None	0/0/0	0.32	0.49
Block 0	Skewback 1	832/0	1248/0	1248/116	1025/116	37122.77	21.58	X/Y/Rot	0/0/0	3.67	0

Key:
X = X direction, Y = Y direction, Rot. = Rotation

Contacts

Label	Position	Point 1	Point 2	Length	Loss A	Loss B	CS	FC	Status	Inter-ring?	Normal	Shear	Moment
Contact 0	Span 1, Ring 1	-192/116	0/0	225.00	0	0	2.20	0.60	S/H/C/-	No	392.03	147.01	-9174.23
Contact 1	Span 1, Ring 1	-173/147	13/21	225.00	0	0	2.20	0.60	S/H/C/-	No	398.94	127.35	-4952.70
Contact 2	Span 1, Ring 1	-151/177	28/41	225.00	0	0	2.20	0.60	S/H/C/-	No	404.84	107.33	-1341.58

Contact 3	Span 1, Ring 1	-129/205	43/61	225.00	0	0	2.20	0.60	S/H/C/-	No	409.70	87.02	1649.19
Contact 4	Span 1, Ring 1	-104/233	60/80	225.00	0	0	2.20	0.60	S/H/C/-	No	413.52	66.47	4011.27
Contact 5	Span 1, Ring 1	-79/259	77/98	225.00	0	0	2.20	0.60	S/H/C/-	No	416.28	45.73	5738.03
Contact 6	Span 1, Ring 1	-52/284	96/114	225.00	0	0	2.20	0.60	S/H/C/-	No	417.98	24.85	6824.55
Contact 7	Span 1, Ring 1	-24/307	115/130	225.00	0	0	2.20	0.60	S/H/C/-	No	418.61	3.90	7267.59
Contact 8	Span 1, Ring 1	5/329	135/145	225.00	0	0	2.20	0.60	S/H/C/-	No	418.17	-17.07	7065.65
Contact 9	Span 1, Ring 1	35/349	156/159	225.00	0	0	2.20	0.60	S/H/C/-	No	416.47	-38.12	6237.35
Contact 10	Span 1, Ring 1	66/368	177/172	225.00	0	0	2.20	0.60	S/H/C/-	No	413.72	-59.05	4761.29
Contact 11	Span 1, Ring 1	99/385	199/184	225.00	0	0	2.20	0.60	S/H/C/-	No	407.40	-74.75	2984.29
Contact 12	Span 1, Ring 1	132/401	222/194	225.00	0	0	2.20	0.60	S/H/C/-	No	398.65	-85.58	1022.62
Contact 13	Span 1, Ring 1	166/414	245/204	225.00	0	0	2.20	0.60	S/H/C/-	No	388.94	-93.23	-1169.53
Contact 14	Span 1, Ring 1	200/426	269/212	225.00	0	0	2.20	0.60	S/H/C/-	No	378.41	-96.70	-3475.19
Contact 15	Span 1, Ring 1	235/437	292/219	225.00	0	0	2.20	0.60	S/H/C/-	No	367.38	-94.89	-5757.68
Contact 16	Span 1, Ring 1	271/445	317/225	225.00	0	0	2.20	0.60	S/H/C/-	No	356.44	-86.91	-7872.03
Contact 17	Span 1, Ring 1	307/452	341/229	225.00	0	0	2.20	0.60	S/H/C/-	No	346.44	-72.57	-9682.63
Contact 18	Span 1, Ring 1	343/456	366/232	225.00	0	0	2.20	0.60	S/H/C/-	No	338.36	-52.60	-11078.12
Contact 19	Span 1, Ring 1	379/459	391/234	225.00	0	0	2.20	0.60	S/H/C/-	No	333.08	-28.54	-11977.48
Contact 20	Span 1, Ring 1	416/460	416/235	225.00	0	0	2.20	0.60	S/H/C/-	No	331.17	-2.31	-12330.66
Contact 21	Span 1, Ring 1	453/459	441/234	225.00	0	0	2.20	0.60	S/H/C/-	No	332.85	24.07	-12118.78
Contact 22	Span 1, Ring 1	489/456	466/232	225.00	0	0	2.20	0.60	S/H/C/-	No	337.95	48.62	-11354.39
Contact 23	Span 1, Ring 1	525/452	491/229	225.00	0	0	2.20	0.60	S/H/C/-	No	345.94	69.39	-10081.13
Contact 24	Span 1, Ring 1	561/445	515/225	225.00	0	0	2.20	0.60	S/H/C/-	No	355.99	84.78	-8373.57
Contact 25	Span 1, Ring 1	597/437	540/219	225.00	0	0	2.20	0.60	S/H/C/-	No	367.11	93.88	-6337.07
Contact 26	Span 1, Ring 1	632/426	563/212	225.00	0	0	2.20	0.60	S/H/C/-	No	378.42	96.75	-4103.25
Contact 27	Span 1, Ring 1	666/414	587/204	225.00	0	0	2.20	0.60	S/H/C/-	No	389.29	94.17	-1816.74
Contact 28	Span 1, Ring 1	700/401	610/194	225.00	0	0	2.20	0.60	S/H/C/-	No	399.36	87.20	382.82
Contact 29	Span 1, Ring 1	733/385	633/184	225.00	0	0	2.20	0.60	S/H/C/-	No	408.46	76.87	2373.73
Contact 30	Span 1, Ring 1	766/368	655/172	225.00	0	0	2.20	0.60	S/H/C/-	No	416.51	63.99	4056.77
Contact 31	Span 1, Ring 1	797/349	676/159	225.00	0	0	2.20	0.60	S/H/C/-	No	419.54	42.95	5680.89
Contact 32	Span 1, Ring 1	827/329	697/145	225.00	0	0	2.20	0.60	S/H/C/-	No	421.48	21.74	6655.26
Contact 33	Span 1, Ring 1	856/307	717/130	225.00	0	0	2.20	0.60	S/H/C/-	No	422.35	0.44	6973.68
Contact 34	Span 1, Ring 1	885/284	736/114	225.00	0	0	2.20	0.60	S/H/C/-	No	422.14	-20.89	6634.52
Contact 35	Span 1, Ring 1	911/259	755/98	225.00	0	0	2.20	0.60	S/H/C/-	No	420.83	-42.17	5641.79
Contact 36	Span 1, Ring 1	937/233	772/80	225.00	0	0	2.20	0.60	S/H/C/-	No	418.25	-63.15	4020.70
Contact 37	Span 1, Ring 1	961/205	789/61	225.00	0	0	2.20	0.60	S/H/C/-	No	414.59	-83.95	1756.82
Contact 38	Span 1, Ring 1	984/177	804/41	225.00	0	0	2.20	0.60	S/H/C/-	No	409.88	-104.52	-1143.47
Contact 39	Span 1, Ring 1	1005/147	819/21	225.00	0	0	2.20	0.60	S/H/C/-	No	404.12	-124.80	-4672.10
Contact 40	Span 1, Ring 1	1025/116	832/0	225.00	0	0	2.20	0.60	S/H/C/-	No	397.34	-144.73	-8819.33

Key:
CS = Crushing Strength, FC = Friction Coefficient, S = Sliding enabled, H = Hinging enabled, C = Crushing enabled, R = Reinforcement present

This report was generated by LimitState:RING 3.2.b.20773

Summary

Details

Bridge name	Location	Reference No.	Map reference
MKT/416 Jack Arch			
Bridge type	**Name of assessor**	**Assessing organization**	**Date of assessment**
Highway	[redacted]	Jacobs	Thursday, August 25, 2022

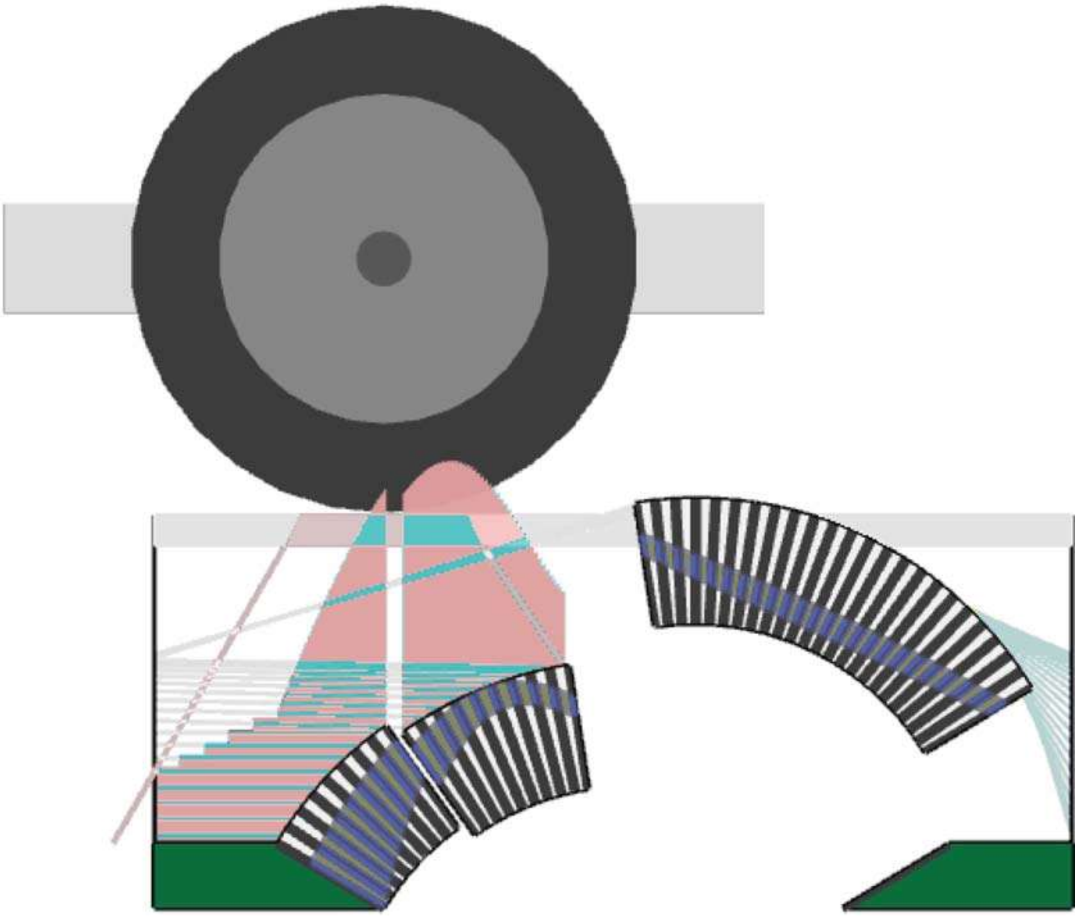
Comments

Results

Adequacy factor
3.53 at load case #1 (this is the critical load case)

Solver used (if not default)
CLP solver

Mode of Response for Current Load Case



Units

Unless specified otherwise, the following units are used throughout this report:

Distance	Force*	Moment*	Angle	Unit weight	Material strength
mm	kN	kNmm	Degrees	kN/m3	N/mm2

* = per metre width

Geometry

Global:	No. Spans	Effective bridge width
	1	1245

Span 1:	Type	Shape	No. Rings	Span	Midspan rise	Auto-calc. abutment angles?	LHS angle	RHS angle
	Multi-ring	Segmental	1	832	235	Yes	31	31

Ring 1:	No. Blocks	Ring thickness
	40	225

Fill Profile Properties

Distances measured from left springing point of left span.

Horizontal distance (x)	Height to surface fill (y)	Surface fill depth (d)	Surface level (y+d)
0	655	60	715

Partial Factors

Loads

Masonry unit weight	Fill unit weight	Surface unit weight	Axle load	Dynamic
1	1	1	1	1

Materials

Masonry strength	Masonry friction
1	1

Fill Properties

Backfill

Unit weight	Angle of friction	Cohesion
21.58	30	0
Model dispersion of live load?	**Model horizontal 'passive' pressures?**	
Yes	Yes	
Dispersion type	**Cutoff angle**	
Boussinesq	30	
Soil arch interface, friction multiplier	**Soil arch interface, cohesion multiplier**	
0.66	0.5	
Mobilisation multiplier on Kp (mp)	**Mobilisation multiplier on cohesion (mpc)**	
0.33	0.05	
Keep mp.Kp > 1?	**Auto identify passive zones?**	
Yes	Yes	

Surface Fill

Unit weight	Load dispersion limiting angle
23.54	26.6

Backing

Position	Backing height	Passive pressures modelled?
Abutment 0	0	Yes
Abutment 1	0	Yes

Vehicles in Project

Name	Axle No.	Load magnitude	Axle position
Default 1kN Single Axle	1	1	0
Single Wheel Load	1	173.62	0

Vehicles in Load Cases

#	Load Case Name	Vehicle(s)	Position	Mirror?	Dynamic Axles
1	Load Case 1	Single Wheel Load	0	No	-
2	Load Case 2	Single Wheel Load	60	No	-
3	Load Case 3	Single Wheel Load	120	No	-
4	Load Case 4	Single Wheel Load	180	No	-
5	Load Case 5	Single Wheel Load	240	No	-
6	Load Case 6	Single Wheel Load	300	No	-
7	Load Case 7	Single Wheel Load	360	No	-
8	Load Case 8	Single Wheel Load	420	No	-
9	Load Case 9	Single Wheel Load	480	No	-
10	Load Case 10	Single Wheel Load	540	No	-
11	Load Case 11	Single Wheel Load	600	No	-
12	Load Case 12	Single Wheel Load	660	No	-
13	Load Case 13	Single Wheel Load	720	No	-
14	Load Case 14	Single Wheel Load	780	No	-
15	Load Case 15	Single Wheel Load	840	No	-

Load Cases

#	Load Case Name	Effective Width	Adequacy Factor
1	Load Case 1	1245	3.53
2	Load Case 2	1245	3.93
3	Load Case 3	1245	3.87
4	Load Case 4	1245	3.9
5	Load Case 5	1245	3.92
6	Load Case 6	1245	3.84
7	Load Case 7	1245	3.67
8	Load Case 8	1245	3.64
9	Load Case 9	1245	3.68
10	Load Case 10	1245	3.87
11	Load Case 11	1245	3.92
12	Load Case 12	1245	3.89
13	Load Case 13	1245	3.87
14	Load Case 14	1245	3.95
15	Load Case 15	1245	3.66

Blocks

Label	Position	Point 1	Point 2	Point 3	Point 4	Area	Unit weight	Support	Support movement X/Y/Rot.	Fill force (V)	Fill force (H)
Block 0	Skewback 0	-416/0	0/0	-192/116	-416/116	37122.77	21.58	X/Y/Rot	0/0/0	2.91	0
Block 1	Span 1, Ring 1	0/0	13/21	-173/147	-192/116	6918.06	21.58	None	0/0/0	0.25	0.39
Block 2	Span 1, Ring 1	13/21	28/41	-151/177	-173/147	6918.06	21.58	None	0/0/0	0.26	0.36
Block 3	Span 1, Ring 1	28/41	43/61	-129/205	-151/177	6918.06	21.58	None	0/0/0	0.26	0.33
Block 4	Span 1, Ring 1	43/61	60/80	-104/233	-129/205	6918.06	21.58	None	0/0/0	0.26	0.30
Block 5	Span 1, Ring 1	60/80	77/98	-79/259	-104/233	6918.06	21.58	None	0/0/0	0.26	0.27
Block 6	Span 1, Ring 1	77/98	96/114	-52/284	-79/259	6918.06	21.58	None	0/0/0	0.26	0.24
Block 7	Span 1, Ring 1	96/114	115/130	-24/307	-52/284	6918.06	21.58	None	0/0/0	0.26	0.21
Block 8	Span 1, Ring 1	115/130	135/145	5/329	-24/307	6918.06	21.58	None	0/0/0	0.25	0.19
Block 9	Span 1, Ring 1	135/145	156/159	35/349	5/329	6918.06	21.58	None	0/0/0	0.25	0
Block 10	Span 1, Ring 1	156/159	177/172	66/368	35/349	6918.06	21.58	None	0/0/0	0.24	0
Block 11	Span 1, Ring 1	177/172	199/184	99/385	66/368	6918.06	21.58	None	0/0/0	0.24	0
Block 12	Span 1, Ring 1	199/184	222/194	132/401	99/385	6918.06	21.58	None	0/0/0	0.23	0
Block 13	Span 1, Ring 1	222/194	245/204	166/414	132/401	6918.06	21.58	None	0/0/0	0.23	0
Block 14	Span 1, Ring 1	245/204	269/212	200/426	166/414	6918.06	21.58	None	0/0/0	0.22	0
Block 15	Span 1, Ring 1	269/212	292/219	235/437	200/426	6918.06	21.58	None	0/0/0	0.22	0
Block 16	Span 1, Ring 1	292/219	317/225	271/445	235/437	6918.06	21.58	None	0/0/0	0.21	0
Block 17	Span 1, Ring 1	317/225	341/229	307/452	271/445	6918.06	21.58	None	0/0/0	0.21	0
Block 18	Span 1, Ring 1	341/229	366/232	343/456	307/452	6918.06	21.58	None	0/0/0	0.21	0
Block 19	Span 1, Ring 1	366/232	391/234	379/459	343/456	6918.06	21.58	None	0/0/0	0.21	0
Block 20	Span 1, Ring 1	391/234	416/235	416/460	379/459	6918.06	21.58	None	0/0/0	0.21	0
Block 21	Span 1, Ring 1	416/235	441/234	453/459	416/460	6918.06	21.58	None	0/0/0	0.21	0.01
Block 22	Span 1, Ring 1	441/234	466/232	489/456	453/459	6918.06	21.58	None	0/0/0	0.21	0.02
Block 23	Span 1, Ring 1	466/232	491/229	525/452	489/456	6918.06	21.58	None	0/0/0	0.21	0.03
Block 24	Span 1, Ring 1	491/229	515/225	561/445	525/452	6918.06	21.58	None	0/0/0	0.21	0.04
Block 25	Span 1, Ring 1	515/225	540/219	597/437	561/445	6918.06	21.58	None	0/0/0	0.21	0.05
Block 26	Span 1, Ring 1	540/219	563/212	632/426	597/437	6918.06	21.58	None	0/0/0	0.22	0.06
Block 27	Span 1, Ring 1	563/212	587/204	666/414	632/426	6918.06	21.58	None	0/0/0	0.22	0.08
Block 28	Span 1, Ring 1	587/204	610/194	700/401	666/414	6918.06	21.58	None	0/0/0	0.23	0.09
Block 29	Span 1, Ring 1	610/194	633/184	733/385	700/401	6918.06	21.58	None	0/0/0	0.23	0.11
Block 30	Span 1, Ring 1	633/184	655/172	766/368	733/385	6918.06	21.58	None	0/0/0	0.24	0.13
Block 31	Span 1, Ring 1	655/172	676/159	797/349	766/368	6918.06	21.58	None	0/0/0	0.24	0.15
Block 32	Span 1, Ring 1	676/159	697/145	827/329	797/349	6918.06	21.58	None	0/0/0	0.25	0.17
Block 33	Span 1, Ring 1	697/145	717/130	856/307	827/329	6918.06	21.58	None	0/0/0	0.25	0.19
Block 34	Span 1, Ring 1	717/130	736/114	885/284	856/307	6918.06	21.58	None	0/0/0	0.26	0.21
Block 35	Span 1, Ring 1	736/114	755/98	911/259	885/284	6918.06	21.58	None	0/0/0	0.26	0.24
Block 36	Span 1, Ring 1	755/98	772/80	937/233	911/259	6918.06	21.58	None	0/0/0	0.26	0.27
Block 37	Span 1, Ring 1	772/80	789/61	961/205	937/233	6918.06	21.58	None	0/0/0	0.26	0.30
Block 38	Span 1, Ring 1	789/61	804/41	984/177	961/205	6918.06	21.58	None	0/0/0	0.26	0.33
Block 39	Span 1, Ring 1	804/41	819/21	1005/147	984/177	6918.06	21.58	None	0/0/0	0.26	0.36

Block 40	Span 1, Ring 1	819/21	832/0	1025/116	1005/147	6918.06	21.58	None	0/0/0	0.25	0.39
Block 0	Skewback 1	832/0	1248/0	1248/116	1025/116	37122.77	21.58	X/Y/Rot	0/0/0	2.91	0

Key:
X = X direction, Y = Y direction, Rot. = Rotation

Contacts

Label	Position	Point 1	Point 2	Length	Loss A	Loss B	CS	FC	Status	Inter-ring?	Normal	Shear	Moment
Contact 0	Span 1, Ring 1	-192/116	0/0	225.00	0	0	2.20	0.60	S/H/C/-	No	336.94	-107.05	11881.63
Contact 1	Span 1, Ring 1	-173/147	13/21	225.00	0	0	2.20	0.60	S/H/C/-	No	321.95	-117.64	9443.17
Contact 2	Span 1, Ring 1	-151/177	28/41	225.00	0	0	2.20	0.60	S/H/C/-	No	305.34	-125.96	6839.49
Contact 3	Span 1, Ring 1	-129/205	43/61	225.00	0	0	2.20	0.60	S/H/C/-	No	287.20	-131.64	4152.10
Contact 4	Span 1, Ring 1	-104/233	60/80	225.00	0	0	2.20	0.60	S/H/C/-	No	267.65	-134.31	1471.19
Contact 5	Span 1, Ring 1	-79/259	77/98	225.00	0	0	2.20	0.60	S/H/C/-	No	246.85	-133.58	-1106.52
Contact 6	Span 1, Ring 1	-52/284	96/114	225.00	0	0	2.20	0.60	S/H/C/-	No	225.10	-129.12	-3481.72
Contact 7	Span 1, Ring 1	-24/307	115/130	225.00	0	0	2.20	0.60	S/H/C/-	No	202.78	-120.70	-5558.23
Contact 8	Span 1, Ring 1	5/329	135/145	225.00	0	0	2.20	0.60	S/H/C/-	No	180.44	-108.27	-7250.27
Contact 9	Span 1, Ring 1	35/349	156/159	225.00	0	0	2.20	0.60	S/H/C/-	No	158.61	-92.15	-8475.76
Contact 10	Span 1, Ring 1	66/368	177/172	225.00	0	0	2.20	0.60	S/H/C/-	No	138.19	-72.79	-9209.49
Contact 11	Span 1, Ring 1	99/385	199/184	225.00	0	0	2.20	0.60	S/H/C/-	No	119.97	-51.14	-9437.08
Contact 12	Span 1, Ring 1	132/401	222/194	225.00	0	0	2.20	0.60	S/H/C/-	No	104.65	-28.52	-9176.82
Contact 13	Span 1, Ring 1	166/414	245/204	225.00	0	0	2.20	0.60	S/H/C/-	No	92.75	-6.49	-8479.17
Contact 14	Span 1, Ring 1	200/426	269/212	225.00	0	0	2.20	0.60	S/H/C/-	No	84.47	13.29	-7420.14
Contact 15	Span 1, Ring 1	235/437	292/219	225.00	0	0	2.20	0.60	S/H/C/-	No	79.64	29.49	-6087.34
Contact 16	Span 1, Ring 1	271/445	317/225	225.00	0	0	2.20	0.60	S/H/C/-	No	77.69	41.44	-4563.61
Contact 17	Span 1, Ring 1	307/452	341/229	225.00	0	0	2.20	0.60	S/H/C/-	No	78.24	46.94	-2968.33
Contact 18	Span 1, Ring 1	343/456	366/232	225.00	0	0	2.20	0.60	S/H/C/-	No	80.51	43.21	-1578.28
Contact 19	Span 1, Ring 1	379/459	391/234	225.00	0	0	2.20	0.60	S/H/C/-	No	82.61	39.37	-305.91
Contact 20	Span 1, Ring 1	416/460	416/235	225.00	0	0	2.20	0.60	S/H/C/-	No	84.52	35.43	845.50
Contact 21	Span 1, Ring 1	453/459	441/234	225.00	0	0	2.20	0.60	S/H/C/-	No	86.24	31.40	1873.52
Contact 22	Span 1, Ring 1	489/456	466/232	225.00	0	0	2.20	0.60	S/H/C/-	No	87.76	27.28	2776.03
Contact 23	Span 1, Ring 1	525/452	491/229	225.00	0	0	2.20	0.60	S/H/C/-	No	89.08	23.09	3550.69
Contact 24	Span 1, Ring 1	561/445	515/225	225.00	0	0	2.20	0.60	S/H/C/-	No	90.18	18.84	4195.50
Contact 25	Span 1, Ring 1	597/437	540/219	225.00	0	0	2.20	0.60	S/H/C/-	No	91.07	14.54	4708.82
Contact 26	Span 1, Ring 1	632/426	563/212	225.00	0	0	2.20	0.60	S/H/C/-	No	91.75	10.21	5089.41
Contact 27	Span 1, Ring 1	666/414	587/204	225.00	0	0	2.20	0.60	S/H/C/-	No	92.22	5.86	5336.36
Contact 28	Span 1, Ring 1	700/401	610/194	225.00	0	0	2.20	0.60	S/H/C/-	No	92.46	1.50	5449.16
Contact 29	Span 1, Ring 1	733/385	633/184	225.00	0	0	2.20	0.60	S/H/C/-	No	92.49	-2.87	5427.66
Contact 30	Span 1, Ring 1	766/368	655/172	225.00	0	0	2.20	0.60	S/H/C/-	No	92.30	-7.22	5272.10
Contact 31	Span 1, Ring 1	797/349	676/159	225.00	0	0	2.20	0.60	S/H/C/-	No	91.89	-11.54	4983.09
Contact 32	Span 1, Ring 1	827/329	697/145	225.00	0	0	2.20	0.60	S/H/C/-	No	91.27	-15.82	4561.59
Contact 33	Span 1, Ring 1	856/307	717/130	225.00	0	0	2.20	0.60	S/H/C/-	No	90.44	-20.06	4008.97

Contact 34	Span 1, Ring 1	885/284	736/114	225.00	0	0	2.20	0.60	S/H/C/-	No	89.40	-24.23	3326.91
Contact 35	Span 1, Ring 1	911/259	755/98	225.00	0	0	2.20	0.60	S/H/C/-	No	88.15	-28.34	2517.50
Contact 36	Span 1, Ring 1	937/233	772/80	225.00	0	0	2.20	0.60	S/H/C/-	No	86.69	-32.35	1583.15
Contact 37	Span 1, Ring 1	961/205	789/61	225.00	0	0	2.20	0.60	S/H/C/-	No	85.04	-36.28	526.62
Contact 38	Span 1, Ring 1	984/177	804/41	225.00	0	0	2.20	0.60	S/H/C/-	No	83.19	-40.09	-648.97
Contact 39	Span 1, Ring 1	1005/147	819/21	225.00	0	0	2.20	0.60	S/H/C/-	No	81.16	-43.79	-1940.20
Contact 40	Span 1, Ring 1	1025/116	832/0	225.00	0	0	2.20	0.60	S/H/C/-	No	78.94	-47.36	-3343.31

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