

SMB/2062 CS454 Assessment and Inspection Report

Braughing, Puckeridge, East Hertfordshire

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National Highways – Historical Railways Estate
VAR9/7070

HRE Assessment Programme



SMB/2062 CS454 Assessment and Inspection Report

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Executive summary

Structure Type: Single span overbridge

Superstructure Form: Double joist compound girders with jack arches

Substructure Form: Brick wingwalls, abutments and parapets

Span: 9.14m (30' – 0")

Assessment Code: CS454

Live Load Capacity:

- Internal Girder – Normal traffic loading
- South Edge Girder – Dead Loads only
- North Edge Girder – 40/44 Tonnes Accidental Vehicle Loading

Capacity Critical Element: South Edge Girders (Midspan bending)

Capacity Factor: 0.98

Condition: Poor to Fair Condition

Local Authority: Hertfordshire

OS Reference: TL 390 241

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 span 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 the site visits described.

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:

- Topside of the internal and edge girders were not exposed.
- No investigation was made of the internal condition of the pseudo-box girders.
- The extrados of the Jack arches were not exposed.

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1. General Description and Structural Details

1.1 Introduction

Jacobs was appointed by National Highways – Historical Railways Estate (HRE) to undertake a CS454 assessment of structure SMB/2062.

1.2 Location and General Description

Structure SMB/2062 carries the B1368 over the trackbed of the former Buntingford branch line part of the Great Eastern Railway. The structure is located 10.5km west of Hockerill. The OS Grid reference is TL 390 241.

The B1368 has a carriageway width of 5.97m over the structure. A footpath runs along the north verge and there is a grass verges on the southern side. The north footpath has a width of 1.38m, the south verge has a width of 0.79m. The overall width between the parapets is 8.13m.

The Buntingford branch line was first opened in 1863, it is likely that a structure was originally constructed around that time, but the present bridge probably dates from the 1920s.

1.3 Construction Type

The structure is a heavily skewed (40°) single-span bridge that comprises six longitudinally spanning double joist compound girders with brick jack arches spanning between them. The skew span of the structure is 9.14m. The internal and edge girders are of a similar construction. The wingwalls, parapets, and abutment are constructed from brindle brick.

Edge/Internal Girders

The double joist compound girders consist of two RSJs which have been riveted together with a top and bottom flange plate. The RSJ dimensions are consistent with the sections properties for a "BSB 135 18" x 7" x 75 lb per foot" girder. The dimension of "BSB 135 18" x 7" x 75 lb per foot" girder are as follows. The web depth of 411mm [$16\frac{1}{4}$ "] and a web thickness of 14mm [$\frac{9}{16}$ "]. The width of the top and bottom flange is 178mm [7"] and the thickness is 23mm [$\frac{7}{8}$ "]. The centre-to-centre spacing between the RSJs is 216mm [$8\frac{1}{2}$ "].

The flange plates are 470mm [$18\frac{1}{2}$ "] wide and have a thickness of 11.4mm [$\frac{7}{16}$ "]. The rivet head diameter is 33.6mm and has a 339mm spacing and 102mm [4"] pitch.

The external RSJ were stamped with a logo for 'CARGO-FLEET-ENGLAND'; the girders likely to have originated from 'Cargo Fleet Iron Co', which was operational between 1883 and 1944.

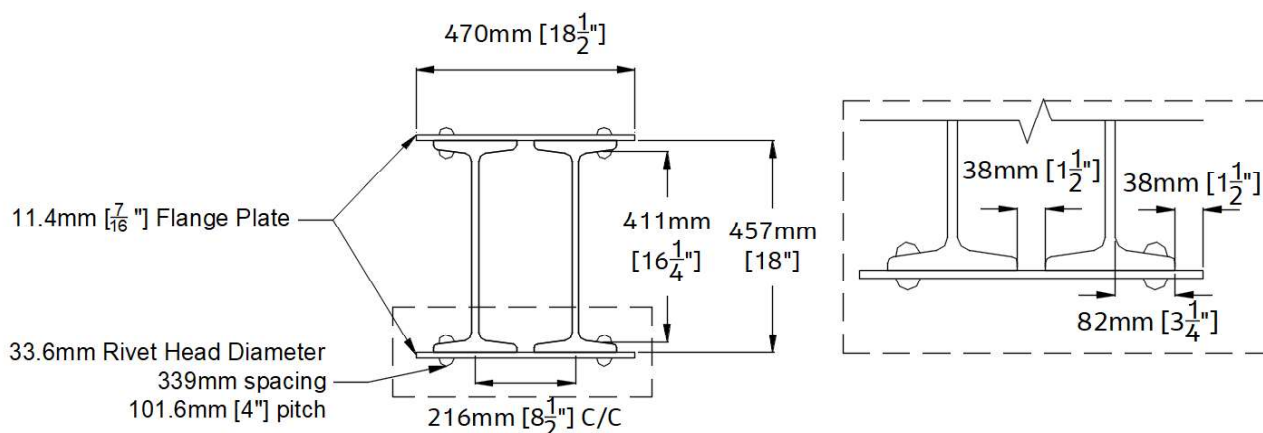


Figure 1. Double joist compound dimensions

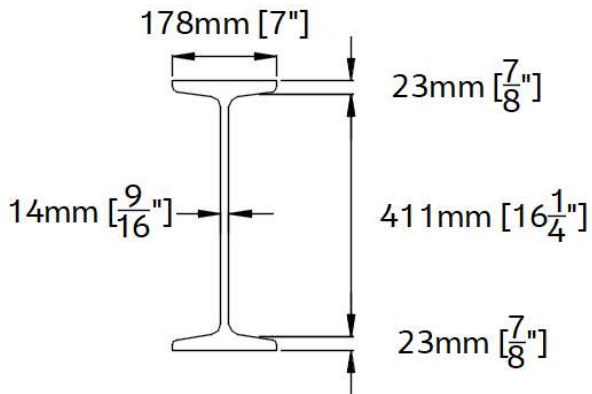


Figure 2. RSJ Dimensions

Jack Arches

The jack arches are constructed from brindle brick. During the previous assessment of the structure, a trial pit confirmed that the jack arches were covered in a concrete infill level to the top of the longitudinal compound girders. The average jack arch rise is 211mm and the depth of the backing is approximately 443mm. The jack arches have an average span of 1.235m.

Tie Bars

The structure was retrofitted with six pairs of tie bars (twelve ties in total) in 2008. The tie bars are 32mm in diameter and are spaced at 2.98m (approximately 3.0m) intervals.

2. Information Search

2.1 Services Search

No service searches were undertaken.

2.2 Site Investigation Description and Results

A survey of the structure was completed on the 4th April 2023.

2.3 Existing Drawings

Refer to Appendix D for a copy of these drawings.

3. Structure Condition

3.1 General

Jacobs undertook an inspection survey of the structure on the 4th April 2023 to inform the CS454 assessment. Weather conditions were sunny with average temperature was 12°C. Parking was available on the west verge north of the structure and access to the solum was obtained through the land to the north surrounding the old station building.

3.2 Structure Condition

3.2.1 Edge Girders

South Edge Girder

Midspan

At midspan, the girder has sustained a severe amount of corrosion; at a minimum, the exposed external faces have at least 2mm of section loss.

On the external RSJ, total section loss of the web has occurred from the base of the web extending up to a 60mm depth over a 1.75m length. Additionally, to accommodate new tie bars, the bottom flange of the external RSJ has been cut/removed over a 50mm width.

The bottom flange plate has sustained total section loss over a width of 225mm (including the cut-out made to accommodate the retrofitted tie bars). The remainder of the bottom flange plate has a residual thickness of 5.4mm over a 245mm width.

The section loss that has occurred around the bottom flange of the external RSJ has separated the bottom flange from the rest of the double joist compound girder and is rendering it ineffective in supporting loading.

Calcite deposits obscure the underside of the top flange plate and the top flange of the external girder; both are assumed to have sustained up to 2mm of corrosion, similar as seen elsewhere on the girder. There was also 1mm of pitting and corrosion to the internal faces of the double joist compound girder.

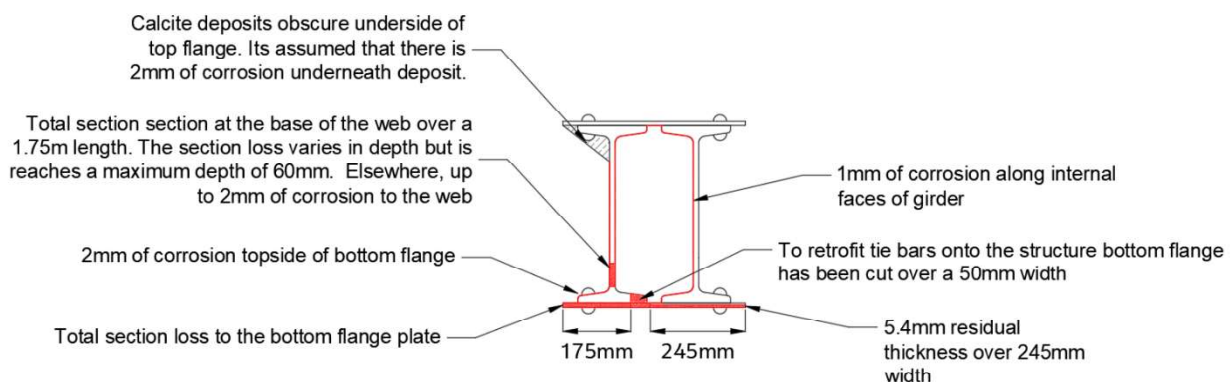


Figure 3. South Edge Girder – Midspan corrosion

Support

At support, there is total section loss of the bottom flange plate over a width of 38mm. The underside of the top flange plate has a 6.4mm residual thickness over a 38mm width. Elsewhere there is 1mm of corrosion across the exposed faces of the girder.

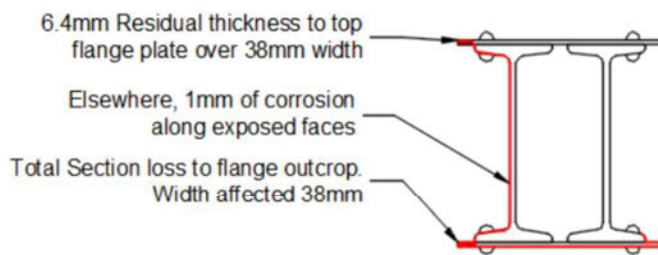


Figure 4. South Edge Girder – Support corrosion

North Edge Girder

The northern edge girder exhibits a similar pattern of corrosion as the southern edge girder.

Midspan

On the external RSJ, the web has become severely corroded, and the corrosion is concentrated toward the base of the web. UTG measurements indicate that the web thickness has reduced to a residual thickness of 7mm from the mid-depth of the girder to the base. There is total section loss of the bottom flange plate over a width of 38mm. Elsewhere, the girder has sustained up to 2mm of corrosion on the exposed faces of the girder. There was also 1mm of pitting and corrosion to the internal faces of the double joist compound girder.

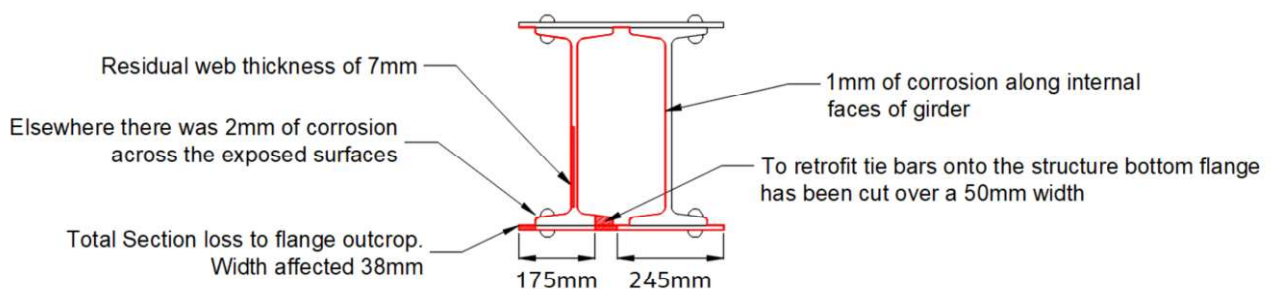


Figure 5. North Edge Girder – Midspan corrosion

Support

At support, there is total section loss of the bottom flange plate over a width of 38mm. Elsewhere there is 1mm of corrosion across the exposed faces of the girder.

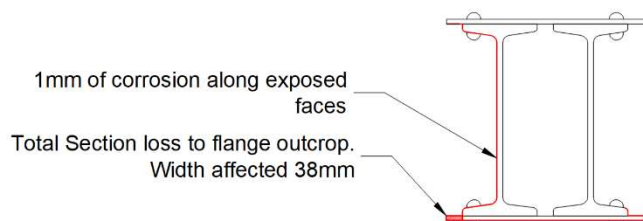


Figure 6. North Edge Girder – Support corrosion

3.2.2 Internal Girders

Internal Girders

All of the internal girders were in a similar condition. At midspan, total rivet head loss has caused the bottom flange plate to become detached from the steel RSJs at the approximate location of the blast-line. Where the plate has detached, it has slightly sagged rendering it ineffective.

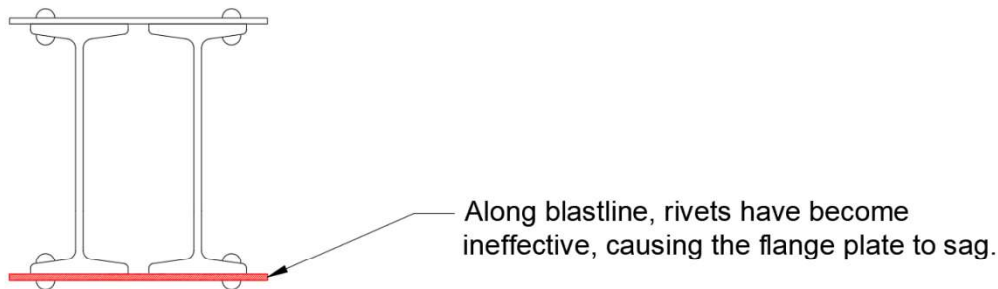


Figure 7. Internal Girders – Midspan corrosion

At the support, only paint loss has occurred to the underside of the bottom flange plate.

3.2.3 Jack Arches

All the jack arches are similar. They are in fair condition, with a 1m band of damp along the midspan of the arches.

3.2.4 Abutments

West abutment

There are a series of three repointed vertical fractures connecting through the full height of the abutment below the bedstone for the second internal girder from the south. Three mortar tabs dated 5/68 were placed, and all appear to have broken and then been repointed at a later date. The repointed mortar has remained intact. There is also another repointed vertical fracture beginning below the north edge girder. The fracture begins 2m from ground level, extending vertically into the bedstone mortar joint and then continuing to the jack arch. A mortar tab dated 58 was installed but has broken. Additionally, a strain gauge has been placed, although one half has broken off.

East abutment

There is a full-height repointed vertical fracture underneath the bedstone for the second internal girder from the north. There is a broken mortar tab dated 5/68 on the fracture. At the broken mortar tab, the fracture is 1mm wide. At the quarter points of the wall, there is damp and algae staining around the bedstones.

3.2.5 Wingwalls

Northwest Wingwall

There is a repointed diagonal fracture extending from the bearing stone, passing through the buttress and connecting onto the wingwall. Along the fracture, there is up to 10mm displacement of the brickwork around the bearing stone and there are isolated areas of spalling in brickwork adjacent to the fracture.

The buttress has spalled up to 15mm along its base at ground level

Northeast Wingwall

There is a full height vertical fracture, which has been previously repointed, begins at the interface between the wingwall and buttress before stepping through the buttress and string course before halting at the interface between the buttress and parapet walls. Along the repointing mortar, a new 1mm fracture appears to have

developed, and there is some mortar loss in the repointing. The new 1mm fracture has also stepped through a single brick in an isolated area of reskinned brickwork.

Southwest Wingwall

There is a stepped fracture on the buttress which has been filled. Joining the step fracture there are two hairline fractures. The Pilaster has been displaced outward by 5 to 10mm at the stringcourse

Southeast Wingwall

There is a full height 10mm vertical fracture at the centre which has been previously filled. Along the fracture, the material used in the repointing of the fracture appears to be loose and spalled away in isolated areas. There is a broken mortar tab on the fracture, dated 5/68.

On the buttress, the bearing stones have multiple fractures and channel reinforcement has been added.

3.2.6 Parapets

North Parapet

On the internal face of the parapet wall, there is a series of four full-height hairline vertical fractures spread along the wall. These fractures are located at the interface between the northwest pilaster, a 4m offset from the northwest pilaster, a 4m offset from the northeast pilaster and at the interface at the northeast pilaster.

There is also an additional hairline fracture, which extends through three bricks at a 1m offset from the northwest pilaster.

South Parapet

At the interface between the parapet wall and southwest pilaster, there is a full-height vertical fracture, which appears to have been filled with a polyethylene filler.

At the interface between the parapet wall and southeast pilaster, there is a full-height 3mm vertical fracture. Along the fracture, there is a broken mortar tab. There is also 2mm vertical fracture extending over a 1m length; the fracture is located 2m east of the Southeast pilaster. One the fracture there is a broken mortar tab.

3.2.7 Formation

The formation to the north west joins onto a landowner property which housed a former station for the railway line.

The land to the southeast is used for agriculture

3.2.8 Road Surface and Traffic Flow

The carriageway appears to be in a good condition. It cannot be guaranteed that the 'good' road condition will be maintained by the local highway authority, and it may degrade over time. As a conservative approach to the assessment, the road condition will be considered 'poor'.

During the site visit, HGV were observed crossing the structure, and the road accommodates a local bus route. In 2008, a traffic census was taken adjacent to the structure. At the time of the recording, the annual average daily flow rate for HGVs was 111, equating to an Annual average hourly HGV flow (AAHHGVF) of 4.6. It is unlikely that the AAHHGVF will have increased by 52% to be above the 7.0 threshold for a 'medium' traffic flow. Therefore, the traffic flow will be assumed to be 'low', for the purpose of the assessment.

4. Assessment to CS 454

4.1 Methodology

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

The girders are constructed from steel. As evidence suggests the bridge was constructed post BS 15:1905, this is assumed to be an early steel with a yield strength, F_y of 230 N/mm² as indicated in the Network Rail NR/GN/CIV/025 Table A2.

The internal girders were checked for normal traffic loading using ALL model 2 (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.

The edge girders were checked for accidental vehicle loads utilising ALL model 1 applied by simple statics.

Determination of the adequacy of the jack arches was 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 was assessed qualitatively.

4.2 Results

Element: South Edge Girders

Action	Location	DL & SDL Effects	Live Loads	Total Load Effect	Assessed Resistance	Live Load Capacity
Bending Moment – 44/40 tonnes AVL	Midspan	378.69 kNm	326.86 kNm	705.55 kNm	403.54 kNm	Failed for 40/44 Tonnes AVL
Bending Moment – 3 tonnes AVL	Midspan	378.69 kNm	49.33 kNm	428.02 kNm	403.54 kNm	Failed for 3 Tonnes AVL
Shear Force - 44/40 tonnes AVL	Support	161.49 kN	149.62 kN	311.11 kN	661.54 kN	Pass for 40/44 Tonnes AVL

The southern edge girders are not able to support the midspan bending moments induced by any form of accidental vehicle loading. After accounting for the dead loads, there is only a residual moment capacity of 24.85 kNm to support live loading.

Element: North Edge Girders

Action	Location	DL & SDL Effects	Live Loads	Total Load Effect	Assessed Resistance	Live Load Capacity
Bending Moment – 44/40 tonnes AVL	Midspan	374.49 kNm	306.13 kNm	680.61 kNm	1099.45 kNm	Passed for 40/44 Tonnes AVL
Shear Force - 44/40 tonnes AVL	Support	161.49 kN	149.62 kN	311.11 kN	661.54 kN	Pass for 44/40 Tonnes AVL

Element: Internal Girders

Action	Location	DL & SDL Effects	Live Loads	Total Load Effect	Assessed Resistance	Live Load Capacity	Capacity Factor 1,2	K Factor 2
Bending Moment – Normal Traffic Loading	Midspan	390.79 kNm	614.61 kNm	1005.40 kNm	1074.21 kNm	Marginal Pass for Normal Traffic Loading	0.98	0.88
Shear Force - Normal Traffic Loading	Support	166.65 kN	224.12 kN	390.77 kN	661.54 kN	Pass for Normal Traffic Loading	1.94	0.88

1. Capacity Factor = (Assessed Resistance – DL & SDL Effect) / (ALL2 Live Load Effect / 'K' factor)
2. Capacity and K Factors are included to allow comparison with BD21

4.2.1 Jack Arches

The jack arches pass for 40 tonnes of traffic loading based upon the empirical method as described in Bridgeguard 3 Current Information Sheet No. 22. These results should be taken with the caveat that CIS No.22 does not account for the conditions of the tie bar connection to the edge girder. The south edge girder has sustained a severe amount of corrosion; the continuation of this corrosion may compromise the integrity of the ties' anchorage, potentially rendering the ties ineffective.

4.2.2 Substructure

On the east and west abutment, a full-height vertical fracture was observed connecting with the bearing stone below the second internal girder from the north and the third internal girder from the north, respectively. The central girders are expected to experience the greatest amount of live loading, and the fractures may have developed from the cyclical loading from passing vehicles. The fractures have been repointed, making it difficult to determine if the fractures have been continuously widening. Additionally, all of the broken mortar tabs placed were at least 55 years old at the time of the inspection, further adding to the ambiguity of whether there has been continuing movement within the wall. A new set of strain gauges and/or mortar tabs should be installed on the fractures, and if there is no evidence of movement, the abutments are considered suitable by qualitative assessment for vehicles conforming to Normal Traffic Loading as defined by CS454.

The northwest wingwall has a potential separation fracture between the buttress and wingwalls. Along the fracture, there is up to 15mm of displacement and spalling. On the northeast wingwall, there is also a potential separation fracture between the buttress and wingwalls and the repointed fracture appears to have widened by up to 1mm. Vertical fractures were also present in the southwest and southeast wingwalls. A new set of strain gauges and/or mortar tabs should be installed on the fractures, and if there is no evidence of movement, the wingwalls are considered suitable by qualitative assessment for vehicles conforming to Normal Traffic Loading (40/44 tonnes) as defined by CS454.

5. Conclusions and Recommendations

While the internal girders supporting the carriageway do pass for 'normal traffic' loading as defined by CS454, the pass was only marginal, and if additional section loss were to occur to the midspan section, the consequence would be restricted loading.

The southern edge girders do not have the capacity to sustain any form of live loading. It is recommended that a road restraint barrier be installed, as soon as is practical, to protect the southern edge girder from any accidental vehicle loading.

The north edge girders passes for 40/44 tonnes accidental vehicle loading. The condition of the north girder should also be monitored as it displayed a similar pattern of section loss to that seen in the south edge girder. If section loss develops in the web of the external RSJ, it may become ineffective, reducing the bending resistance to approximately match the south girder's section.

The jack arches pass for 40 tonnes of traffic loading based upon the empirical method as described in Bridgeguard 3 Current Information Sheet No. 22. To ensure the tie bars remain effective, the ties' connections to the edge girders should be monitored as the continuation of corrosion to the bottom flange of the RSJs in these areas may compromise the integrity of the ties' anchorage.

On the substructure, there are several vertical fractures, which may indicate that there has been continuous movement within the walls of the structure. The fractures within the structure appear to have a long history as there were a number of broken mortar tabs which were dated between 1958 to 1968. As the mortar tabs have not been renewed, it was impossible to determine whether the movement within the wall is still occurring or whether the structure has 'settled'. A new set of strain gauges and/or mortar tabs should be installed on the fractures, and if there is no evidence of movement, the substructures are considered suitable by qualitative assessment for vehicles conforming to Normal Traffic Loading (40/44 tonnes) as defined by CS454.

Due to the arrangement of the double joist compound girder, replacing the external RSJ to repair the unit does not appear practical. There may be an option to replace the entire edge girder unit with a new girder but any modification to the edge girder will also affect the construction of the retrofitted tie bars and jack arches.

Appendix A. Photographs



Photograph 1. North Elevation



Photograph 2. South Elevation



Photograph 3. Southwest approach



Photograph 4. Northeast approach



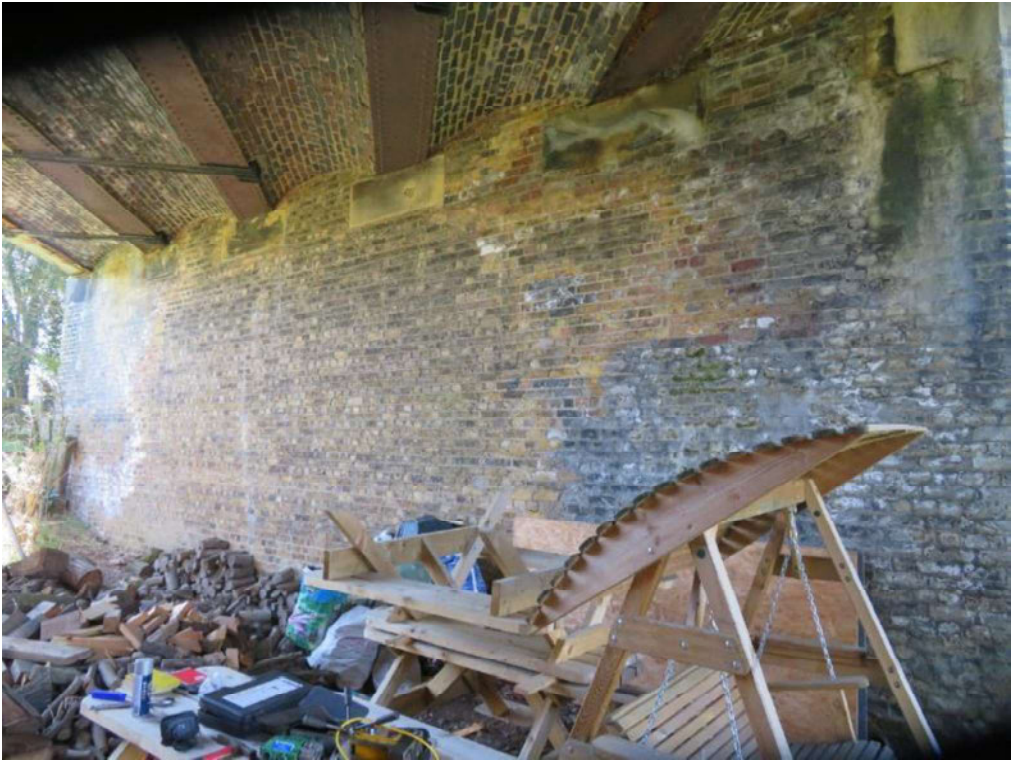
Photograph 5. View over structure, view northeastward



Photograph 6. Soffit



Photograph 7. Soffit (View South along east edge girder)



Photograph 8. West abutment



Photograph 9. East abutment



Photograph 10. Northwest Wingwall



Photograph 11. Northeast Wingwall



Photograph 12. Southwest Wingwall



Photograph 13. Southeast Wingwall



Photograph 14. South Edge Girder at East Support - Web



Photograph 15. South Edge Girder at East Support – Underside of bottom flange plate



Photograph 16. South Edge Girder at Midspan – External face and web



Photograph 17. South Edge Girder at Midspan – Underside of Bottom Flange



Photograph 18. South Edge Girder at Midspan – Internal Faces



Photograph 19. North Edge Girder at East Support - Web



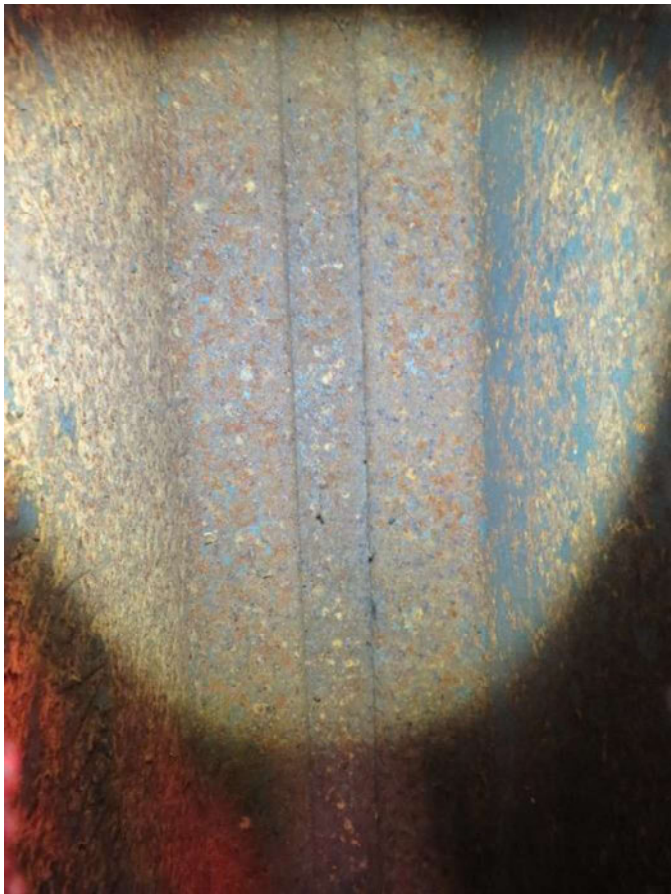
Photograph 20. North Edge Girder at East Support – Underside of bottom flange plate



Photograph 21. North Edge Girder at Midspan – External face and web



Photograph 22. North Edge Girder at Midspan – Underside of Bottom Flange



Photograph 23. North Edge Girder at Midspan – Internal Faces



Photograph 24. Second Internal Girder from South - Support



Photograph 25. Second Internal Girder from South - Midspan



Photograph 26. West abutment – Fracture below second internal girder from south



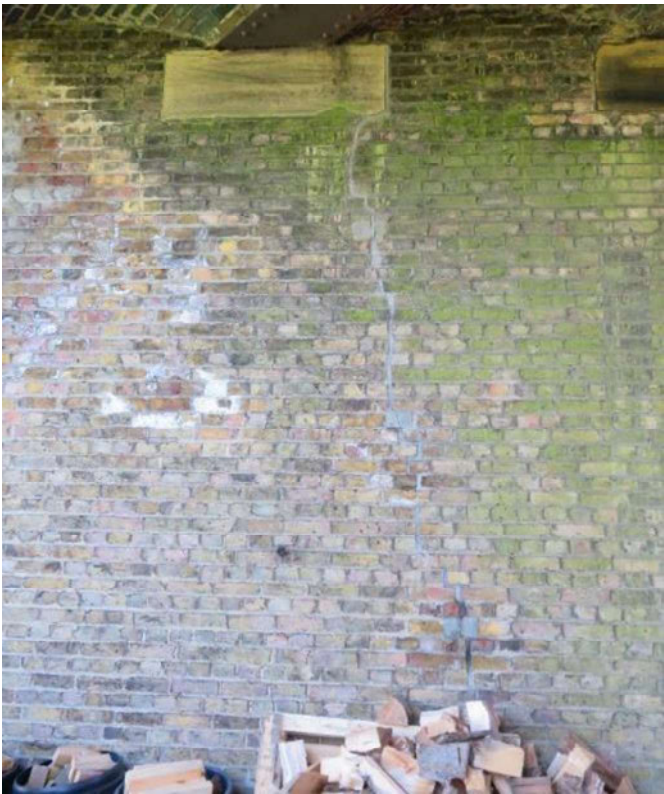
Photograph 27. West abutment – Fracture below second internal girder from south



Photograph 28. West abutment – Fracture below second internal girder from south



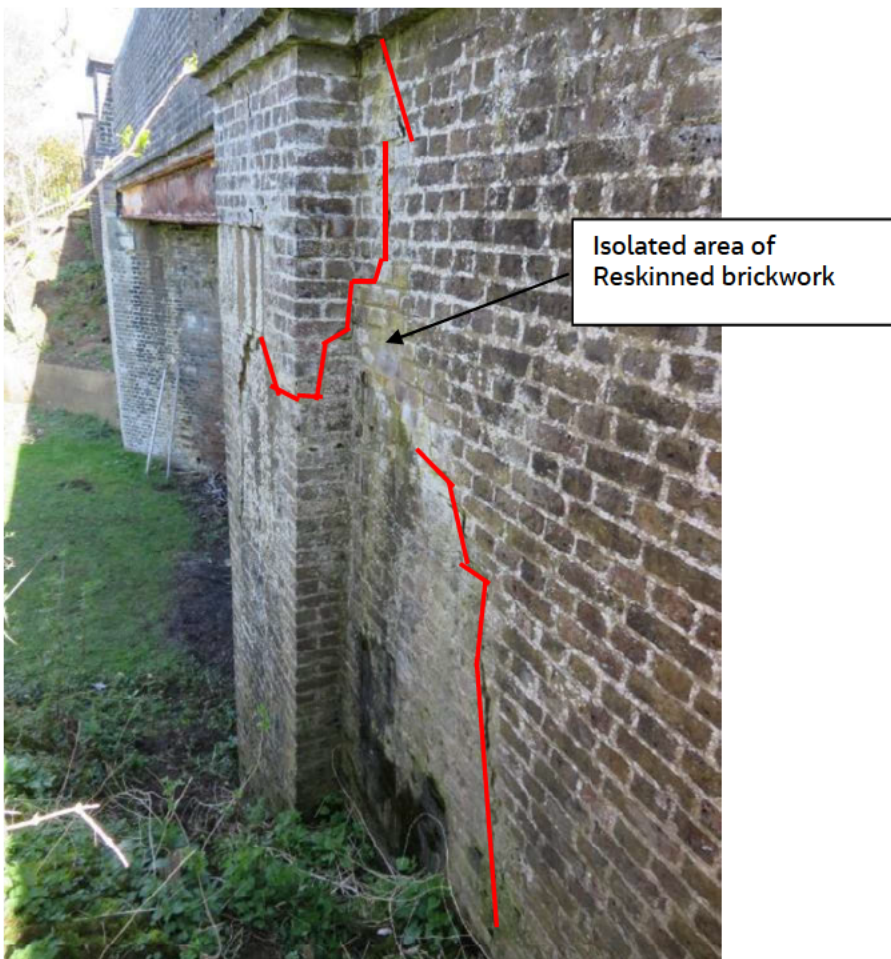
Photograph 29. West abutment – north edge fracture



Photograph 30. East Abutment – Vertical fracture under second internal girder from north



Photograph 31. East Abutment – Vertical fracture under second internal girder from north -broken mortar tab



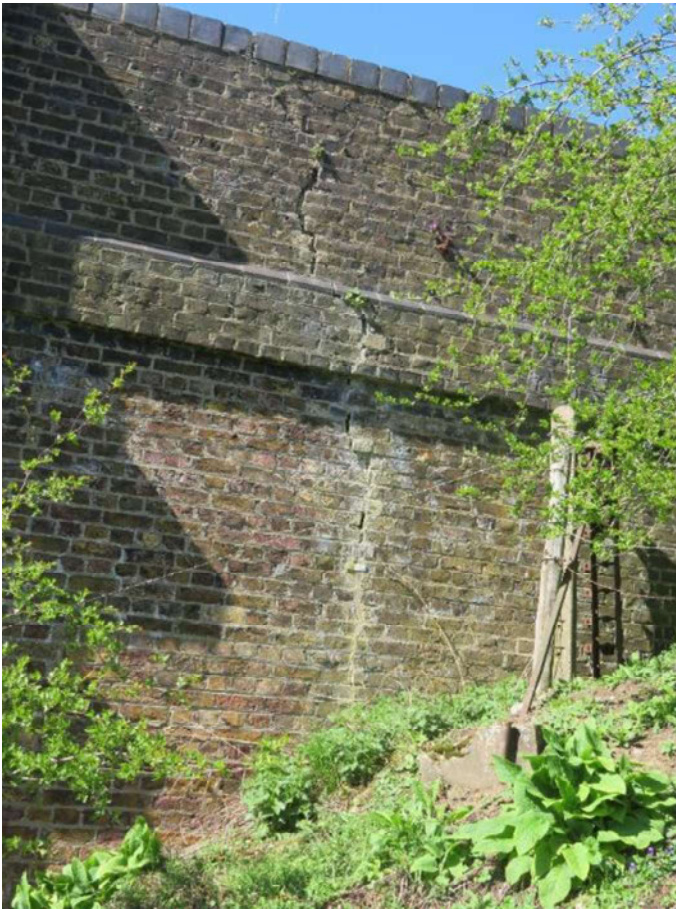
Photograph 32. Northwest wingwall – Diagonal fracture



Photograph 33. Northeast wingwall – Vertical fracture



Photograph 34. Southwest wingwall – Fracture



Photograph 35. Southeast wingwall – Fracture



Photograph 36. Southeast wingwall – fracture mortar tab



Photograph 37. Southeast wingwall - buttress



Photograph 38. North Parapet



Photograph 39. South Parapet



Photograph 40. South Parapet – broken mortar tab



Photograph 41. Southwest Parapet – Filled fracture



Photograph 42. North Formation



Photograph 43. South Formation

Appendix B. Form AA

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

ELR / Bridge No: SMB/2062

Appendix: 4

Issue: 1

Revision: B (Nov 2000)

APPROVAL IN PRINCIPLE FOR ASSESSMENT**Bridge/Line Name: Braughing Station / Buntingford branch****ELR/Bridge No. SMB/2062****Brief Description of Existing Bridge:**

Dimensions and condition factors are obtained from site measurements and inspection. (See Jacobs report "VAR9-7070 Assessment Programme – Assessment and Inspection Report – Bridge Ref.: SMB/2062 – March 2025).

(a) Span Arrangement

The structure is a heavily skewed (40°) single-span bridge. The skewed span of the structure is 9.14m.

(b) Superstructure Type

The superstructure comprises six longitudinally spanning double joist compound girders at 1.665m centres with brick jack arches spanning between them. The internal and edge girders are of a similar construction.

(c) Substructure Type

The wingwalls, parapets, and abutment are constructed from brindle brick.

(d) Planned highway works/modifications at this site

None

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

The structure carries the B1368 over the former trackbed. The B1368 has a carriageway width of 5.97m over the structure. A footpath runs along the north verge and there is a grass verges on the southern side. The north footpath has a width of 1.38m, the south verge has a width of 0.79m. The overall width between the parapets is 8.13m.

(f) Any other requirements

None

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

ELR / Bridge No: SMB/2062

Appendix: 4

Issue: 1

Revision: B (Nov 2000)

APPROVAL IN PRINCIPLE FOR ASSESSMENT**Assessment Criteria****(a) Loadings and Speed**

The bridge is to be assessed with vehicle loading obtained from and applied in accordance with CS 454. 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 – 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.

The girders are constructed from steel. As evidence suggests the bridge was constructed post BS 15:1905, this is assumed to be an early steel with a yield strength, F_y of 230 N/mm² as indicated in the Network Rail NR/GN/CIV/025 Table A2.

The internal 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.

The edge girders will be checked for accidental vehicle loads utilising ALL model 1 applied by simple statics.

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 following factors will be applied to loadings:

- An impact factor of 1.80 for 'Poor' surface category shall be applied to the critical axle.
- A traffic flow factor of 0.90 for 'Low' traffic flow category shall be applied.
- A lane factor of 1.0 shall be applied.

The substructure will be assessed qualitatively.

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

ELR / Bridge No: SMB/2062

Appendix: 4

Issue: 1

Revision: B (Nov 2000)

APPROVAL IN PRINCIPLE FOR ASSESSMENT**Senior Civil Engineer's Comments**

None

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.....

.....

.....

.....

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

Title

Date

21/05/2025

Category 2 and 3

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

Signed

Title

Date

Signed

Title

Date

Appendix C. Form BA

FORM 'BA' (BRIDGES)**GC/TP0356**ELR/ Bridge No **SMB/2062**

Appendix: 4

Issue: 1

Revision: A (Dec 2005)

CERTIFICATION FOR ASSESSMENT CHECK**Assessment Group:** **Jacobs UK Ltd****Bridge/Line Name:** **Braughing Station / Buntingford branch****Category of Check:** **1****ELR/ Bridge No:** **SMB/2062**

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

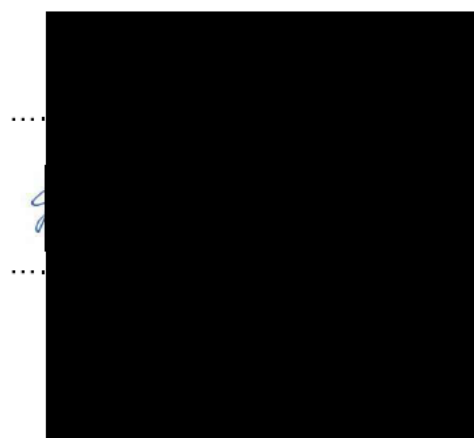
- (1) It has been assessed in accordance with the Approval in Principle as recorded on Form AA approved on 21st May 2025.
- (2) It has been checked for compliance with the following principal British Standards, Codes of Practice, BRB (Residuary) Limited technical notes and Assessment standards:

CS 454 – “Assessment of highway bridges and structures”

CS 456 – “The assessment of steel 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 1NameSignatureDate

Assessor

Assessment Checker

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

FORM 'BA' (BRIDGES)

GC/TP0356

ELR/ Bridge No **SMB/2062**

Appendix: 4

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.

Checker is responsible.

(b) Check

Name

Signature

Date

Assessor

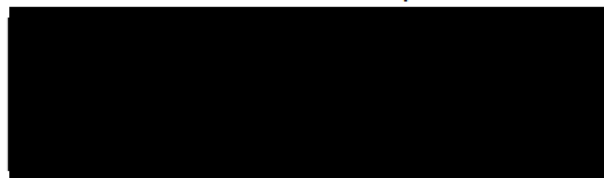
.....

Assessment Checker

.....

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

This Certificate is accepted by....



.....

FORM 'BAA' (BRIDGES)

GC/TP0356

ELR/ Bridge No SMB/2062

Appendix: 4

Issue: 1

Revision: A (Dec 2005)

CERTIFICATION FOR ASSESSMENT CHECK

Notification of Assessment Check

Assessment Group	Jacobs UK Ltd
Bridge Name/Road No.	Braughing Station
Line Name	Buntingford branch
ELR Code/Structure No.	SMB/2062

The above bridge 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 is as follows: -

STATEMENT OF CAPACITY

Internal Girders:	Normal Traffic
North Edge Girder:	40 tonnes GVW accidental vehicle loading
South Edge Girder:	Dead Loads Only
Substructure:	Normal traffic (by qualitative assessment)*

*The vertical fractures in the substructure should be monitored to determine if there is any further widening. If there is no evidence of movement, the substructures are considered suitable by qualitative assessment for vehicles conforming to Normal Traffic Loading (40/44 tonnes) as defined by CS454.

Recommended Loading Restrictions

The south edge girder should be protected from any form of live loading. It's recommended that a road restraint barrier be installed, as soon as is practical.

No restrictions are required for the internal and north edge girder.

Description of Structural Deficiencies and Recommended Strengthening

The assessment of the structure has demonstrated that the southern edge girders do not have the capacity to sustain any form of live loading. Due to the arrangement of the double joist compound girder, replacing the external RSJ to repair the unit does not appear practical. There may be an option to replace the entire edge girder unit with a new girder but any modification to the edge girder will also affect the construction of the retrofitted tie bars and jack arches.

As the internal girder has only marginally passed for normal traffic loading, and the north edge girders are also displaying a similar corrosion pattern to the south girders, the most cost-effective solution is likely to be the infill of the structure.

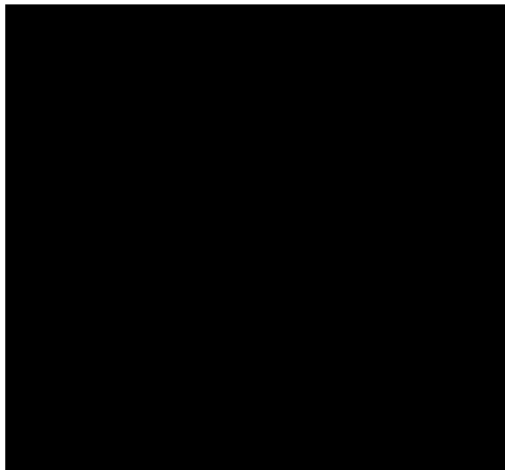
FORM 'BAA' (BRIDGES)**GC/TP0356**

ELR/ Bridge No SMB/2062

Appendix: 4

Issue: 1

Revision: A (Dec 2005)

CERTIFICATION FOR ASSESSMENT CHECKNameSignatureDate

Assessor

.....

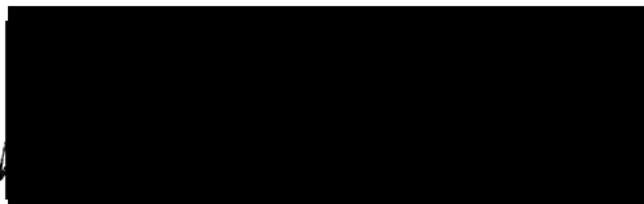
Assessment Checker

.....

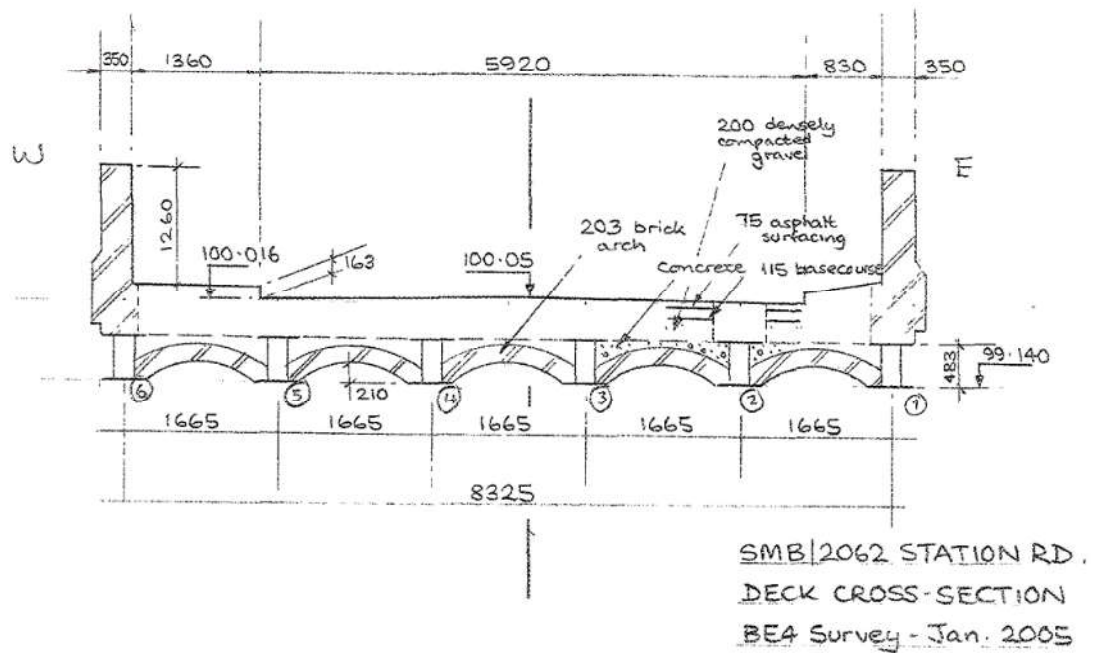
Authorised signatory of
the firm of Consulting
Engineers to whom
Assessor/Checker is
responsible.

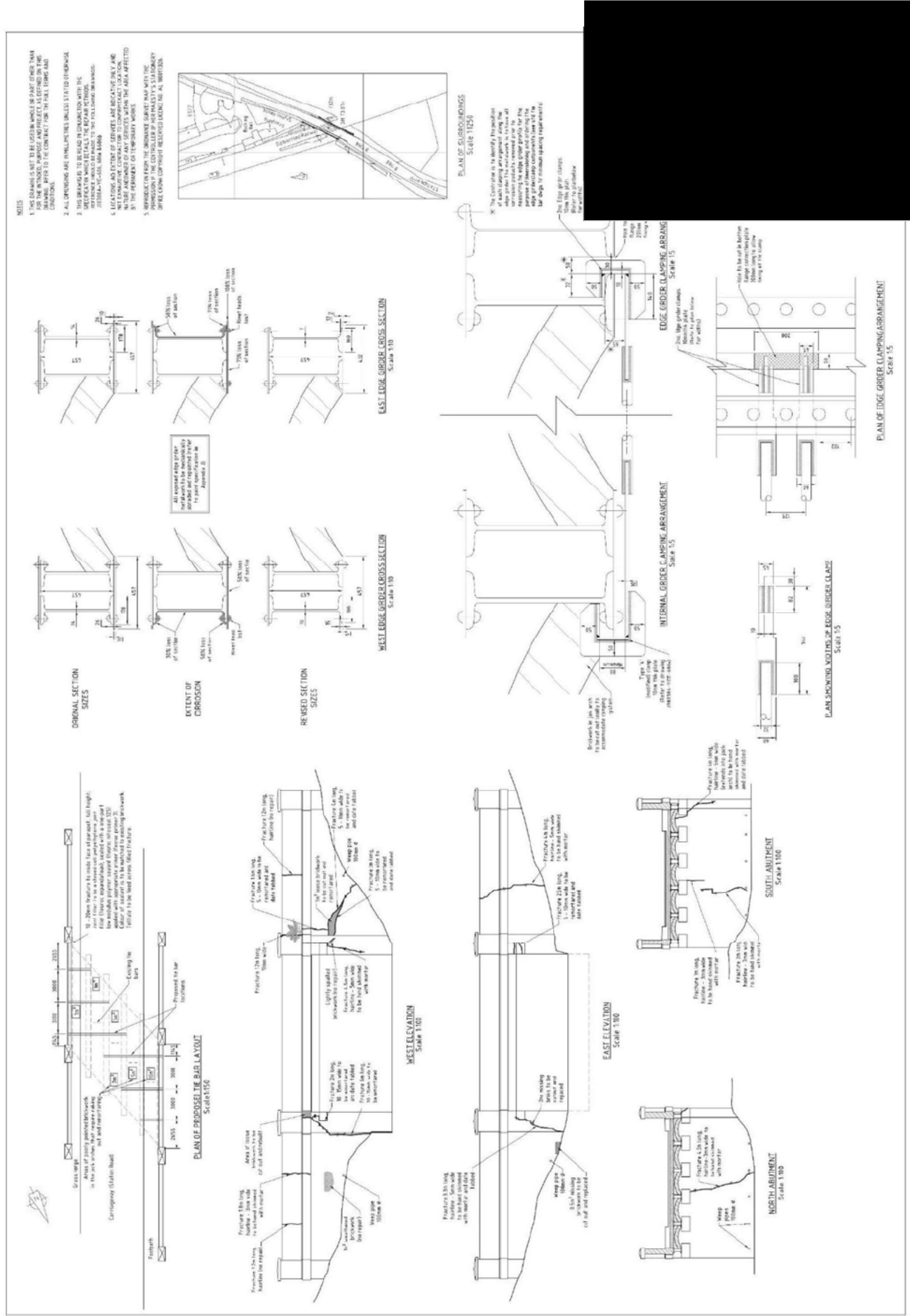
.....

This Certificate is accepted by...

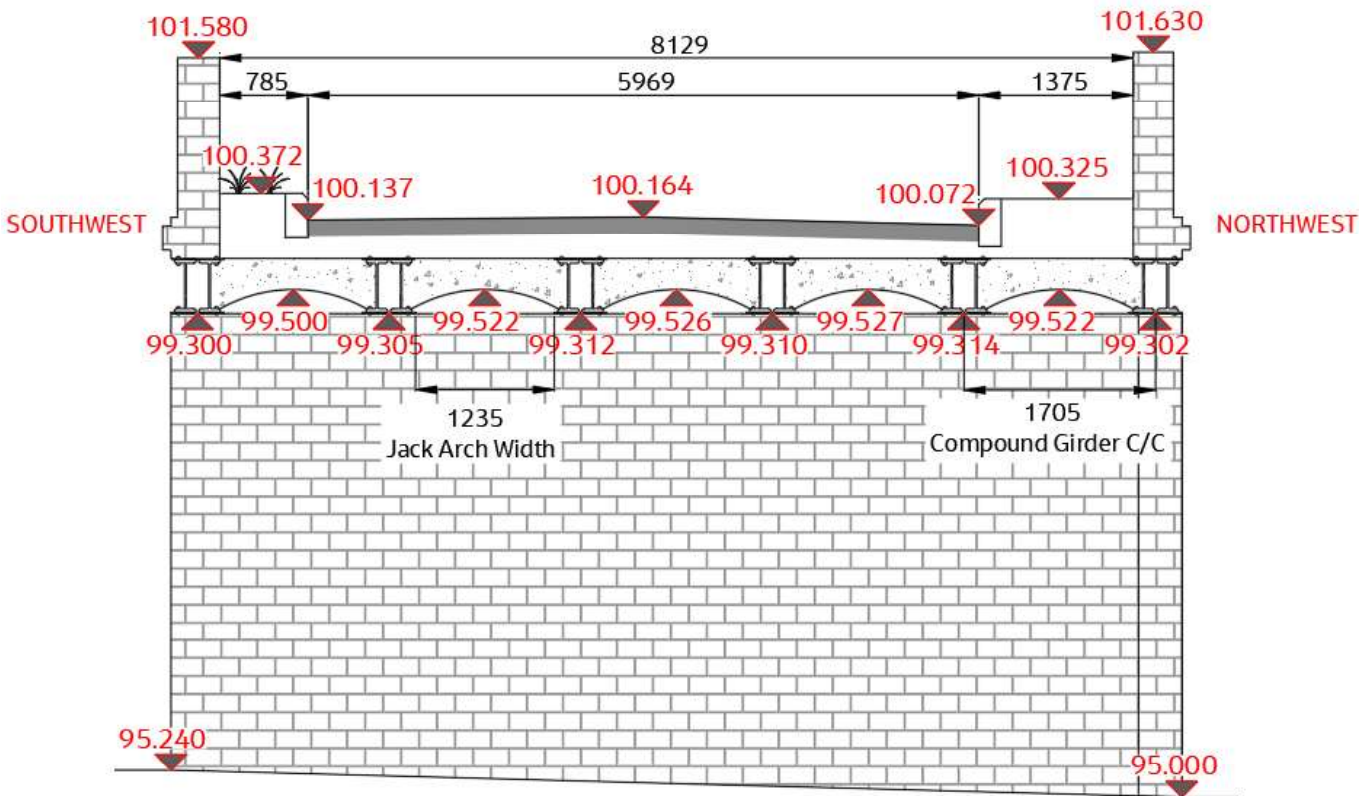


Appendix D. Existing Drawings

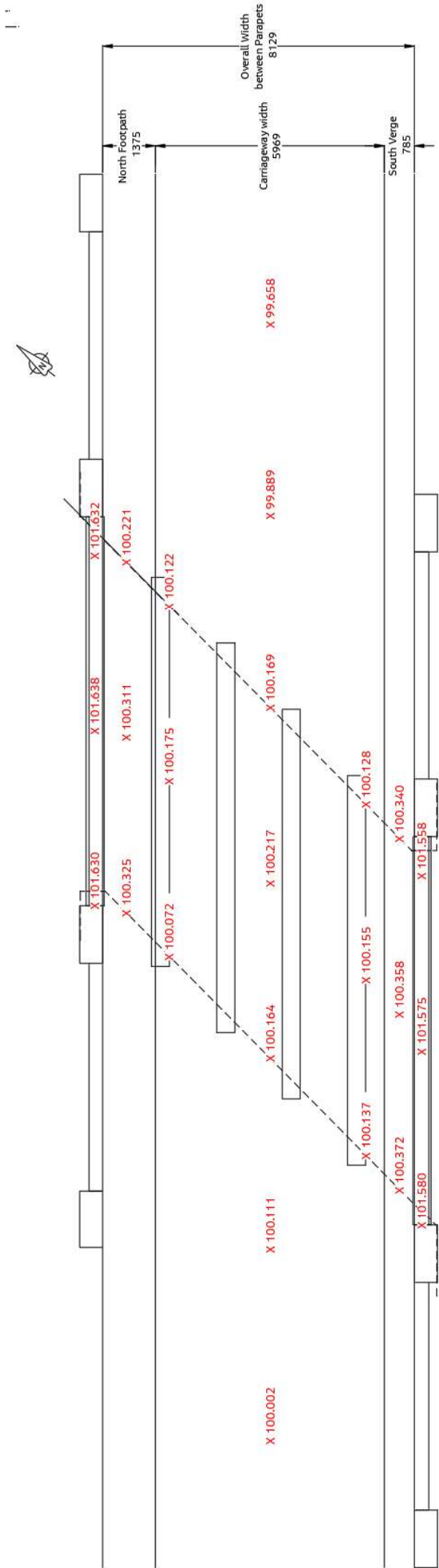




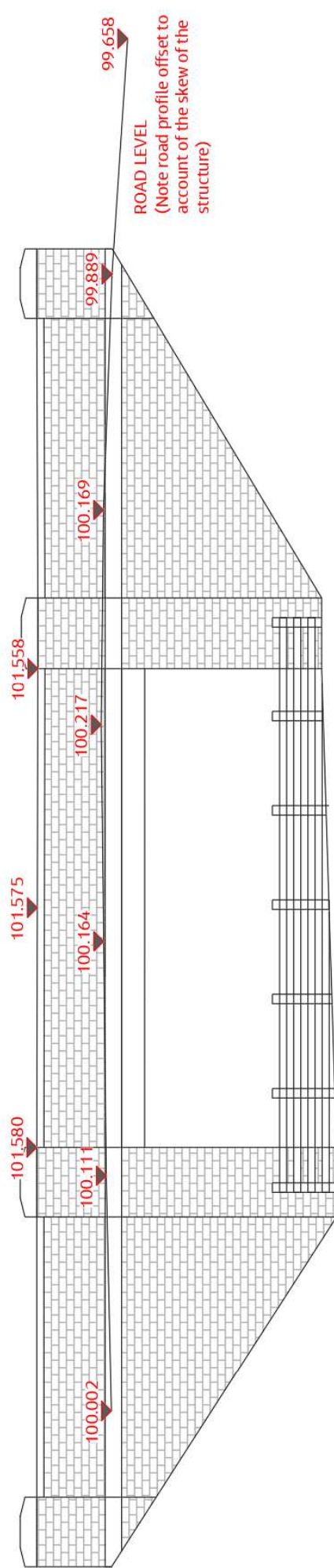
Appendix E. Survey Sketches



Cross section



Plan



South Elevation

Appendix F. Calculations

1. The first step in the process is to identify the problem or issue that needs to be addressed. This involves gathering information and understanding the context of the problem.

Jacobs
York

Project Title: HRE	Programme	Calc. No.: 0451898
Job No:		File: VAR9-7070
Project Manager		Subject: SMB-2062
Assessor		
Project Group	31200	
		CS 454 Assessment

[illegible]

Superseded by Calculation No.

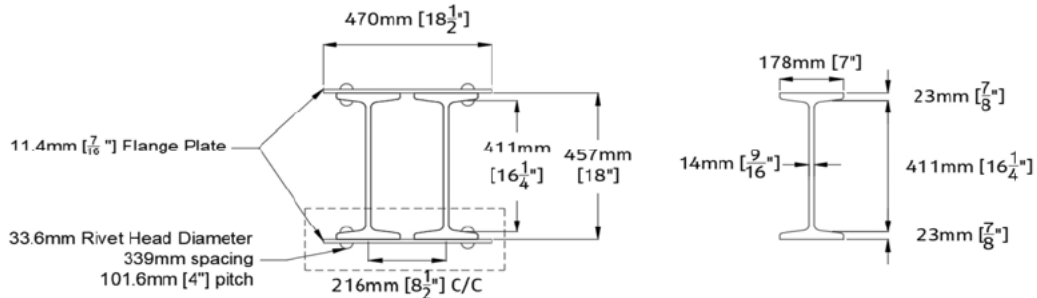
Date

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

Office	York Office	Page No.		Calc No.	0451898
Job No. & Title	VAR9-7070 HRE Assessment Programme SMB-2062 CS 454 Assessment	Calcs by		Date	May-25
Section	Section Properties	Checker		Date	May-25

Uncorroded section properties of girders

Internal/Edge Girder elastic section properties about x-x



Rivet head diameter was measure to be 33.6 mm
 Rivets therefore have a 21.0 mm, or 7/8 " shaft diameter
 The Rivet hole is assumed to be 22.57 mm
 (rivet hole = rivet diameter + 1/16")

No	Section	B mm	H mm	A mm	y mm	A.y mm3	A(y-y _t) ² mm4	I _{xx, Sec} mm4
1	Top flange Plate	470.00	11.40	5.36E+03	5.70	3.05E+04	2.71E+08	5.80E+04
2	RSJ T Flange (Left)	178.00	23.00	4.09E+03	22.90	9.38E+04	1.77E+08	1.80E+05
3	RSJ T Flange (Right)	178.00	23.00	4.09E+03	22.90	9.38E+04	1.77E+08	1.80E+05
4	RSJ Web (Left)	14.00	411.00	5.75E+03	239.90	1.38E+06	5.01E+05	8.10E+07
5	RSJ Web (Right)	14.00	411.00	5.75E+03	239.90	1.38E+06	5.01E+05	8.10E+07
6	RSJ B. Flange (Left)	178.00	23.00	4.09E+03	456.90	1.87E+06	2.10E+08	1.80E+05
7	RSJ B Flange (Right)	178.00	23.00	4.09E+03	456.90	1.87E+06	2.10E+08	1.80E+05
8	Bottom Flange Plate	470.00	11.40	5.36E+03	474.10	2.54E+06	3.18E+08	5.80E+04
9	Bot Rivet Hole (left)	-22.57	34.40	-7.76E+02	462.60	-3.59E+05	-4.18E+07	-7.66E+04
10	Bot Rivet Hole (right)	-22.57	34.40	-7.76E+02	462.60	-3.59E+05	-4.18E+07	-7.66E+04
Total				3.70E+04		8.54E+06	1.28E+09	1.63E+08

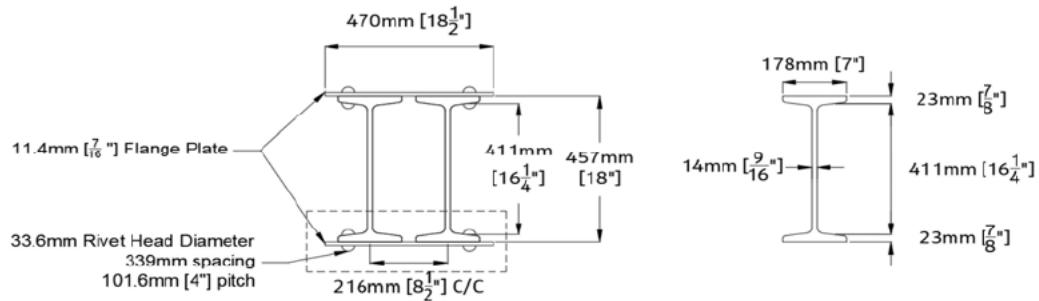
* Rivet shaft widths have been subtracted from bottom flange width

Overall Height of Girder	D =	479.80	mm
Depth of Web	d _w =	411.00	mm
Distance to the NA from the bottom of section	y _b =	249.23	mm
Distance to the NA from the top of section	y _t =	230.57	mm
Second Moment of Area of the section	I _{xx} =	1.44E+09	mm ⁴

Elastic section modulus (Bottom Flange)	Z _{XT} =	5.78E+06	mm ³
Elastic section modulus (Top Flange)	Z _{XC} =	6.25E+06	mm ³

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Job No. & Title	VAR9-7070 HRE Assessment Programme SMB-2062 CS 454 Assessment	Calcs by		Date	May-25
Section	Section Properties	Checker		Date	May-25

Internal / Edge Girder, plastic section properties about x-x

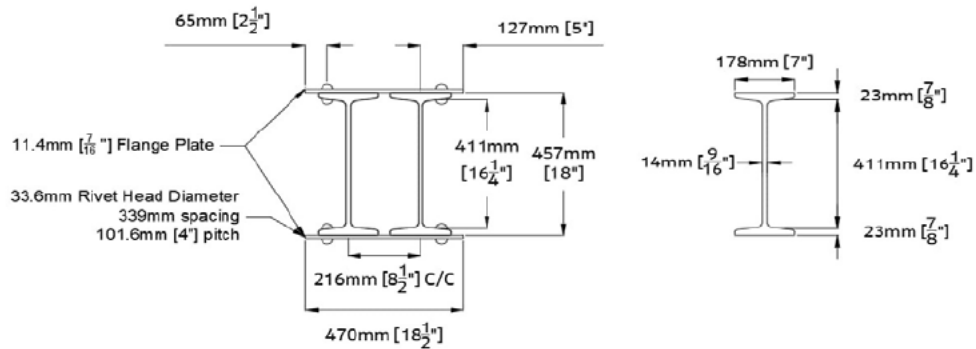


No	Section	B mm	H mm	A mm	y mm	A (y _p -y) mm ³
1	Top flange Plate	470.00	11.40	5.36E+03	5.70	1.11E+06
2	RSJ T Flange (Left)	178.00	23.00	4.09E+03	22.90	7.75E+05
3	RSJ T Flange (Right)	178.00	23.00	4.09E+03	22.90	7.75E+05
4	RSJ Web (Left) Above PNA	14.00	177.77	2.49E+03	123.29	2.21E+05
5	RSJ Web (Left) Below PNA	14.00	233.23	3.27E+03	328.79	3.81E+05
6	RSJ Web (Right) Above PNA	14.00	177.77	2.49E+03	123.29	2.21E+05
7	RSJ Web (Right) Below PNA	14.00	233.23	3.27E+03	328.79	3.81E+05
8	RSJ B. Flange (Left)	178.00	23.00	4.09E+03	456.90	1.00E+06
9	RSJ B Flange (Right)	178.00	23.00	4.09E+03	456.90	1.00E+06
10	Bottom Flange Plate	470.00	11.40	5.36E+03	474.10	1.40E+06
11	Bot Rivet Hole (left)	-22.57	34.40	-7.76E+02	462.60	-1.94E+05
12	Bot Rivet Hole (right)	-22.57	34.40	-7.76E+02	462.60	-1.94E+05
Total				3.70E+04		6.88E+06

Half Area	=	18523.59	mm ²
Area above PNA excluding web	=	13546.00	mm ²
Area above PNA in web	=	4977.59	mm ²
Depth of web above PNA	=	177.77	mm
Distance between top to web to top of section	=	34.40	mm
Depth of PNA from top of section	yp =	212.17	mm
Z _{pe} = ΣAy	=	6.88E+06	mm ³

Office	York Office	Page No.		Calc No.	0451898
Job No. & Title	VAR9-7070 HRE Assessment Programme SMB-2062 CS 454 Assessment	Calcs by		Date	May-25
Section	Section Properties	Checker		Date	May-25

Edge/Internal Girder section properties about y-y



Axis taken from the left edge

No	Section	B mm	H mm	A mm ²	x mm	A.x mm ³	A(x-x _t) ² mm ⁴	I _{yy, Sec} mm ⁴
1	Top flange Plate	11.40	470.00	5.36E+03	235.00	1.26E+06	0.00E+00	9.86E+07
2	RSJ T Flange (Left)	23.00	178.00	4.09E+03	127.00	5.20E+05	4.78E+07	1.08E+07
3	RSJ T Flange (Right)	23.00	178.00	4.09E+03	343.00	1.40E+06	4.78E+07	1.08E+07
4	RSJ Web (Left)	411.00	14.00	5.75E+03	127.00	7.31E+05	6.71E+07	9.40E+04
5	RSJ Web (Right)	411.00	14.00	5.75E+03	343.00	1.97E+06	6.71E+07	9.40E+04
6	RSJ B. Flange (Left)	23.00	178.00	4.09E+03	127.00	5.20E+05	4.78E+07	1.08E+07
7	RSJ B Flange (Right)	23.00	178.00	4.09E+03	343.00	1.40E+06	4.78E+07	1.08E+07
8	Bottom Flange Plate	11.40	470.00	5.36E+03	235.00	1.26E+06	0.00E+00	9.86E+07
Total				3.86E+04		9.07E+06	3.25E+08	2.41E+08

Distance to the NA from the left of section

$$x_{NA} = 235.00 \text{ mm}$$

Second Moment of Area of the section

$$I_{yy} = 5.66E+08 \text{ mm}^4$$

Radius of Gyration

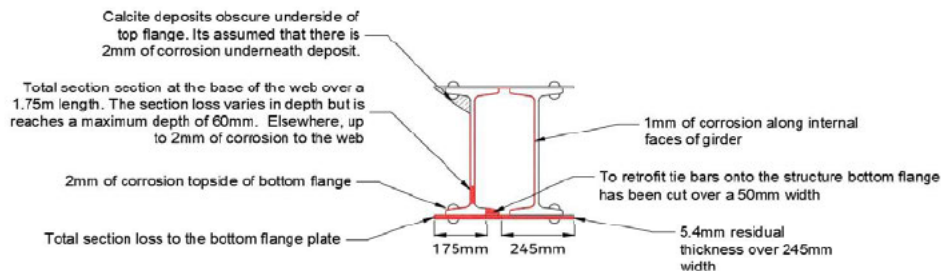
$$r_y = 121.08 \text{ mm}$$

Office	York Office	Page No.		Calc No.	0451898
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Section	Section Properties	Checker		Date	May-25

Corroded Section Properties

South Edge Girder - Midspan

South Edge Girder corroded elastic section properties about x-x, at midspan



Due to the pattern of corrosion, the external RSJ (positioned left in the diagram) is assumed not to contribute to the section properties of the double-joist compound girder. As the external RSJ is not longer effective, the top and bottom flange plates are only assumed to be effective above the internal girder, reducing their section reduced by approximately half.

No	Section	B mm	H mm	A mm	y mm	A.y mm ³	A(y-y _t) ⁴ mm ⁴	I _{xx, Sec} mm ⁴
1	Top flange Plate	235.00	11.40	2.68E+03	5.70	1.53E+04	1.14E+08	2.90E+04
2	RSJ T Flange (Left)	178.00	23.00	-	-	-	-	-
3	RSJ T Flange (Right)	178.00	23.00	4.09E+03	22.90	9.38E+04	1.46E+08	1.80E+05
4	RSJ Web (Left)	14.00	411.00	-	-	-	-	-
5	RSJ Web (Right)	14.00	411.00	5.75E+03	239.90	1.38E+06	4.51E+06	8.10E+07
6	RSJ B. Flange (Left)	178.00	23.00	-	-	-	-	-
7	RSJ B Flange (Right)	178.00	23.00	4.09E+03	456.90	1.87E+06	2.46E+08	1.80E+05
8	Bottom Flange Plate *3	235.00	5.40	1.27E+03	471.10	5.98E+05	8.53E+07	3.08E+03
9	Bot Rivet Hole (left)	22.57	-34.40	-	-	-	-	-
10	Bot Rivet Hole (right) *2	22.57	-28.40	-6.41E+02	459.60	-2.95E+05	-3.93E+07	-4.31E+04
Section Loss due to corrosion								
11	SL TFP underside	19.00	-1.00	-1.90E+01	10.90	-2.07E+02	-7.68E+05	-1.58E+00
12	SL RSJ T Flange (Right) Vertical SL	-1.00	23.00	-2.30E+01	22.90	-5.27E+02	-8.22E+05	-1.01E+03
13	SL RSJ T Flange (Right) Underside	81.00	-1.00	-8.10E+01	33.90	-2.75E+03	-2.57E+06	-6.75E+00
14	SL RSJ Web (Right) Vertical	-1.00	411.00	-4.11E+02	239.90	-9.86E+04	-3.22E+05	-5.79E+06
15	SL RSJ B Flange (Right) Vertical SL	-1.00	23.00	-2.30E+01	456.90	-1.05E+04	-1.38E+06	-1.01E+03
16	SL RSJ B Flange (Right) Topside SL *1	58.43	-1.00	-5.84E+01	445.90	-2.61E+04	-3.20E+06	-4.87E+00
Total				1.66E+04		3.52E+06	5.47E+08	7.56E+07

*1 - Rivet shaft width has been subtracted from bottom flange width

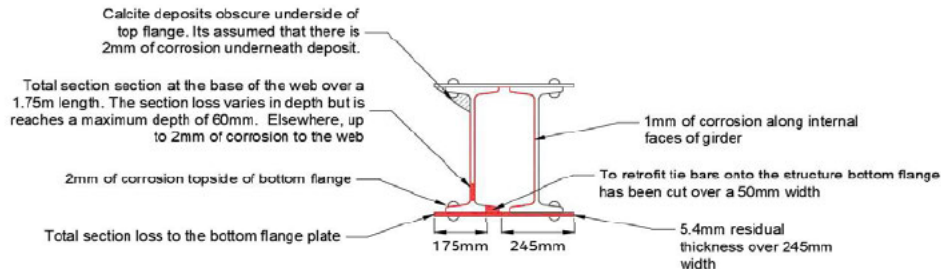
*2 - Rivet hole depth reduced to avoid double subtraction of bottom flange plate

*3 - Thickness has been reduced to account for section loss

JACOBS CALCULATION SHEET					
Office	York Office	Page No.		Calc No.	0451898
Job No. & Title	VAR9-7070 HRE Assessment Programme SMB-2062 CS 454 Assessment	Calcs by		Date	May-25
Section	Section Properties	Checker		Date	May-25
	Overall Height of Girder D = 479.80 mm Depth of Web d_w = 411.00 mm Distance to the NA from the bottom of section y_b = 267.91 mm Distance to the NA from the top of section y_t = 211.89 mm Second Moment of Area of the section I_{xx} = 6.23E+08 mm⁴ Elastic section modulus (Bottom Flange) Z_{XT} = 2.32E+06 mm³ Elastic section modulus (Top Flange) Z_{XC} = 2.94E+06 mm³				

Office	York Office	Page No.		Calc No.	0451898
Job No. & Title	VAR9-7070 HRE Assessment Programme SMB-2062 CS 454 Assessment	Calcs by		Date	May-25
Section	Section Properties	Checker		Date	May-25

South Edge Girder corroded plastic section properties about x-x, at midspan



No	Section	B mm	H mm	A mm	y mm	A (y _p -y) mm ³
1	Top flange Plate	235.00	11.40	2.68E+03	5.70	4.20E+05
2	RSJ T Flange (Left)	178.00	23.00	-	-	-
3	RSJ T Flange (Right)	178.00	23.00	4.09E+03	22.90	5.72E+05
4	RSJ Web (Left)	14.00	411.00	-	-	-
	RSJ Web (Right)					
5	Above PNA	14.00	128.21	1.80E+03	98.51	1.15E+05
6	Below PNA	14.00	282.79	3.96E+03	304.01	5.60E+05
7	RSJ B. Flange (Left)	178.00	23.00	-	-	-
8	RSJ B Flange (Right)	178.00	23.00	4.09E+03	456.90	1.20E+06
9	Bottom Flange Plate *3	235.00	5.40	1.27E+03	471.10	3.91E+05
10	Bot Rivet Hole (left)	-22.57	34.40	-	-	-
11	Bot Rivet Hole (right) *2	-22.57	28.40	-6.41E+02	174.11	-7.37E+03
Section Loss due to corrosion						
12	SL TFP underside	19.00	-1.00	-1.90E+01	10.90	-2.88E+03
13	SL RSJ T Flange (Right) Vertical SL	-1.00	23.00	-2.30E+01	22.90	-3.21E+03
14	SL RSJ T Flange (Right) Underside	81.00	-1.00	-8.10E+01	33.90	-1.04E+04
15	SL RSJ Web (Right) Vertical Above PNA	-1.00	128.21	-1.28E+02	98.51	-1.26E+04
15	SL RSJ Web (Right) Vertical Below PNA	-1.00	282.79	-2.83E+02	304.01	-8.60E+04
16	SL RSJ B Flange (Right) Vertical SL	-1.00	23.00	-2.30E+01	456.90	-6.77E+03
17	SL RSJ B Flange (Right) Topside SL *1	58.43	-1.00	-5.84E+01	445.90	-1.66E+04
	Total			1.66E+04		3.12E+06

*1 - Rivet shaft width has been subtracted from bottom flange width

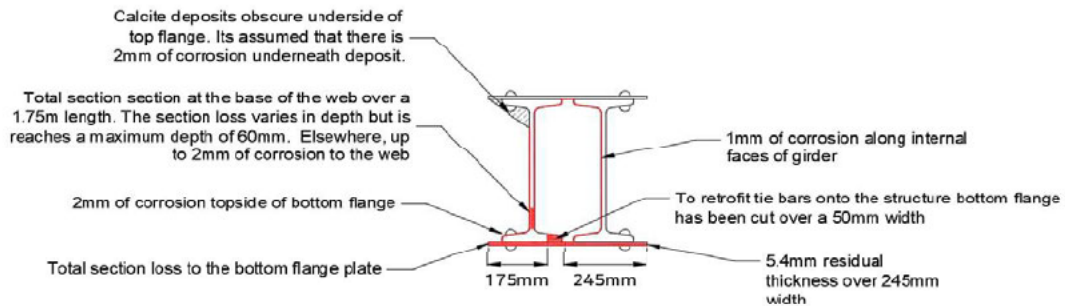
*2 - Rivet hole depth reduced to avoid double subtraction of bottom flange plate

*3 - Thickness has been reduced to account for section loss

Half Area	=	8316.79	mm ²
Area above PNA excluding web	=	6650.00	mm ²
Area above PNA in web	=	1666.79	mm ²
Depth of web above PNA	=	128.21	mm
Distance between top to web to top of section	=	34.40	mm
Depth of PNA from top of section	yp	162.61	mm
	Z _{pe} =ΣAy	3.12E+06	mm ³

Office	York Office	Page No.		Calc No.	0451898
Job No. & Title	VAR9-7070 HRE Assessment Programme SMB-2062 CS 454 Assessment	Calcs by		Date	May-25
Section	Section Properties	Checker		Date	May-25

South edge girder corroded section properties about y-y, at midspan



Axis taken from the left edge

No	Section	B mm	H mm	A mm ²	x mm	A.x mm ³	A(x-x _t) ² mm ⁴	I _{yy, Sec} mm ⁴
1	Top flange Plate	11.40	235.00	2.68E+03	352.50	9.44E+05	1.05E+05	1.23E+07
2	RSJ T Flange (Left)	23.00	178.00	-	-	-	-	-
3	RSJ T Flange (Right)	23.00	178.00	4.09E+03	343.00	1.40E+06	4.32E+04	1.08E+07
4	RSJ Web (Left)	411.00	14.00	-	-	-	-	-
5	RSJ Web (Right)	411.00	14.00	5.75E+03	343.00	1.97E+06	6.06E+04	9.40E+04
6	RSJ B. Flange (Left)	23.00	178.00	-	-	-	-	-
7	RSJ B Flange (Right)	23.00	178.00	4.09E+03	343.00	1.40E+06	4.32E+04	1.08E+07
8	Bottom Flange Plate *3	5.40	235.00	1.27E+03	352.50	4.47E+05	4.96E+04	5.84E+06

Section Loss due to corrosion

11	SL TFP underside	19.00	-1.00	-1.90E+01	235.00	-4.47E+03	-2.35E+05	-1.58E+00
12	SL RSJ T Flange (Right) Vertical SL	23.00	-1.00	-2.30E+01	254.50	-5.85E+03	-1.94E+05	-1.92E+00
13	SL RSJ T Flange (Right) Underside	-1.00	81.00	-8.10E+01	299.00	-2.42E+04	-1.81E+05	-4.43E+04
14	SL RSJ Web (Right) Vertical	411.00	-1.00	-4.11E+02	336.50	-1.38E+05	-3.90E+04	-3.43E+01
15	SL RSJ B Flange (Right) Vertical SL	23.00	-1.00	-2.30E+01	299.00	-6.88E+03	-5.13E+04	-1.92E+00
16	SL RSJ B Flange (Right) Topside SL	-1.00	81.00	-8.10E+01	254.50	-2.06E+04	-6.82E+05	-4.43E+04
Total				1.73E+04		5.97E+06	-1.08E+06	3.98E+07

*1 - Rivet shaft width has been subtracted from bottom flange width

*2 - Rivet hole depth reduced to avoid double subtraction of bottom flange plate

*3 - Thickness has been reduced to account for section loss

Distance to the NA from the left of section

Second Moment of Area of the section

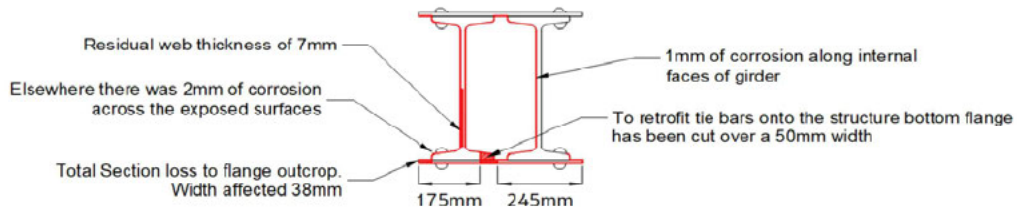
Radius of Gyration

X _{NA}	=	346.25	mm
I _{yy}	=	3.87E+07	mm ⁴
r _y	=	47.37	mm

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North Edge Girder - Midspan

North Edge Girder corroded elastic section properties about x-x, at midspan



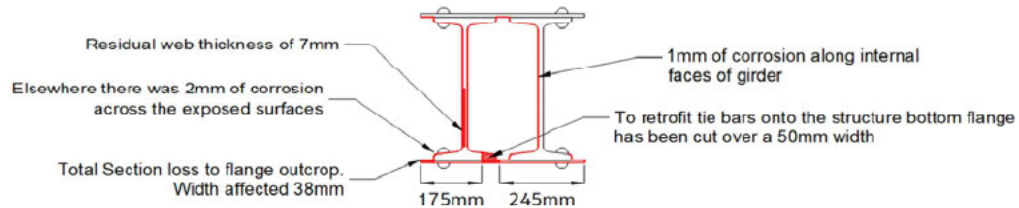
No	Section	B mm	H mm	A mm	y mm	A.y mm ³	A(y-y _t) ² mm ⁴	I _{xx, Sec} mm ⁴
0	Uncorroded Section	-	-	3.70E+04	-	8.54E+06	1.28E+09	1.63E+08
Section Loss due to corrosion								
1	Top plate vertical	-2.00	11.40	-2.28E+01	5.70	-1.30E+02	-9.69E+05	-2.47E+02
2	Top plate underside Ext	36.00	-2.00	-7.20E+01	10.40	-7.49E+02	-2.92E+06	-2.40E+01
3	Top plate underside Int	38.00	-1.00	-3.80E+01	10.90	-4.14E+02	-1.54E+06	-3.17E+00
4	TF (Left) Vertical ext	-2.00	23.00	-4.60E+01	22.90	-1.05E+03	-1.64E+06	-2.03E+03
5	TF (Left) Vertical int	-1.00	23.00	-2.30E+01	22.90	-5.27E+02	-8.22E+05	-1.01E+03
6	TF (Left) underside external	80.00	-2.00	-1.60E+02	33.40	-5.34E+03	-5.10E+06	-5.33E+01
7	TF (Left) underside internal	81.00	-1.00	-8.10E+01	33.90	-2.75E+03	-2.57E+06	-6.75E+00
8	TF (Right) Vert. int	-1.00	23.00	-2.30E+01	22.90	-5.27E+02	-8.22E+05	-1.01E+03
9	TF (Right) underside internal	81.00	-1.00	-8.10E+01	33.90	-2.75E+03	-2.57E+06	-6.75E+00
10	Web (Left) external	-2.00	205.50	-4.11E+02	137.15	-5.64E+04	-2.30E+06	-1.45E+06
11	Web (Left) external	-6.00	205.50	-1.23E+03	342.65	-4.22E+05	-2.11E+07	-4.34E+06
12	Web (Left) internal	-1.00	411.00	-4.11E+02	239.90	-9.86E+04	-3.22E+05	-5.79E+06
13	Web (Right) Internal	-1.00	411.00	-4.11E+02	239.90	-9.86E+04	-3.22E+05	-5.79E+06
14	BF (Left) topside external*1	57.43	-2.00	-1.15E+02	446.40	-5.13E+04	-6.32E+06	-3.83E+01
15	BF (Left) topside internal	55.00	-1.00	-5.50E+01	445.90	-2.45E+04	-3.01E+06	-4.58E+00
16	BF (Left) topside internal	27.00	-23.00	-6.21E+02	456.90	-2.84E+05	-3.73E+07	-2.74E+04
17	BF (Left) Vertical ext	-2.00	23.00	-4.60E+01	456.90	-2.10E+04	-2.76E+06	-2.03E+03
18	BF (right) topside	81.00	-1.00	-8.10E+01	445.90	-3.61E+04	-4.44E+06	-6.75E+00
19	BF (Right) Vert. int	-1.00	23.00	-2.30E+01	456.90	-1.05E+04	-1.38E+06	-1.01E+03
20	Bot plate ext outcrop	38.00	-11.40	-4.33E+02	474.10	-2.05E+05	-2.98E+07	-4.69E+03
21	Bot plate topside	29.00	-1.00	-2.90E+01	468.90	-1.36E+04	-1.92E+06	-2.42E+00
22	Bot plate cut out tie	50.00	-11.40	-5.70E+02	474.10	-2.70E+05	-3.92E+07	-6.17E+03
23	Bot plate underside *1	336.86	-2.00	-6.74E+02	477.80	-3.22E+05	-4.76E+07	-2.25E+02
Total				3.14E+04		6.61E+06	1.06E+09	1.45E+08

*1 - Rivet shaft width has been subtracted from bottom flange width

Overall Height of Girder	D	=	479.80	mm
Depth of Web	d _w	=	411.00	mm
Distance to the NA from the bottom of section	y _b	=	269.10	mm
Distance to the NA from the top of section	y _t	=	210.70	mm
Second Moment of Area of the section	I _{xx}	=	1.21E+09	mm ⁴
Elastic section modulus (Bottom Flange)	Z _{XT}	=	4.49E+06	mm ³
Elastic section modulus (Top Flange)	Z _{XC}	=	5.73E+06	mm ³

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North Edge Girder corroded plastic section properties about x-x. at midspan



No	Section	B mm	H mm	A mm	y mm	A (y _p -y) mm ³
1	Top flange Plate	470.00	11.40	5.36E+03	5.70	7.55E+05
2	RSJ T Flange (Left)	178.00	23.00	4.09E+03	22.90	5.07E+05
3	RSJ T Flange (Right)	178.00	23.00	4.09E+03	22.90	5.07E+05
4	RSJ Web (Left) Above PNA	14.00	112.28	1.57E+03	90.54	8.82E+04
5	RSJ Web (Left) Below PNA	14.00	298.72	4.18E+03	296.04	6.25E+05
6	RSJ Web (Right) Above PNA	14.00	112.28	1.57E+03	90.54	8.82E+04
7	RSJ Web (Right) Below PNA	14.00	298.72	4.18E+03	296.04	6.25E+05
8	RSJ B. Flange (Left)	178.00	23.00	4.09E+03	456.90	1.27E+06
9	RSJ B Flange (Right)	178.00	23.00	4.09E+03	456.90	1.27E+06
10	Bottom Flange Plate	470.00	11.40	5.36E+03	474.10	1.75E+06
11	Bot Rivet Hole (left)	-22.57	34.40	-7.76E+02	462.60	-2.45E+05
12	Bot Rivet Hole (right)	-22.57	34.40	-7.76E+02	462.60	-2.45E+05
Section Loss due to corrosion						
13	Top plate vertical	-2.00	11.40	-2.28E+01	5.70	-3.21E+03
14	Top plate underside Ext	36.00	-2.00	-7.20E+01	10.40	-9.81E+03
15	Top plate underside Int	38.00	-1.00	-3.80E+01	10.90	-5.16E+03
16	TF (Left) Vertical ext	-2.00	23.00	-4.60E+01	22.90	-5.69E+03
17	TF (Left) Vertical int	-1.00	23.00	-2.30E+01	22.90	-2.85E+03
18	TF (Left) underside external	80.00	-2.00	-1.60E+02	33.40	-1.81E+04
19	TF (Left) underside internal	81.00	-1.00	-8.10E+01	33.90	-9.13E+03
20	TF (Right) Vert. int	-1.00	23.00	-2.30E+01	22.90	-2.85E+03
21	TF (Right) underside internal	81.00	-1.00	-8.10E+01	33.90	-9.13E+03
22	Web (Left) external Above PNA	-2.00	112.28	-2.25E+02	90.54	-1.26E+04
23	Web (Left) external Below PNA	-2.00	93.22	-1.86E+02	193.29	-8.69E+03
24	Web (Left) external (Below PNA)	-6.00	205.50	-1.23E+03	342.65	-2.42E+05
25	Web (Left) internal Above PNA	-1.00	112.28	-1.12E+02	90.54	-6.30E+03
26	Web (Left) internal Below PNA	-1.00	298.72	-2.99E+02	183.76	-1.11E+04
27	Web (Right) Internal Above PNA	-1.00	112.28	-1.12E+02	90.54	-6.30E+03
28	Web (Right) Internal Below PNA	-1.00	298.72	-2.99E+02	296.04	-4.46E+04

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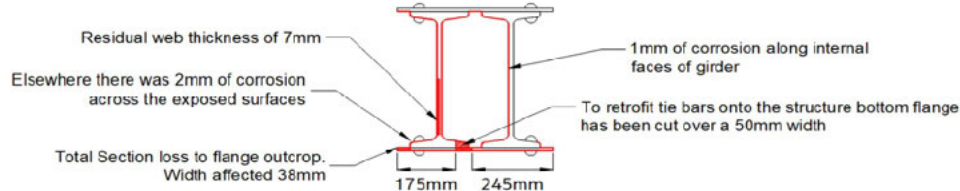
29	BF (Left) topside external*1	57.43	-2.00	-1.15E+02	446.40	-3.44E+04
30	BF (Left) topside internal	55.00	-1.00	-5.50E+01	445.90	-1.65E+04
31	BF (Left) topside internal	27.00	-23.00	-6.21E+02	456.90	-1.93E+05
32	BF (Left) Vertical ext	-2.00	23.00	-4.60E+01	456.90	-1.43E+04
33	BF (right) topside internal	81.00	-1.00	-8.10E+01	445.90	-2.42E+04
34	BF (Right) Vert. int	-1.00	23.00	-2.30E+01	456.90	-7.14E+03
35	Bot plate ext outcrop	38.00	-11.40	-4.33E+02	474.10	-1.42E+05
36	Bot plate topside	29.00	-1.00	-2.90E+01	468.90	-9.34E+03
37	Bot plate cut out tie	50.00	-11.40	-5.70E+02	474.10	-1.87E+05
38	Bot plate underside	336.86	-2.00	-6.74E+02	477.80	-2.23E+05
Total				3.14E+04		5.75E+06

*1 - Rivet shaft width has been subtracted from bottom flange width

Half Area	=	15693.80	mm ²
Area above PNA excluding web	=	12999.20	mm ²
Area above PNA in web	=	2694.60	mm ²
Depth of web above PNA	=	112.28	mm
Distance between top to web to top of section	=	34.40	mm
Depth of PNA from top of section	yp =	146.68	mm
Zpe=ΣAy	=	5.75E+06	mm ³

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Edge girder corroded section properties about y-y, at midspan



Axis taken from the left edge

No	Section	B mm	H mm	A mm ²	x mm	A.x mm ³	A(x-x _t) ² mm ⁴	I _{yy, Sec} mm ⁴
0	Uncorroded Section	-	-	3.86E+04	-	9.07E+06	3.25E+08	2.41E+08
Section Loss due to corrosion								
1	Top plate vertical	11.40	-2.00	-2.28E+01	1.00	-2.28E+01	-1.38E+06	-7.60E+00
2	Top plate underside Ext	-2.00	36.00	-7.20E+01	20.00	-1.44E+03	-3.71E+06	-7.78E+03
3	Top plate underside Int	-1.00	38.00	-3.80E+01	235.00	-8.93E+03	-5.38E+03	-4.57E+03
4	TF (Left) Vertical ext	23.00	-2.00	-4.60E+01	39.00	-1.79E+03	-1.99E+06	-1.53E+01
5	TF (Left) Vertical int	23.00	-1.00	-2.30E+01	215.50	-4.96E+03	-2.27E+04	-1.92E+00
6	TF (Left) underside external	-2.00	80.00	-1.60E+02	80.00	-1.28E+04	-4.46E+06	-8.53E+04
7	TF (Left) underside internal	-1.00	81.00	-8.10E+01	174.50	-1.41E+04	-4.25E+05	-4.43E+04
8	TF (Right) Vert. int	23.00	-1.00	-2.30E+01	254.50	-5.85E+03	-1.33E+03	-1.92E+00
9	TF (Right) underside internal	-1.00	81.00	-8.10E+01	295.50	-2.39E+04	-1.91E+05	-4.43E+04
10	Web (Left) external	205.50	-2.00	-4.11E+02	121.00	-4.97E+04	-6.51E+06	-1.37E+02
11	Web (Left) external	205.50	-6.00	-1.23E+03	123.00	-1.52E+05	-1.89E+07	-3.70E+03
12	Web (Left) internal	411.00	-1.00	-4.11E+02	133.50	-5.49E+04	-5.28E+06	-3.43E+01
13	Web (Right) Internal	411.00	-1.00	-4.11E+02	336.50	-1.38E+05	-3.30E+06	-3.43E+01
14	BF (Left) topside external	-2.00	80.00	-1.60E+02	80.00	-1.28E+04	-4.46E+06	-8.53E+04
15	BF (Left) topside internal	-1.00	55.00	-5.50E+01	161.50	-8.88E+03	-4.01E+05	-1.39E+04
16	BF (Left) topside internal	-23.00	27.00	-6.21E+02	195.50	-1.21E+05	-1.64E+06	-3.77E+04
17	BF (Left) Vertical ext	23.00	-2.00	-4.60E+01	39.00	-1.79E+03	-1.99E+06	-1.53E+01
18	BF (right) topside internal	-1.00	81.00	-8.10E+01	295.50	-2.39E+04	-1.91E+05	-4.43E+04
19	BF (Right) Vert. int	23.00	-1.00	-2.30E+01	254.50	-5.85E+03	-1.33E+03	-1.92E+00
20	Bot plate ext outcrop	-11.40	38.00	-4.33E+02	19.00	-8.23E+03	-2.25E+07	-5.21E+04
21	Bot plate topside	-1.00	29.00	-2.90E+01	235.00	-6.82E+03	-4.10E+03	-2.03E+03
22	Bot plate cut out tie *1	-9.40	50.00	-4.70E+02	200.00	-9.40E+04	-1.03E+06	-9.79E+04
23	Bot plate underside	-2.00	432.00	-8.64E+02	254.00	-2.19E+05	-4.36E+04	-1.34E+07
Total				3.28E+04		8.10E+06	2.47E+08	2.27E+08

*1 - Cut out size reduced to avoid double subjection with item 23

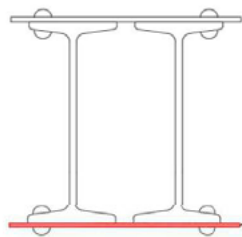
Distance to the NA from the left of section
Second Moment of Area of the section
Radius of Gyration

x _{NA}	=	246.90	mm
I _{yy}	=	4.74E+08	mm ⁴
r _y	=	120.14	mm

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Internal Girder @ midspan

Internal Girder corroded elastic section properties about x-x, at midspan



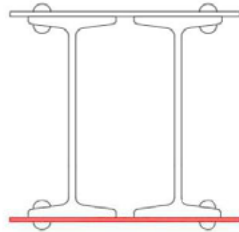
No	Section	B mm	H mm	A mm	y mm	A.y mm ³	A(y-y _t) ² mm ⁴	I _{xx, Sec} mm ⁴
0	Uncorroded Section	-	-	3.70E+04	-	8.54E+06	1.28E+09	1.63E+08
8	Bottom Flange Plate	424.86	-11.40	-4.84E+03	474.10	-2.30E+06	-2.87E+08	-5.25E+04
	Total			3.22E+04		6.25E+06	9.91E+08	1.63E+08

* Rivet shaft widths have been subtracted from bottom flange width

Overall Height of Girder	D	=	479.80	mm
Depth of Web	d _w	=	411.00	mm
Distance to the NA from the bottom of section	y _b	=	285.86	mm
Distance to the NA from the top of section	y _t	=	193.94	mm
Second Moment of Area of the section	I _{xx}	=	1.15E+09	mm ⁴
Elastic section modulus (Bottom Flange)	Z _{XT}	=	4.04E+06	mm ³
Elastic section modulus (Top Flange)	Z _{XC}	=	5.95E+06	mm ³

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Internal Girder, corroded plastic section properties about x-x



Along blastline, rivets have become ineffective, causing the flange plate to sag.

No	Section	B mm	H mm	A mm	y mm	A (y _p -y) mm ³
1	Top flange Plate	470.00	11.40	5.36E+03	5.70	1.11E+06
2	RSJ T Flange (Left)	178.00	23.00	4.09E+03	22.90	7.75E+05
3	RSJ T Flange (Right)	178.00	23.00	4.09E+03	22.90	7.75E+05
4	RSJ Web (Left)	14.00	91.28	1.28E+03	80.04	1.69E+05
5	RSJ Web (Left)	14.00	319.72	4.48E+03	285.54	3.28E+05
6	RSJ Web (Right)	14.00	91.28	1.28E+03	80.04	1.69E+05
7	RSJ Web (Right)	14.00	319.72	4.48E+03	285.54	3.28E+05
8	RSJ B. Flange (Left)	178.00	23.00	4.09E+03	456.90	1.00E+06
9	RSJ B Flange (Right)	178.00	23.00	4.09E+03	456.90	1.00E+06
10	Bottom Flange Plate*	470.00	11.40	-	-	-
11	Bot Rivet Hole (left)	-22.57	23.00	-5.19E+02	462.60	-1.30E+05
12	Bot Rivet Hole (right)	-22.57	23.00	-5.19E+02	462.60	-1.30E+05
Total				3.22E+04		5.39E+06

*section properties of bottom flange plate removed

Half Area	=	16101.89	mm ²
Area above PNA excluding web	=	13546.00	mm ²
Area above PNA in web	=	2555.89	mm ²
Depth of web above PNA	=	91.28	mm
Distance between top to web to top of section	=	34.40	mm
Depth of PNA from top of section	yp =	125.68	mm
Z _{pe} = ΣAy	=	5.39E+06	mm ³

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BS5400:3-2000 Cl. 9.3.7.3.1	<u>Compactness Check - Double joist compound girder</u>		
	The projection of the compression flange outstand (bfo) should not exceed:		
			$7t_{fo}\sqrt{355/\sigma_{yf}}$
	Where:		
	bfo =	the width of the outstand measured from the edge to the nearest line of rivets connecting it to the supporting part of the member	
	tfo =	the mean thickness of the outstand	
	σyf =	the minimum yield strength of the flange material	
Form AA		bfo =	65.00 mm
		tfo =	11.40 mm
		σyf =	230 N/mm²
		$7t_{fo}\sqrt{355/\sigma_{yf}}$	99.14 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 d1 ≤ 0.5dw	$28t_w\sqrt{\frac{355}{\sigma_{yw}}}$	
	For d1 > 0.5dw	the greater of	$\left\{ \begin{array}{l} \left(32 - \frac{8d_1}{d_w}\right)t_w\sqrt{\frac{355}{\sigma_{yw}}} \\ 24t_w\sqrt{\frac{355}{\sigma_{yw}}} \end{array} \right.$
	Where:		
	tw =	the thickness of the web plate	14.00 mm
	dw =	the depth of the web plate	411.00 mm
Form AA	σyw =	Minimum yeild strength of web material	230.00 N/mm²
	d1 =	depth between plastic neutral axis and top of web	177.77 mm*
		0.5dw =	205.50 mm
	*girder is symmetrical		
	Therefore the depth of the web should not exceed =		487.01 mm
	Girder is considered to be compact		

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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.

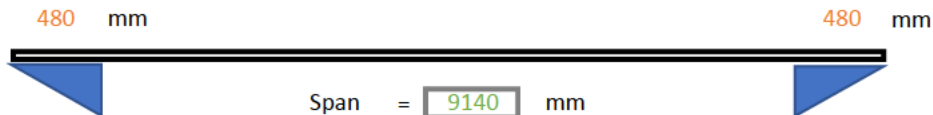
CS 454
Cl. 6.6.3

Where the support is concrete or granite, the length of the bearing area should be taken as no greater than one quarter of the depth of the member.

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

Internal / Edge Girder

Depth of the Girder = 479.80 mm



Effective Span = Span + ((1/4 Bearing Length) x 2)

Effective Span = 9379.90 mm

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Dead Loads and Superimposed Dead Loads

Unit Weights of Materials

Material	kg/m ³	kN/m ³
Steel	7850	77.01
Other solid bricks	2100	20.60
Plain concrete	2300	22.56
Bituminous macadam (tar)	2400	23.54
Miscellaneous fill	2200	21.58

Partial Factors for Actions

Structure Type	1
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Actions	ULS Partial Factor
Steel dead load	1.05
Concrete, stone, masonry or timber dead load	1.15
Surfacing superimposed dead load	1.75
Other super imposed dead loads	1.20

Double Joist Compound Unit Self Weight

Area of girder = 0.037 m²
Girder Self Weight UDL $77.01 \times 1.05 \times 0.037 = 2.996$ kN/m²

Surfacing

Depth of surfacing (assumed as top 100mm) = 0.100 m

Girder beneath the carriageway road (2nd/3rd internal girder from South)

Width of surfacing (equal to girder spacing) = 1.705 m
 Area of surfacing = 0.171 m²

Surfacing UDL $23.54 \times 1.75 \times 0.171 = 7.025$ kN/m²

1st Internal Girder from south

Width of surfacing affecting girder = 1.575 m
 Area of surfacing = 0.158 m²

Surfacing UDL $23.54 \times 1.75 \times 0.158 = 6.489$ kN/m²

4th Internal Girder from south

Width of surfacing affecting girder = 0.985 m
 Area of surfacing = 0.099 m²

Surfacing UDL $23.54 \times 1.75 \times 0.099 = 4.058$ kN/m²

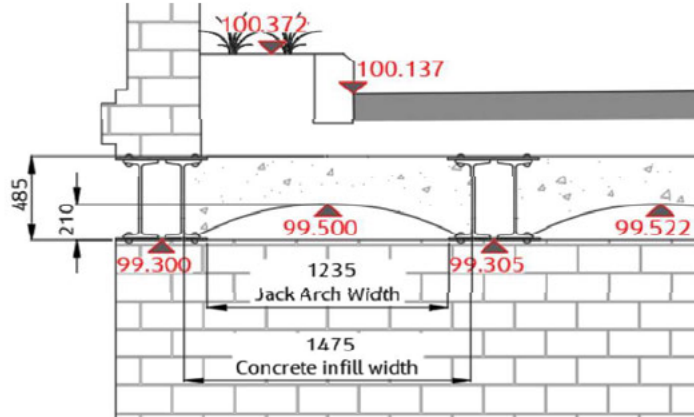
CS 454
Table 4.1.1a

CS 454
Appendix A

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Jack arch and concrete infill

* Jack arch dimensions are constant throughout the structure



Arch Barrier Thickness

$$t_{Arch} = 203 \text{ mm}$$

Arch Barrier Rise

$$a = 210 \text{ mm}$$

Jack Arch Span

$$2b = 1235 \text{ mm}$$

Jack Arch Radius

Intrados

$$R_I = (a^2 + b^2) / 2a = 1013 \text{ mm}$$

Extrados

$$R_E = R_I + t_{Arch} = 1216 \text{ mm}$$

Area of Jack Arch

Angle of Sector

$$\theta = 2 \times \sin^{-1}(b/R_I) = 75^\circ$$

Length of Extrados Chord

$$c = (R_E^2 - (R_I - a)^2)^{0.5} - b = 296 \text{ mm}$$

$$2 \times (b + c) = 1826 \text{ mm}$$

Angle of Sector, adjusted for Extrados

$$\alpha = 2 \times \sin^{-1}((b + c) / R_E) = 97^\circ$$

Area of Segment, under Intrados

$$A_I = ((\theta \times \pi) / 360) - (\sin(\theta) / 2) \times R_I^2 = 0.177 \text{ m}^2$$

Area of Segment, under Extrados

$$A_E = ((\alpha \times \pi) / 360) - (\sin(\alpha) / 2) \times R_E^2 = 0.523 \text{ m}^2$$

Total Jack Arch

$$A = 0.346 \text{ m}^2$$

Area of Concrete Infill

Maximum Thickness of concrete infill

$$= 485 \text{ mm}$$

Concrete infill width

$$= 1475 \text{ mm}$$

Area of Concrete infill

$$= 0.193 \text{ m}^2$$

UDL of Jack Arch + Concrete infill

Jack arch + Infill UDL on edge girder

$$0.5 \times ((0.346 \times 20.60 \times 1.15) + (0.193 \times 22.56 \times 1.15)) = 6.60 \text{ kN/m}$$

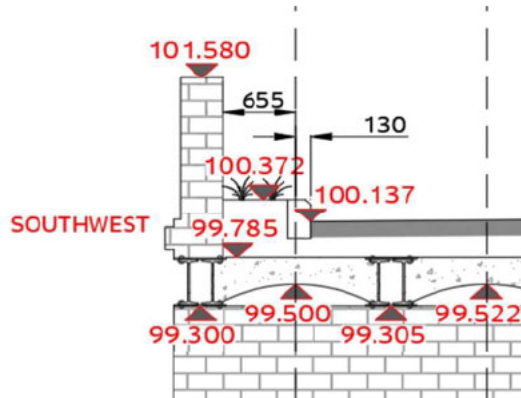
Jack arch + Infill UDL on internal girder

$$(0.346 \times 20.60 \times 1.15) + (0.193 \times 22.56 \times 1.15) = 13.19 \text{ kN/m}$$

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Miscellaneous Fill

South Edge Girder

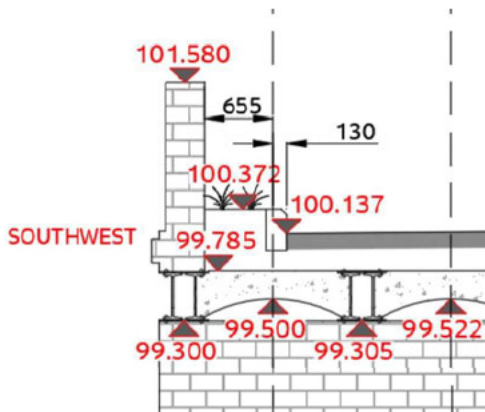


Total depth of fill = 0.587 m
Width of the fill affecting edge girder = 0.655 m

Area of Fill $0.587 \times 0.655 = 0.384 \text{ m}^2$

Miscellaneous Fill UDL $0.384 \times 21.58 \times 1.20 = 9.96 \text{ kN/m}$

1st Internal Girder from South



Spacing between girders = 1.705 m

Total depth of fill at verge = 0.587 m
Width of the fill in verge affecting girder = 0.130 m

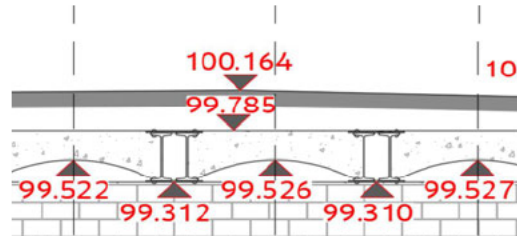
Total depth of fill under road surface = 0.252 m
Width of the fill under surfacing affecting girder = 1.575 m

Area of Fill $0.587 \times 0.130 + 0.252 \times 1.575 = 0.473 \text{ m}^2$

Miscellaneous Fill UDL $0.473 \times 21.58 \times 1.20 = 12.26 \text{ kN/m}$

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2nd/3rd Internal Girder from South

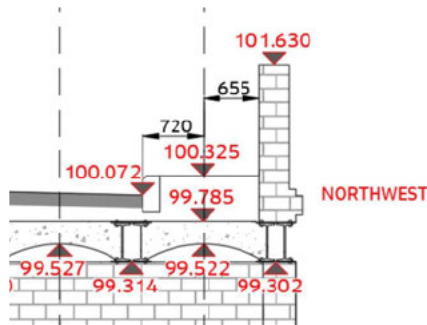


Total depth of fill = 0.279 m
Width of the fill under surfacing affecting girder (equal to girder spacing) = 1.705 m

Area of Fill $0.279 \times 1.705 = 0.476 \text{ m}^2$

Miscellaneous Fill UDL $0.476 \times 21.58 \times 1.20 = 12.32 \text{ kN/m}$

4th Internal Girder from South



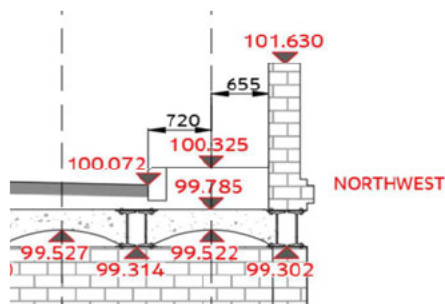
Total depth of fill under surfacing = 0.187 m
Width of the fill under surfacing affecting girder = 0.985 m

Total depth of fill @ verge = 0.540 m
Width of the fill @ verge affecting girder = 0.720 m

Area of Fill $0.540 \times 0.720 + 0.187 \times 0.985 = 0.573 \text{ m}^2$

Miscellaneous Fill UDL $0.573 \times 21.58 \times 1.20 = 14.84 \text{ kN/m}$

North Edge Girder



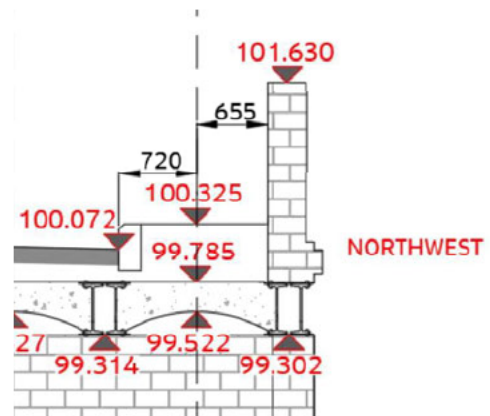
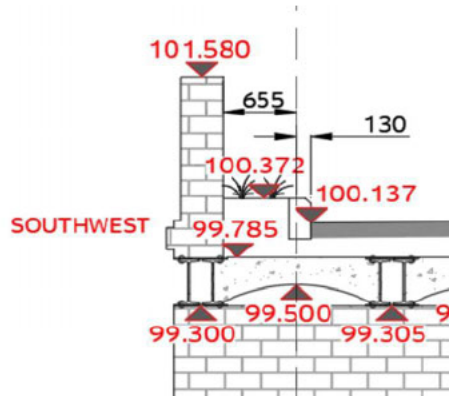
Total depth of fill = 0.540 m
Width of the fill affecting edge girder = 0.655 m

Area of Fill $0.540 \times 0.655 = 0.354 \text{ m}^2$

Miscellaneous Fill UDL $0.354 \times 21.58 \times 1.20 = 9.16 \text{ kN/m}$

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Parapet



South Parapet Height
North Parapet Height

= 1.795 m
= 1.845 m

Parapet Width

= 0.3500 m

Area of South Parapet

= 0.628 m²

Area of North Parapet

= 0.646 m²

South Parapet UDL

0.628 x 20.60 x 1.15 = 14.88 kN/m

North Parapet UDL

0.646 x 20.60 x 1.15 = 15.30 kN/m

Cumulative UDLs total affecting girder per m

South Edge Girder

3.00 + 6.60 + 9.96 + 14.88 = 34.43 kN/m

1st Internal Girder from South

3.00 + 6.49 + 13.19 + 12.26 = 34.93 kN/m

2nd/3rd Internal Girder from South

3.00 + 7.02 + 13.19 + 12.32 = 35.53 kN/m

4th Internal Girder from South

3.00 + 4.06 + 13.19 + 14.84 = 35.09 kN/m

North Edge Girder

3.00 + 6.60 + 9.16 + 15.30 = 34.05 kN/m

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Dead Load Effects

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 $V_{max} = \frac{wl}{2}$

South Edge Girder UDL

Dead Load + Superimposed Dead Load = 34.43 kN/m

MDL = 378.69 kNm

VDL = 161.49 kN



1st Internal Girder from South

Dead Load + Superimposed Dead Load = 34.93 kN/m

MDL = 384.19 kNm

VDL = 163.84 kN

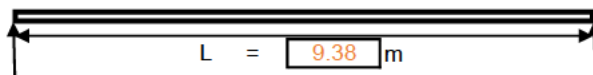


2nd/3rd Internal Girder from South

Dead Load + Superimposed Dead Load = 35.53 kN/m

MDL = 390.79 kNm

VDL = 166.65 kN



4th Internal Girder from South

Dead Load + Superimposed Dead Load = 35.09 kN/m

MDL = 385.88 kNm

VDL = 164.55 kN



North Edge Girder

Dead Load + Superimposed Dead Load = 34.05 kN/m

MDL = 374.49 kNm

VDL = 159.70 kN



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Live Loads - Internal Girders

The internal 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.

CS 454
Table 5.18

The carriageway width shall be divided into a number of notional lanes

Total Carriageway Width = 5.969 m

Number of notional lanes used in assessment n = 2

CS 454
Cl. 5.19

The UDL and KEL, determined in accordance with Table 5.19a, will modified by the following factors:

Lane Factor L_{FACTOR} = 1

K Factor (40 tonne loading) K = 0.88

*Surface condition taken as 'Poor', Traffic flow taken as 'low'

Safety Factor

CS 454
Table 3.4 /
Appendix A

Actions	ULS Partial Factor
Traffic actions for normal traffic and restricted traffic	1.50

CS 454
Table 5.19a

The UDL and KEL applied in each are derived by the table 5.19a, see below:

Load Length, L (m)	UDL(kN/m)	KEL (kN)
L ≤ 20m	230 / L ^{0.67}	82

Load Length (Effective Span of Internal Girder) L = 9.38 m

Uniformly Distributed Load UDL = 51.33 kN/m
Knife-edge load KEL = 82.00 kN

Proportion Factor

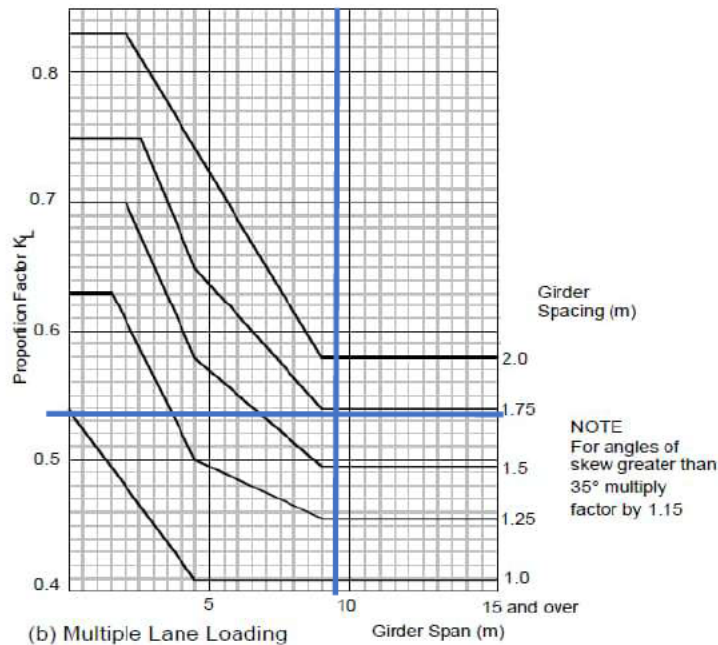
BA16/97
Cl. 2.6

The nominal live load bending moment applied to an internal girder under a traffic lane can be obtained by multiplying the gross moment due to the effects of the live load from one notional lane of width 2.5m by the appropriate factor from figure Figure 2/2.

Girder Spacing = 1.71 m

Proportion Factor K_L = 0.535

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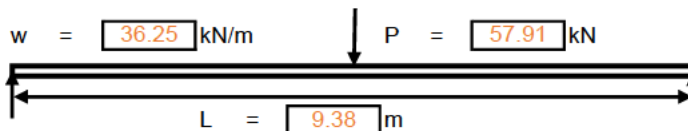


Applied Load

Applied UDL (40 tonnes loading) $1.00 \times 0.88 \times 1.5 \times 51.33 \times 0.54 = 36.25$ kN/m

Applied KEL (40 tonnes loading) $1.00 \times 0.88 \times 1.5 \times 82.00 \times 0.54 = 57.91$ kN

Bending, at midspan



Maximum Bending moment $(wL^2/8) + (PL/4) = 534.44$ kNm

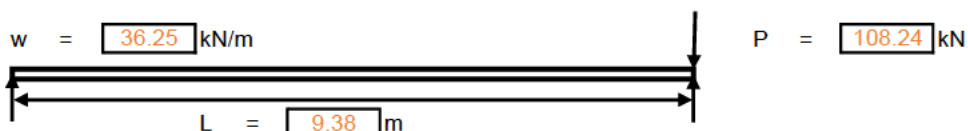
As the skew angle of the structure is greater than 35° an additional factor of 1.15 will also be applied

Maximum Bending moment = 614.61 kNm

Shear, at Support

The proportion fraction is not considered for the KEL in shear. Therefore the applied KEL will be reduced by a factor of 1/0.535

Therefore KEL would be equal to 108.24 kN with the proportion factor removed



Maximum Shear $(wL/2) + (P/2) = 224.12$ kNm

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Accidental Vehicle Loading

The edge girders will be checked for accidental vehicle loading on the verges.

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 low traffic flow.

CS 454
Cl. 5.27.4

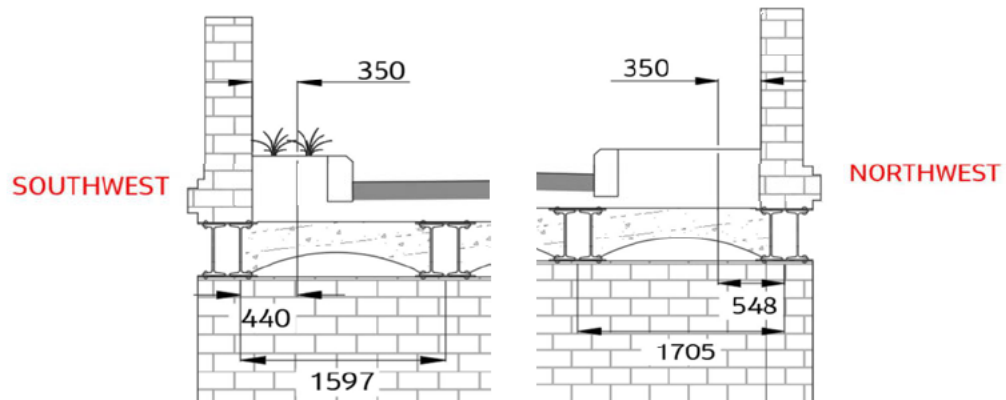
CS 454
Table 5.9a
Table 5.9b

CS 454
Appendix A

Impact factor	*Only effects maximum axel load	=	1.8
Traffic flow factor		=	0.9

Actions	ULS Partial Factor
Traffic actions for normal traffic and restricted traffic	1.50

Consider the arrangment below



South Edge Gider is considered to carry 72.45% of a single wheel load
*Contribution from external girder discounted

North Edge Gider is considered to carry 67.86% of a single wheel load

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CS 454
Appendix B

Vehicle load models

Normal traffic loading

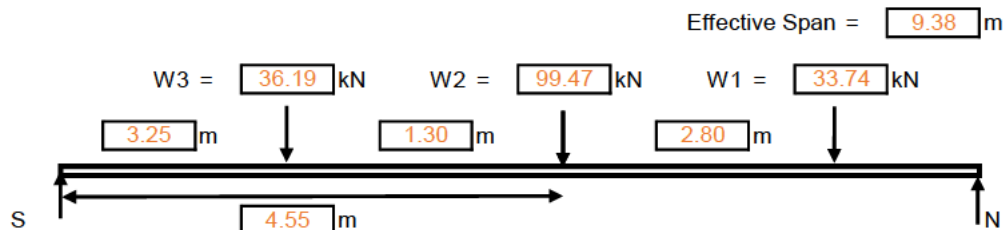
Vehicle Ref	=	H	W1	W2	W3	W4	W5	W6
Nominal Axle Load (kN)			69	113	74	88	88	0
Wheel Load (0.5 x AL x TFF x PF)			46.575	76.275	49.95	59.4	59.4	0
Wheel Load (including impact)			46.575	137.3	49.95	59.4	59.4	0
Spacing (m)			2.8	1.3	7.6	1.35	0	

*W2 and W3 are interchangeable for worst case senerio

	W1	W2	W3	W4	W5	W6
Wheel Load, on S edge girder (kN)	33.743	99.468	36.188	43.034	43.034	0
Wheel Load, on N edge girder (kN)	31.605	93.167	33.896	40.308	40.308	0

Bending moment - South Edge Girder

The worst case bending moment occurs when the W1 is positioned 7.354 m from the support in the direction of travel



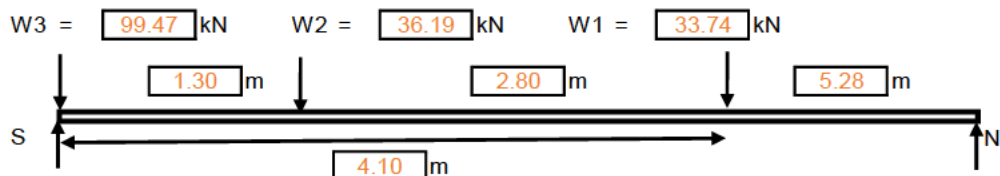
$$\begin{aligned} \text{Reaction at N} &= ((3.25 \times 36.19) + (4.55 \times 99.47) + (7.35 \times 33.74)) / 9.38 = 87.30 \text{ kN} \\ \text{Reaction at S} &= 82.10 \text{ kN} \end{aligned}$$

Maximum Bending moment occurs at 4.55 m from the support in the direction of travel
(Or at W2 position)

$$\text{Maximum Bending Moment} = 4.83 \times 87.30 - 2.80 \times 33.74 = 326.83 \text{ kNm}$$

Shear Force

The worst case shear force occurs when the W1 is positioned 4.101 m from the support in the direction of travel and the critical load is on the W3 axle

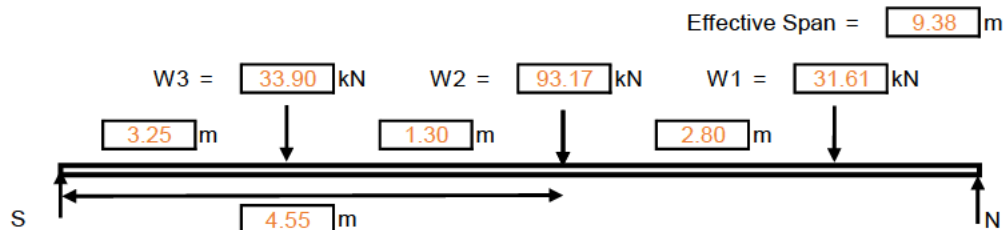


$$\text{Reaction at S} = ((5.28 \times 33.74) + (8.08 \times 36.19) + (9.38 \times 99.47)) / 9.38 = 149.62 \text{ kN}$$

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Bending moment - North Edge Girder

The worst case bending moment occurs when the W1 is positioned **7.354 m** from the support in the direction of travel



Reaction at N $((3.25 \times 33.90) + (4.55 \times 93.17) + (7.35 \times 31.61)) / 9.38 =$ **81.77** kN

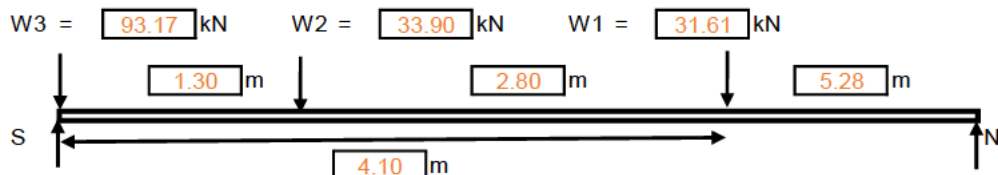
Reaction at S $=$ **76.90** kN

Maximum Bending moment occurs at **4.55 m** from the support in the direction of travel
(Or at W2 position)

Maximum Bending Moment $4.83 \times 81.77 - 2.80 \times 31.61 =$ **306.13** kNm

Shear Force

The worst case shear force occurs when the W1 is positioned **4.101 m** from the support in the direction of travel and the critical load is on the **W3** axle



Reaction at S $((5.28 \times 31.61) + (8.08 \times 33.90) + (9.38 \times 93.17)) / 9.38 =$ **140.14** kN

Reduced Live Loading - South Edge Girder

The southern edge girder has failed to support the loading induced by 44/40t of live loading. Additional calculation has been undertaken to determine the reduced live load capacity.

After accounting for the moments induced by dead loads, there is only a residual capacity of 24.85 kNm.

RLL - 3 tonnes Bending moment

Vehicle Ref = O	W1	W2	W3	W4	W5	W6
Nominal Axle Load (kN)	21	9	0	0	0	0
Wheel Load (0.5 x AL x Tff x PF)	14.175	6.075	0	0	0	0
Wheel Load (including impact)	25.515	6.075	0	0	0	0
Spacing (m)	2	0	0	0	0	0

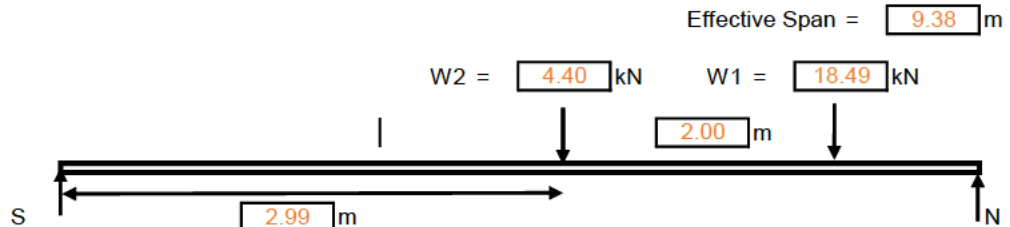
*W2 and W3 are interchangeable for worst case senerio

	W1	W2	W3	W4	W5	W6
Wheel Load, on S edge girder (kN)	18.485	4.4012	0	0	0	0

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The worst case bending moment occurs when the W1 is positioned from the support in the direction of travel

4.99 m



Reaction at N $((2.99 \times 4.40) + (4.99 \times 18.49)) / 9.38 = 11.24$ kN

Reaction at S = 11.65 kN

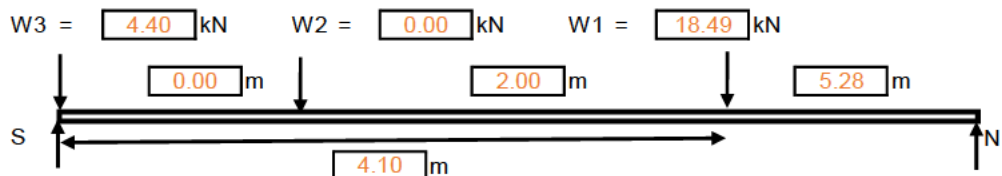
Maximum Bending moment occurs at 4.99 m from the support in the direction of travel (Or at W1 position)

Maximum Bending Moment $6.39 \times 11.24 - 2.00 \times 18.49 = 49.33$ kNm

Shear Force

The worst case shear force occurs when the W1 is positioned in the direction of travel and the critical load is on the W3 axle

4.101 m



Reaction at S $((5.28 \times 18.49) + (7.28 \times 0.00) + (7.28 \times 4.40)) / 9.38 = 13.82$ kN

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BS 5400-3
CL9.9.1.2

CS 456
CI 4.3.3 Table 2
CS 454 CI.3.9

CS 456
CI 9.8

Form AA

Internal Girders

Bending Capacity

The bending resistance of a beam should be taken as: $M_D = \frac{M_R}{\gamma_m \gamma_{f3}}$

γ_m = partial factor for material strength = 1.05
 γ_{f3} = partial load factor = 1.10
 M_R = limiting moment of resistance

M_R is obtained from Figure 11 b) in CS 456 using the value of to provide the ratio of M_R/M_{ult}

$$\lambda_{LT} \sqrt{\frac{\sigma_{yc} M_{ult}}{355 M_{pe}}}$$

Where

σ_{yc} = 230.00 N/mm²

Mult = moment of resistance of cross section if lateral torsional buckling is prevented
 M_{pe} for compact sections
or the least of $Z_{xc}\sigma_{yc}$, $Z_{xt}\sigma_{yt}$ or $Z_{pe}\sigma_y$ for non compact sections

		Midspan - Internal Girder	
		Uncorroded	Corroded
Z_{xc}	(mm ³)	6.25E+06	5.95E+06
Z_{xt}	(mm ³)	5.78E+06	4.04E+06
Z_{pe}	(mm ³)	6.88E+06	5.39E+06
$Z_{xc}\sigma_{yc}$	(kNm)	1.44E+03	1.37E+03
$Z_{xt}\sigma_{yt}$	(kNm)	1.33E+03	9.29E+02
$Z_{pe}\sigma_y$	(kNm)	1.58E+03	1.24E+03

As the girders are condered to be Compact Mult will therefore equal

		Midspan - Internal Girder	
		Uncorroded	Corroded
Mult	(kNm)	1.58E+03	1.24E+03

BS 5400-3
CI 9.7.2

Slenderness parameter, taken as:

$$\lambda_{LT} = \frac{l_e}{r_y} k_4 \eta \nu$$

Concrete backed jack arches span between the internal girders and act to restrain the girders.
Therefore, the effective length for lateral torsional buckling, l_e , will be equal to 0.

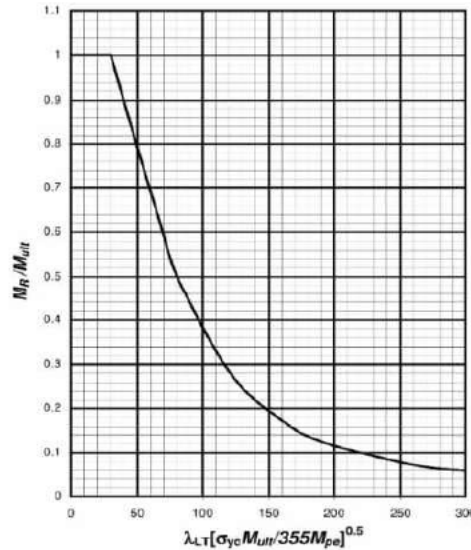
As l_e = 0 m

Therefore λ_{LT} = 0 m and $\lambda_{LT} \sqrt{\frac{\sigma_{yc} M_{ult}}{355 M_{pe}}} = 0$ m

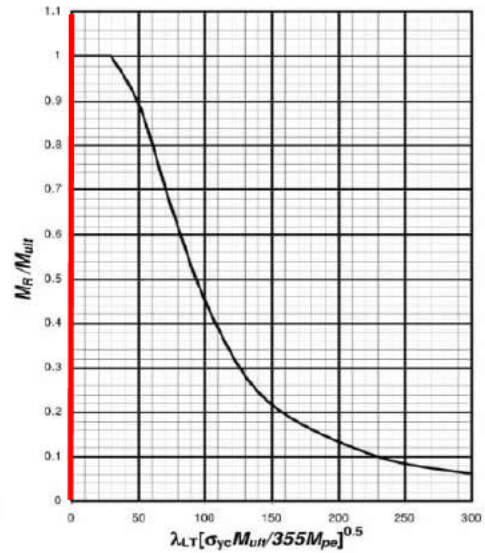
CS456
9.8

From Figure 11 M_R/M_{ult} = 1.0

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a) Beams fabricated by welding



b) All other sections

Note Expressions for replacement curves are given in Annex G.

Figure 11 – Limiting moment of resistance, M_R

Therefore, M_R , will be equal to

		Midspan - Internal Girder	
		Uncorroded	Corroded
Mult	(kNm)	1582.03	1240.71
M_R	(kNm)	1582.03	1240.71
M_D	(kNm)	1369.72	1074.21

The imposed load on the structure will be equal to:

Imposed load		Internal Girder
Live load moment	(kNm)	614.61
Dead load	(kNm)	390.79
Total Moment	(kNm)	1005.40

(Maximum dead load occurs on the 2nd/3rd internal girders)

The internal girders marginally pass to support the bending moment induced by normal traffic loading.

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BS 5400-3
Cl. 9.9.2.2

Form AA

Shear Capacity

Shear resistance of a girder $V_D = \left[\frac{t_w(d_w - h_h)}{\gamma_m \gamma_{f3}} \right] \tau_l$

γ_m	=	partial factor for material strength	=	1.05	
γ_{f3}	=	partial load factor	=	1.10	
t_w	=	thickness of web	=	14.00	mm
d_w	=	the depth of the web panel	=	411.00	mm
d_{we}	=	depth of the web panel between flange plates	=	368.30	mm

d_{we} has been taken to be the distance between the fillets

h_h	=	height of largest hole or cut out	=	0.00	mm
σ_{yw}	=	minimum yield strength of web material	=	230.00	N/mm ²
τ_l	=	limit shear strength of web panel			

Limiting shear strength of web panel is found from BS 5400-3 Figures 12-18. corresponding to values of t_y , Ψ , m_{fw} and the slenderness ratio λ .

τ_y	=	Shear strength	$\sigma_{yw} / \sqrt{3} =$	132.79	N/mm ²
φ	=	Aspect ratio of panel	$a/d_{we} =$	0.00	
a	=	the clear length of panel between transverse stiffeners			
	=	0 mm			No transverse stiffeners were present

The value of m_{fw} is determined through the following equation

$$m_{fw} = \frac{\sigma_{yf} b_{fe} t_f^2}{2 \sigma_{yw} d_{we}^2 t_w}$$

σ_{yf}	=	minimum yield strength of flange material	=	230.00	
t_f	=	flange plate thickness	=	11.40	
b_{fe}	=	the smallest of:			

- $10 t_f \sqrt{355 / \sigma_{yf}} = 141.63$ mm
- the distance between the mid-plane of the web to the nearer edge of the flange. i.e. =0 where there is no flange outstand.
= 184.15 mm taken as half d_{we}
- if there are two or more webs, half the clear distance between webs;
clear distance between webs is 216.00 mm
half clear distance is 108 mm

$m_{fw} = 0.0037$ Therefore use Figure 13 for $m_{fw} = 0.005$

The slenderness ratio is equal to:

$\lambda =$ Slenderness ratio $(d_{we}/t_w) \cdot \sqrt{(\sigma_{yw}/355)} = 21.18$

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Therefore the ratio of T_l / T_y is equal to :

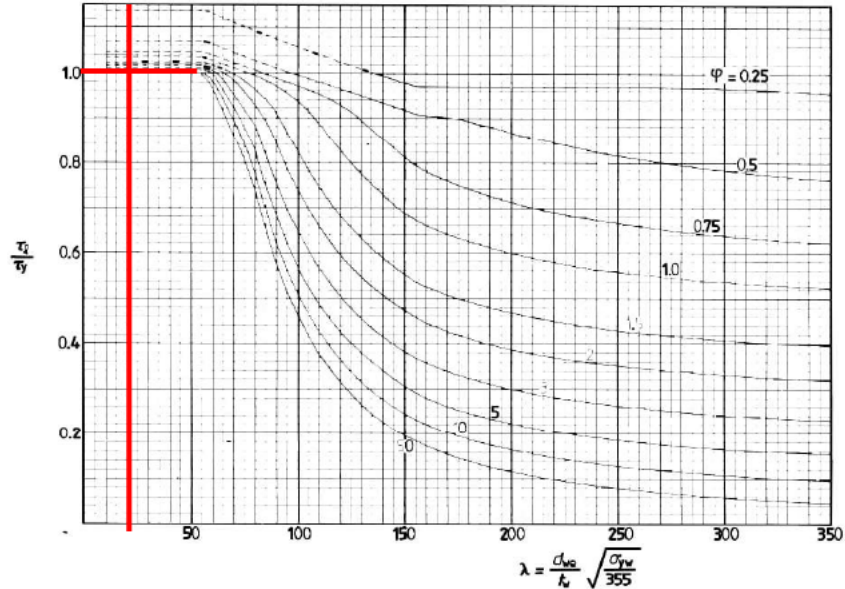


Figure 13 — Limiting shear strength τ_l for $m_{fw} = 0.005$

As the slenderness ratio is below 50, and the aspect ratio is 0; T_l / T_y is assumed to be equal to 1

$$\begin{aligned} T_l / T_y &= 1 \\ T_l &= 132.79 \text{ N/mm}^2 \\ V_D &= 661.54 \text{ kN} \end{aligned}$$

Therefore the Shear resistance will be equal to:

Imposed load		Internal Girder
Live load	(kN)	224.12
Dead load	(kN)	166.65
Total Load	(kN)	390.77

(Maximum dead load occurs on the 2nd/3rd internal girders)

Internal Girders pass for the shear induced by normal traffic loading

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BS 5400-3
CL9.9.1.2

CS 456
Table 2
Cl.3.9

CS 456
Cl.9.8

Form AA

Edge Girder

Bending Capacity

The bending resistance of a beam should be taken as: $M_D = \frac{M_R}{\gamma_m \gamma_{f3}}$

γ_m = partial factor for material strength = 1.05
 γ_{f3} = partial load factor = 1.10
 M_R = limiting moment of resistance

M_R is obtained from Figure 11 b) in CS 456 using the value of to provide the ratio of M_R/M_{ult}

$$\lambda_{LT} \sqrt{\frac{\sigma_{yc} M_{ult}}{355 M_{pe}}}$$

Where

σ_{yc} = 230.00 N/mm²

M_{ult} = moment of resistance of cross section if lateral torsional buckling is prevented
 M_{pe} for compact sections
or the least of $Z_{xc}\sigma_{yc}$, $Z_{xt}\sigma_{yt}$ or $Z_{pe}\sigma_{y}$ for non compact sections

		Midspan - Edge Girder		
		Uncorroded	South Corroded	North Corroded
Z_{xc}	(mm ³)	6.25E+06	2.94E+06	5.73E+06
Z_{xt}	(mm ³)	5.78E+06	2.32E+06	4.49E+06
Z_{pe}	(mm ³)	6.88E+06	3.12E+06	5.75E+06
$Z_{xc}\sigma_{yc}$	(kNm)	1.44E+03	6.76E+02	1.32E+03
$Z_{xt}\sigma_{yt}$	(kNm)	1.33E+03	5.35E+02	1.03E+03
$Z_{pe}\sigma_y$	(kNm)	1.58E+03	7.17E+02	1.32E+03

As the girders are condered to be Compact M_{ult} will therefore equal

		Midspan - Edge Girder		
		Uncorroded	South Corroded	North Corroded
M_{ult}	(kNm)	1.58E+03	7.17E+02	1.32E+03

BS 5400-3
Cl. 9.7.2

Slenderness parameter, taken as:

$$\lambda_{LT} = \frac{l_e}{r_y} k_4 \eta \nu$$

Cl.9.6.2 a)

Where the effective length l_e , is:

$$l_e = k_1 k_2 k_3 L$$

L = The effective span of the beam = 9.38 m

k_1 = 1.00
 k_2 = 1.0
 k_e = 1.0

assumed for a level 1 assessment

Therefore l_e = 9.38 m

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CS 456
Cl. 9.7.2

$$k_4 = \left[\frac{4Z_{pe}^2 \left(1 - \frac{I_y}{I_x} \right)}{A^2 h^2} \right]^{0.25} \quad \text{for flanged beams symmetrical about the minor axis}$$

$$k_4 = \left[\frac{I_y Z_{pe}^2 \left(1 - \frac{I_y}{I_x} \right)}{A^2 C_w} \right]^{0.25} \quad \text{for flanged beams symmetrical about the major axis}$$

$$k_4 = 1.0 \quad \text{for all other beams}$$

Girders are symmetrical in both axes, therefore use k_4 about the minor axis

Uncorroded Section

North EG Midspan Section

South ED Midspan Section

$I_y =$	5.7E+08	mm ⁴	$I_y =$	4.7E+08	mm ⁴	$I_y =$	3.9E+07	mm ⁴
$I_x =$	1.4E+09	mm ⁴	$I_x =$	1.2E+09	mm ⁴	$I_x =$	6.2E+08	mm ⁴
$A =$	3.7E+04	mm ²	$A =$	3.1E+04	mm ²	$A =$	1.7E+04	mm ²

Distance between centroids of two flanges

$$h = \text{distance between the centroids of the two flanges} = 440.80 \text{ mm}$$

Centroid of top flange

Section	B mm	H mm	A mm ²	y mm	A.y mm ³
Top flange Plate	470.00	11.40	5.36E+03	5.70	3.05E+04
RSJ T Flange (Left)	178.00	23.00	4.09E+03	22.90	9.38E+04
RSJ T Flange (Right)	178.00	23.00	4.09E+03	22.90	9.38E+04
			1.35E+04		2.18E+05

$$\text{Location of centroid in top flange} = 16.10 \text{ mm}$$

Centroid of bottom flange (including corrosion on first internal girder)

Section	B mm	H mm	A mm ²	y mm	A.y mm ³
RSJ B. Flange (Left)	178.00	23.00	4.09E+03	456.90	1.87E+06
RSJ B. Flange (Right)	178.00	23.00	4.09E+03	456.90	1.87E+06
Bottom Flange Plate	470.00	11.40	5.36E+03	474.10	2.54E+06
			8.19E+03		3.74E+06

$$\text{Location of centroid in top flange} = 456.90 \text{ mm}$$

Note: The distance between the centroids will be minimally changed between the uncorroded and corroded sections. For the purpose of the assessment, the distance is assumed to remain the same. Therefore:

$$k_4 \text{ for uncorroded section will be equal to } k_{4, \text{ uncorroded}} = 0.81$$

$$k_4 \text{ for south midspan corroded section will be equal to } k_{4, \text{ S EG corroded}} = 0.91$$

$$k_4 \text{ for north midspan corroded section will be equal to } k_{4, \text{ N EG corroded}} = 0.81$$

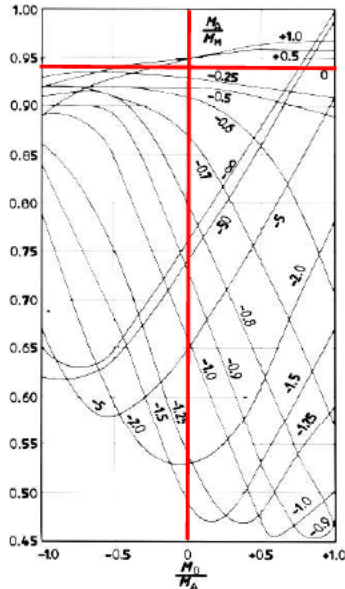
$\eta = 1.0$, but where the bending moment varies substantially within the half-wavelength of buckling of the compression flange, advantage may be obtained by using η from Figure 10a) or 10b)

As the girders are simply supported, the moments at the supports are zero.

MB/MA	=	0
MB/MM	=	0

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BS 5400-3
Figure 10



$$\eta = 0.94$$

BS 5400-3
Cl. 9.7.2

v is dependent on the shape of the beam and can be obtained from Table 9, using the parameters:

$$\lambda_F = \frac{l_w}{r_y} \cdot \frac{t_f}{D} \quad i = \frac{I_c}{I_c + I_t}$$

BS 5400-3
Cl. 9.7.1a

l_w = half-wavelength of buckling of the compression flange. (equal to l_e per Cl.9.7.1 a))

$$= 9380 \text{ mm}$$

t_f = mean thickness of the two flanges

$$= 34.40 \text{ mm}$$

D = depth of cross section.

$$= 479.80 \text{ mm}$$

r_y = radius of gyration

	Uncorroded	South Corroded	North Corroded
r_y	121.08	47.37	120.14

All units in mm

	Uncorroded	South Corroded	North Corroded
λ_F	5.55	14.20	5.60

Girders are symmetrical, therefore $i = 0.5$

BS 5400-3
Table 9

$v = 0.793$ For uncorroded and north corroded section

$v = 0.549$ For South corroded section

BS 5400-3
Cl. 9.7.2

Slenderness parameter, taken as:

$$\lambda_{LT} = \frac{l_e}{r_y} k_4 \eta v$$

For uncorroded and north corroded section

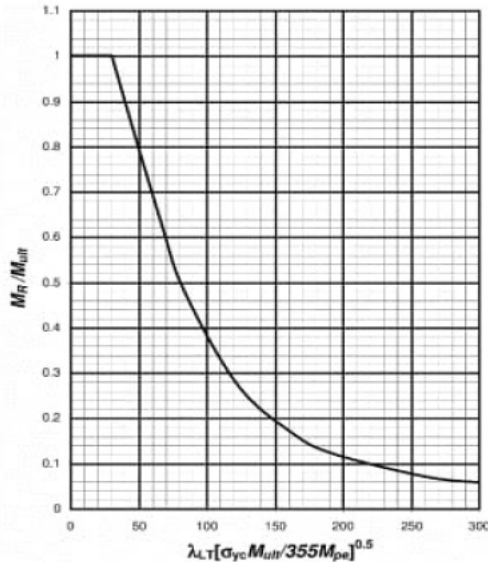
$$\lambda_{LT} = 46.788$$

For South corroded section

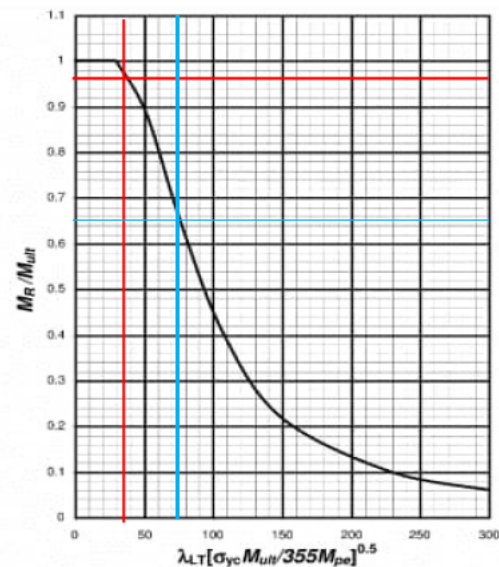
$$\lambda_{LT} = 92.734$$

JACOBS			CALCULATION SHEET		
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CS456 Cl.9.8	The limiting moment of resistance, MR, shall be obtained from Figure 11a) for beams fabricated by welding (excluding local welding of stiffeners) or Figure b) for all other sections (including stress relieved welded sections) according to the value of: $\lambda_{LT} \sqrt{\frac{\sigma_{yc} M_{ult}}{355 M_{pe}}}$ 37.660 74.643 which is equal to For uncorroded and north corroded section For South corroded section				

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a) Beams fabricated by welding



b) All other sections

The ratio of M_R/M_{ULT} is therefore:

$M_R/M_{ULT} =$	0.960	For uncorroded and north corroded section
	0.650	For South corroded section

Therefore, M_R , will be equal to

		Midspan - Edge Girder		
		Uncorroded	South	North
Mult	(kNm)	1582.03	717.06	1322.78
M_R	(kNm)	1518.75	466.09	1269.87
M_D	(kNm)	1314.93	403.54	1099.45

The imposed load on the structure will be equal to:

		Midspan - Edge Girder		
Imposed load		Uncorroded*	South	North
Live load moment	(kNm)	326.83	326.83	306.13
Dead load	(kNm)	378.69	378.69	374.49
Total Moment	(kNm)	705.52	705.52	680.61

*maximum live load occurs on south

The uncorroded section and the north edge girder both pass for 40/44t accidental live loading.

Due to the corrosion the southern edge girder has sustained the section no longer able to carry 40/44t accidental live loading. After accounting for the dead loads, there is only a residual capacity of 24.85 kNm to support live loading.

Additionally, a reduced live loading of a 3-tonne GVW was also considered but the south edge girder is not able to support the resultant maximum bending moment of 49.33 kNm, therefore the girder is only able to support the dead load

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BS 5400-3
Cl. 9.9.2.2

Form AA

Shear Capacity

Shear resistance of a girder $V_D = \left[\frac{t_w (d_w - h_h)}{\gamma_m \gamma_{f3}} \right] \tau_l$

γ_m	=	partial factor for material strength	=	1.05	
γ_{f3}	=	partial load factor	=	1.10	
t_w	=	thickness of web	=	14.00	mm
d_w	=	the depth of the web panel	=	411.00	mm
d_{we}	=	depth of the web panel between angles	=	368.30	mm

d_{we} has been taken to be the distance between the fillets

h_h	=	height of largest hole or cut out	=	0.00	mm
σ_{yw}	=	minimum yield strength of web material	=	230.00	N/mm ²
τ_l	=	limit shear strength of web panel			

Limiting shear strength of web panel is found from BS 5400-3 Figures 12-18. corresponding to values of t_y , ϕ , m_{fw} and the slenderness ratio λ . The limiting shear strength is dependent on:

τ_y	=	Shear strength	$\sigma_{yw} / \sqrt{3} =$	132.79	N/mm ²
λ	=	Slenderness ratio	$(d_{we}/t_w) \cdot \sqrt{(\sigma_{yw}/355)} =$	21.18	
ϕ	=	Aspect ratio of panel	$a/d_{we} =$	0.00	
a	=	the clear length of panel between transverse stiffeners			
	=	0 mm			No transverse stiffeners were present

The value of m_{fw} is needed to determine the graph to be used to find τ_l/τ_y

$$m_{fw} = \frac{\sigma_{yf} b_{fe} t_f^2}{2 \sigma_{yw} d_{we}^2 t_w}$$

σ_{yf}	=	minimum yield strength of flange material	=	230.00	
t_f	=	flange plate thickness	=	11.40	
b_{fe}	=	the smallest of:			

a) $10 t_f \sqrt{355/\sigma_{yf}} = 141.63$ mm

b) the distance between the mid-plane of the web to the nearer edge of the flange. i.e. =0 where there is no flange outstand.
= 184.15 mm taken as half d_{we}

$m_{fw} = 0.0048$ Therefore use Figure 13 for $m_{fw} = 0.005$

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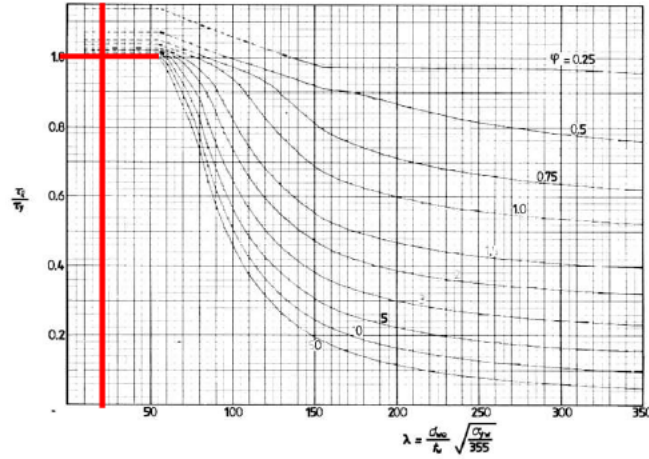


Figure 13 — Limiting shear strength τ_l for $m_{fw} = 0.005$

As the slenderness ratio is below 50, and the aspect ratio is 0; T_l/T_y is assumed to be equal to 1

$$\begin{aligned} T_l/T_y &= 1 \\ T_l &= 132.79 \text{ N/mm}^2 \end{aligned}$$

Therefore the Shear resistance will be equal to:

$$V_D = 661.54 \text{ kN}$$

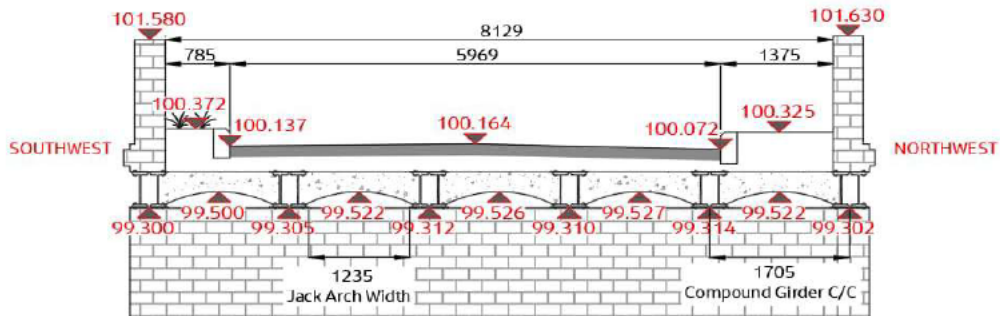
Imposed load		Edge Girder
Live load (normal traffic)	(kN)	149.62
Dead load	(kN)	161.49
Total Load	(kN)	311.11

Girders passes for normal traffic loading

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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?				1.235	m	Yes
*Non-compliant if greater than 2.0m						
Do jack arches spring from bottom flanges of beams?				Yes		Yes
*If not, non-compliant						
What is the beam spacing?	(web - web) b	=	1.705	m	Yes	
What is the rise of the arch?	r _c	=	0.21	m		
Gross aspect ratio?	b/r _c	=	8.12			
*Non-compliant if b/r _c greater than 10						
What is the arch barrel thickness?	d	=	275	mm	Yes	
(include concrete fill above)						
How is thickness derived? » From previous assessment calculations						
*Non-compliant if thickness is less than 220mm						

SECTION 2 Check for Deficiency			Pass/ Fail
Type No.	Deficiency		
1	What is the backing material? Is it structural? > Concrete - yes (based on log of trial pit) Does the structural backing extend to at least the crown level of the arch extrados? > Yes *If not, fail What is effective shear depth of deck? D_s = > arch rise + barrel thickness + height of structural fill above crown of extrados fill above crown of extrados Is D_s ≥ minimum requirement Fig1. Fig1. Min = *Fail if D_s < Fig 1	664 mm 210 mm 275 mm 179 mm 376.4 mm	Pass
2	Do jack arches span longitudinally or transversely between longitudinal girders? > Transversely For Longitunal spanning jack arches, ignore following questions onties/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

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SECTION 2 Check for Deficiency (continued)					
Type No.	Deficiency			Pass/ Fail	
3a CI	What is the cross sectional area of one tie? (Allowing for corrosion losses)	A =	-	mm ²	N/A
	What is the number of ties per beam length?	n =	-		
	What is the clear skew span?	L =	-	m	
	Specific area of tie (A _s) = ((n+1) x A) / L	A _s =	N/A	mm ² /m	
	*Non-compliant if less than 260mm ² /m				
3b WI/ST	What is the cross sectional area of one tie? (Allowing for corrosion losses)	A =	804.2477	mm ²	Pass
	What is the number of ties per beam length?	n =	3		
	What is the clear skew span?	L =	9.14	m	
	Specific area of tie (A _s) = ((n+1) x A) / L	A _s =	351.97	mm ² /m	
	*Non-compliant if less than 260mm ² /m				
4	What is the maximum tie spacing?	S =	-	m	N/A
	*Non compliant if greater than 3.0m for cast iron				
4	Are ties located within crown of external arch?		No		Pass
	*If so, then fail CI or possible fail for wrought iron/steel				
5	Does external bay construction provide alternative lateral restraint? (ie not soft edge)				N/A
	*If so, pass *If not, are ties provided in first Jack Arch bay?				
5	*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	
7	Horizontal Displacement of supporting beam	Fail	Fail	Pass	
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	
9	Transversely bowed bottom flange of supporting beam	Fail	Fail	Pass	
10	Cracking at crown of arch owing to spreading of springings (other than 12, 13)	Fail	Fail	Pass	
11	Distortion and any associated cracking of jack arch barrel	Fail	Fail	Pass	
12	Arch crack resulting in substructure crack	Fail	Fail ⁽⁵⁾	Pass	
13	Substructure crack or other distress resulting in crack to jack arch	Possible fail ⁽³⁾	Possible Fail ^{(3) (5)}	Pass	

Notes

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

(5) Not applicable in general to longitudinally spanning arches

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Capacity Factors

The north edge girder and internal girders have passed for normal traffic loading. The capacity factor has also to be determined using the maximum bending capacity as the limiting factor.

As the southern edge girder fails for all live loading scenarios, the capacity factor has not been determined.

North Edge Girder

The maximum bending moment imposed by the dead load effects is MDL = 374.49 kNm
The bending resistance of a corroded internal girder was derived to be M_D = 1099.45 kNm

Therefore the maximum allowable bending moment that can be induced by live loading is:

$$1,099.45 - 374.49 = 724.97 \text{ kNm}$$

*Live loading induced by normal traffic loading is equal to = 306.13 kNm

Therefore the maximum C Factor allowed would be:

$$C_{Factor} = \frac{M_{Maximum \text{ allowable for live load}}}{M_{Applied}} \quad 724.97 / 306.13 = 2.37$$

For the Internal

The maximum bending moment imposed by the dead load effects is MDL = 390.79 kNm
The bending resistance of a corroded internal girder was derived to be M_D = 1074.21 kNm

Therefore the maximum allowable bending moment that can be induced by live loading is:

$$1,074.21 - 390.79 = 683.42 \text{ kNm}$$

*Live loading induced by normal traffic loading is equal to = 698.42 kNm

*K Factor has been removed

Therefore the maximum C Factor allowed would be:

$$C_{Factor} = \frac{M_{Maximum \text{ allowable for live load}}}{M_{Applied}} \quad 683.42 / 306.13 = 0.98$$

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From Figures 5.19a to 5.19f, the acceptable K Factor for each loading category is as follows:

From Figure 5.19a for high traffic flow and poor surface, the K factors are:

	K Factor	Pass/Fail
Normal Traffic	0.91	Pass
26 Tonne	0.86	Pass
18 Tonne	0.72	Pass
Group 1 FE	0.47	Pass
7.5 tonne	0.43	Pass
Group 2 FE	0.25	Pass
3 tonne	0.20	Pass

The Internal Girders are suitable for Normal Traffic GVW.

From Figure 5.19b for medium traffic flow and poor surface, the K factors are:

	K Factor	Pass/Fail
Normal Traffic	0.90	Pass
26 Tonne	0.85	Pass
18 Tonne	0.70	Pass
Group 1 FE	0.47	Pass
7.5 tonne	0.43	Pass
Group 2 FE	0.25	Pass
3 tonne	0.20	Pass

The Internal Girders are suitable for Normal Traffic GVW.

From Figure 5.19c for low traffic flow and poor surface, the K factors are:

	K Factor	Pass/Fail
Normal Traffic	0.88	Pass
26 Tonne	0.82	Pass
18 Tonne	0.66	Pass
Group 1 FE	0.47	Pass
7.5 tonne	0.40	Pass
Group 2 FE	0.25	Pass
3 tonne	0.20	Pass

The Internal Girders are suitable for Normal Traffic GVW.

From Figure 5.19d for high traffic flow and good surface, the K factors are:

	K Factor	Pass/Fail
Normal Traffic	0.81	Pass
26 Tonne	0.76	Pass
18 Tonne	0.63	Pass
Group 1 FE	0.47	Pass
7.5 tonne	0.37	Pass
Group 2 FE	0.25	Pass
3 tonne	0.20	Pass

The Internal Girders are suitable for Normal Traffic GVW.

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From Figure 5.19e for medium traffic flow and good surface, the K factors are:

	K Factor	Pass/Fail
Normal Traffic	0.79	Pass
26 Tonne	0.75	Pass
18 Tonne	0.61	Pass
Group 1 FE	0.47	Pass
7.5 tonne	0.35	Pass
Group 2 FE	0.25	Pass
3 tonne	0.20	Pass

The Internal Girders are suitable for Normal Traffic GVW.

From Figure 5.19f for low traffic flow and good surface, the K factors are:

	K Factor	Pass/Fail
Normal Traffic	0.77	Pass
26 Tonne	0.73	Pass
18 Tonne	0.58	Pass
Group 1 FE	0.47	Pass
7.5 tonne	0.35	Pass
Group 2 FE	0.25	Pass
3 tonne	0.20	Pass

The Internal Girders are suitable for Normal Traffic GVW.