Essex Highways



ASSESSMENT REPORT

Boxted Bridge ECC Br. No. 0059

October 2026

RINGWAY JACOBS | Essex County Council

Document Control Sheet

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Executive Summary

This Assessment Report outlines the results of the 2026 Assessment of 0059 Boxted Bridge, which is a structure owned and maintained by Essex County Council (ECC). The assessment has been carried out in accordance with the Approval in Principle (AIP) reference 0059-A-AS-AIP-001-F00. The structure is a single span half-through bridge with steel plate girders and a concrete jack arch deck with secondary steel beams, supported by brick abutments.

The previous assessment undertaken in 1992 found the structure to be suitable for 3 tonnes restricted traffic only. The structure has been closed to pedestrians and vehicular traffic due to significant defects found in the 2023 Principal Inspection, including severe corrosion, section losses and girder rotation. A Proving Load Test was carried out on the 10th of September 2025 to test the structure for pedestrian traffic loads. The test proved the bridge can withstand the pedestrian loading at the time of the test. Hence the bridge was reopened for pedestrians in January 2026.

The aim of the new assessment was to use a more refined analysis method (when compared with the previous assessment), making use of the findings from the 2025 Proving Load Test and the Inspection for Assessment (IFA) to determine the current safe load carrying capacity of the bridge for vehicular traffic.

The new assessment found the bridge to be substandard for any vehicular traffic loads in accordance with CS 454.

The structure is considered adequate for the current pedestrian footpath arrangement subject to a monitoring regime, given the results of the 2025 Proving Load Test and the outcome of the recent monitoring inspections.

1. Introduction

This Assessment Report presents the results of the 2026 Assessment of 0059 Boxted Bridge, which is a structure owned and maintained by Essex County Council (ECC). The assessment has been carried out in accordance with the AIP reference 0059-A-AS-AIP-001-F00.

Boxted Bridge was constructed in 1897 and is located within the Dedham Vale National Landscape (OS grid reference: TM 01251 34422; What3Words: caskets.waged.boxing) and carries Wick Road, a single unclassified carriageway over the River Stour. The bridge comprises a single span half-through bridge with steel plate girders and a concrete jack arch deck with secondary steel beams. The bridge is supported by brick abutments. The structure has a clear square span of 11.9m. The superstructure is approximately 6.75m wide at the south end and 5.55m wide at the north end.

The structure was previously assessed in 1992 and deemed adequate for 3 tonnes traffic only. The structure was closed to pedestrians and vehicular traffic in 2023 due to significant defects found in the 2023 Principal Inspection, including severe corrosion, section losses and girder rotation.

A Proving Load Test was carried out on the 10th of September 2025 to test the structure for pedestrian loads. The test proved the bridge can withstand the pedestrian loading at the time of the test. Hence the bridge was reopened for pedestrians in January 2026.

Essex Highways have been commissioned by Essex County Council to carry out a new structural assessment. The aim of the new assessment is to determine the current safe load carrying capacity of the bridge for vehicular traffic.

2. Structure Description

2.1. Structural Type

The superstructure is a simply supported single span half-through bridge, comprising two plate edge girders. Secondary steel beams span longitudinally from the abutments and transversely in the middle of the bridge. The transverse crossbeams comprise built-up I-sections. The longitudinal beams next to the girders comprise rolled channel sections and there are rolled I-sections in-between. Steel hogging plates span between the bottom flanges of the secondary steel beams, and the deck is filled with mass concrete (jack arch construction). The main plate girders comprise I-sections, which are built up using rivet connections with angles. Vertical and longitudinal stiffeners are formed from tee sections, which are riveted to the girders. The plate girders act as parapets.

The substructure comprises brick abutments and brick wing walls. The foundation type is unknown as the foundations are buried and there are no records available.

See general arrangement drawing BR0059-01-1101A attached in Appendix D for more details.

2.2. Span Arrangements

The bridge has a clear square span of 11.9m.

2.3. Condition

An IFA of the structure was undertaken in May 2026. Refer to IFA report reference 0059-A-AS-IFA-001-F00 for details regarding the condition of the structure.

The inspection found the primary and secondary steel deck elements in poor condition, with extensive corrosion, paint loss and significant section loss recoded throughout. The critical defects include:

- Areas of local reduction in flange width and thickness of the main girders, near supports and the mid-span region,

- Localised significant corrosion and section loss in girder webs and vertical stiffeners,
- Missing rivets in structural connections,
- Significant deterioration of crossbeams, channel sections and hogging plates,
- Local deformation, damage and section loss to end plates.

Several defects have been noted within the abutments, including vertical cracks at the east ends of each abutment and minor erosion of brickwork. There are no signs of differential settlement along the vertical cracks, and no obvious signs of foundation settlement have been noted. Shrinkage cracks, spalling and hollow sounding areas have been noted throughout the mass concrete at the top of the abutments, below the jack arches.

Significant displacement of brickwork, cracking and mortar loss have been observed within the wing walls and pilasters.

Evidence of minor scour was observed below the bridge but is not currently considered critical.

The defects identified in the IFA have been incorporated into this assessment, including any associated section losses.

3. Previous Assessment Summary

The previous assessment was undertaken in 1992 by Taywood Engineering Limited. The findings are presented in the assessment report reference 1302C/92/2375/B59/5-A.

The 1992 assessment found the structure to be suitable for 3 tonnes restricted traffic only, in accordance with BD 21/84. The plate girders governed the capacity of the bridge. In order to achieve resistance for 3 tonne traffic, steel yield stress of 300 MPa was adopted based on test results from a single steel sample and it was assumed that the crossbeams and vertical stiffeners act as u-frames restraining the top flange against buckling. Without the u-frame restraints, the capacity was determined to be less than dead load.

The 1992 assessment found severe localised corrosion and section loss to the crossbeams and channels. Impact damage was noted to the end plates in the plate girders, but the girders were found to be in good condition overall. The abutments were found to be in reasonable condition overall.

4. Interim Measures Summary

The structure is currently closed to vehicular traffic, with access permitted to pedestrians only.

A full bridge closure was implemented on the 30th of June 2023 following the 2023 Principal Inspection which found the structure to be in poor condition, with significant section losses, detached rivet and significant rotation of the girders. The bridge was reopened to pedestrians in January 2026, following the 2025 Proving Load Test for pedestrian loads.

5. Monitoring Summary (in Accordance with CS 470)

Following the opening of the bridge to pedestrians in January 2026, the structure is subject to monitoring inspections every 6 months, comprising visual observations and photography.

6. Assessment Inspection Summary

The IFA comprised an inspection of the visible elements within touching distance, including inspection of the topside and the underside of the deck, tapping of rivets, tapping of abutments and corroded steelwork, measurements of crack widths on abutments and measurements of section losses on

exposed steel elements. Close inspection of the underside of the deck and the elevations of the edge girders was carried out from an access tower.

The defects found in the IFA are presented in the report reference 0059-A-AS-IFA-001-F00 and are summarised in section 2.3 of this assessment report.

The section losses have been quantified where feasible and listed in the IFA report. In addition to the quantified section losses, a condition factor of 0.85 has been used in all resistance calculations to account for the overall significant corrosion and distortions observed throughout the structure, potential strength loss since the steel testing carried out in 1992, and to account for areas which cannot be readily inspected.

7. Assessment Method

7.1. Scope

The assessment method and assumptions are detailed in the AIP reference 0059-A-AS-AIP-001-F00.

The superstructure (except the jack arches) has been assessed quantitatively in accordance with CS 454, CS 456, CS 457 and other relevant standards as listed in the AIP. The jack arches have been assessed qualitatively in accordance with Bridge guard 3 - Current Information Sheet No. 22 (CIS 22).

The substructure has been assessed qualitatively in accordance with CS 459.

A risk assessment of the plate girders acting as parapets has been carried out in accordance with CS 461.

7.2. Material Properties

Steel

The nominal yield strength of the steel has been calculated in accordance with CS 456 Annex H using the yield strength of 300 MPa obtained from the material testing results dated 4th of March 1992, attached to the 1992 Assessment Report. This has resulted in a nominal yield strength of 246MPa, which was used for the assessment of all built-up members including the plate girders and the crossbeams, considering that the steel sample was taken from the plate girder (from the bearing stiffener specifically). A characteristic yield strength of 230MPa has been used in the assessment of all other steel elements including the channels, the longitudinal I-beams and the rivets.

It should be noted that the 246 MPa result is based on a single test sample only, and the use of fewer than five test results is not recommended by CS 456. The yield strength of 246MPa has been adopted considering it is significantly smaller than 300MPa and only slightly higher than the characteristic yield strength of 230 MPa.

The characteristic yield strength of 230 MPa is based on common practice for steel manufactured pre 1948, based on BS 15 (1948) which requires a yield strength of 232 MPa for steel plates less than 19mm thick.

Concrete

The strength of the mass concrete infill to the jack arches has been assumed as 5 MPa which is considered a conservative value.

7.3. Analysis

The superstructure has been analysed using a 3D linear elastic finite element model using a combination of plate and beam (line) elements. The main edge girders have been modelled using plate elements, including the stiffeners. The secondary longitudinal and transverse beams have been

modelled using beam (line elements). The mass concrete jak arches have been modelled using plate elements with orthotropic properties with CS 457 cl. 13.2.2. Refer to the AIP reference 0059-A-AS-AIP-001-F00 for the idealised diagram of the structure.

8. Assessment Commentary

8.1. Plate Girders

The plate girders were found to be just about adequate for permanent loads only, governed by lateral torsional buckling of the top flange. In order to obtain accurate slenderness of the girder, eigenvalue analysis has been carried out in accordance with BS 5400-3 cl. 9.7.5.

The north-east bearing stiffener has been found to be inadequate for the required end-restraint force in accordance with BS 5400-3 cl. 9.12.5.2. Therefore, a factor has been applied in accordance with CS 456 cl. 9.6.1, which increases the slenderness of the plate girder and thus reduces the capacity for lateral torsional buckling.

The assessment used information from the 2025 Proving Load Test to calibrate the analysis model to verify the deflections observed against the predicted values. The correlation indicates the presence of moment-resisting connections between the crossbeams and plate girders that were capable of sustaining the loads applied during the Proving Load Test and have therefore been considered in the assessment up to the proven load level. However, the behaviour and capacity of these connections at higher vehicular loading levels have not been verified and would require further investigation.

8.2. Secondary Beams

The crossbeams have been found to be just about adequate for 44 tonne normal traffic loads, governed by bending.

The longitudinal channels exhibit significant corrosion of the bottom flange. The south east corner (IFA photo 30) has been identified as the most onerous case with approximately 45mm wide section loss where the bending capacity limits the channel to 7.5 tonne restricted traffic only. The longitudinal RSJs have also been found to be adequate for 7.5 tonne restricted traffic only governed by bending capacity.

The rivets in the crossbeams were found to be adequate for 3 tonne restricted traffic loads only, for shear stress in the rivet induced by the shear flow in the cross-section.

8.3. Jack Arches

The concrete jack arches have been assessed in accordance with CIS 22 and were found to be generally satisfactory against the relevant assessment criteria. However, the arrangement of the edge channels supporting the jack arch deck is atypical. Based on engineering judgement, it has been assumed that adequate horizontal restraint is provided, considering the channels are partially restrained by the adjacent plate girders. In addition, the channels are embedded within the abutments, and single tie bars are present near the abutments, providing further restraint.

8.4. Substructure

The vertical cracks at the east ends of the abutments have been present since at least 1992, as reported in the 1992 assessment report. It is possible that the cracks have been caused by vehicle impact to the east plate girder, where the transverse movement of the girder could have introduced the cracks. There are no signs of settlement along the cracks, and they do not appear to have worsened significantly based on findings from past inspections. Furthermore, no obvious signs of foundation settlement have been identified.

The significant cracking and displacement of brickwork within the south wing walls is likely caused by tree growth. However, the displacement of the north east wing wall could be also indicative of

vehicular traffic surcharge loads exceeding the capacity of the wing wall or could be evidence of impact damage.

8.5. Assessment Check

The assessment has been subject to a category 2 check in accordance with CG 300. The results presented in section 9 of this assessment report have been checked and the check calculations are acceptably comparable with the assessment results.

8.6. General comments

During the 2025 Proving Load Test for pedestrian loads, acceptable displacements have been observed in the plate girders, and the recent monitoring inspections have not reported any concerns since the bridge was opened to pedestrians. Therefore, the result of the 2025 Proving Load Test is considered to take precedence over the deficiencies found in this assessment.

The inadequacies identified in this assessment suggest that the bridge would be at a high risk of overstress if reopened to vehicular loading. It should also be noted that the 2023 PI found significant rotations of the girders when the bridge was last open to vehicular traffic - such rotations have not been noted in the 2026 IFA or the 2025 Proving Load Test for pedestrian loads.

9. Assessment Result

9.1. Superstructure

The calculations show that the superstructure is just about adequate for permanent loads only, governed by the capacity of the plate girders. The results are summarised in Table 1 below. Refer to the summary at the end of Appendix C for more details.

The concrete jack arches have been found to be adequate for normal traffic loads (44 tonnes) in accordance with the empirical assessment method to CIS 22.

Table 1. Superstructure assessment summary

Element	Effect type	Capacity	Load combination	Load effect	Utilisation ratio
Plate girder, mid-span	Bending moment (lateral torsional buckling)	1053 kNm	Permanent loads	1059 kNm	1.01
Intermediate transverse stiffener in plate girders	Stress in vertical stiffener	181 MPa	Permanent loads (including forces from u-frame restraints)	480 MPa	2.65
South-east, south-west and north-west bearing stiffeners	Buckling of the effective stiffener section (ratio)	0.64	Permanent loads (including end-restraint force)	0.33	0.51
North-east bearing stiffener	Buckling of the effective stiffener section (ratio)	0.64	Permanent loads (including end-restraint force)	1.22	1.89
Plate girder	Shear force	1692 kN	Permanent + 44t	551 kN	0.33

Crossbeam (2 additional plates)	Bending moment	860	kNm	Permanent + 44t	485	kNm	0.56
Crossbeam (1 additional plate)	Bending moment	622	kNm	Permanent + 44t	457	kNm	0.73
Crossbeam (no additional plate)	Bending moment	398	kNm	Permanent + 44t	400	kNm	1.01
Crossbeam	Shear force	418	kN	Permanent + 44t	349	kN	0.83
Longitudinal RSJ	Bending moment	88	kNm	Permanent + 18t	95	kNm	1.08
Longitudinal RSJ	Bending moment	88	kNm	Permanent + 7.5t	59	kNm	0.67
Longitudinal RSJ	Shear force	189	kN	Permanent + 44t	174	kN	0.92
Channel	Bending moment	45	kNm	Permanent + 18t	64	kNm	1.41
Channel	Bending moment	45	kNm	Permanent + 7.5t	40	kNm	0.88
Channel	Shear force	184	kN	Permanent + 44t	68	kN	0.37
Rivets in plate girders - general	Shear stress in rivet	97	MPa	Permanent + 44t	65	MPa	0.67
Rivets in crossbeams	Shear stress in rivet	97	MPa	Permanent + 7.5t	113	MPa	1.16
Rivets in crossbeams	Shear stress in rivet	97	MPa	Permanent + 3t	77	MPa	0.80

9.2. Substructure

The abutments are considered adequate for normal traffic loads given their condition and their past performance when the bridge was open to vehicular traffic prior to the closure in 2023.

Both south wing walls, and the north east wing wall are considered inadequate for any vehicular traffic surcharge considering the significant cracks and displaced brickwork. The north west wing wall is considered adequate for normal traffic loads.

9.3. Parapets

The CS 461 risk assessment recommends an N1/N2 parapet upgrade.

10. Recommendations

The structure has been found to be substandard for any vehicular traffic loads and is subject to widespread corrosion, section loss, impact damage and local distortions.

It is recommended that the current traffic management and monitoring regime enabling pedestrian access are maintained until further works are completed or until any concerns arise during the monitoring inspections.

Further steel sampling and strength testing is very unlikely to raise the capacity of the structure to vehicular loading when using the current assessment method. Further assessment including non-

linear analysis is also not expected to improve the capacity to vehicular loading considering the magnitude of overstress found in this assessment. Nonetheless, should any further analysis be undertaken, intrusive investigations to confirm the connection details between the crossbeams and the plate girders would be considered necessary to enable a full assessment of the structure for vehicular loading.

Appendix A. Assessment & Check Certificates

ASSESSMENT CERTIFICATE

Project Details

Structure Name: Boxted Bridge

Structure Ref: 0059

Structure Category: CAT 2

Declaration of Assessment

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

1. **It has been assessed in accordance with:**
    a. The Approval in Principle dated 02/09/2026 (AIP document ref. no. 0059-A-AS-AIP-001-F00).
2. **It has been checked for compliance with:**
    a. The assessed capacity of the structure is as follows:

    The assessed capacity of the structure is permanent loads (refer to Table 1 below).

Table 1: Summary of Assessed Capacity.

Structural Elements	Vehicle Rating	Limitations
Plate girders	Permanent loads only	
Crossbeams	44t normal traffic	
Rivets in crossbeams	3t restricted traffic	
Longitudinal RSJs	7.5t restricted traffic	
Longitudinal Channels	7.5t restricted traffic	
Jack arches	44t normal traffic	Qualitative assessment to CIS 22
Abutments, north west wingwall	44t normal traffic	Qualitative assessment to CS 459
North east, south west and south east wing walls	Permanent loads only	Qualitative assessment to CS 459

NOTE: In accordance with CS 463, the 2025 Proving Load Test (see technical note reference 0059-F-AM-TNO-004-F00) is considered to take precedence over the result of this assessment.

Signed:	[redacted]
Name:	
Name of Organisation:	Ringway Jacobs Ltd
Engineering Qualifications:	BEng, CEng, MIStructE
Signed:	[redacted]
Name:	
Name of Organisation:	Ringway Jacobs Ltd
Position Held:	Head of Design Services
Date:	As above

Acceptance of Assessment Certification

The certificate is recommended for acceptance by the TAA:

Signed:	[redacted]
Name:	
Name of Organisation:	Ringway Jacobs Ltd.
Position Held:	Technical Approval Team Leader
Engineering Qualifications:	BEng, CEng, MICE
Date:	As above

The certificate is accepted by ECC:

Signed:	[redacted]
Name:	
TAA:	Essex County Council
Date:	As above

ASSESSMENT CHECK CERTIFICATE

Project Details

Structure Name: Boxted Bridge

Structure Ref: 0059

Structure Category: CAT 2

Declaration of Check of Assessment

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

1. **It has been checked in accordance with:**
    a. The Approval in Principle dated 02/09/2026 (AIP document ref. no. 0059-A-AS-AIP-001-F00).
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Longitudinal Channels	7.5t restricted traffic	
Jack arches	44t normal traffic	Qualitative assessment to CIS 22
Abutments, north west wingwall	44t normal traffic	Qualitative assessment to CS 459
North east, south west and south east wing walls	Permanent loads	Qualitative assessment to CS 459

NOTE: In accordance with CS 463, the 2025 Proving Load Test (see technical note reference 0059-F-AM-TNO-004-F00) is considered to take precedence over the result of this assessment.

Signed:	[redacted]
Name:	
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Engineering Qualifications:	BEng, CEng, MIStructE
Signed:	[redacted]
Name:	
Name of Organisation:	Ringway Jacobs Ltd
Position Held:	Head of Design Services
Date:	As above

Acceptance of Check Certification

The certificate is recommended for acceptance by the TAA:

Signed:	[redacted]
Name:	
Name of Organisation:	Ringway Jacobs Ltd.
Position Held:	Technical Approval Team Leader
Engineering Qualifications:	BEng, CEng, MICE
Date:	As above

The certificate is accepted by ECC:

Signed:	[redacted]
Name:	
TAA:	Essex County Council
Date:	As above

Appendix B. Copy of Approved AIP

RESTRICTED



APPROVAL IN PRINCIPLE FOR ASSESSMENT (NON-EUROCODE)

Boxted Bridge ECC Br. No. 0059

August 2026

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Name of Project: Boxted Bridge Assessment
Name of Structure: Boxted Bridge
Structure reference no.: ECC Br. No. 0059

Summary:

This Approval in Principle (AIP) covers the methodology of the proposed assessment of 0059 Boxted Bridge, which is a structure owned and maintained by Essex County Council (ECC).

The previous assessment undertaken in 1992 found the structure to be suitable for 3 tonnes restricted traffic only. The structure has been closed to pedestrians and vehicle traffic due to significant defects found in the 2023 Principal Inspection, including severe corrosion, section losses and girder rotation.

A Proving Load Test was carried out on the 10th of September 2025 to test the structure for pedestrian traffic loads. The test proved the bridge can withstand the pedestrian loading at the time of the test. Hence the bridge was reopened for pedestrians in January 2026.

The aim of the new assessment is to use a more refined analysis method (when compared with the previous assessment), making use of the findings from the 2025 Proving Load Test and the Inspection for Assessment to determine the current safe load carrying capacity of the bridge for vehicle traffic.

1. HIGHWAY DETAILS



Figure 1.a: Carriageway Carried by Structure.

1.1. Type of highway

Single carriageway (unclassified)

1.2. Design traffic speed

The permitted speed over the bridge was 60mph when it was open.

1.3. Existing restrictions

The structure is currently closed to vehicular traffic, with access permitted for pedestrians only.

A full bridge closure was implemented on the 30th of June 2023 following findings of the 2023 Principal Inspection. The bridge was reopened to pedestrians in January 2026, following the Proving Load Test for pedestrian load effects.

2. SITE DETAILS

2.1. Obstacles crossed

River Stour

3. EXISTING STRUCTURE

3.1. Description of structure and design working life

Boxted Bridge (ECC Structure No. 0059) is located in Boxted, Colchester (OS Grid Reference TM 01244 34419) and carries Wick Road, an unclassified road over the River Stour. The design working life of the structure is unknown. The bridge was constructed in 1897.

3.2. Structural type

The superstructure comprises a simply supported single span half-through bridge, comprising two main plate girder edge beams with secondary steel beams spanning transversely in the middle of the bridge and longitudinally at both ends, adjacent to the abutments. The transverse cross-beams comprise built-

up I-sections. The longitudinal beams comprise rolled angle sections at both edges and rolled I-sections in-between. Steel hogging plates (approximately 5mm thick) span between the bottom flanges of the secondary steel beams, and the deck is filled in with mass concrete (jack arch construction). The steel members are generally connected using rivets with steel angles and tee sections forming connecting plates and stiffeners for the plate girders. The substructure comprises brick abutments and brick wing walls. See general arrangement drawing BR0059-01-1101A attached in Appendix C for more details.

3.3. Foundation type

The foundation type is unknown.

3.4. Span arrangements

The bridge has a clear square span of 11.9m.

3.5. Articulation arrangements

The bridge is simply supported.

3.6. Road restraint systems requirements

The edge girders act as parapets on the structure.

3.7. Proposals for water management

Not applicable for assessment.

3.8. Proposed arrangements for future maintenance and inspection for assessment

3.8.1. Traffic Management

The structure is currently open for pedestrians only. The topside elements can be inspected on foot without additional traffic management being necessary.

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

An Inspection for Assessment (IfA) was undertaken on 11th-13th of May 2026. As part of the IfA, section losses within the exposed steel elements have been measured (refer to IfA Report ref. 0059-A-AS-IFA-001-F00).

The topside elements were inspected on foot. The underside elements were inspected from the embankments and on foot when the water levels were low. Close inspection of the underside of the structure was undertaken using a temporary access tower to measure all section losses throughout the deck.

3.8.3. Intrusive or further investigations proposed

A Proving Load Test for pedestrian loads was undertaken on the 10th of September 2025. The assessment will use the information from the 1992 Assessment Report (which included strength testing of the steel), the 2025 Proving Load Test and the IfA.

If the initial assessment finds the structure to be adequate for any vehicular traffic loads (3 tonnes restricted traffic or higher) the following further investigations are proposed:

- Exposure of the main girder to cross-beam connection to confirm the hidden detail (no information available in current records).
- If the assessment finds that the assumed strength of the concrete infill in the deck governs the assessment, compressive strength testing will be proposed.

The intrusive investigations listed above are considered unnecessary unless the remainder of the structure can be proven to be adequate for vehicular traffic loads.

3.9. Environment and Sustainability

Not applicable.

3.10. Durability – Material strengths assumed and basis of assumptions

Steel

The nominal yield strength of the steel will be calculated in accordance with CS 456 Annex H using the yield strength of 300 MPa obtained from the material testing results dated 4th of March 1992, attached to the 1992 Assessment Report.

Concrete

The strength of the mass concrete infill to the jack arches will be assumed as 5 MPa which is considered a conservative value. A sensitivity analysis will be carried out and potentially further investigation recommended should this govern the assessment.

Condition

The IfA found the structure to be in poor condition. In addition to the section losses quantified in the IfA, an overall condition factor of 0.85 is proposed to be adopted in the assessment (refer to IfA Report ref. 0059-A-AS-IFA-001-F00).

3.11. Risks and hazards considered for design, execution, maintenance and demolition. Consultation with and/or agreement from Overseeing Organisations.

Not applicable for assessment.

3.12. Resilience and Security

Not applicable.

3.13. Year of construction

The structure was built in 1897.

3.14. Reason for Assessment

Ringway Jacobs have been commissioned by Essex County Council to carry out an assessment of the structure to determine the load carrying capacity based on the latest information on the current condition and the findings from the 2025 Proving Load Test in order to determine if the bridge can be reopened to vehicular traffic. The new assessment will use a more refined analysis method when compared with the previous assessment, which used hand calculations. The structure will be assessed for vehicular traffic loads to verify the capacity beyond the scope of the 2025 Proving Load Test.

3.15. Part of structure to be assessed

The superstructure will be assessed quantitatively in accordance with CS 454.

The substructure will be assessed qualitatively in accordance with CS 459.

4. ASSESSMENT CRITERIA

4.1. Actions

4.1.1. Permanent actions

Dead load and superimposed dead loads will be applied in accordance with CS 454, using the unit weights as listed below unless otherwise stated.

Steel	7850 kg/m^3
Plain concrete	2300 kg/m^3
Surfacing	2400 kg/m^3

4.1.2. Snow, wind and thermal actions

Not considered.

4.1.3. Actions relating to normal traffic under AW regulations and C&U regulations

The structure will be assessed for normal traffic using ALL model 1 in accordance with CS 454, which is particularly suitable for spans under 20m (CS 454 cl. 5.5.2 Note 1). Furthermore, ALL model 2 may be unsuitable for this structure as in the middle of the deck, the main beams span transversely and CS 454 cl. 5.6 (3) may apply. In accordance with CS 454 cl. 5.5.2, either one of the models should be used to assess for normal traffic, therefore using ALL model 1 on its own is considered to be sufficient.

4.1.4. Actions relating to General Order Traffic under STGO regulations

Special Vehicles are not considered, which has been agreed with the Essex Highways Asset Manager.

4.1.5. Footway or footbridge variable actions

The bridge will be assessed for the pedestrian live loading model in accordance with CS 454.

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

Not considered.

4.1.7. Accidental actions

Accidental collision loads will be considered in accordance with CS 461, treating the edge girders as parapets.

4.1.8. Actions during construction

Not considered.

4.1.9. Any special action not covered above

In addition to the loads described above, to check for model correlation, the structure will be analysed for the loads applied during the 2025 Proving Load Test, including:

- 5kN/m^2 applied over a path of 2m width, positioned along the centreline of the bridge and applied to the full span length (with ultimate limit state safety factors of 1.1 x 1.5). This was the

load that the Proving Load Test was aiming to simulate in order to verify the structure for pedestrian traffic loads over a 2m wide path.

- 12.2 and 16 tonnes uniformly distributed mass over a 3.4m x 3.4m patch in the centre of the bridge – these were the actual loads applied during the 2025 Proving Load Test.

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

The load testing and assessment will help determine the capacity of the bridge and actions to enable continued use of the structure if possible.

4.3. Minimum headroom to be provided

Not applicable.

4.4. Authorities consulted and any special conditions required

Essex County Council has been consulted with regards to the proposed assessment. No special conditions are required.

4.5. Standards and documents listed in the Technical Approval Schedule (TAS)

See *APPENDIX A* for the list of relevant documents.

4.6. Proposed departures from standards listed in 4.5

No departures from standards are proposed. The structure will be assessed for normal traffic and pedestrian traffic actions in accordance with CS 454 (see sections 4.1.3 and 4.1.5 of this AIP).

4.7. Proposed departures from standards concerning methods for dealing with aspects not covered by standards listed in 4.5

Not applicable.

4.8. Proposals for assessment of safety critical fixings

Not applicable.

5. STRUCTURAL ANALYSIS

5.1. Method of analysis proposed for superstructure, substructure and foundations

The superstructure will be analysed using a 3D linear elastic finite element model using a combination of plate and beam (line) elements. The main edge girders will be modelled using plate elements and the jack arch deck will be modelled using a combination of beam and plate elements. Modelling the main girders using plate elements will enable to simulate the deflections of the bridge more accurately in order to compare the behaviour of the bridge with the load test, and to determine elastic critical buckling moments more accurately. The jack arches will be assessed in accordance with Bridgeguard 3: Current Information Sheet No. 22 (CIS 22).

The substructure will be assessed qualitatively in accordance with CS 459.

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

See Appendix B for the idealised diagram. The superstructure will be modelled using plate elements for the main girders and the concrete within the jack arch deck. Beam elements will be used for the secondary transverse and longitudinal beams within the jack arch deck. Plate elements will also be used to model the stiffeners in the main girders. The connections of the cross beams to the edge girders will be assumed as rigid moment-resisting connections considering the correlation between the full u-frame model and the top flange lateral movement noted in the 2025 Proving Load Test (see technical note ref. 0059-F-AM-TNO-004-F00).

5.3. Assumptions intended for calculation of structural element stiffness

The structure will be analysed using linear elastic analysis with the concrete assumed to be uncracked.

The sectional properties of the steel members in the global analysis will be calculated without reduction due to corrosion, in accordance with CS 456 cl. 7.2. The IfA (report ref. 0059-A-AS-IFA-001-F00) has not identified defects that will have a significant effect on the global modelling. However, section losses measured during the IfA will be considered in the capacity calculations.

The modulus of elasticity of steel will be taken as 205 GPa in accordance with CS 456 (BS 5400-3 cl. 6.6). The yield strength of the steel will be based on previous material testing as per section 3.10 of this AIP.

For global analysis, the effective depth of the mass concrete jack arch elements will be calculated in accordance with CS 457 cl. 13.2.2. The hogging plates will be applied as dead loads only and will not be included as elements in the analysis model. The modulus of elasticity of the concrete infill will be calculated in accordance with CS 457 Appendix C. The jack arches will be modelled as transverse elements, using plate elements with orthotropic properties with stiffness defined only in the direction of the span of the jack arches.

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

Not applicable.

6. GEOTECHNICAL CONDITIONS

6.1. Acceptance of recommendations of the ground investigation report (reference/date) to be used in the assessment and reasons for any proposed changes

Not applicable.

6.2. Summary of design for highway structure in the ground investigation report

Not applicable.

6.3. Differential settlement to be allowed for in the assessment of the structure

Not applicable.

7. CHECK

7.1. Proposed category

Category 2.

7.2. If Category 3, name of proposed independent checker

Not applicable.

8. DRAWINGS AND DOCUMENTS

8.1. List of drawings (including numbers) and documents accompanying the submission

Table 8.1.a: List of drawings (including numbers) and documents accompanying the submission

Document Reference	Document Title	Document Revision / Date
BR0059-01-1101	Boxted Bridge ECC Bridge No. 059 Existing Bridge General Arrangement	A

8.2. List of construction and record drawings (including numbers) to be used in the assessment

Table 8.2.a: List of construction and record drawings (including numbers) to be used in the assessment

Document Reference	Document Title	Document Revision / Date
BR0059-01-1101	Boxted Bridge ECC Bridge No. 059 Existing Bridge General Arrangement	A
TEL/2375/B59/301/B	Boxted Bridge (B59) General Arrangement Sheet 1 of 1	B (March 1992)

8.3. List of pile driving or other construction records

None.

8.4. List of previous inspection and assessment reports

Table 8.4.a: List of previous inspection and assessment reports

Document Reference	Document Title	Document Revision / Date
1302C/92/2375/B59/5-A	Assessment Report for Boxted Bridge	May 1992
None	2023 Principal Inspection Report	14 July 2023
0059-A-AS-IFA-001-F00	2026 Inspection for Assessment Report	Revision F00

9. THE ABOVE IS SUBMITTED FOR ACCEPTANCE

Signed: [redacted]

Name:

Engineering Qualification: BEng(Hons), CEng, MIStructE

Name of Organisation: Ringway Jacobs

Date: As above

Signed: [redacted]

Name:

Engineering Qualification: BEng, CEng, MIStructE

Name of Organisation: Ringway Jacobs

Date: As above

10. THE ABOVE IS ~~REJECTED~~ / ACCEPTED AND RECOMMENDED FOR ACCEPTANCE

Signed: [redacted]

Name:

Engineering Qualifications BEng, CEng, MICE

Name of Organisation: Ringway Jacobs

Date: As above

11. THE ABOVE IS ~~REJECTED~~ / AGREED ~~SUBJECT TO THE AMENDMENTS AND CONDITIONS SHOWN BELOW~~

Signed: [redacted]

Name:

TAA: Essex County Council

Date: As above

Appendix A. Technical Approval Schedule

Technical Approval Schedule (TAS)

Schedule of Documents Relating to Assessment of Highway Bridges and Structures

(All documents are taken to include revisions current as of 25 March 2026)

The Design Manual for Roads and Bridges (DMRB)		
DMRB reference	**Title**	**Notes**
~~CD 360 Revision 2~~	~~Use of compressive membrane action in bridge decks~~	~~Supersedes BD 81/02~~
CG 300 Revision 0.2.2	Technical approval of highway structures	Supersedes CG 300 Revision 0.2.1
CS 450 Revision 0.1.0	Inspection of highway structures	Supersedes BD 63/17
~~CS 451 Revision 0~~	~~Structural review and assessment of highway structures~~	~~Supersedes BD 101/11~~
~~CS 453 Revision 0~~	~~The assessment of highway bridge supports~~	~~Supersedes BD 48/93, BD 60/04 and IAN 91/07~~
CS 454 Revision 1.1.0	Assessment of highway bridges and structures	Supersedes CS 454 Revision 1
CS 455 Revision 1.1.1	The assessment of concrete highway bridges and structures	Supersedes CS 455 Revision 1.1.0
CS 456 Revision 0	The assessment of steel highway bridges and structures	Supersedes BD 56/10
CS 457 Revision 1	The assessment of composite highway bridges and structures	Supersedes BD 61/10
~~CS 458 Revision 0~~	~~The assessment of highway bridges and structures for the effects of special type general order (STGO) and special order (SO) vehicles~~	~~Supersedes BD 86/11~~
CS 459 Revision 1	The assessment of bridge substructures, retaining structures and buried structures	Supersedes BA 55/06
CS 461 Revision 0.2.0	Assessment and upgrading of in-service parapets	Supersedes CS 461 Revision 0.1.0
CS 462 Revision 0.1.0	Repair and management of deteriorated concrete highway structures	Supersedes CS 462 Revision 0
CS 463 Revision 0	Load testing for bridge assessment	Supersedes BA 54/94
~~CS 466 Revision 1.0.0~~	~~Risk management and structural assessment of concrete half-joint deck structures~~	~~Supersedes CS 466 Revision 0~~
CS 469 Revision 1.0.0	Management of scour and other hydraulic actions at highway structures.	Supersedes BD 97/12
GG 104 Revision 0.1.0	Requirements for safety risk assessment	Supersedes GG 104 Revision 0

Interim Advice Notes		
IAN reference	**Title**	**Notes**
~~IAN 173/13~~	~~Implementation of BD 97/12 – The Assessment of Scour and Other Hydraulic Actions at Highway Structures~~	Superseded by CS 469 Revision 1.0.0

Eurocodes and associated UK National Annexes			
Eurocode part	**Title**	**Amendment / Corrigenda**	***Notes***
Eurocode 0	**Basis of structural design**		
~~BS EN 1990:2002 +A1:2005~~	~~Eurocode 0: Basis of structural design~~	~~+A1:2005~~ ~~Incorporating corrigenda December 2008 and April 2010~~	~~See CD 350 section 7 for additional guidance.~~ ~~This document is to be used until 30 March 2028. After which it will be superseded by BS EN 1990:2023+A1:2026~~
~~NA to BS EN 1990:2002 + A1:2005~~	~~UK National Annex to Eurocode 0 Basis of structural design~~	~~National Amendment No.1~~	~~See CD 350 Section 7 for additional guidance.~~
Eurocode 1	**Actions on structures**		
~~BS EN 1991-2:2003~~	~~Eurocode 1: Actions on structures. Traffic loads on bridges~~	~~Corrigenda December 2004 and February 2010~~	~~See CD 350 section 7 for additional guidance.~~ ~~This document is to be used until 30 March 2028. After which it will be superseded by BS EN 1991-2:2023~~

Eurocodes and associated UK National Annexes			
Eurocode part	**Title**	**Amendment / Corrigenda**	***Notes***
~~NA +A1:2020 to BS EN 1991-2:2003~~	~~UK National Annex to Eurocode 1: Actions on structures. Traffic loads on bridges~~	~~Corrigendum No.1 Amendment June 2020~~	~~See CD 350 section 7 for additional guidance.~~
Eurocode 2	**Design of concrete structures**		
~~BS EN 1992-1-1:2004 + A1:2014~~	~~Eurocode 2: Design of concrete structures – Part 1-1: General rules and rules for buildings~~	~~Incorporating corrigendum January 2008, November 2010 and January 2014~~	~~This document is to be used until 30 March 2028. After which it will be superseded by BS EN 1992-1-1:2023~~
~~NA + A2:2014 to BS EN 1992-1-1:2004 + A1:2014~~	~~UK National Annex to Eurocode 2: Design of concrete structures – Part 1-1: General rules and rules for buildings~~		~~This document is to be used until 30 March 2028. After which it will be superseded by NA to BS EN 1992-1-1:2023~~
~~BS EN 1992-2:2005~~	~~Eurocode 2: Design of concrete structures – Part 2: Concrete bridges – Design and detailing rules~~	~~Corrigendum July 2008~~	~~This document is to be used until 30 March 2028. After which it will be superseded by BS EN 1992-1-1:2023~~
~~NA to BS EN 1992-2:2005~~	~~UK National Annex to Eurocode 2: Design of concrete structure – Part 2: Concrete bridges – Design and detailing rules~~	~~-~~	~~This document is to be used until 30 March 2028. After which it will be superseded by NA to BS EN 1992-1-1:2023~~
Eurocode 3	**Design of steel structures**		
~~BS EN 1993-2:2006~~	~~Eurocode 3: Design of steel structures – Part 2 Steel bridges~~	~~Corrigendum July 2009~~	~~This document is to be used until 30 March 2028. After which it will be superseded by NA to BS EN 1993-2:2026~~

Eurocodes and associated UK National Annexes			
Eurocode part	**Title**	**Amendment / Corrigenda**	***Notes***
~~NA + A1:2012 to BS EN 1993-2:2006~~	~~UK National Annex to Eurocode 3: Design of steel structures – Part 2 Steel bridges~~	~~+ A1:2012~~	
Eurocode 4	**Design of composite steel and concrete structures**		
~~BS EN 1994-2:2005~~	~~Eurocode 4: Design of composite steel and concrete structures – Part 2 General rules and rules for bridges~~	~~Corrigendum July 2008~~	
~~NA to BS EN 1994-2:2005~~	~~UK National Annex to Eurocode 4: Design of composite steel and concrete structures – Part 2 General rules and rules for bridges~~	~~-~~	
Eurocode 7	**Geotechnical design**		
~~BS EN 1997-1:2004+A1:2013~~	~~Eurocode 7: Geotechnical design – Part 1 General rules~~	~~+A1:2013 Corrigendum February 2009~~	~~This document is to be used until 30 March 2028. After which it will be superseded by BS EN 1990:2023, BS EN 1997-1:2024, BS EN 1997-3:2025~~
~~NA+A2:2022 to BS EN 1997-1:2004+A1:2013~~	~~UK National Annex to Eurocode 7: Geotechnical design – Part 1 General rules~~	~~+A1:2013 Incorporating Corrigendum No.1, Amendment 1 – July 2014 and Amendment 2 - 2022~~	~~Supersedes NA+A1:2014 to BS EN 1997-1:2004+A1:2013~~

Bsi Published Documents		
Published Document reference	**Title**	**Notes**
~~PD 6694-1:2011 + A1:2020~~	~~Recommendations for the design of structures subject to traffic loading to BS EN 1997-1~~	~~Incorporating Corrigendum Jan 2022 & Jul 2023~~

		~~See CD 350 Appendix A for additional guidance.~~

Product Standards referenced in British Standards or Eurocodes		
Product Standard reference	**Title**	**Notes**
~~BS EN 1317-1:2010~~	~~Road Restraint Systems – Part 1 – Terminology and general criteria for test methods~~	
~~DD ENV 1317-4:2002~~	~~Road Restraint Systems – Part 4 – Performance classes, impact test acceptance criteria and test methods for terminals and transitions of safety barriers.~~	~~*Draft BS EN 1317-4 for public comment published in June 2012*~~

British Standards		
British Standard reference	**Title**	**Notes**
BS 4-1	BS 4 - Structural Steel Sections Part 1 - Hot rolled sections	Replaced by BS EN 10365:2017
BS 4360	Weldable Structural Steel	Replaced by BS EN 10029:1991, BS 7668:1994, BS EN 10113-1:1993, BS EN 10113-3:1993, BS EN 10210-1:1994, BS EN 10113-2:1993
~~BS 4395~~	~~High Strength Friction Bolts and Associated Nuts and Washers for Structural Engineering Metric Series Part 1 General Grade~~	~~Withdrawn, nothing replaces it~~
~~BS 4604-1~~	~~Specification for The use of high strength friction grip bolts in structural steelwork metric series - Part 1: General grade~~	~~Replaced by BS EN 1993-1-8:2005~~
~~BS 4848-2~~	~~Hot-rolled structural steel sections Part 2. Specification for hot-finished hollow sections~~	~~Replaced by BS EN 10210-2:1997~~
BS 5400-3	Steel, concrete and composite bridges. Part 3: Code of practice for design of steel bridges	Replaced by BS EN 1993-1-10:2005, BS EN 1993-2:2006, BS EN 1993-1-8:2005, BS EN 1993-1-1:2005+A1:2014, BS EN 1993-1-5:2006+A1:2017
BS 5400-4	Steel, concrete and composite bridges. Code of practice for design of concrete bridges	Replaced by BS EN 1992-2:2005
BS 5400-5	Steel, concrete and composite bridges. Part 5: Code of practice for design of composite bridges	Replaced by BS EN 1994-2:2005
BS 5400-10	Steel, concrete and composite bridges. Part 10: Code of practice for fatigue	Replaced by BS EN 1993-1-9:2005

British Standards		
BS 5628	Code of practice for the use of masonry	Replaced by BS EN 1996-1-2:2005, BS EN 1996-2:2006, BS EN 1996-3:2006, PD 6697:2010
~~BS 5975~~	~~Code of practice for temporary works procedures and the permissible stress design of falsework~~	~~Current version is BS 5975-1:2024 and BS 5975-2:2024~~
BS 8002	Code of practice for earth retaining structures	Current version is BS 8002:2015 but there is also BS 8002:2015 - TC
BS 8004	Code of practice for foundations	Current version is BS 8004:2015+A1:2020 but there is also BS 8004:2015 – TC
~~BS 8006-1~~	~~Code of practice for strengthened / reinforced soils and other fills~~	~~Current version is BS 8006-1:2010+A1:2016~~
~~BS 8006-2~~	~~Code of practice for strengthened / reinforced soils Part 2: Soil nail design~~	~~Current version is BS 8006-2:2011+A1:2017~~
~~BS 8081~~	~~Code of practice for grouted anchors~~	~~Current version is BS 8081:2015+A2:2018~~

The Manual Contract Document for Highway Works (MCHW)		
MCHW reference	**Title**	***Notes***
MCHW Volume 1: October 2022	Specification for Highway Works	*Specification compliant with the execution standards must be used. A Departure is necessary for the parts where a compliant revision has not been published. Amendments October 2022 Supersedes April 2022 version*
MCHW Volume 2: October 2022	Notes for guidance on the Specification for Highway Works	*Notes for guidance compliant with the execution standards must be used. A Departure is necessary for the parts where a compliant revision has not been published. Amendments October 2022 Supersedes November 2021 version*
MCHW Volume 3: February 2017	Highway Construction Details	

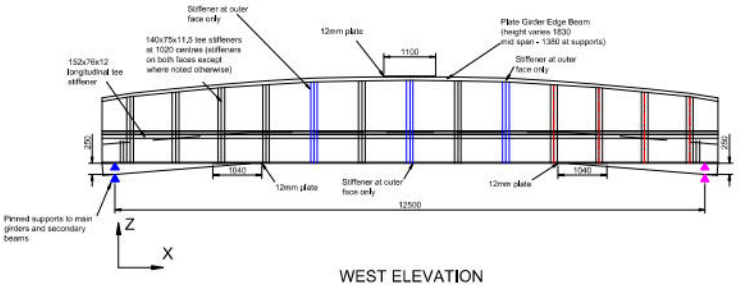
Miscellaneous		
Standard reference	**Title**	**Notes**
CIRIA - C764	Hidden defects in bridges. Guidance for detection and maintenance	

Miscellaneous		
Standard reference	**Title**	**Notes**
~~CIRIA - C778~~	~~Management of safety critical fixings in-service. Guidance for the management and design of safety-critical fixings~~	
CIRIA - C800	Guidance on the assessment of masonry arch bridges	
~~DfT Masonry Parapets Guidance~~	~~Guidance on the Design, Assessment and Strengthening of Masonry Parapets on Highway Structures~~	
DfT UK Roads Liaison Group. MHS CoP	Management of Highway Structures - Code of Practice	

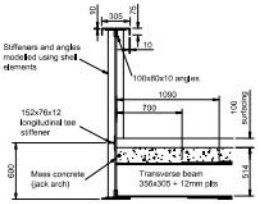
Additional Standards **Additional standards needed for a particular assessment should be listed here.**		
Reference	**Title**	**Notes**
Bridgeguard 3: CIS 22	Assessment of Jack Arches, Metal Arch Plates and Associated Ties in Metal Beam Bridge Decks	

Appendix B. Idealised Model for Analysis

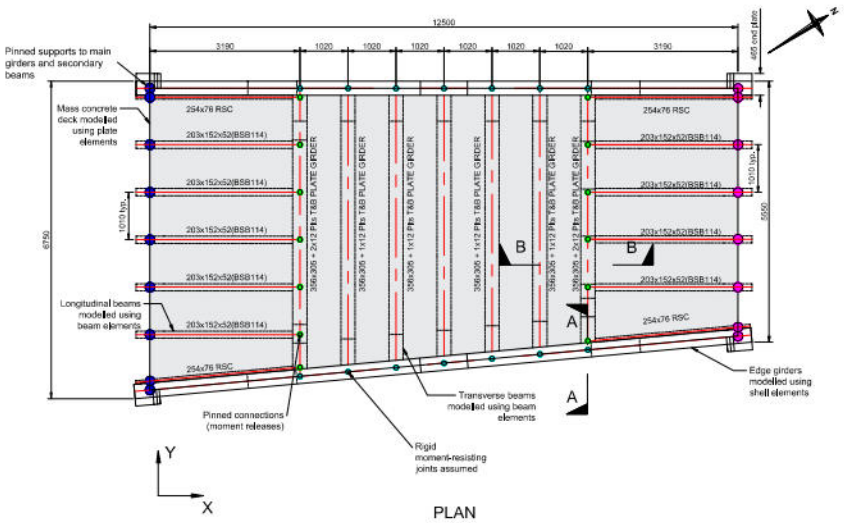
IDEALISED DIAGRAM FOR ANALYSIS
(NOT TO SCALE, DIMENSIONS IN MILLIMETRES)



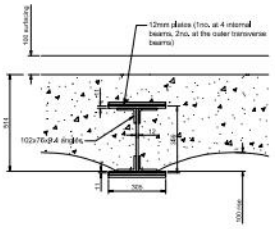
WEST ELEVATION



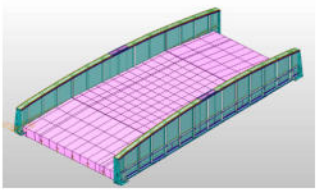
SECTION A-A



PLAN



SECTION B-B



ISOMETRIC VIEW

Key

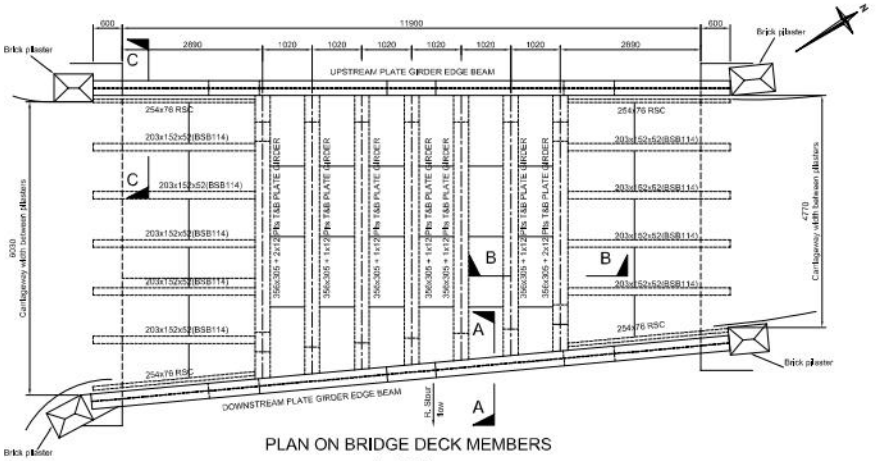
Supports:

- Translation fixed in X,Y,Z
- Translation fixed in Y,Z

Connections:

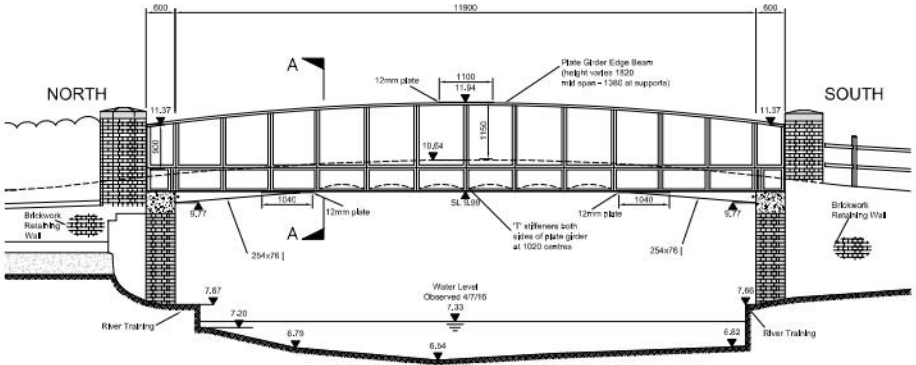
- Pinned connection (translation X,Y,Z)
- Rigid moment-resisting joint (translation and rotation X,Y,Z)

Appendix C. Drawings



PLAN ON BRIDGE DECK MEMBERS

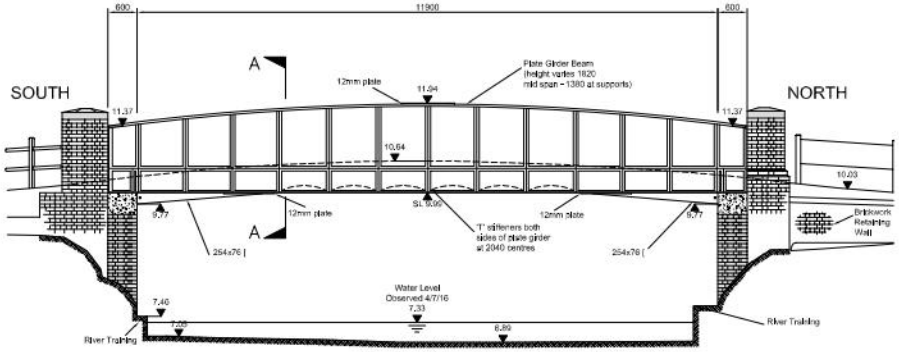
Scale 1:50



UPSTREAM (WEST) ELEVATION

WATERWAY AREA= 37.70 m²

Scale 1:50

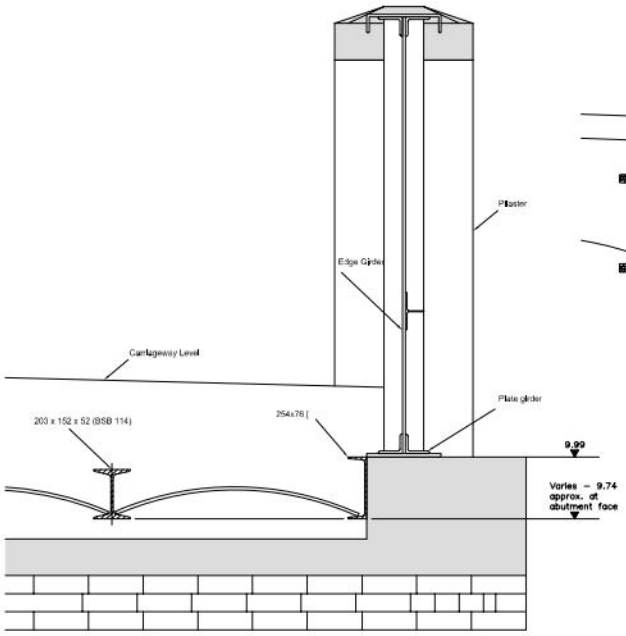


DOWNSTREAM (EAST) ELEVATION

WATERWAY AREA= 36.15 m²

Scale 1:50

[redacted]

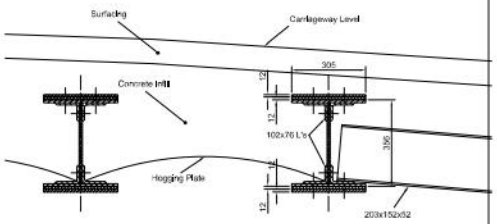


SECTION C-C

Scale 1:10

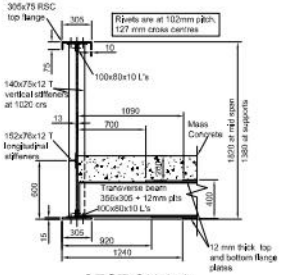
NOTES

1. All dimensions are in millimetres unless noted otherwise.
2. All levels are in metres above Ordnance Datum unless noted otherwise.



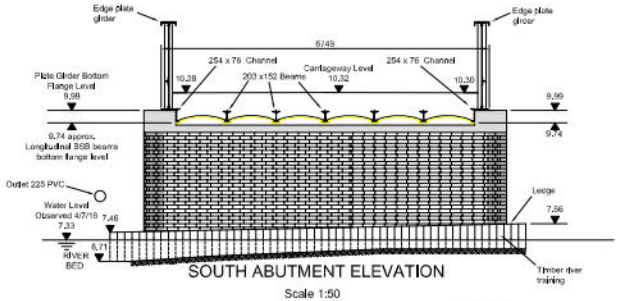
SECTION B-B

Scale 1:10



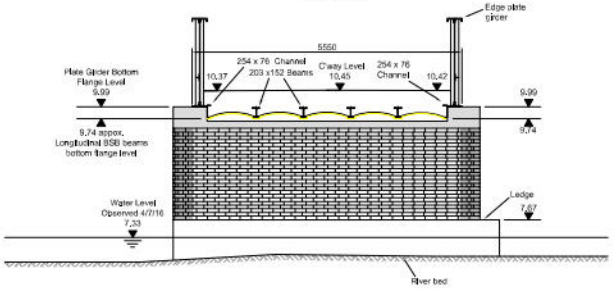
SECTION A-A

Scale 1:25



SOUTH ABUTMENT ELEVATION

Scale 1:50



NORTH ABUTMENT ELEVATION

Scale 1:50

Rev.	Date	Description of revision	Drawn	Checked	Re-Ver'd	Approved
A	08/18	River bed profile amended and other minor amendments	[redacted]			

DRAWING STATUS

INFORMATION

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SCHEME TITLE

BOXTED BRIDGE
ECC BRIDGE No. 059

DRAWING TITLE

EXISTING BRIDGE
GENERAL ARRANGEMENT

DESIGNED [redacted]

DATE

DRAWING UNITS: DIMENSIONS IN MILLIMETRES, LEVELS IN METRES

SCALE: As Shown

DRAWING No.: BR0059-01-1101

REV.: A

Appendix D. Photographs



Figure 1. Boxted Bridge – Deck



Figure 2. Boxted Bridge - Carriageway



Figure 3. Boxted Bridge - Bearing Stiffeners

Appendix C. Calculations

Boxted Bridge ECC Br. No. 0059

Assessment

October 2026

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Introduction

These calculations cover the assessment of 0059 Boxted Bridge, which is a structure owned and maintained by Essex County Council (ECC). The assessment has been carried out in accordance with the Approval in Principle (AIP) reference 0059-A-AS-AIP-001 and the Inspection for Assessment (IFA) report reference 0059-A-AS-IFA-001.

The previous assessment undertaken in 1992 found the structure to be suitable for 3 tonnes restricted traffic only. The structure has been closed to pedestrians and vehicle traffic due to significant defects found in the 2023 Principal Inspection, including severe corrosion, section losses and girder rotation.

A Proving Load Test was carried out on the 10th of September 2025 to test the structure for pedestrian traffic loads. The test proved the bridge can withstand the pedestrian loading at the time of the test. Hence the bridge was reopened for pedestrians in January 2026.

The aim of the new assessment is to use a more refined analysis method (when compared with the previous assessment), making use of the findings from the 2025 Proving Load Test and the Inspection for Assessment to determine the current safe load carrying capacity of the bridge for vehicle traffic.

REFERENCE	CALCULATIONS	OUTPUT

Material properties

Concrete

AIP — Concrete cube strength f_{cu} = **5.0** **N / mm²**

CS 457 cl. C1 — Elastic modulus of concrete E_f = 5000 MPa

CS 457 Eq. C.2 — Young's modulus of jack arches E = 0.35 E_f = 1750 MPa

Poisson's ratio v = 0.2

Shear modulus G = E / (2(1+v)) = 729 MPa

Steel

1992 assessment test results — Steel yield strength mean from testing σ_{ym} = 300 MPa

Number of tests n = 1

CS 456 Eq. H.4.1a — $\sigma_y = \sigma_{ym}\left(1 - 0.128\left(\frac{n+1}{n}\right)^{0.5}\right)$ = 246 MPa

Characteristic yield stress for rolled sections σ_y = 230 MPa

Note: eq. H.4.1b is not used as only one test result is available. 246MPa is considered conservative compared to 300MPa.

BS 5400-3 cl. 6.6 — Young's modulus of steel E_s = 205000 Mpa

AIP — Condition factor 0.85

Partial factors at ULS for materials

CS 455 Table 2.13a — Concrete strength type → Characteristic

Concrete partial factor γ_{mc} = 1.5

Bond strength factor γ_{mb} = 1.25

Shear in concrete partial factor γ_{mv} = 1.25

CS 454 cl. 3.9 — Partial factor for load effects γ_{f3} = 1.1

CS 456 Table 2 — Steel partial factor

γ_m = 1.05 general

γ_m = 1.2 buckling, fastener in tension, long. stiffener in compression

γ_m = 1.1 fasteners in shear

γ_m = $0.95 + \frac{1.8}{(\frac{L}{r}+5)}$ for compression members cl. 10.6.1.1, 10.6.3 CS 456

but not greater than 1.05

Table 3.4 — **Partial factors for actions at ULS, γ_{fL}**

Self-weights

Steel γ_{fL} = 1.05

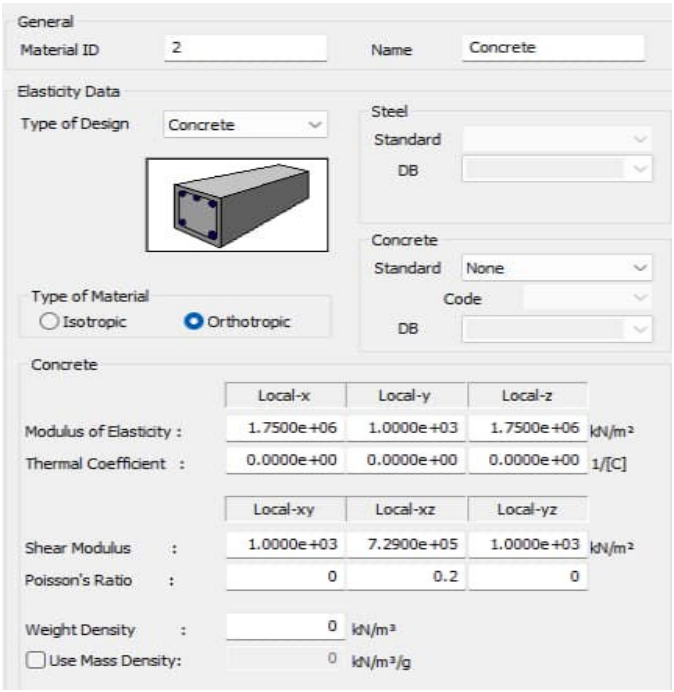
Concrete γ_{fL} = 1.15

cl. 3.4.1 Appendix A — Surfacing (top 100mm) γ_{fL} = 1.75

Vertical traffic loads for normal and restricted traffic γ_{fL} = 1.5 Comb. 1 γ_{fL} = 1.25 Comb. 2-4

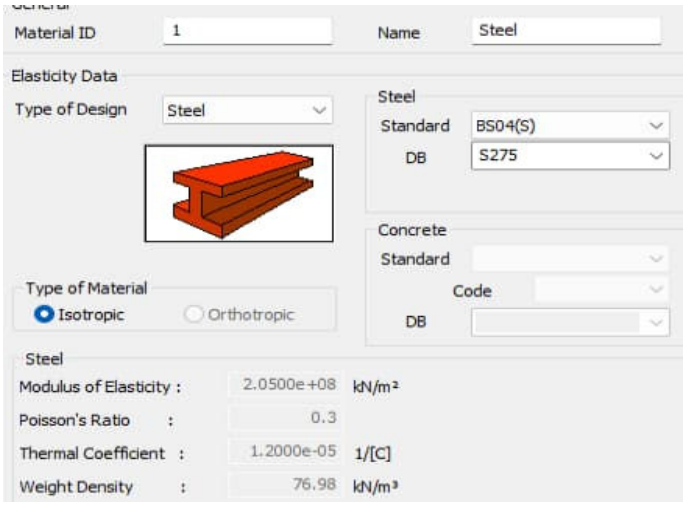
REFERENCE	CALCULATIONS				OUTPUT
	Analysis				
	Jack arches				
Topo survey	Assume that the top 100mm is the tarmac.				
2023	Nominal depth of concrete fill at the cross-beam flange	514	mm	approx.	
	Arch rise	100	mm		
	Depth of concrete at the crown	414	mm	approx.	
	Average depth along the arch	464	mm		
CS 457 cl. 13.2.2	Effective depth as per CS 457 cl. 13.2.2	434	mm		

Orthotropic material properties:



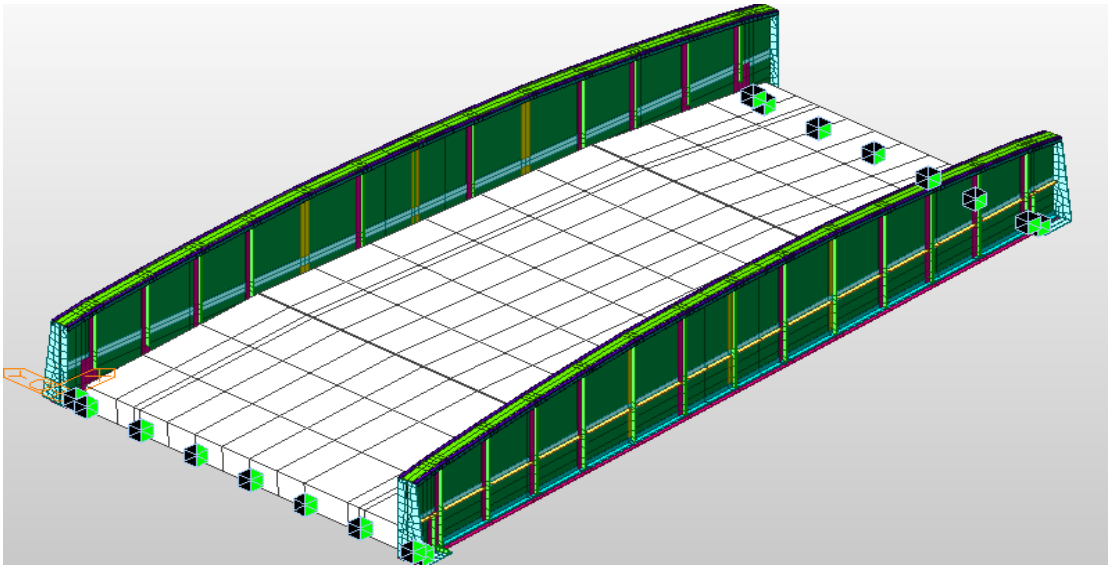
where local-x direction is in the direction of the span of each jack arch

Steel properties



REFERENCE	CALCULATIONS	OUTPUT

View of analysis model



The structure has been modelled in accordance with the AIP reference 0059-A-AS-AIP-001. Refer to Appendix B of the AIP for the idealised diagram dimensions.

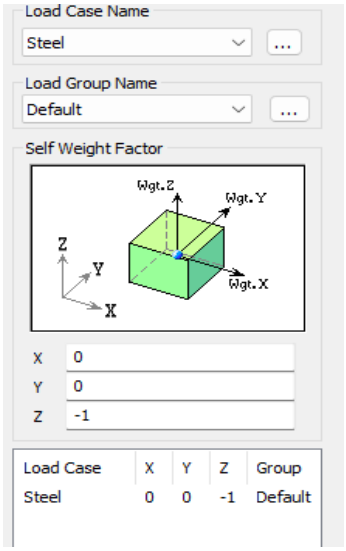
REFERENCE	CALCULATIONS	OUTPUT

Loads

CS 454
Table 4.1.1a

Plain concrete unit weight	=	2300	kg/m^3		
Steel unit weight	=	7850	kg/m^3	→ 77.0	kN/m^3
Tarmac unit weight	=	2400	kg/m^3		

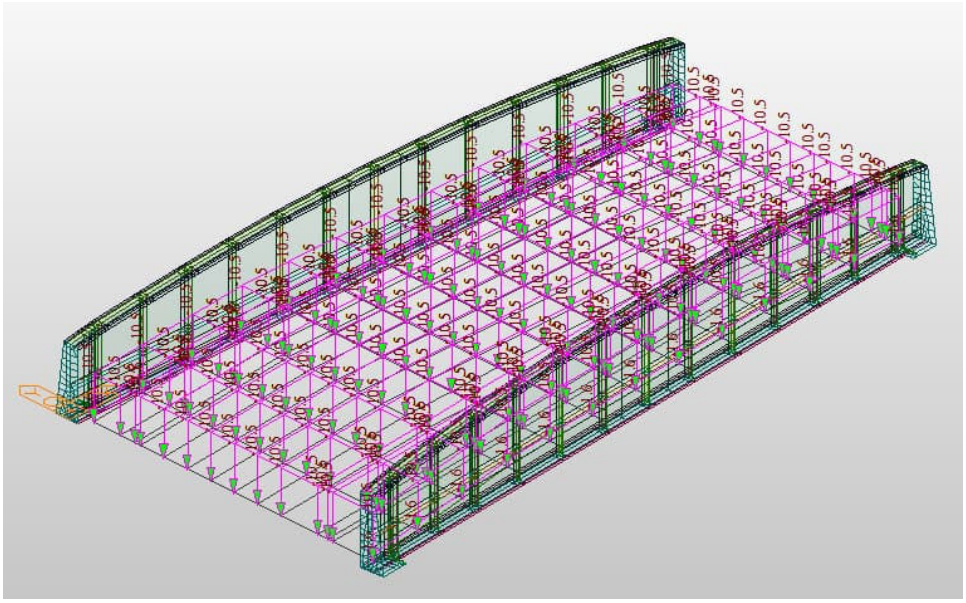
The steel self-wieight is applied automatically through the self-weight feature in Midas



Section "Analysis" of these calculations

Average concrete fill depth	=	464	mm
Average udl from concrete	=	10.5	kN/m^2

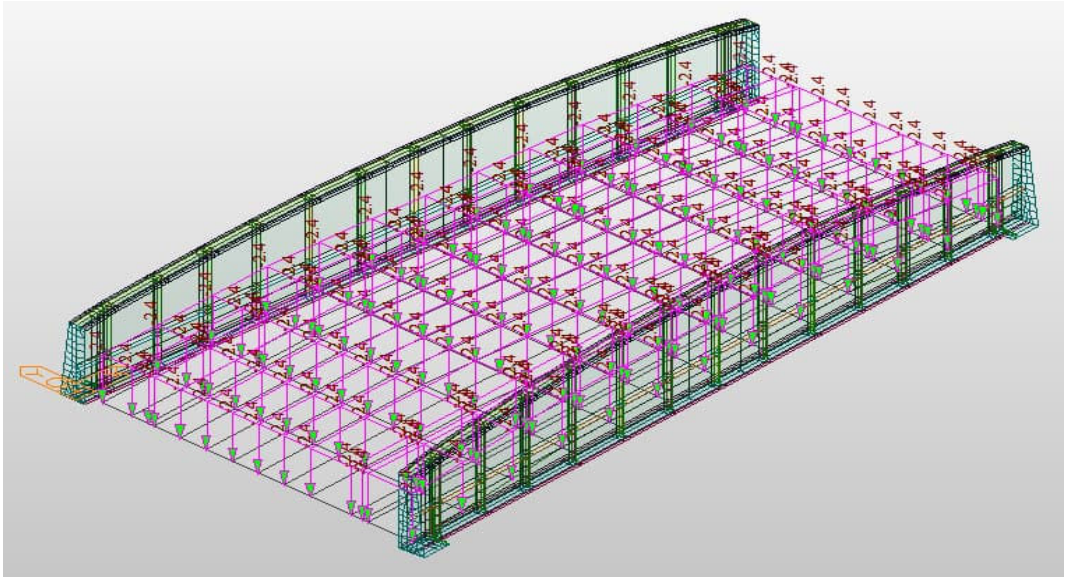
Applied UDL:



Assumed surfacing depth	=	100	mm

UDL from surfacing = 2.4 kN/m^2

Applied UDL:



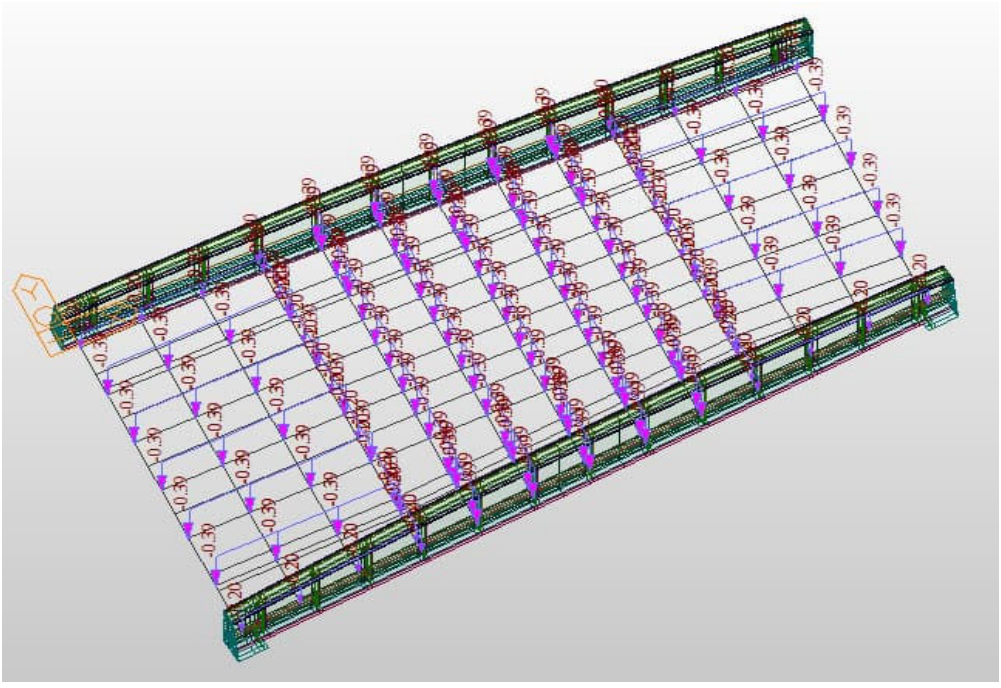
Site measurement

Steel hogging plate thickness 5 mm

Hogging plate UDL 0.005 x 77.0 = 0.39 kN/m^2

Span between crossbeams	1.02	m
UDL per cross beam (inner)	0.39	kN/m
UDL per cross beam (edge)	0.20	kN/m
Span between longitudinal. beams	1.01	m
UDL per long beam (inner)	0.39	kN/m
UDL per long beam (edge)	0.19	kN/m

Applied UDLs:



Moving load analysis

REFERENCE	CALCULATIONS	OUTPUT

CS 454 cl. 5.9

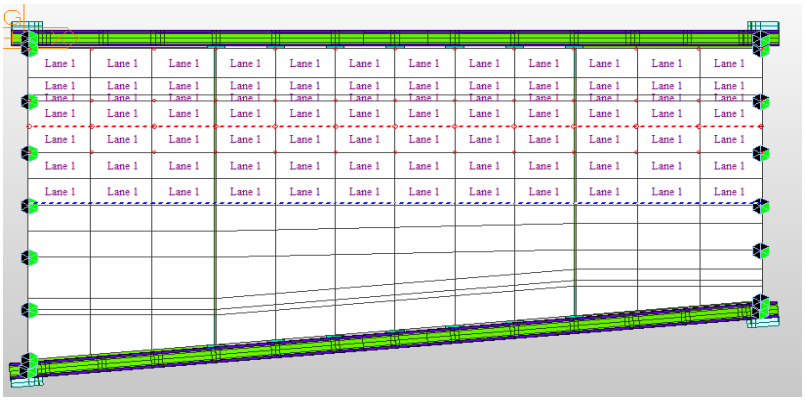
Traffic loads have been applied using the Midas Moving Load feature in accordance with CS 454 ALL model 1 with all vehicles from Appendix B.

Lanes are defined in accordance with CS 454, comprising a single lane and remaining area(s).

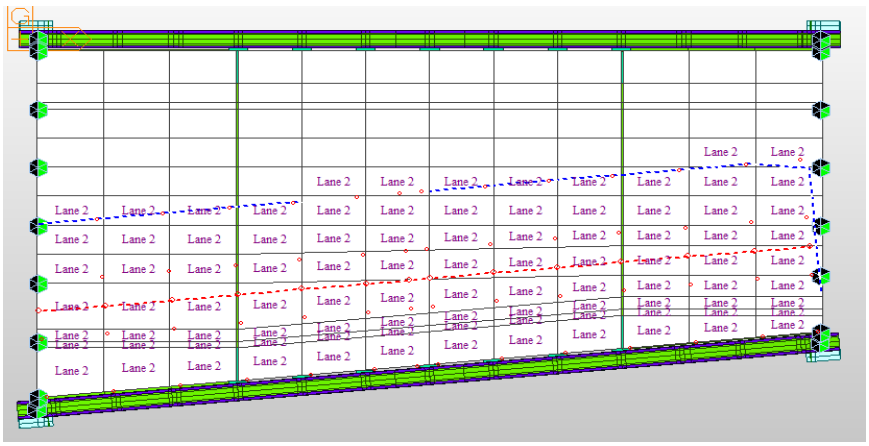
Physical carriageway width	4.95	m	(north end)
Notional lane width	3	m	
Number of notional lanes which fit within the carriageway			n = 1
Remaining area	1.95	m	

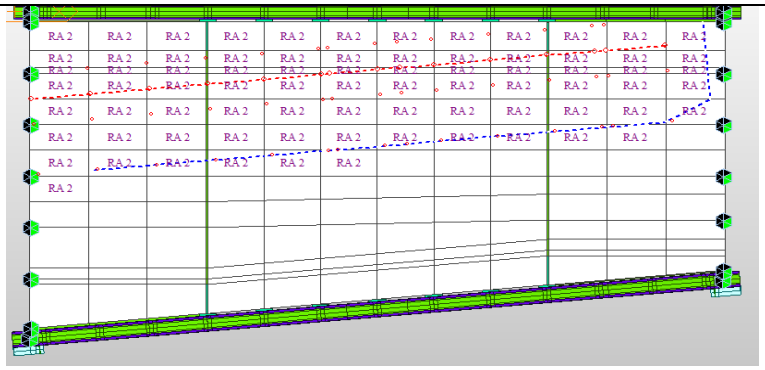
Three main configurations have been analysed with the lane placed against each edge girder and in the middle.

Lane against west edge

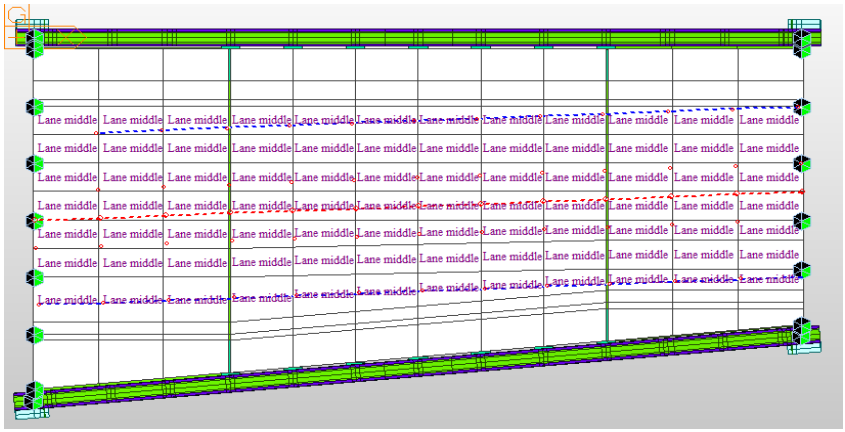


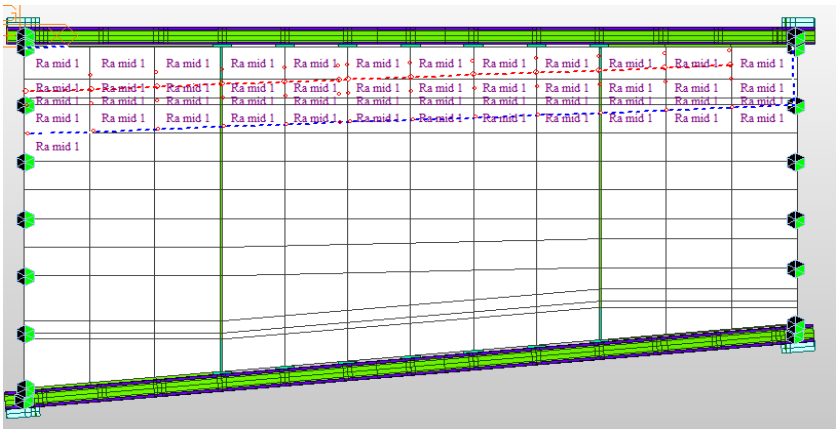
Lane against east edge





Lane in the middle





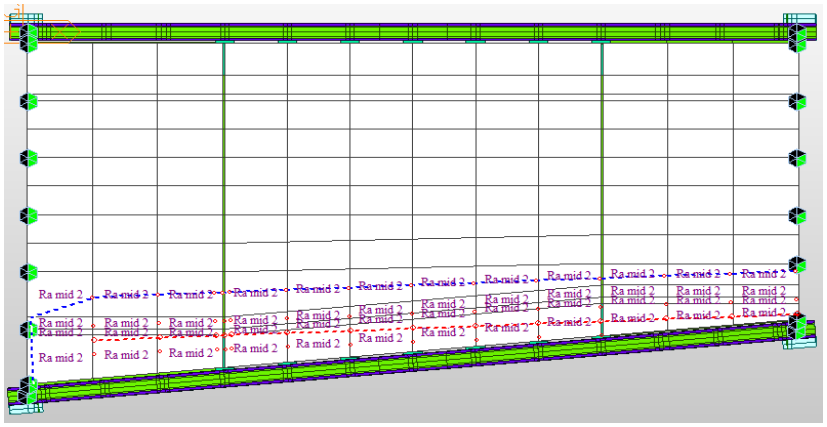
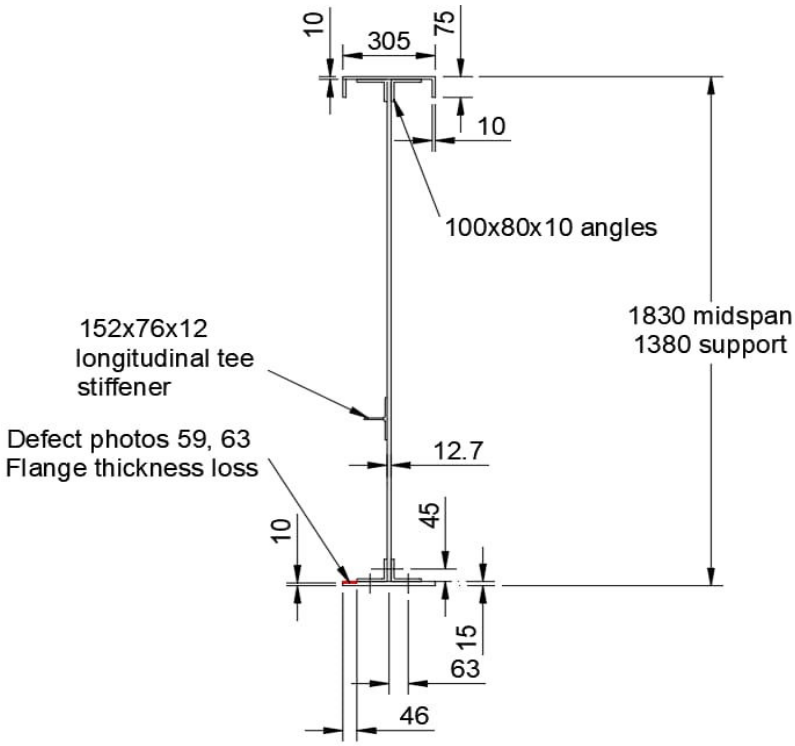
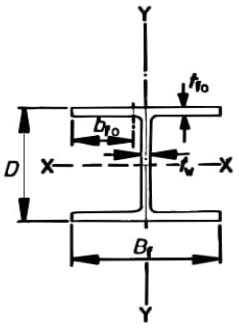


Plate Girders





Height of section - midspan	D =	1830	mm
Height of section - at supports	D =	1380	mm
Height of section - at quarter span	D =	1740	mm
Top flange thickness		10	mm
Top flange width		305	mm
Top flange vertical plate thickness		10	mm
Top flange vertical depth		75	mm
Angle horizontal length		100	mm
Angle vertical length		80	mm
Angle thickness		10	mm
Web thickness		12.7	mm
Bottom flange width overall		305	mm
Bottom flange thickness		15	mm
Bottom flange corroded thickness (photo 59, 63)		10	mm
Bottom flange corroded width (photo 59, 63)		46	mm
Rivet opening diameter		18	mm
Bottom web rivet offset from bottom flange's top surface		45	mm

Gross section properties

Second moment of area $I = \sum (b d^3 / 12 + A y'^2)$

$y' = y_E - y$

where
- b - breadth of element
- d - depth of element
- A - Area of element
- $y_{xc} = y_E$ - Distance between top fibre and elastic neutral axis
- y - Distance from top fibre to centroid of element

Note: the longitudinal stiffeners are curtailed and not connected to transverse stiffeners hence ignored.

Mid-span about major (x-x) axis:

Element	b mm	d mm	A mm^2	y mm	Ay mm^3	y' mm	I mm^4
Flange top	305.0	10.0	3.1E+03	5.0	1.53E+04	870.7	2.31E+09
Flange top vert. x2	20.0	65.0	1.3E+03	42.5	5.53E+04	833.2	9.03E+08
Top angle flange x2	200.0	10.0	2.0E+03	15.0	3.00E+04	860.7	1.48E+09
Top angle web x2	20.0	70.0	1.4E+03	55.0	7.70E+04	820.7	9.44E+08
Web	12.7	1805.0	2.3E+04	912.5	2.09E+07	-36.8	6.25E+09
Web loss	-4.7	80.0	-3.8E+02	1129	-4.25E+05	-253.3	-2.43E+07
Bot angle flange x2	200.0	10.0	2.0E+03	1810	3.62E+06	-934.3	1.75E+09
Bot angle web x2	20.0	70.0	1.4E+03	1770.0	2.48E+06	-894.3	1.12E+09
Flange bottom 1	259.0	15.0	3.9E+03	1822.5	7.08E+06	-946.8	3.48E+09
Flange bottom 2	46.0	10.0	4.6E+02	1825	8.40E+05	-949.3	4.15E+08
Bottom rivet web	-32.7	18.0	-5.9E+02	1770	-1.04E+06	-894.3	-4.71E+08
Bot. rivet flange x2	-36.0	25	-9.0E+02	1817.5	-1.64E+06	-941.8	-7.98E+08
Total			**3.7E+04**		3.20E+07		**1.737E+10**

Distance from ENA to comp fibre: $y_{xc} = \sum Ay / \sum A$ = 875.7 mm

Second moment of area: I_x = 1.74E+10 mm^4

Radius of gyration: $r_x = (I_x / A)^{0.5}$ = (1.74E+10 / 3.7E+04)$^{0.5}$ = 689.2 mm

Distance from ENA to tensile fibre: y_{xt} = 954.3 mm

Plastic neutral axis: y_P = (∑A / 2 - (area of top flange and top angles)) / (thickness of web) + thickness of top flange

y_P = 838.89 mm

Minimum distance from ENA to top/bottom of web: y_{xw} = 865.7 mm

Quarter-span about major (x-x) axis:

Element	b mm	d mm	A mm^2	y mm	Ay mm^3	y' mm	I mm^4
Flange top	305.0	10.0	3.1E+03	5.0	1.53E+04	827.1	2.09E+09
Flange top vert. x2	20.0	65.0	1.3E+03	42.5	5.53E+04	789.6	8.11E+08
Top angle flange x2	200.0	10.0	2.0E+03	15.0	3.00E+04	817.1	1.34E+09
Top angle web x2	20.0	70.0	1.4E+03	55.0	7.70E+04	777.1	8.46E+08
Web	12.7	1715.0	2.2E+04	867.5	1.89E+07	-35.4	5.37E+09
IFA photo 85 Web loss	-5.7	60.0	-3.4E+02	1039	-3.55E+05	-206.9	-1.47E+07
Bot angle flange x2	200.0	10.0	2.0E+03	1720	3.44E+06	-887.9	1.58E+09
Bot angle web x2	20.0	70.0	1.4E+03	1680.0	2.35E+06	-847.9	1.01E+09
Flange bottom 1	259.0	15.0	3.9E+03	1732.5	6.73E+06	-900.4	3.15E+09
IFA Photo 72 Flange bottom 2	46.0	10.0	4.6E+02	1735	7.98E+05	-902.9	3.75E+08
Bottom rivet web	-32.7	18.0	-5.9E+02	1680	-9.89E+05	-847.9	-4.23E+08
Bot. rivet flange x2	-36.0	25	-9.0E+02	1727.5	-1.55E+06	-895.4	-7.22E+08
Total			**3.5E+04**		2.95E+07		**1.539E+10**

Distance from ENA to comp fibre: $y_{xc,2} = \sum Ay / \sum A$ = 832.1 mm

Second moment of area: $I_{x,2}$ = 1.54E+10 mm^4

Radius of gyration: $r_{x,2} = (I_{x,2} / A_2)^{0.5}$ = (1.54E+10 / 3.5E+04)$^{0.5}$ = 659.0 mm

Distance from ENA to tensile fibre: $y_{xt,2}$ = 907.9 mm

Plastic neutral axis: $y_{P,2}$ = (∑A / 2 - (area of top flange and top angles)) / (thickness of web) + thickness of top flange

$y_{P,2}$ = 795.23 mm

Minimum distance from ENA to top/bottom of web: $y_{xw,2}$ = 822.1 mm

At south east support about major (x-x) axis:

Element	b mm	d mm	A mm^2	y mm	Ay mm^3	y' mm	I mm^4
IFA photo 81 Flange top	265.0	10.0	2.7E+03	5.0	1.33E+04	637.9	1.08E+09
Flange top vert. x2	20.0	65.0	1.3E+03	42.5	5.53E+04	600.4	4.69E+08
Top angle flange x2	200.0	10.0	2.0E+03	15.0	3.00E+04	627.9	7.89E+08
Top angle web x2	20.0	70.0	1.4E+03	55.0	7.70E+04	587.9	4.84E+08
Web	12.7	1355.0	1.7E+04	687.5	1.18E+07	-44.6	2.67E+09
			0.0E+00		0.00E+00	642.9	0.00E+00
Bot angle flange x2	200.0	10.0	2.0E+03	1360	2.72E+06	-717.1	1.03E+09
Bot angle web x2	20.0	70.0	1.4E+03	1320	1.85E+06	-677.1	6.42E+08
IFA photo 27 and 76 Flange bottom	224.0	15.0	3.4E+03	1373	4.61E+06	-729.6	1.79E+09
			0.0E+00		0.00E+00	642.9	0.00E+00
Bottom rivet web	-32.7	18.0	-5.9E+02	1320	-7.77E+05	-677.1	-2.70E+08
Bot. rivet flange x2	-36.0	25	-9.0E+02	1368	-1.23E+06	-724.6	-4.73E+08
Total			**3.0E+04**		1.92E+07		**8.205E+09**

Distance from ENA to comp fibre $y_{xc,3}$ = ∑Ay / ∑A = 642.9 mm

Second moment of area $I_{x,3}$ = 8.20E+09 mm^4

Radius of gyration $r_{x,3} = (I_{x,3} / A_3)^{0.5}$ = (8.20E+09 / 3.0E+04)$^{0.5}$ = 524.5 mm

Distance from ENA to tensile fibre $y_{xt,3}$ = 737.1 mm

Plastic neutral axis $y_{P,3}$ = (∑A / 2 - (area of top flange and top angles)) / (thickness of web) + thickness of top flange

$y_{P,3}$ = 605.67 mm

Minimum distance from ENA to top/bottom of web $y_{xw,3}$ = 632.9 mm

Mid-span about minor (y-y) axis:

	Element	b mm	d mm	A mm^2	y mm	Ay mm^3	y' mm	I mm^4
1	top flange vert. Left	65.0	10.0	6.5E+02	5.0	3.25E+03	146.9	1.40E+07
2	top flange hor.	10.0	305.0	3.1E+03	152.5	4.65E+05	-0.6	2.36E+07
3	top flange vert. Right	65.0	10.0	6.5E+02	300.0	1.95E+05	-148.1	1.43E+07
4	angles hor. Left x2	20.0	100.0	2.0E+03	96.0	1.92E+05	55.9	7.92E+06
5	angles ver. Left x2	140.0	10.0	1.4E+03	141	1.97E+05	10.9	1.78E+05
6	angles hor. right x2	20.0	100.0	2.0E+03	209.0	4.18E+05	-57.1	8.19E+06
7	angles ver. right x2	140.0	10.0	1.4E+03	164.0	2.30E+05	-12.1	2.16E+05
8	red. flange bot left	10.0	46.0	4.6E+02	23	1.06E+04	128.9	7.73E+06
9	flange bot. full thick	15.0	259.0	3.9E+03	175.5	6.82E+05	-23.6	2.39E+07
10	Web rivets x2	-36.0	32.7	-1.2E+03	152.5	-1.80E+05	-0.6	-1.05E+05
11	Flange rivets top&bot	-45.0	18.0	-8.1E+02	215.5	-1.75E+05	-63.6	-3.30E+06
12	Web	1805.0	12.7	2.3E+04	152.5	3.50E+06	-0.6	3.16E+05
13	Web loss	-80.0	4.7	-3.8E+02	152.5	-5.73E+04	-0.6	-8.23E+02
	Total			**3.6E+04**		5.48E+06		**9.696E+07**

Distance from ENA to comp fibre y_{yc} = ∑Ay / ∑A = 151.9 mm

Second moment of area I_y = 9.70E+07 mm^4

Radius of gyration $r_y = (I_y / A)^{0.5}$ = (9.70E+07 / 3.6E+04)$^{0.5}$ = 51.9 mm

Second moment of area of top flange I_c = 5.19E+07 mm^4

Second moment of area of bottom flange I_t = 3.16E+07 mm^4 (items 8 + 9)

REFERENCE	CALCULATIONS	OUTPUT

Section classification

Web

Depth of web plate on the compressive side of the plastic NA = y_P - thickness of top flange = 828.89 mm

Depth of the web plate = 1805 mm

BS 5400-3 cl. 9.3.7.2

Ratio m = 0.46

Nominal yield stress σ_{yw} = 246 MPa

Thickness of web plate t_w = 12.7 mm

Limiting depth $\frac{34t_w}{m}\sqrt{\frac{355}{\sigma_{yw}}}$ when m does not exceed 0.5; and $\frac{374t_w}{(13m-1)}\sqrt{\frac{355}{\sigma_{yw}}}$ when m exceeds 0.5; → 1130.26 mm < 1805 mm

i.e. section is not compact

Effective section

BS 5400-3 cl. 8.2

Shear lag

Breadth of outstand to centre of web b = 152.5 mm

Span length L = 12.5 m

k = (1 - 0.15 b / L) = 1.00

b / L = 0.0122

Ψ = 1.00

Effective breadth k Ψ b = 151.5 mm i.e. approx. full flange width is effective

cl. 9.4.2.3 Effective area of tension flange

Net area with holes deducted is already considered in properties calculated above.

Effective areas of compression flanges or plates in compression

cl. 9.4.2.4 Holes in compression flange are filled with rivets hence not deducted. Shear lag as above.

cl. 9.4.2.5 Effective web

cl. 9.4.2.5.1 Note: the longitudinal stiffeners are curtailed and not connected to transverse stiffeners hence ignored.

Depth of web measured in its plane from the elastic neutral axis of the gross section of the beam to the compressive edge of the web y_c = 865.7 mm

$\frac{y_c}{t_w}\sqrt{(\sigma_{yw}/355)}$ = 56.7 < 68 OK

t_{we} = t_w = 12.7 mm

Shape limitations

cl. 9.3.2.1 Flange

σ_y = 246 MPa

$12\sqrt{355/\sigma_y}$ = 14.42

b_{f0} = 89.5 mm

t_{f0} = 10 mm

b_{f0} / t_{f0} = 8.95 < 14.42

i.e. OK OK

cl. 9.3.1 σ_{ys} = 246 MPa

REFERENCE	CALCULATIONS	OUTPUT

cl. 9.3.4 The top flange downstands are treated as stiffeners

h_s = 65 mm

t_s = 10 mm

σ_{ys} = 246 MPa

$\frac{h_s}{t_s}\sqrt{\frac{\sigma_{ys}}{355}}$ = 5.4 < 10

i.e. OK, Kc = 1 — OK

cl. 9.4.2.4 k_h = 1 otherwise $k_h A_c$ would exceed the gross area.

Transverse stiffeners

cl. 9.3.4

Take vertical stiffener 140 x 75 x 11.5

h_s = 63.5 mm

t_s = 11.5 mm

σ_{ys} = 246 MPa

$\frac{h_s}{t_s}\sqrt{\frac{\sigma_{ys}}{355}}$ = 4.6 < 10 i.e. OK — OK

cl. 9.3.1 σ_{ys} = 246 MPa

Plastic moment of resistance

y'_p = ABS($y_P - y$)

Mid-span about major (x-x) axis:

Element	b mm	d mm	A mm^2	y mm	y'_p mm	Ay'_p mm^3
Flange top	305.0	10.0	3.1E+03	5	833.9	2.54E+06
Flange top vert. x2	20.0	65.0	1.3E+03	43	796.4	1.04E+06
Top angle flange x2	200.0	10.0	2.0E+03	15	823.9	1.65E+06
Top angle web x2	20.0	70.0	1.4E+03	55	783.9	1.10E+06
Web top	12.7	828.9	1.1E+04	424	414.4	4.36E+06
Web bottom	12.7	976.1	1.2E+04	1327	488.1	6.05E+06
Web loss	-4.7	80.0	-3.8E+02	1129	290.1	-1.09E+05
Bot angle flange x2	200.0	10.0	2.0E+03	1810	971.1	1.94E+06
Bot angle web x2	20.0	70.0	1.4E+03	1770	931.1	1.30E+06
Flange bottom 1	259.0	15.0	3.9E+03	1823	983.6	3.82E+06
Flange bottom 2	46.0	10.0	4.6E+02	1825	986.1	4.54E+05
Bottom rivet web	-32.7	18.0	-5.9E+02	1770	931.1	-5.48E+05
Bot. rivet flange x2	-36.0	25.0	-9.0E+02	1818	978.6	-8.81E+05
totals:			3.7E+04			2.27E+07

Plastic moment of resistance $M_{pe} = \Sigma A y'_p \sigma_{ys}$ = 5582 kNm

cl. 9.7.1 Plastic modulus $Z_{pe} = M_{pe} / \sigma_{ys} = \Sigma A y'_p$ = 2.27E+07 mm^3

Quarter-span about major (x-x) axis:

Element	b mm	d mm	A mm^2	y mm	y'_p mm	Ay'_p mm^3
Flange top	305.0	10.0	3.1E+03	5	790.2	2.41E+06
Flange top vert. x2	20.0	65.0	1.3E+03	43	752.7	9.79E+05
Top angle flange x2	200.0	10.0	2.0E+03	15	780.2	1.56E+06
Top angle web x2	20.0	70.0	1.4E+03	55	740.2	1.04E+06
Web top	12.7	785.2	1.0E+04	403	392.6	3.92E+06
Web bottom	12.7	929.8	1.2E+04	1260	464.9	5.49E+06
Web loss	-4.7	80.0	-3.8E+02	1039	243.8	-9.17E+04
Bot angle flange x2	200.0	10.0	2.0E+03	1720	924.8	1.85E+06
Bot angle web x2	20.0	70.0	1.4E+03	1680	884.8	1.24E+06
Flange bottom 1	259.0	15.0	3.9E+03	1733	937.3	3.64E+06
Flange bottom 2	46.0	10.0	4.6E+02	1735	939.8	4.32E+05
Bottom rivet web	-32.7	18.0	-5.9E+02	1680	884.8	-5.21E+05
Bot. rivet flange x2	-36.0	25.0	-9.0E+02	1728	932.3	-8.39E+05
totals:			3.5E+04			2.11E+07

REFERENCE	CALCULATIONS	OUTPUT

Plastic moment of resistance $M_{pe} = \Sigma A\, y'_p \quad \sigma_{ys} = 5184$ kNm

cl. 9.7.1 Plastic modulus $Z_{pe} = M_{pe} / \sigma_{ys} = \Sigma A\, y'_p = 2.11E+07$ mm^3

Elastic modulus of effective section

From above the elastic moduli of the effective section are:

Compression fibre	$Z_{xc} = I_{xc} / y_{xc} = 1.98E+07$ mm^3	Mid-span	
	$Z_{xc} = I_{xc} / y_{xc} = 1.85E+07$ mm^3	Quarter span	
Tension fibre	$Z_{xt} = I_{xt} / y_{xt} = 1.82E+07$ mm^3	Mid-span	
With respect to web	$Z_{xw} = I_{xw} / y_{xw} = 2.01E+07$ mm^3	Mid-span	

see above Nominal yield stresses

$\sigma_{yc} = 246$ MPa

$\sigma_{yt} = 246$ MPa

$\sigma_{yw} = 246$ MPa

Beam with intermediate restraints by u-frames

cl. 9.6.4.1.3 u-frame spacing $l_R = 2040$ mm

flexibility of the joint $f = 5E-11$ rad/Nmm

$d_1 = 1438$ mm

$d_2 = 1616$ mm

width of web plate in the vertical 12.7 mm x 32 = 406.4 mm

take vertical stiffener 140 x 75 x 11.5

Consider section loss to roadside vertical stiffener (photo 87 for example)
Consider thickness loss to web (IFA photo 85 worst case)

Properties of vertical stiffener with section loss

Element	b mm	d mm	A mm^2	y mm	Ay mm^3	y' mm	I mm^4
stiffener fl top	140.0	11.5	1.6E+03	5.8	9.26E+03	15.2	3.91E+05
rivets stiffener 2x	-36.0	11.5	-4.1E+02	5.8	-2.38E+03	15.2	-1.01E+05
girder web	406.4	7.0	2.8E+03	15.0	4.27E+04	6.0	1.13E+05
rivets web 2x	-36.0	7.0	-2.5E+02	15.0	-3.78E+03	6.0	-1.00E+04
stiffener fl bottom	140.0	11.5	1.6E+03	24.3	3.90E+04	-3.3	3.50E+04
rivets stiffener 2x	-36.0	11.5	-4.1E+02	24.3	-1.00E+04	-3.3	-9.01E+03
stiffener web	11.5	63.5	7.3E+02	61.8	4.51E+04	-40.8	1.46E+06
Total			**5.7E+03**		1.20E+05		**1.879E+06**

Distance from ENA to top fibre $\Sigma Ay / \Sigma A = 21.0$ mm

Total depth 93.5 mm

Distance from ENA to bottom fibre 72.5 mm

Maximum distance from ENA to extreme fibre 72.5 mm

Second moment of area of the vertical 1.88E+06 mm^4

Radius of gyration of effective stiffener $(I / A)^{0.5} = 18$ mm

Elastic modulus of the vertical stiffener 2.6E+04 mm^3

Properties of vertical stiffener without section loss

Element	b mm	d mm	A mm^2	y mm	Ay mm^3	y' mm	I mm^4
stiffener web top	11.5	63.5	7.3E+02	31.8	2.32E+04	49.6	2.04E+06
stiffener fl top	140.0	11.5	1.6E+03	69.3	1.11E+05	12.1	2.53E+05
rivets stiffener 2x	-36.0	11.5	-4.1E+02	69.3	-2.87E+04	12.1	-6.52E+04
girder web	406.4	12.7	5.2E+03	81.4	4.20E+05	0.0	6.94E+04
rivets web 2x	-36.0	12.7	-4.6E+02	81.4	-3.72E+04	0.0	-6.15E+03
stiffener fl bottom	140.0	11.5	1.6E+03	93.5	1.50E+05	-12.1	2.53E+05
rivets stiffener 2x	-36.0	11.5	-4.1E+02	93.5	-3.87E+04	-12.1	-6.52E+04
stiffener web bot.	11.5	63.5	7.3E+02	131.0	9.56E+04	-49.6	2.04E+06
Total			**8.6E+03**		6.96E+05		**4.524E+06**

Distance from ENA to top fibre $y_{xc} = \sum Ay / \sum A$ = 81.4 mm

Total depth 162.7 mm

Distance from ENA to bottom fibre 81.4 mm

Maximum distance from ENA to extreme fibre 81.4 mm

Radius of gyration of effective stiffener $r_{se,2} = (I_{se} / A)^{0.5}$ = 23 mm

Second moment of area of the vertical $I_{se,2}$ = 4.52E+06 mm^4 = I_1

Elastic modulus of the vertical stiffener $Z_{se,2}$ = 5.6E+04 mm^3

Area $A_{se,2}$ = 8.6E+03 mm^2

Area of web plate only $A_{se,w,2}$ = 5.2E+03 mm^2

Second moment of area of the crossbeam I_2 = 3.8E+08 mm^4

u = 0.50

B = 6120 mm

E = 205000 MPa

As the section losses are local, significant reduction in deflection is not expected and hence full stiffener is assumed for the calculation of the deflection. Section losses will be considered in strength verifications of the stiffeners.

Lateral deflection due to unit force:

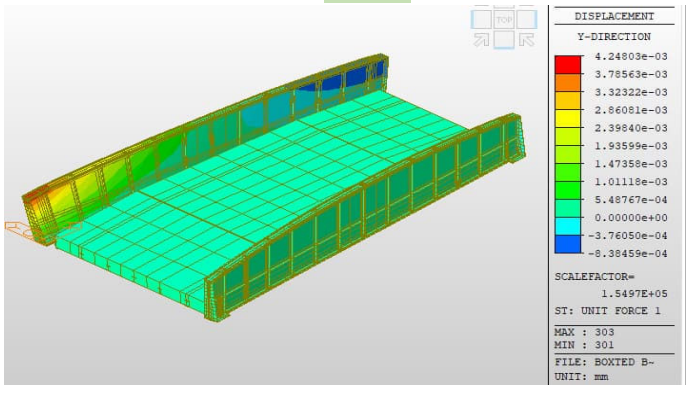
$$\delta_R = \frac{{d_1}^3}{3EI_1} + \frac{uB{d_2}^2}{EI_2} + fd_2{}^2 = 0.0011 + 0.0001 + 0.0001 = 0.0013 \text{ mm}$$

cl. 9.6.4.1.1.2 k_3 = 1

Model D09 δ_{e1} = 0.004 mm

δ_{e2} = 0.004 mm

$\delta_{e,max}$ = 0.004 mm



REFERENCE	CALCULATIONS	OUTPUT

σ_{yc} = 246 MPa

I_c = 5.19E+07 mm^4

$$\ell_1 = (EI_c\ell_R\delta_R)^{0.25} = 2306 \text{ mm}$$

$$X = \frac{\ell_1^{\,3}}{\sqrt{2}EI_c\delta_{e,\,max.}} = 0.204$$

$$k_5 = 2.22 + \frac{0.69}{X + 0.5} = 3.2$$

The effective length is the greater of:

$\ell_e = k_2k_3k_5\ell_1$ but not less than $k_3\ell_R$ nor greater than L

for simply supported spans $\ell_e = \pi k_2 \sqrt{\frac{EI_c}{L}\left(\delta_{e1} + \delta_{e2}\right)}$ or

for continuous beams, the value of ℓ_e given by **9.6.2**b)

BS 5400-3 cl. 9.6.2

L = 12.5 m

k2 = 1

$k_2k_3k_5\ell_1$ = 7382 mm

$k_3\ell_R$ = 2040 mm

$\pi k_2 \sqrt{\frac{EI_c}{L}\left(\delta_{e1} + \delta_{e2}\right)}$ = 8201 mm

Hence l_e = 8201 mm > 2040 mm = l_R

cl. 9.7.2 below note 2 Hence λ_{LT} should be taken as l_e / r_{yc}

cl. 9.7.2 below note 2 Properties of compression flange to cl. 9.7.2 - About minor (y-y) axis:

	Element	b mm	d mm	A mm^2	y mm	Ay mm^3	y' mm	I mm^4
1	top flange vert. Left	65.0	10.0	6.5E+02	5.0	3.25E+03	145.9	1.38E+07
2	top flange hor.	10.0	305.0	3.1E+03	152.5	4.65E+05	-1.6	2.37E+07
3	top flange vert. Right	65.0	10.0	6.5E+02	300.0	1.95E+05	-149.1	1.44E+07
4	angles hor. Left	10.0	100.0	1.0E+03	96.0	9.60E+04	54.9	3.85E+06
5	angles ver. Left	70.0	10.0	7.0E+02	141	9.87E+04	9.9	7.49E+04
6	angles hor. Right	10.0	100.0	1.0E+03	209.0	2.09E+05	-58.1	4.21E+06
7	angles ver. Right	70.0	10.0	7.0E+02	164.0	1.15E+05	-13.1	1.25E+05
8	Web 1/3	601.7	12.7	7.6E+03	152.5	1.17E+06	-1.6	1.22E+05
9	Web rivet	-18.0	32.7	-5.9E+02	152.5	-8.98E+04	-1.6	-5.39E+04
10	Flange rivet top	-20.0	18.0	-3.6E+02	215.5	-7.76E+04	-64.6	-1.51E+06
	Total			**1.4E+04**		2.18E+06		**5.876E+07**

Distance from ENA to comp fibre $y_{yc} = \Sigma Ay / \Sigma A$ = 150.9 mm

Second moment of area I_{yc} = 5.88E+07 mm^4

Radius of gyration $r_{yc} = (I_{yc} / A)^{0.5}$ = (5.88E+07 / 1.4E+04) $^{0.5}$ = 63.8 mm

$\lambda_{LT} = l_e / r_{yc}$ = 129

Plastic moment of resistance M_{pe} = 5582 kNm

BS 5400-3 cl. 9.8

M_{ult} = min ($Z_{xc}\,\sigma_{yc}$, $Z_{xt}\,\sigma_{yt}$, Z_{xw} σ_{yw})

REFERENCE	CALCULATIONS	OUTPUT

$$M_{ult} = \min \begin{cases} 1.98E+07 \times 246 \\ 1.82E+07 \times 246 \\ 2.01E+07 \times 246 \end{cases} = 4471 \text{ kNm}$$

$$\lambda_{LT}\sqrt{\left(\frac{\sigma_{yc}}{355}\right)\left(\frac{M_{ult}}{M_{pe}}\right)} = 96$$

BS 5400-3 cl. 9.7.1 c)

$L / l_w = \max(\text{Integer}(L / l_e - 1); 1) = 1$

$l_w = 12.5$ m

CS 456 Appendix A Figure 11

$M_R / M_{ult} = 0.45$

$M_R = 2012$ kNm

$\gamma_{f3} = 1.1$

$\gamma_m = 1.05$

BS 5400-3 cl. 9.9.1.2

Bending moment resistance
Assuming u-frames are adequate

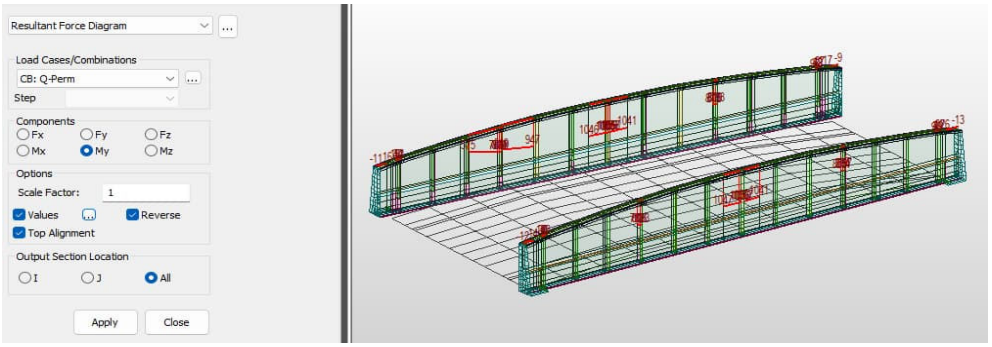
$$M_D = \frac{M_R}{\gamma_m \gamma_{f3}} = 1742 \text{ kNm}$$

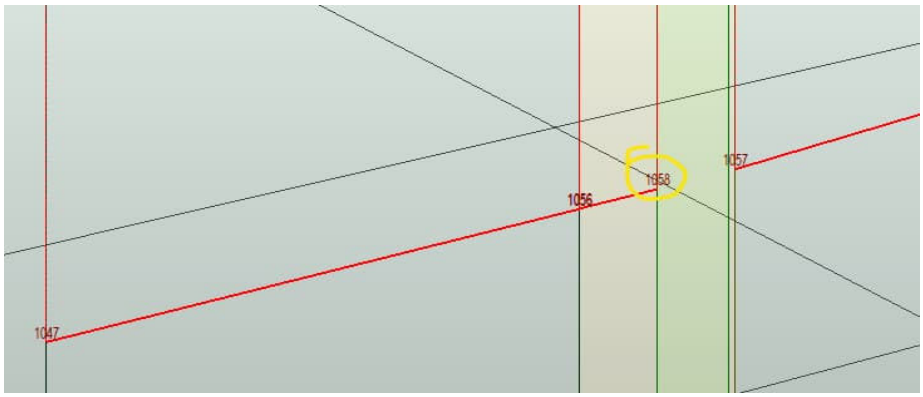
With condition factor 0.85 M_D = 1481 kNm

Model rev. D08

Bending moment M = 1058 kNm Permanent loads

Utilisation = 0.71 OK





Flange strength

cl. 9.10.2 Strength of stiffened flanges

As the top flange has vertical downstands, it is checked as a stiffened flange.

cl. 9.10.2.1 Determine the longitudinal stress at the mid-plane of the flange plate

Height of mid-plane of the flange plate from top fibre 5 mm

REFERENCE	CALCULATIONS	OUTPUT

Height with respect to ENA y_f = 875.721 - 5 = 870.721 mm — Mid-span

Second moment of area I_x = 1.74E+10 mm^4 — Mid-span

Height with respect to ENA y_f = 832.1 - 5 = 827.109 mm — Quarter-span

Second moment of area I_x = 1.54E+10 mm^4 — Quarter-span

Longitudinal stress at the mid-plane of the flange plate

$$\sigma_f = \frac{M\ y_f}{I_x} =$$ 53.1 MPa — Permanent loads, Mid-span

Longitudinal stress at the mid-plane of the flange plate

$$\sigma_f = \frac{M\ y_f}{I_x} =$$ 43.8 MPa — Permanent loads, Quarter-span

cl. 9.5.7.1 Distance between successive long. stiffeners and/or webs b = 147.5 mm

Thickness of the flange t_f = 10 mm

As defined in Figure 1 of BS 5400-3 t_{f0} = 10 mm

b_{f0} = 89.5 mm

Radius of curvature R_f = 46662 mm

Transverse bending stress due to curvature

$$\sigma_2 = \frac{3\sigma_f b_{fo}^2}{R_f t_{fo}} =$$ 2.7 MPa — Permanent loads, Mid-span

$$\sigma_2 = \frac{3\sigma_f b_{fo}^2}{R_f t_{fo}} =$$ 2.3 MPa — Permanent loads, Mid-span

cl. 9.10.2.1 In-plane shear stress (box beam) τ_1 = 0 MPa

Determine the shear stress in the flange plate at the junction with the web of the beam

Model ver. D08 Shear force on the beam at mid-span V = 34 kN — Permanent loads

Properties of top flange about x-x axis at mid-span

Element	b mm	d mm	A mm^2	y mm	Ay mm^3	y' mm	Ay' mm^3
Flange top	305.0	10.0	3.1E+03	5.0	1.53E+04	870.7	2.66E+06
Flange top vert. x2	20.0	65.0	1.3E+03	42.5	5.53E+04	833.2	1.08E+06
Top angle flange x2	200.0	10.0	2.0E+03	15.0	3.00E+04	860.7	1.72E+06
Top angle web x2	20.0	70.0	1.4E+03	55.0	7.70E+04	820.7	1.15E+06
Total			**7.8E+03**		1.78E+05		6.61E+06

Distance from ENA to comp fibre (whole section) y_{xc} = 875.7 mm

Ay' = 6.6E+06 mm^3

cl. 9.5.8 Additional shear force from sloping flange

Already accounted for hence taken as 0 here = 0 kN — Permanent loads

Total shear V = 34 kN — Permanent loads

Breadth of flange b_f = 305 mm

Shear stress in flange $\tau_2 = V\,Ay' / b_f I_x$ = 0.04 MPa

REFERENCE	CALCULATIONS	OUTPUT

$\tau = \tau_1 + 0.5\ \tau_2 = 0.02$ MPa — Permanent loads

cl. 9.10.2.1

$$\sigma_1^2 + \sigma_2^2 - \sigma_1\sigma_2 + 3\tau^2$$

$= 53.1^2 + 2.7^2 + 53.1 \times 2.7 + 3 \times 0.02^2 = 2967$ $(MPa)^2$ — Permanent loads

Section "'Materials and Factors" of these calculations

$\sigma_{yf} = 246$ MPa

cl. 9.10.2.1

$$\left(\frac{\sigma_{yf}}{\gamma_m \gamma_{f3}}\right)^2 = 45251 \ (MPa)^2$$

With condition factor:

$0.85 \times 45251 = 38463$ $(MPa)^2$ — OK

Utilisation $= \frac{2967}{38463} = 0.08$ — Permanent loads

Strength of longitudinal stiffener

The vertical plates in the top flange are checked as longitudinal stiffeners.

cl. 9.10.2.2 Spacing between logitudinal stiffeners $b = 295$ mm

cl. 9.4.2.4 Slenderness parameter $\lambda = \frac{b}{t_f}\sqrt{\frac{\sigma_{yf}}{355}} = 24.5$

Figure 5 $K_c = 0.5$

Effective width of flange for the stiffener $0.5\ K_c\ b = 74$ mm — (flange is to one side of stiffener)

Properties of effective stiffener

Element	b mm	d mm	A mm^2	y mm	Ay mm^3	y' mm	I mm^4
Stiffener vertical	10.0	65.0	6.5E+02	42.5	2.76E+04	833.2	4.51E+08
Flange portion	73.8	10.0	7.4E+02	5.0	3.69E+03	870.7	5.59E+08
Totals			1.4E+03		3.13E+04		1.01E+09

Centroid of stiffener from comp. fibre $\Sigma Ay / \Sigma A = 23$ mm

Centroid of entire section $y_{xc} = 876$ mm

Distance from section centroid to stiffener centroid $y_{Bs} = 853$ mm

cl. 9.10.2.3 Stress in stiffener $\sigma_a = \frac{M\ y_{Bs}}{I_x} = 52.0$ MPa — Permanent loads

cl. 9.10.4 Second moment of area of effective stiffener $I_{se} = 1.01E+09$ mm^4

Total width of the stiffened flange between the main beam webs $B = 146.2$ mm

Number of longitudinal stiffeners in width B $n = 1$

l = $\ell_e = 1.5\left(\frac{B}{t_f}\right)^{0.75}\left[I_{se}(n+1)\right]^{0.25} = 2377$ mm

Figure 19 Radius of gyration of effective stiffener $r_{se} = (I_{se}/A)^{0.5} = 853$ mm

Slenderness factor $\lambda = \frac{\ell}{r_{se}}\sqrt{\frac{\sigma_{ys}}{355}} = 2.3 = \frac{\ell}{r_{se}}\sqrt{\frac{\sigma_{ye}}{355}}$

REFERENCE	CALCULATIONS	OUTPUT

e_f = 450 mm

$\sigma_E = \pi^2 E/(\ell/r_{se})^2$ = 2.6E+05 MPa

$\zeta = -(1 - 1.35\sigma_a/\sigma_E)$ when obtaining $k_{\ell 1}$ = -1.0

$\zeta = (1 + 1.35\sigma_a/\sigma_E)$ when obtaining $k_{\ell 2}$ = 1.0

For obtaining k_{l1} $\Delta = \frac{\ell}{625} + \frac{\zeta r_{se}^2}{y_{Bs}} + \frac{e_f}{2}$ = -625 mm

Figure 19 NOTE

The middle component gives a relieving effect, hence it is ignored

$\Delta = \frac{\ell}{625} + \frac{e_f}{2}$ = 229 mm

For obtaining k_{l2} $\Delta = \frac{\ell}{625} + \frac{\zeta r_{se}^2}{y_{Bs}} + \frac{e_f}{2}$ = 1083 mm

Distance from the centroid of the effective stiffener section to the point on the stiffener furthest from the plate:

y_0 = 52 mm

Distance from the centroid of the effective stiffener section to the mid-plane of the flange plate:

y_z = 18 mm

$\eta = \frac{y_o \Delta}{r_{se}^2}$ when obtaining $k_{\ell 1}$ = 0.016

$\eta = \frac{y_z \Delta}{r_{se}^2}$ when obtaining $k_{\ell 2}$ = 0.026

k_{l1} = 1

k_{l2} = 0.98

cl. 9.10.2.3

τ_1 = 0 MPa

$\sigma_a + 2.5\tau_1 k_{s1}$ = σ_a = 52.0 MPa = $\sigma_a + 2.5\tau_1 k_{s2}$

$\sigma_{ye} = \sqrt{\sigma_{yf}^2 - 3\tau^2\gamma_m^2\gamma_{f3}^2}$ = 246 MPa

Material factor for longitudinal stiffeners in compression γ_m = 1.2

$\frac{k_{\ell 1}\sigma_{ys}}{\gamma_m\gamma_{f3}}$ = 186 MPa

With condition factor:

0.85 x 186 MPa = 158 MPa > 52 MPa OK OK

Utilisation = 0.33 Permanent loads

$\frac{k_{\ell 2}\sigma_{ye}}{\gamma_m\gamma_{f3}}$ = 182 MPa > 52 MPa

0.85 x 182 MPa = 155 MPa > 52 MPa OK OK

Utilisation = 0.34 Permanent loads

REFERENCE	CALCULATIONS	OUTPUT

Strength of u-frame restraints

cl. 9.12.3.1 Intermediate u-frames should be designed in accordance with cl. 9.12.3.2 and cl. 9.12.3.3

cl.9.12.3.2 l_e = 8201 mm

cl. 9.12.2 l_R = 2040 mm

δ_R = 0.0013 mm

l_w = 12500 mm

Maximum compressive stress in the flange from bending about the major axis of the girder (as above)

My/I = 53.4 MPa — Permanent loads

Stress from bending about the minor axis due to curvature (as shown previously)

σ_2 = 2.7 MPa — Permanent loads

Maximum compressive stress in the flange

σ_{fc} = 53.4 + 2.7 = 56.1 MPa — Permanent loads, Mid-span

σ_{fc} = 43.8 + 2.3 = 46.0 MPa — Permanent loads, Quarter-span

Note: BS 5400-3 refers to σ_{fc} as "strength" but it is assumed it refers to the "stress" applied in the flange (as clarified in CS 456)

$S = Z_{pc}/Z_{xc}$ = 1.15 — Mid-span

= 1.14 — Quarter-span

λ_{LT} = 128.6

$\sigma_{ci} = \frac{\pi^2 ES}{\lambda_{LT}^2}$ = 140 MPa — Mid-span

= 140 MPa — Quarter-span

cl. 9.6.4.1.1.2 **Properties of compression flange to cl. 9.6.4.1.1.2 - About minor (y-y) axis:**

	Element	b mm	d mm	A mm^2	y mm	Ay mm^3	y' mm	I mm^4
1	top flange vert. Left	65.0	10.0	6.5E+02	5.0	3.25E+03	144.4	1.36E+07
2	top flange hor.	10.0	305.0	3.1E+03	152.5	4.65E+05	-3.1	2.37E+07
3	top flange vert. Right	65.0	10.0	6.5E+02	300.0	1.95E+05	-150.6	1.47E+07
4	angles hor. Left	10.0	100.0	1.0E+03	96.0	9.60E+04	53.4	3.69E+06
5	angles ver. Left	70.0	10.0	7.0E+02	141	9.87E+04	8.4	5.56E+04
6	angles hor. Right	10.0	100.0	1.0E+03	209.0	2.09E+05	-59.6	4.38E+06
7	angles ver. Right	70.0	10.0	7.0E+02	164.0	1.15E+05	-14.6	1.54E+05
8	Flange rivet top	-20.0	18.0	-3.6E+02	215.5	-7.76E+04	-66.1	-1.58E+06
	Total			**7.4E+03**		1.1E+06		**5.9E+07**

Distance from ENA to comp fibre $y_c = \sum Ay / \sum A$ = 149.4 mm

Distance from ENA to tension fibre y_t = 155.6 mm

Second moment of area I_c = 5.87E+07 mm^4

Radius of gyration $r_c = (I_c/A)^{0.5}$ = (5.87E+07 / 7.4E+03)$^{0.5}$ = 89.1 mm

cl. 9.6.4.1.2 n = 4

CS 456 App. A m = n (typo in the standard, confirmed in CS 456 Appendix A)

BS 5400-3

$$F_R = \left(\frac{\sigma_{fc}}{\sigma_{ci} - \sigma_{fc}}\right)\frac{\ell_w}{667\delta_R}$$

= 9.6 kN — Permanent loads, Mid-span

cl. 9.12.3 = 7.1 kN — Quarter-span

cl. 9.12.2

but not greater than

$$F_R = \left(\frac{\sigma_{fc}}{\sigma_{ci} - \sigma_{fc}}\right)\frac{(n+1)EI_c}{16.7n\ell_R^2}$$

= 144.2 kN — Permanent loads, Mid-span

= 106 kN — Quarter-span

REFERENCE	CALCULATIONS	OUTPUT
	Hence F_R = 9.6 kN	Permanent loads Mid-span
	F_R = 7.1 kN	Quarter-span
	U-frames with cross members subjected to vertical loading	
cl. 9.12.3.3	Rotation of crossbeam 0.008 rad	Permanent loads
Model D11	Mean of rotations either side $\frac{0.008 + 0.008}{2}$ = 0.008	Permanent loads
	Difference in rotation Θ = 0.008 - 0.008 = 0 rad	Permanent loads
	$F_c = \frac{\theta d_2}{1.5\delta_R + \left\{ \ell_R^3 / \left(12EI_c \right) \right\}}$ = 0.00 kN	Permanent loads
	Total force per u-frame F_R + F_C = 9.6 kN	Permanent loads
cl. 9.13.3.3 b) Annex E	**Transverse moments in compression flanges: u-frame restraints**	

I_c = 5.19E+07 mm^4
d_2 = 1616 mm
Θ = 0 rad — Permanent loads
l_e = 8201 mm
σ_{fc} = 56 MPa — Permanent loads
l_w = 12500 mm
L = 12500 mm
E = 205000 MPa

σ'_{ci} is taken as follows:

a) if ℓ_w is less than three times the spacing of U-frames, then $\sigma'_{ci} = \sigma_{ci}$, as defined in **9.12.2**;
b) if ℓ_w is more than four times the spacing of U-frames, or if ℓ_e has been calculated in accordance with **9.6.4.1.1.2**, then $\sigma'_{ci} = 1.25\sigma_{ci}$; or
c) for intermediate values of ℓ_e, σ'_{ci} is obtained by linear interpolation;

Spacing of u-frames l_R = 2040 mm

$\frac{l_w}{l_R} = \frac{12500}{2040}$ = 6.1 > 4

σ_{ci} = 140 MPa

σ_{ci}' = 1.25 σ_{ci} = 175 MPa

$$M_y = \frac{5EI_c\theta d_2}{L\ell_e\left(1-\sigma_{fc}/\sigma'_{ci}\right)}\left\{1+\frac{\left(L/\ell_e\right)-1.25}{2.8+3.5\left(\sigma_{fc}/\sigma'_{ci}\right)^2}\right\}$$

M_y = 0.0 kNm — Permanent loads

Maximum compressive stress in the flange from bending about the major axis of the beam (as shown above)

σ_f = 53.4 MPa — Permanent loads

Longitudinal stress from My' $\sigma_b = \sigma_2 + \frac{M_y\, y}{I_c}$ = 2.7 + $\frac{0.00 \times 155.6 \times 1.0E+06}{5.9E+07}$

σ_b = 2.7 MPa — Permanent loads

REFERENCE	CALCULATIONS	OUTPUT

As above Z_{xc} = 1.98E+07 mm^3

M_{ult} = 4471 kNm

M_R = 2012 kNm

The following must be satisfied:

$$\frac{\sigma_f Z_{xc}}{M_R} + \frac{\sigma_b Z_{xc}}{M_{ult}} \leq \frac{1}{\gamma_m \gamma_{f3}}$$

$\frac{\sigma_f Z_{xc}}{M_R} + \frac{\sigma_b Z_{xc}}{M_{ult}}$ = 0.54 Permanent loads

$\frac{1}{\gamma_m \gamma_{f3}}$ = 0.87

With condition factor:

0.85 x 0.87 = 0.74 > 0.54 OK Permanent loads OK

Utilisation = 0.73

Shear resistance of plate girder under pure shear

Ignore longitudinal stiffeners due to their curtailment and check shear to cl. 9.9.2.2.

cl. 9.9.2.2

Depth of web d_w = 1380 - 10 - 15 = 1355 mm

Thickness of the web

Consider section loss from photo 104 of the IFA.

t_w = 8 mm over a height of 70 mm

t_w = 12.7 mm over a height of 1285 mm

Average web thickness t_w = 12.5 mm

Largest hole or cut-out h_h = 0 mm

Clear depth of web d_{we} = 1380 - 10 - 15 - 2 x 80 = 1195 mm

Nominal yield stress of web material σ_{yw} = 246 MPa

Nominal yield stress of flange material σ_{yf} = 246 MPa

Clear length of panel between transverse stiffeners a = 960 - 11.5 = 949 mm

Flange thickness t_f = 10 mm

b_{fe} is the smallest of:

a) $10 t_f \sqrt{\frac{355}{\sigma_{yf}}}$; or

b) the distance from the mid-plane of the web to the nearer edge of the flange; i.e. taken as zero if there is no flange outstand; or

c) if there are two or more webs, half the clear distance between webs;

a) $10 t_f \sqrt{\frac{355}{\sigma_{yf}}}$ = 136 mm

b) 305 / 2 = 152.5 mm

c) N/A

Hence b_{fe} = 136 mm

$m_{fw} = \frac{\sigma_{yf} b_{fe} t_f^2}{2 \sigma_{yw} d_{we}^2 t_w}$ = 0.0004

REFERENCE	CALCULATIONS	OUTPUT

$$\lambda = \frac{d_{we}}{t_w}\sqrt{\frac{\sigma_{yw}}{355}} = 80$$

$$\tau_y = \frac{\sigma_{yw}}{\sqrt{3}} = 142 \text{ MPa}$$

Aspect ratio of the panel $\varphi = \frac{a}{d_{we}} = 0.79$

Figure 12 to 18 $\tau_l / \tau_y = 0.96$

$\tau_l = 136.2$ MPa

$$V_D = \left[\frac{t_w\left(d_w - h_h\right)}{\gamma_m \gamma_{f3}}\right]\tau_\ell = 1990 \text{ kN}$$

With condition factor 0.85 x 1990 = 1692 kN

Applied shear force V = 551 kN Utilisation = 0.33 Permanent + 44 tonnes OK

Restraint at supports

9.12.5.2.5 Bending moment in the span M = 1058 kNm Permanent loads

cl. 9.6.4.1.2 d_f = 1367.5 mm

9.12.5.2.5 σ_{fc} = 56 MPa Permanent loads

σ_{ci} = 140 MPa

β = 1

D = 1380 mm

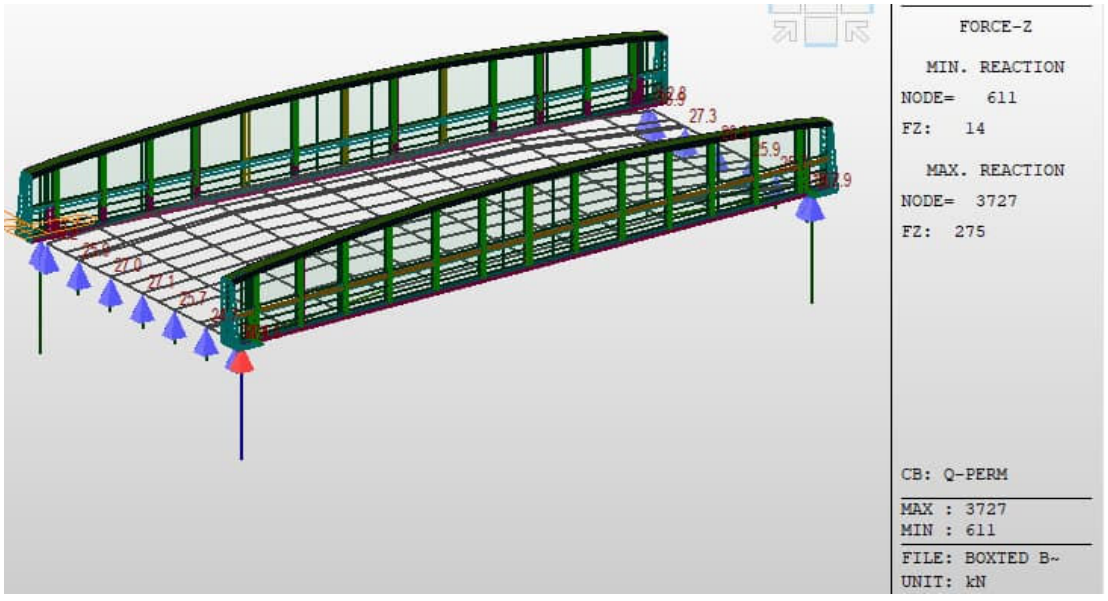
Δ = D / 200 = 6.9

$\Delta_{e1} = \Delta_{e2}$ = 6.9

Model rev. D09 $\delta_{t1} = \delta_{t2}$ = 0.004 mm per 1N

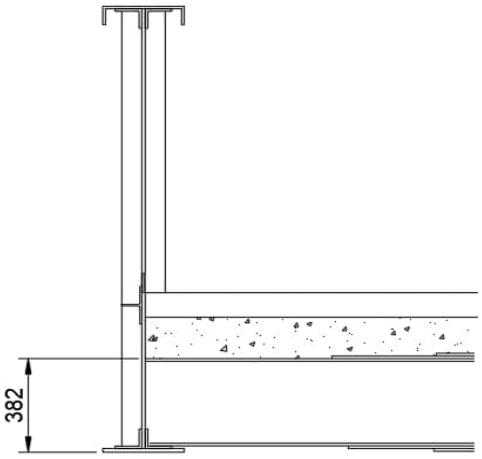
$\Sigma\ \delta$ = 0.008 mm per 1N

Model rev. D08 Bearing reaction R = 275 kN Permanent loads



REFERENCE	CALCULATIONS	OUTPUT

d_L = 382 mm



$\theta_L \tan\alpha$ = 0

9.12.5.2.2 Force due to bow of compression flange

$$F_{S1} = 0.005 \frac{M}{d_f \{1 - (\sigma_{fc}/\sigma_{ci})^2\}}$$ = 4.6 kN — Permanent loads

9.12.5.2.3 Force due to non-verticality of web at supports

$$F_{S2} = \frac{\beta(\Delta_{e1} + \Delta_{e2})\,\sigma_{fc}}{(\sigma_{ci} - \sigma_{fc})\,\Sigma\delta}$$ = 1.2 kN — Permanent loads

9.12.5.2.4 Force due to eccentricity of bearing reaction

$$F_{S3} = \frac{R d_L (\Delta/D + \theta_L \tan\alpha)}{D}$$ = 0.4 kN — Permanent loads

9.12.5.2.6 Additional force due to cross members in U-frames subjected to vertical loading

Distance of the loaded frame from the support

x_1 = 3190 mm

x_2 = 5230 mm

x_3 = 7270 mm

x_4 = 9310 mm

l_w = 12500 mm

$$F_L = \sum F_c \frac{(\ell_w - x)}{\ell_w}$$

= 0.00 x (0.74 + 0.58 + 0.42 + 0.26) = 0.00 kN — Permanent loads

cl. 9.12.5.2.1 $F_S = F_{S1} + F_{S2} + F_{S3} + F_{S4}$ + F_L = 6.2 kN — Permanent loads

cl. 9.12.5.3 The bearing stiffener is to be verified for the horizontal force Fs above. See next sections.

REFERENCE	CALCULATIONS	OUTPUT

cl. 9.13

Transverse web stiffeners other than at supports

Effective section of transverse web stiffeners

Portion of web plate $\min (16\ t_w ; a/2) =$ 203.2 mm

Effective width of web $b_{we} =$ 406.4 mm

Properties of vertical stiffener with section losses as determined in previous sections

Second moment of area the vertical stiffener	$I_{se} = I_1 =$	1.9E+06	mm^3
Elastic modulus of the vertical stiffener	$Z_{se} =$	2.6E+04	mm^3
Area	$A_{se} =$	5.7E+03	mm^2
Area of web plate only	$A_{se,w} =$	2.8E+03	mm^2
Distance from ENA to top fibre of stiffener		21	mm
Distance from web centre to top fibre of stiffener		15	mm

Eccentricity with respect to the web $e_1 =$ 21 - 15 = 6 mm

Properties of the vertical stiffener near the top of the crossbeam with no section loss as determined in previous sections

Second moment of area of the vertical	$I_{se,2} =$	4.52E+06	mm^4
Elastic modulus of the vertical stiffener	$Z_{se,2} =$	5.6E+04	mm^3
Area	$A_{se,2} =$	8.6E+03	mm^2
Area of web plate only	$A_{se,w,2} =$	5.2E+03	mm^2
Radius of gyration of effective stiffener	$r_{se} =$	23	mm

Eccentricity with respect to the web $e_2 =$ 81 - 81 = 0 mm

Axial force due to tension field action

$t_w =$ 12.5 mm average

$E =$ 205000 MPa

Check mid-span

Spacing of stiffeners	a =	1020	mm	
Depth of web	b =	1805	mm	
Area of web panel	=	2.2E+04	mm^2	
Distance from ENA to top of web		866	mm	
Distance from ENA to bottom of web		939	mm	
Modulus top		2.0E+07	mm^3	
Modulus bottom		1.8E+07	mm^3	
Axial force	F =	56	kN	Permanent loads
Bending moment	M =	1058	kNm	
Shear force	V =	34	kN	
Longitudinal stress at top of web	=	53	MPa	
Longitudinal stress at bottom of web	=	-57	MPa	
Average stress from bending	=	-2.2	MPa	
Stress from axial force	=	1.5	MPa	
Average stress	σ_1 =	-0.7	MPa	

$$2.9E\left(\frac{t_w}{b}\right)^2 = 28.3 \text{ MPa}$$

-0.7 MPa < 28.32 MPa

τ_0 is given by:

$$\tau_o = 3.6E\left\{1+\left(\frac{b}{a}\right)^2\right\}\left(\frac{t_w}{b}\right)^2\sqrt{1-\frac{\sigma_1}{2.9E}\left(\frac{b}{t_w}\right)^2} = 147 \text{ MPa}$$

when $\sigma_1 < 2.9E\left(\frac{t_w}{b}\right)^2$, or

$\tau_o = 0$

when $\sigma_1 \geq 2.9E\left(\frac{t_w}{b}\right)^2$

Average shear stress $\tau =$ 1.5 MPa < 147 MPa

tension field action does not occur — OK

REFERENCE	CALCULATIONS	OUTPUT

Check quarter-span

Spacing of stiffeners	a	=	2029	mm	
Depth of web	b	=	1715	mm	
Area of web panel		=	21364	mm^2	
Distance from ENA to top of web			822	mm	
Distance from ENA to bottom of web			893	mm	
Modulus top			1.9E+07	mm^3	
Modulus bottom			1.7E+07	mm^3	
Axial force	F	=	64	kN	Permanent loads
Bending moment	M	=	815	kNm	
Shear force	V	=	253	kN	
Longitudinal stress at top of web		=	44	MPa	
Longitudinal stress at bottom of web		=	-47	MPa	
Average stress from bending		=	-1.9	MPa	
Stress from axial force		=	1.8	MPa	
Average stress		σ_1 =	-0.1	MPa	

$$2.9E\left(\frac{t_w}{b}\right)^2 = 31.4 \text{ MPa}$$

-0.1 MPa < 31.37 MPa

τ_0 is given by:

$$\tau_o = 3.6E\left\{1+\left(\frac{b}{a}\right)^2\right\}\left(\frac{t_w}{b}\right)^2\sqrt{1-\frac{\sigma_1}{2.9E}\left(\frac{b}{t_w}\right)^2} = 67 \text{ MPa}$$

when $\sigma_1 < 2.9E\left(\frac{t_w}{b}\right)^2$, or

$\tau_o = 0$

when $\sigma_1 \geq 2.9E\left(\frac{t_w}{b}\right)^2$

Average shear stress τ = 11.8 MPa < 67 MPa

tension field action does not occur — OK

cl. 9.13.3.3 **Axial force representing the destabilizing influence of the web**

Nominal yield stress σ_{ys} = 246 MPa

t_w = 12.5 mm average

Check mid-span

I = $I_{se,2}$ = 4523623 mm^4 — without section losses

a = 1020 mm

l_s = 1805 mm

$r_{se,2}$ = 23 mm — without section losses

Slenderness factor $\lambda = \frac{\ell_s}{r_{se}}\sqrt{\frac{\sigma_{ys}}{355}}$ = 65 — without section losses

Figure 24

k_s = 0.27 — without section losses

τ_R = min(τ , τ_0) = 2 MPa — Permanent loads

ΣA_s = 0

Av. long stress from axial force σ_1 = 1.5 MPa — Permanent loads

Bending stress σ_b = 57.2 MPa

$$\sigma_R = \tau_R + \left(1+\frac{\Sigma A_s}{\ell_s t_w}\right)\left(\sigma_1 + \frac{\sigma_b}{6}\right) = 13 \text{ MPa}$$

$$F_{wi} = \frac{\ell_s{}^2}{a} t_w k_s \sigma_R = 135 \text{ kN}$$

REFERENCE	CALCULATIONS	OUTPUT

Second moment of area of long. stiffeners ΣI_s = 0

Reduction factor

$$\eta_s = \frac{1}{1+\left(0.5\ell_s{}^3/a^3 I\right)\left\{\Sigma I_s+\left(\ell_s t_w{}^3/12\right)\right\}} = 0.85$$ without section losses

$\eta_s F_{wi}$ = 115 kN without section losses

Check quarter span

I = $I_{se,2}$ = 4523623 mm^4 without section losses

a = 1020 mm

l_s = 1715 mm

$r_{se,2}$ = 23 mm without section losses

Slenderness factor $\lambda = \frac{\ell_s}{r_{se}}\sqrt{\frac{\sigma_{ys}}{355}}$ = 62 without section losses

Figure 24

k_s = 0.26 without section losses

τ_R = min (τ , τ_0) = 12 MPa Permanent loads

ΣA_s = 0

Av. long stress from axial force σ_1 = 1.8 MPa Permanent loads

Bending stress σ_b = 47.3 MPa

$$\sigma_R = \tau_R + \left(1+\frac{\Sigma A_s}{\ell_s t_w}\right)\left(\sigma_1+\frac{\sigma_b}{6}\right) = 22 \text{ MPa}$$

$$F_{wi} = \frac{\ell_s{}^2}{a} t_w k_s \sigma_R = 201 \text{ kN}$$

Second moment of area of long. stiffeners ΣI_s = 0

Reduction factor

$$\eta_s = \frac{1}{1+\left(0.5\ell_s{}^3/a^3 I\right)\left\{\Sigma I_s+\left(\ell_s t_w{}^3/12\right)\right\}} = 0.87$$ without section losses

$\eta_s F_{wi}$ = 176 kN without section losses

cl. 9.13.3.4 **Axial force due to curvature**

σ_f = 53.1 MPa Permanent loads

B_f = 305 mm

t_f = 10 mm

R_f = 46662 mm

β = 0 deg

b_{we} = 406.4 mm

$$F_f = \frac{\sigma_f B_f t_f b_{we}}{R_f \cos\beta} = 1.4 \text{ kN}$$ Permanent loads

to be applied to the web plate only

Strength of transverse web stiffeners (without u-frame actions)

cl. 9.13.5.1 **Transverse stress in web plate**

Stress from axial force representing the destabilizing influence of the web

The stresses are verified for two cases: with and without section losses in the stiffener.

Check mid-span

Stress from axial force

$\frac{\eta_s F_{wi}}{A_{se}} = 20$ MPa — Permanent loads with section losses

$\frac{\eta_s F_{wi}}{A_{se,2}} = 13$ MPa — Permanent loads without section losses

Bending moment on stiffener due to eccentricity

$M_{wi} = \eta_s F_{wi} e_1 = 0.7$ kNm — Permanent loads with section losses

$M_{wi,2} = \eta_s F_{wi} e_2 = 0$ kNm — Permanent loads without section losses

Vertical stress at mid-thickness of the web, from bending of stiffener

$\frac{M_{wi} e_1}{I_{se}} = 2$ MPa — Permanent loads with section losses

$\frac{M_{wi,2} e_2}{I_{se,2}} = 0$ MPa — Permanent loads without section losses

Total stress in web from axial force representing the destabilizing influence of the web

$20 + 2 = 22$ MPa — Permanent loads with section losses

$13 + 0 = 13$ MPa — Permanent loads without section losses

Stress from axial force due to curvature

$\frac{F_f}{A_{se,w}} = 0.5$ MPa — Permanent loads with section losses

$\frac{F_f}{A_{se,w}} = 0.2$ MPa — Permanent loads without section losses

Bending moment on stiffener due to eccentricity

$M_f = F_f e_1 = 8E\text{-}03$ kNm — Permanent loads with section losses

$M_{f,2} = F_f e_2 = 0E+00$ kNm — Permanent loads without section losses

Vertical stress sat mid-thickness of the web, from bending of stiffener

$\frac{M_f e_1}{I_{se}} = 3E\text{-}02$ MPa — Permanent loads with section losses

$\frac{M_f e_2}{I_{se,2}} = 0E+00$ MPa — Permanent loads without section losses

Total stress from axial force due to curvature

$5E\text{-}01 + 8E\text{-}03 = 1$ MPa — Permanent loads with section losses

$2E\text{-}01 + 0E+00 = 0$ MPa — Permanent loads without section losses

Total "transverse" stress on the web part of the vertical stiffener

$\sigma_{e2} = 22 + 1 = 23$ Mpa — Permanent loads with section losses

REFERENCE	CALCULATIONS	OUTPUT

cl. 9.13.5.1 **Yielding of web plate**

σ_1 = 1.5 MPa — Permanent loads with section losses

σ_b = 57.2 MPa — Permanent loads with section losses

τ_R = 2 MPa — Permanent loads with section losses

y = 903 mm

b = 1805 mm

k = $2y/b$ or 0.77, whichever is smaller = 0.77

$$\sigma_e = \sqrt{(\sigma_1 + k\sigma_b)^2 + \sigma_{es2}^2 - \sigma_{es2}(\sigma_1 + k\sigma_b) + 3\tau_R^2} = 40 \text{ MPa}$$

Resistance with condition factor

0.85 x $\frac{\sigma_{ys}}{\gamma_m \gamma_{f3}}$ = 181 MPa > 40 MPa OK — Permanent loads with section losses — OK

Utilisation = 0.22

9.13.5.2 **Yielding of stiffener**

Check mid-span stiffeners

Stress from axial force representing the destabilizing influence of the web

Stress from axial force as determined previously 20 MPa — Permanent loads with section losses

Stress from axial force as determined previously 13 MPa — Permanent loads without section losses

Vertical stress from bending of stiffener

$\frac{M_{wi}}{Z_{se}}$ = 26 MPa — Permanent loads with section losses

$\frac{M_{wi,2}}{Z_{se,2}}$ = 0 MPa — Permanent loads without section losses

Total longitudinal stress in stiffener 20 + 26 = 47 MPa — Permanent loads with section losses

13 + 0 = 13 MPa — Permanent loads without section losses

Check quarter-span stiffeners

Stress from axial force representing the destabilizing influence of the web

$\frac{\eta_s F_{wi}}{A_{se,2}}$ = 21 MPa — Permanent loads without section losses

Resistance with condition factor

0.85 x $\frac{\sigma_{ys}}{\gamma_m \gamma_{f3}}$ = 181 MPa > 47 MPa OK — Permanent loads with section losses — OK

Utilisation = 0.26

181 MPa > 21 MPa OK — Permanent loads without section losses — OK

Utilisation = 0.11

cl 9.13.5.3 **Buckling of effective stiffener section**

σ_y = 246 MPa

l_s = 1805 mm worst-case at mid-span

$r_{se,2}$ = 23 mm

$$\lambda = \frac{\ell_s}{r_{se}}\sqrt{\frac{\sigma_y}{355}} = 65$$

σ_c / σ_y = 0.57

σ_{ls} = σ_c = 140 MPa

$A_{se,2}$ = 8557 mm^2

REFERENCE	CALCULATIONS	OUTPUT

$Z_x = Z_{se,2} = 55607 \text{ mm}^3$

Maximum axial force in stiffener $P = \eta_s F_{wi} = 115$ kN

Maximum bending moment in stiffener $M_{xs} = 0.0$ kNm

Material factor for longitudinal stiffeners in compression $\gamma_m = 1.2$

With condition factor 0.85 x $\frac{1}{\gamma_m \gamma_{f3}} = 0.64$

$$\frac{P}{A_{se}\sigma_{\ell s}} + \frac{M_{xs}}{Z_x \sigma_y} = 0.096 < 0.64$$ OK

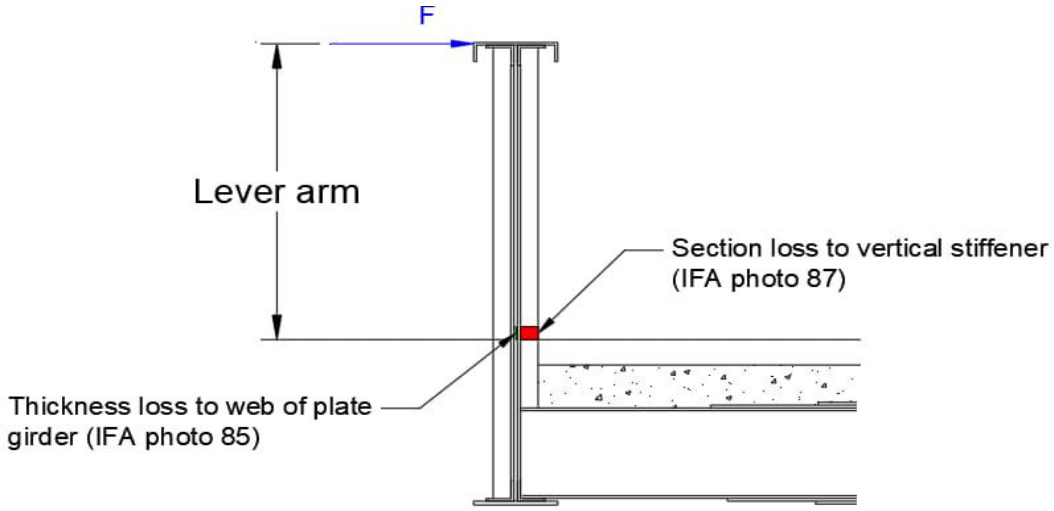
Utilisation = 0.15

Permanent loads without section losses

OK

Verification of vertical stiffener for u-frame actions

Check stresses near carriageway level where there is section loss to the vertical stiffener



Lever arm: 1170 mm

Bending moment due to the u-frame force 9.6 x 1170 = 11.2 kNm

Bending stress M / Z_{se} = 433 MPa

Permanent loads with section losses

Include stresses from the destabilising force in the web from previous sections

Total longitudinal stress 433 + 47 = 480 MPa

Permanent loads with section losses

Resistance

cl. 9.13.5.2 $\frac{\sigma_{ys}}{\gamma_m \gamma_{f3}} = 213$ MPa

With condition factor:

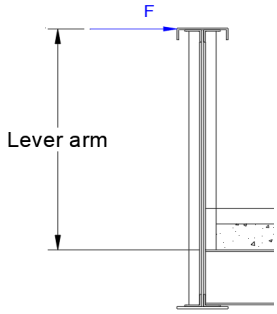
0.85 x 213 MPa = 181 MPa < 480 MPa NOT OK

Utilisation = 2.65

Permanent loads with section losses

NOT OK

Check stresses in the vertical near the top of the crossbeam, assume no section losses in stiffener



REFERENCE	CALCULATIONS	OUTPUT

Lever arm of force: 1416 mm — Mid-span region

1336 mm — Quarter span

Bending moment due to the u-frame force: 9.6 x 1416 = 13.6 kNm — Permanent loads Mid-span region

7.1 x 1336 = 9.5 kNm — Quarter span

Bending stress: $M / Z_{se,2}$ = 244 MPa — Permanent loads Mid-span region

$M / Z_{se,2}$ = 170 MPa — Quarter span

Include stresses from the destabilising force in the web from previous sections

Total longitudinal stress: 244 + 13 = 258 MPa — Permanent loads Mid-span region

170 + 21 = 191 MPa — Quarter span

cl. 9.13.5.2 Resistance $\frac{\sigma_{ys}}{\gamma_m \gamma_{f3}}$ = 213 MPa

With condition factor:

0.85 x 213 MPa = 181 MPa < 258 MPa NOT OK — Permanent loads without section losses Closest to mid-span — NOT OK

Utilisation = 1.42

181 MPa < 191 MPa NOT OK — NOT OK

Utilisation = 1.05 — Quarter span

cl 9.13.5.3 **Buckling of effective stiffener section**

σ_y = 246 MPa

l_s = 1715 mm — Quarter span

$r_{se,2}$ = 23 mm

$\lambda = \frac{\ell_s}{r_{se}} \sqrt{\frac{\sigma_y}{355}}$ = 62

σ_c / σ_y = 0.60

$\sigma_{ls} = \sigma_c$ = 147 MPa

$A_{se,2}$ = 8557 mm^2

$Z_x = Z_{se,2}$ = 55607 mm^3

Maximum axial force in stiffener: $P = \eta_s F_{wi}$ = 176 kN

Maximum bending moment in stiffener: M_{xs} = 9.5 kNm

Material factor for longitudinal stiffeners in compression: γ_m = 1.2

With condition factor: 0.85 x $\frac{1}{\gamma_m \gamma_{f3}}$ = 0.64

$\frac{P}{A_{se}\sigma_{\ell s}} + \frac{M_{xs}}{Z_x \sigma_y}$ = 0.832 < 0.64 NOT OK — Permanent loads without section losses Quarter span — NOT OK

Utilisation = 1.29

Hence the stiffeners are inadequate for u-frame actions

REFERENCE	CALCULATIONS	OUTPUT

Load bearing support stiffeners

Figure 28

25 t_w = 318 mm

cl. 9.14.2

Spacing between the end plate and the next transverse stiffener:

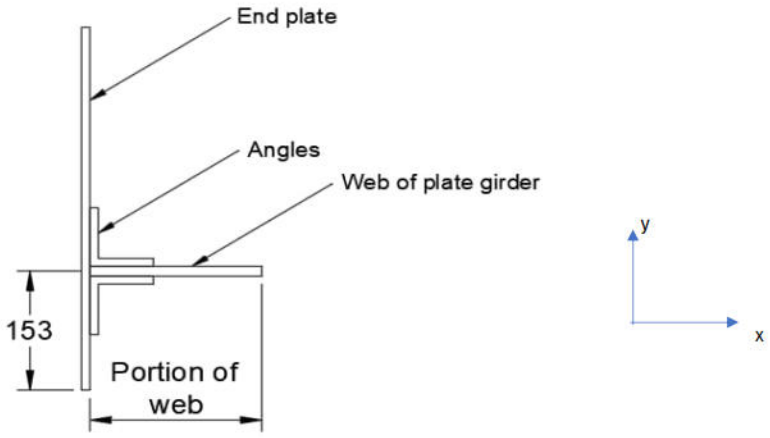
approx. 455 mm > 318 mm treat as single leg stiffener

Transverse stiffener spacing a = 949 mm half spacing a/2 = 474 mm

16 t_w = 203.2 mm

Hence the portion of the web is 203 mm

Sectional plan at the bottom of the bearing stiffener:



Properties of bearing stiffener

About plate girder's web (about X-X axis)

Element	b mm	d mm	A mm^2	y mm	Ay mm^3	y' mm	I mm^4
End plate	9.5	465.0	4.4E+03	232.5	1.03E+06	43.2	8.78E+07
Angles vertical 2x	9.5	150.0	1.4E+03	312.0	4.45E+05	-36.3	4.55E+06
Angles horizontal 2x	65.5	19.0	1.2E+03	312.0	3.88E+05	-36.3	1.68E+06
Web	203.2	12.7	2.6E+03	312.0	8.05E+05	-36.3	3.44E+06
Total			**9.7E+03**		2.67E+06		**9.750E+07**

Distance from ENA to top fibre ∑Ay / ∑A = 275.7 mm

Total depth 465.0 mm

Distance from ENA to bottom fibre 189.3 mm

Maximum distance from ENA to extreme fibre 275.7 mm

Second moment of area of the stiffener I_{bx} = 9.75E+07 mm^4

Elastic modulus of the vertical stiffener Z_{bx} = 3.5E+05 mm^3

Radius of gyration r_{se} = 100 mm

Eccentricity with respect to the web e = 312 - 276 = 36 mm

REFERENCE	CALCULATIONS	OUTPUT

Properties of north east stiffener with section loss (IFA photo 78 & 92)

About plate girder's web (about X-X axis)

Element	b mm	d mm	A mm^2	y mm	Ay mm^3	y' mm	I mm^4
End plate	1.0	430.0	4.3E+02	215.0	9.25E+04	57.3	8.04E+06
Angles vertical 2x	9.5	150.0	1.4E+03	277.0	3.95E+05	-4.7	2.70E+06
Angles horizontal 2x	65.5	19.0	1.2E+03	277.0	3.45E+05	-4.7	6.49E+04
Web	203.2	12.7	2.6E+03	277.0	7.15E+05	-4.7	9.15E+04
Total			**5.7E+03**		1.55E+06		**1.090E+07**

Distance from ENA to top fibre ∑Ay / ∑A = 272.3 mm

Total depth 430.0 mm

Distance from ENA to bottom fibre 157.7 mm

Maximum distance from ENA to extreme fibre 272.3 mm

Second moment of area of the vertical $I_{bx,2}$ = 1.09E+07 mm^4

Elastic modulus of the vertical stiffener $Z_{bx,2}$ = 4.0E+04 mm^3

Radius of gyration r_{se} = 44 mm

Eccentricity with respect to the web e = 277 - 272 = 5 mm

cl. 9.14.3.1 **Loading on bearing stiffeners**

a) Axial force from bearing reaction R = 275 kN — Permanent loads

b) Axial force representing the destabilising influence of the web

Height of web panel l_s = 1355 mm

Girder's section properties at the support:

Girder cross-sectional area A_3 2.98E+04 mm^2

Girder section modulus to edge of web $Z_{w,3} = I / y$ = 8.20E+09 / 633 = 1.30E+07 mm^3

Forces near support

Axial force in girder	73	kN	Permanent loads
Bending moment in girder	27	kNm	
Axial force in girder	146	kN	Permanent loads + 44t
Bending moment in girder	54	kNm	
Average stress in web from axial force σ_1 =	2.4	MPa	Permanent loads
Stress in web from bending of girder σ_b =	2.1	MPa	
Average stress in web from axial force σ_1 =	4.9	MPa	Permanent loads + 44t
Stress in web from bending of girder σ_b =	4.2	MPa	

cl. 9.14.3.2

σ_{ys} = 246 MPa

t_w = 12.7 mm

a = 600 mm

$\sum A_s$ = 0

$$\sigma_R = \left(1 + \frac{\sum A_s}{\ell_s t_w}\right)\left(\sigma_1 + \frac{\sigma_b}{6}\right) = 2.8 \text{ MPa}$$ Permanent loads

$$\sigma_R = \left(1 + \frac{\sum A_s}{\ell_s t_w}\right)\left(\sigma_1 + \frac{\sigma_b}{6}\right) = 5.6 \text{ MPa}$$ Permanent loads + 44t

REFERENCE	CALCULATIONS	OUTPUT

Second moment of area of long. stiffeners ΣI_s = 0

Reduction factor

$$\eta_s = \frac{1}{1 + \left(0.5\ell_s^{\,3}/a^3 I\right)\left\{\Sigma I_s + \left(\ell_s t_w^{\,3}/12\right)\right\}} = 1.00$$

Take south west, north west and south east bearing stiffeners

r_{se} = 100 mm

Slenderness factor $\lambda = \frac{\ell_s}{r_{se}}\sqrt{\frac{\sigma_{ys}}{355}}$ = 11

Figure 24 k_s = 0.01

$F_{wi} = \frac{\ell_s^2}{a} t_w k_s \sigma_R$ = 1.1 kN

$\eta_s F_{wi}$ = 1.1 kN Permanent loads

Take the north east stiffener

r_{se} = 44 mm

Slenderness factor $\lambda = \frac{\ell_s}{r_{se}}\sqrt{\frac{\sigma_{ys}}{355}}$ = 26

k_s = 0.07

$F_{wi} = \frac{\ell_s^2}{a} t_w k_s \sigma_R$ = 7.6 kN

$\eta_s F_{wi}$ = 7.6 kN Permanent loads

$F_{wi} = \frac{\ell_s^2}{a} t_w k_s \sigma_R$ = 15.2 kN

$\eta_s F_{wi}$ = 15.2 kN Permanent loads +44t

c) N/A

d) Eccentricity

cl. 9.14.3.3 Eccentricity due to uneven seating etc. = 0 mm

Assumed as 0 as the inspection found no concerns.

Lever arm of the F_S force 1367.5 mm

Total stress on bearing stiffener

Take south west, north west and south east bearing stiffeners which do not have significant section loss

Total vertical force P = 275 + 1.1 = 276 kN Permanent loads

Total eccentricity 0 + 36 = 36 mm

Total bending moment about the X-X axis

M = F_S x 1367.5 + P x 36

= 8.4 + 10 = 18 kNm

Total stress M / Z + P / A = 81 MPa

Bending moment without the F_S force

M = P x 36 = 10 kNm

Stress M / Z + P / A = 57 MPa

REFERENCE	CALCULATIONS	OUTPUT

cl. 9.14.4.2 Yielding of stiffener

$$\frac{\sigma_{ys}}{\gamma_m \gamma_{f3}} = 213 \text{ MPa}$$

With condition factor:

0.85 x 213 MPa = 181 MPa > 81 MPa OK — Permanent loads With the Fs force — OK

Utilisation = 0.45

181 MPa > 57 MPa OK — Permanent loads Without the Fs force — OK

Utilisation = 0.31

Take the north east stiffener with significant section loss

Total vertical force P = 275 + 0 + 7.6 = 283 kN — Permanent loads

Total eccentricity 0 + 5 = 5 mm

Total bending moment about the X-X axis

M = F_S x 1367.5 + P x 5 = 10 kNm

Total stress M / Z + P / A = 294 MPa

Bending moment without the F_S force

M = P x 5 = 1.33 kNm

Stress M / Z + P / A = 83 MPa

cl. 9.14.4.2 Yielding of stiffener

$$\frac{\sigma_{ys}}{\gamma_m \gamma_{f3}} = 213 \text{ MPa}$$

With condition factor:

0.85 x 213 MPa = 181 MPa < 294 MPa NOT OK — Permanent loads With the Fs force — NOT OK

Utilisation = 1.62

181 MPa > 83 MPa OK — Permanent loads Without the Fs force — OK

Utilisation = 0.46

cl 9.14.4.3 **Buckling of effective bearing stiffener section**

Take south west, north west and south east bearing stiffeners which do not have significant section loss

σ_y = 246 MPa

l_s = 1355 mm

r_{se} = 100 mm

$$\lambda = \frac{\ell_s}{r_{se}} \sqrt{\frac{\sigma_y}{355}} = 11$$

σ_c / σ_y = 1.00

σ_{ls} = σ_c = 246 MPa

A_{se} = 9668 mm^2

Z_x = Z_{bx} = 353694 mm^3

Maximum axial force in stiffener P = 276 kN — Permanent loads

Maximum bending moment in stiffener M_{xs} = 18.5 kNm — Permanent loads

Material factor for longitudinal stiffeners in compression γ_m = 1.2

REFERENCE	CALCULATIONS	OUTPUT

With condition factor 0.85 x $\frac{1}{\gamma_m \gamma_{f3}}$ = 0.64

$\frac{P}{A_{se}\sigma_{\ell s}} + \frac{M_{xs}}{Z_x\sigma_y} + \frac{M_{ys}}{Z_y\sigma_y} + \frac{M_y}{Z_{cy}\sigma_y}$ = 0.33 < 0.64 OK — Permanent loads with the Fs force — OK

Utilisation = 0.51

M_{ys} and M_y = 0

North east bearing stiffener

σ_y = 246 MPa

l_s = 1355 mm

r_{se} = 44 mm

$\lambda = \frac{\ell_s}{r_{se}}\sqrt{\frac{\sigma_y}{355}}$ = 26

σ_c / σ_y = 0.90

σ_{ls} = σ_c = 221 MPa

A_{se} = 5680 mm^2

Z_x = $Z_{bx,2}$ = 40019 mm^3

Maximum axial force in stiffener P = 283 kN — Permanent loads

Maximum bending moment in stiffener M_{xs} = 9.8 kNm — Permanent loads with the Fs force

Material factor for longitudinal stiffeners in compression γ_m = 1.2

With condition factor 0.85 x $\frac{1}{\gamma_m \gamma_{f3}}$ = 0.64

$\frac{P}{A_{se}\sigma_{\ell s}} + \frac{M_{xs}}{Z_x\sigma_y} + \frac{M_{ys}}{Z_y\sigma_y} + \frac{M_y}{Z_{cy}\sigma_y}$ = 1.217 < 0.64 NOT OK — Permanent loads with the Fs force — NOT OK

Utilisation = 1.89

M_{ys} and M_y = 0

Maximum bending moment in stiffener M_{xs} = 1.3 kNm — Permanent loads without the Fs force

$\frac{P}{A_{se}\sigma_{\ell s}} + \frac{M_{xs}}{Z_x\sigma_y} + \frac{M_{ys}}{Z_y\sigma_y} + \frac{M_y}{Z_{cy}\sigma_y}$ = 0.360 < 0.64 OK — OK

Utilisation = 0.56

M_{ys} and M_y = 0

Increase the slenderness in accordance with CS 456 Appendix A cl. 9.6.1

CS 456 Appendix A cl. 9.6.1

Calculate spare resistance from buckling of stiffener

Rearrange equation: $\frac{P}{A_{se}\sigma_{\ell s}} + \frac{M_{xs}}{Z_x\sigma_y} + \frac{M_{ys}}{Z_y\sigma_y} + \frac{M_y}{Z_{cy}\sigma_y}$ = 0.64

M_{xs} = 1.3 + F_{SD} x 1367.5

Hence

F_{SD} = [(0.64 - $\frac{P}{A_{se}\sigma_{\ell s}}$) x $Z_x\sigma_y$ - 1.3] / 1367.5 = 2.04 kN

New slenderness $\lambda'_{LT} = \frac{\lambda_{LT}}{\sqrt{\frac{1}{8}\left(\frac{5F_{SD}}{F_S}+3\right)}}$ = 1.31 λ_{LT} — Permanent loads

REFERENCE	CALCULATIONS	OUTPUT

Determine slenderness to cl. 9.7.5 as an alternative

E = 2.1E+05 MPa

cl. 9.7.5 Model rev. D09 — Elastic critical buckling moment M_{Cr} = 2969 kNm — Determined through elastic buckling analysis

Z_{PE} = 2.3E+07 mm^3

σ_{yc} = 246 MPa

M_{ult} = 4471 kNm

M_{PE} = 5582 kNm

$\lambda_{LT} = \sqrt{\frac{\pi^2 E Z_{pe}}{M_{cr}}}$ = 124

Increase the slenderness in accordance with CS 456 Appendix A cl. 9.6.1

New slenderness $\lambda'_{LT} = \frac{\lambda_{LT}}{\sqrt{\frac{1}{8}\left(\frac{5F_{SD}}{F_S}+3\right)}}$ = 1.31 λ_{LT} = 163 — Permanent loads

$\lambda_{LT}\sqrt{\left(\frac{\sigma_{yc}}{355}\right)\left(\frac{M_{ult}}{M_{pe}}\right)}$ = 121

CS 456 Appendix A Figure 11 — M_R / M_{ult} = 0.32

M_R = 1431 kNm

cl. 9.9.1.2 — Assessment bending moment resistance $M_D = \frac{M_R}{\gamma_m \gamma_{f3}}$ = 1239 kNm

With condition factor 0.85 M_D = 1053 kNm

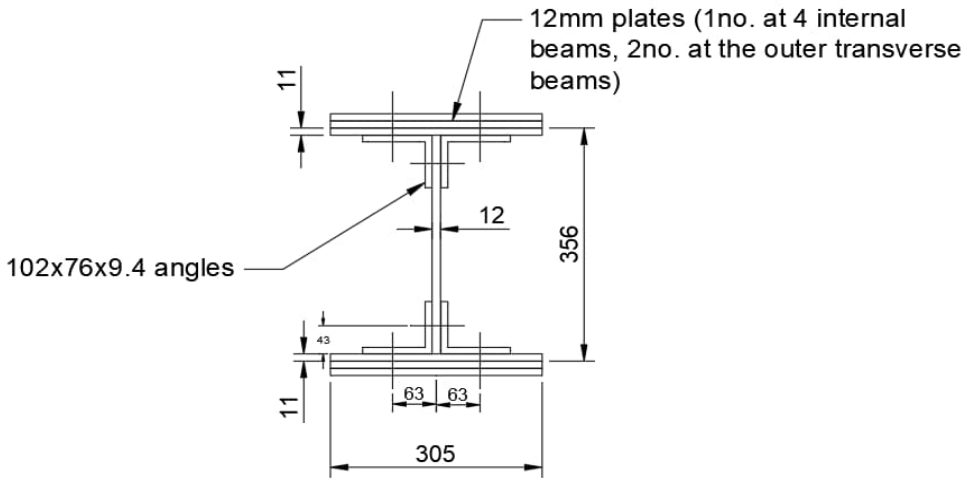
Model rev. D08 — Bending moment M = 1058 kNm — Permanent loads

Utilisation = 1.00

Without safety factors:

Capacity	1431	kNm	
Applied bending moment	1140	kNm	Permanent loads+7.5t
	1380	kNm	Permanent loads+18t
	1527	kNm	Permanent loads+26t
	949	kNm	Permanent loads+pedestrian ALL over 2m

Crossbeams



Height of section without additional plates	D =	356	mm
Height of section with one additional plate	D =	380	mm
Height of section with two additional plates	D =	404	mm
Additional plate thickness		12	mm
Top flange thickness		11	mm
Top flange width		305	mm
Web thickness		12	mm
Bottom flange width		305	mm
Bottom flange thickness		11	mm
Rivet opening diameter		18	mm
Bottom web rivet offset from bottom flange's top surface		43	mm
Angle horizontal length		102	mm
Angle vertical length		76	mm
Angle thickness		9.4	mm

Gross section properties

Crossbeam with two additional plates - about major (x-x) axis:

Element	b mm	d mm	A mm^2	y mm
Flange top	305.0	35.0	1.1E+04	17.5
Top angle flange x2	204.0	9.4	1.9E+03	39.7
Top angle web x2	18.8	66.6	1.3E+03	77.7
Web	12.0	334.0	4.0E+03	202
Flange bottom	305.0	35.0	1.1E+04	386.5
Bot angle flange x2	204.0	9.4	1.9E+03	364.3
Bot angle web x2	18.8	66.6	1.3E+03	326.3
Bottom rivet web	-30.8	18.0	-5.5E+02	326.0
Bot. rivet flange x2	-36.0	44	-1.6E+03	381.8

Total **3.0E+04**

Plastic neutral axis y_P = ∑A / 2 - (area of top flange and top angles) / (thickness of web) + thickness of top flange

y_P = 112.3 mm

Crossbeam with one additional plate - about major (x-x) axis:

Element	b mm	d mm	A mm^2	y mm	Ay mm^3	y' mm	I mm^4
Flange top	305.0	23.0	7.0E+03	11.5	8.07E+04	166.5	1.95E+08
Top angle flange x2	204.0	9.4	1.9E+03	27.7	5.31E+04	150.3	4.33E+07
Top angle web x2	18.8	66.6	1.3E+03	65.7	8.23E+04	112.3	1.63E+07
Web	12.0	334.0	4.0E+03	190	7.62E+05	-12.0	3.78E+07
Flange bottom	305.0	23.0	7.0E+03	368.5	2.59E+06	-190.5	2.55E+08
Bot angle flange x2	204.0	9.4	1.9E+03	352.3	6.76E+05	-174.3	5.83E+07
Bot angle web x2	18.8	66.6	1.3E+03	314.3	3.94E+05	-136.3	2.37E+07
Bottom rivet web	-30.8	18.0	-5.5E+02	314.0	-1.74E+05	-136.0	-1.03E+07
Bot. rivet flange x2	-36.0	32	-1.2E+03	363.8	-4.24E+05	-185.8	-4.04E+07

Total **2.3E+04** 4.03E+06 5.78E+08

REFERENCE	CALCULATIONS	OUTPUT

Distance from ENA to comp fibre $y_{xc} = \sum Ay / \sum A = 178.0$ mm

Second moment of area $I_x = 5.78E+08$ mm^4

Plastic neutral axis $y_P =$ (∑A / 2 - (area of top flange and top angles)) / (thickness of web) + thickness of top flange

$y_P = 130.3$ mm

Crossbeam with no additional plates, with section loss - about major (x-x) axis:

IFA photo 46

Element	b mm	d mm	A mm^2	y mm	Ay mm^3	y' mm	I mm^4
Flange top	305.0	11.0	3.4E+03	5.5	1.85E+04	132.0	5.85E+07
Top angle flange x2	204.0	9.4	1.9E+03	15.7	3.01E+04	121.8	2.85E+07
Top angle web x2	18.8	66.6	1.3E+03	53.7	6.72E+04	83.8	9.25E+06
Web	12.0	334.0	4.0E+03	178	7.13E+05	-40.5	4.38E+07
Flange bottom	290.0	3.0	8.7E+02	354.5	3.08E+05	-217.0	4.10E+07
Bot angle flange x2	204.0	9.4	1.9E+03	348.3	6.68E+05	-210.8	8.52E+07
Bot angle web x2	18.8	66.6	1.3E+03	310.3	3.89E+05	-172.8	3.79E+07
Bottom rivet web	-30.8	18.0	-5.5E+02	310.0	-1.72E+05	-172.5	-1.65E+07
Bot. rivet flange x2	-36.0	12	-4.5E+02	349.8	-1.56E+05	-212.3	-2.01E+07
	Total	**1.4E+04**		1.87E+06		2.67E+08	

Distance from ENA to comp fibre $y_{xc} = \sum Ay / \sum A = 137.5$ mm

Second moment of area $I_x = 2.67E+08$ mm^4

Plastic neutral axis $y_P =$ (∑A / 2 - (area of top flange and top angles)) / (thickness of web) + thickness of top flange

$y_P = 32.8$ mm

Note: IFA photo 46 has been identified as the most onerous case compared with photos 50, 48, 32 and 29.

Crossbeam with no additional plates, without section loss - about major (x-x) axis:

IFA photo 46

Element	b mm	d mm	A mm^2	y mm	Ay mm^3	y' mm	I mm^4
Flange top	305.0	11.0	3.4E+03	5.5	1.85E+04	160.3	8.63E+07
Top angle flange x2	204.0	9.4	1.9E+03	15.7	3.01E+04	150.1	4.32E+07
Top angle web x2	18.8	66.6	1.3E+03	53.7	6.72E+04	112.1	1.62E+07
Web	12.0	334.0	4.0E+03	178	7.13E+05	-12.2	3.79E+07
Flange bottom	305.0	11.0	3.4E+03	350.5	1.18E+06	-184.7	1.14E+08
Bot angle flange x2	204.0	9.4	1.9E+03	340.3	6.53E+05	-174.5	5.84E+07
Bot angle web x2	18.8	66.6	1.3E+03	302.3	3.79E+05	-136.5	2.38E+07
Bottom rivet web	-30.8	18.0	-5.5E+02	302.0	-1.67E+05	-136.2	-1.03E+07
Bot. rivet flange x2	-36.0	20	-7.3E+02	345.8	-2.54E+05	-180.0	-2.38E+07
	Total	**1.6E+04**		2.61E+06		3.46E+08	

Distance from ENA to comp fibre $y_{xc} = \sum Ay / \sum A = 165.8$ mm

Second moment of area $I_x = 3.46E+08$ mm^4

Plastic neutral axis $y_P =$ (∑A / 2 - (area of top flange and top angles)) / (thickness of web) + thickness of top flange

$y_P = 124.3$ mm

REFERENCE	CALCULATIONS	OUTPUT

Section classification

Web

Depth of web plate on the compressive side of the plastic NA = y_P - thickness of top flange = 77.30 mm

Depth of the web plate = 334.0 mm

BS 5400-3 cl. 9.3.7.2

Ratio m = 0.23

Nominal yield stress σ_{yw} = 246 MPa

Thickness of web plate t_w = 12.0 mm

Limiting depth $\frac{34t_w}{m}\sqrt{\frac{355}{\sigma_{yw}}}$ when m does not exceed 0.5; and $\frac{374t_w}{(13m-1)}\sqrt{\frac{355}{\sigma_{yw}}}$ when m exceeds 0.5; → 2119 mm > 334 mm

i.e. web is compact

Compression flange

Compression flange outstand b_{f0} = 146.5 mm

Nominal yield stress σ_{yf} = 246 MPa

Mean thickness of outstand

Flange thickness with one additional plate t_{f0} = 23 mm

$7t_{fo}\sqrt{\frac{355}{\sigma_{yf}}}$ = 194 mm > 147 mm

i.e. flange is compact

Plastic moment of resistance - crossbeam with two additional plates

y'_p = ABS(y_P - y)

Crossbeam with two additional plates - about major (x-x) axis:

Element	b mm	d mm	A mm^2	y mm	y'_p mm	A y'_p mm^3
Flange top	305.0	35.0	1.1E+04	18	94.8	1.01E+06
Top angle flange x2	204.0	9.4	1.9E+03	40	72.6	1.39E+05
Top angle web x2	18.8	66.6	1.3E+03	78	34.6	4.33E+04
Web top	12.0	77.3	9.3E+02	74	38.6	3.59E+04
Web bottom	12.0	256.7	3.1E+03	241	128.4	3.95E+05
Flange bottom	305.0	35.0	1.1E+04	387	274.2	2.93E+06
Bot angle flange x2	204.0	9.4	1.9E+03	364	252.0	4.83E+05
Bot angle web x2	18.8	66.6	1.3E+03	326	214.0	2.68E+05
Bottom rivet web	-30.8	18.0	-5.5E+02	326	213.7	-1.18E+05
Bot. rivet flange x2	-36.0	44.4	-1.6E+03	382	269.5	-4.31E+05
totals:			3.0E+04			4.75E+06

Plastic moment of resistance M_{pe} = $\Sigma A\,y'_p$ σ_{ys} = 1168 kNm

cl. 9.7.1 Plastic modulus Z_{pe} = M_{pe} / σ_{ys} = $\Sigma A\,y'_p$ = 4.75E+06 mm^3

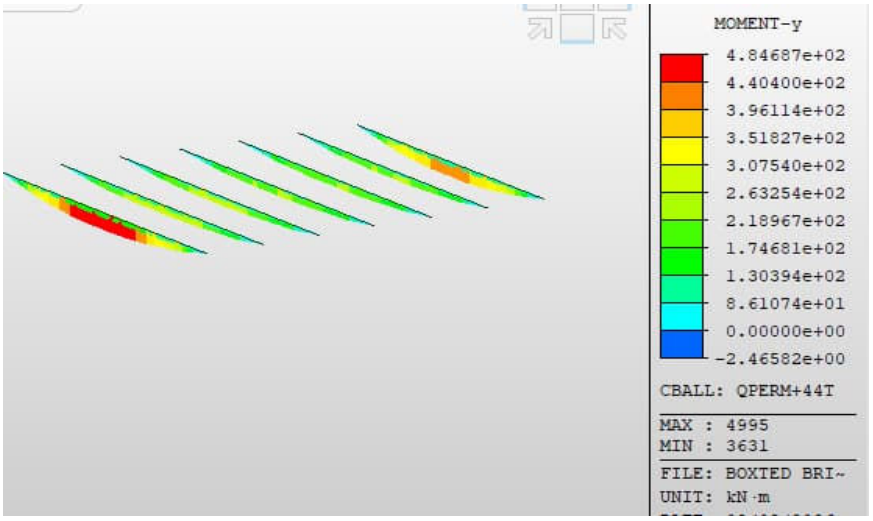
Bending resistance with condition factor

cl. 9.8 The compression flange is restrained by the concrete jack arches hence it is assumed that buckling is prevented and the effective length = 0

cl. 9.9.1.2 M_{ult} = $\frac{0.85}{\gamma_{f3}}$ x $\frac{M_{pe}}{\gamma_m}$ = 860 kNm

Model rev. D24 Applied bending moment M = 485 kNm Permanent loads + 44t

Utilisation = 0.56 OK



Plastic moment of resistance - crossbeam with one additional plate

$$y'_p = ABS(y_P - y)$$

Crossbeam with one additional plate - about major (x-x) axis:

Element	b mm	d mm	A mm^2	y mm	y'_p mm	$A\,y'_p$ mm^3
Flange top	305.0	23.0	7.0E+03	12	118.8	8.33E+05
Top angle flange x2	204.0	9.4	1.9E+03	28	102.6	1.97E+05
Top angle web x2	18.8	66.6	1.3E+03	66	64.6	8.09E+04
Web top	12.0	107.3	1.3E+03	77	53.6	6.91E+04
Web bottom	12.0	226.7	2.7E+03	244	113.4	3.08E+05
Flange bottom	305.0	23.0	7.0E+03	369	238.2	1.67E+06
Bot angle flange x2	204.0	9.4	1.9E+03	352	222.0	4.26E+05
Bot angle web x2	18.8	66.6	1.3E+03	314	184.0	2.30E+05
Bottom rivet web	-30.8	18.0	-5.5E+02	314	183.7	-1.02E+05
Bot. rivet flange x2	-36.0	32.4	-1.2E+03	364	233.5	-2.72E+05
totals:			2.3E+04			3.44E+06

Plastic moment of resistance $M_{pe} = \Sigma A\,y'_p \quad \sigma_{ys} = 846$ kNm

cl. 9.7.1 Plastic modulus $Z_{pe} = M_{pe} / \sigma_{ys} = \Sigma A\,y'_p = 3.44E+06$ mm^3

Bending resistance with condition factor

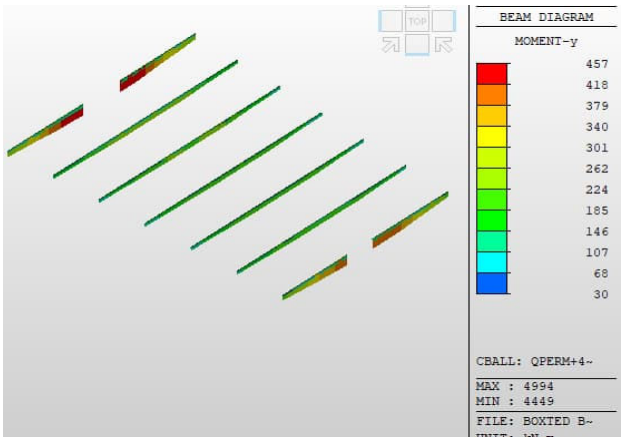
cl. 9.8 The compression flange is restrained by the concrete jack arches hence it is assumed that buckling is prevented and the effective length = 0

cl. 9.9.1.2 $$M_{ult} = \frac{0.85}{\gamma_{f3}} \times \frac{M_{pe}}{\gamma_m} = 622 \text{ kNm}$$

Model rev. D08 $M = 457$ kNm Permanent loads + 44t

Applied bending moment from global model

Utilisation = 0.73 OK



REFERENCE	CALCULATIONS	OUTPUT

Plastic moment of resistance - crossbeam with no additional plates, with section loss

$$y'_p = ABS(y_P - y)$$

Crossbeam with no additional plates - about major (x-x) axis:

Element	b mm	d mm	A mm^2	y mm	y'_p mm	$A\,y'_p$ mm^3
Flange top	305.0	11.0	3.4E+03	6	27.3	9.15E+04
Top angle flange x2	204.0	9.4	1.9E+03	16	17.1	3.27E+04
Top angle web x2	18.8	66.6	1.3E+03	54	20.9	2.62E+04
Web top	12.0	113.3	1.4E+03	68	34.9	4.74E+04
Web bottom	12.0	220.7	2.6E+03	235	201.9	5.35E+05
Flange bottom	290.0	3.0	8.7E+02	355	321.7	2.80E+05
Bot angle flange x2	204.0	9.4	1.9E+03	348	315.5	6.05E+05
Bot angle web x2	18.8	66.6	1.3E+03	310	277.5	3.48E+05
Bottom rivet web	-30.8	18.0	-5.5E+02	310	277.2	-1.54E+05
Bot. rivet flange x2	-36.0	12.4	-4.5E+02	350	317.0	-1.42E+05

totals: 1.4E+04 1.67E+06

Plastic moment of resistance $M_{pe} = \Sigma A\,y'_p \; \sigma_{ys}$ = 410 kNm

cl. 9.7.1 Plastic modulus $Z_{pe} = M_{pe} / \sigma_{ys} = \Sigma A\,y'_p$ = 1.67E+06 mm^3

Bending resistance with condition factor

cl. 9.8 The compression flange is restrained by the concrete jack arches hence it is assumed that buckling is prevented and the effective length = 0

cl. 9.9.1.2 $M_{ult} = \frac{0.85}{\gamma_{f3}} \times \frac{M_{pe}}{\gamma_m}$ = 302 kNm

Applied bending moment from u-frame action M = 14 kNm — Permanent loads

Section "Plate Girders" from these calculations

Utilisation = 0.04 — OK

Model rev. D08

Applied bending moment from global analysis model M = 201 kNm — Permanent +44t

Utilisation = 0.67 — OK

Plastic moment of resistance - crossbeam with no additional plates, without section loss

$$y'_p = ABS(y_P - y)$$

Crossbeam with no additional plates - about major (x-x) axis:

Element	b mm	d mm	A mm^2	y mm	y'_p mm	$A\,y'_p$ mm^3
Flange top	305.0	11.0	3.4E+03	6	118.8	3.99E+05
Top angle flange x2	204.0	9.4	1.9E+03	16	108.6	2.08E+05
Top angle web x2	18.8	66.6	1.3E+03	54	70.6	8.84E+04
Web top	12.0	113.3	1.4E+03	68	56.7	7.70E+04
Web bottom	12.0	220.7	2.6E+03	235	110.4	2.92E+05
Flange bottom	305.0	11.0	3.4E+03	351	226.2	7.59E+05
Bot angle flange x2	204.0	9.4	1.9E+03	340	216.0	4.14E+05
Bot angle web x2	18.8	66.6	1.3E+03	302	178.0	2.23E+05
Bottom rivet web	-30.8	18.0	-5.5E+02	302	177.7	-9.85E+04
Bot. rivet flange x2	-36.0	20.4	-7.3E+02	346	221.5	-1.63E+05

totals: 1.6E+04 2.20E+06

Plastic moment of resistance $M_{pe} = \Sigma A\,y'_p \; \sigma_{ys}$ = 540 kNm

cl. 9.7.1 Plastic modulus $Z_{pe} = M_{pe} / \sigma_{ys} = \Sigma A\,y'_p$ = 2.20E+06 mm^3

REFERENCE	CALCULATIONS	OUTPUT

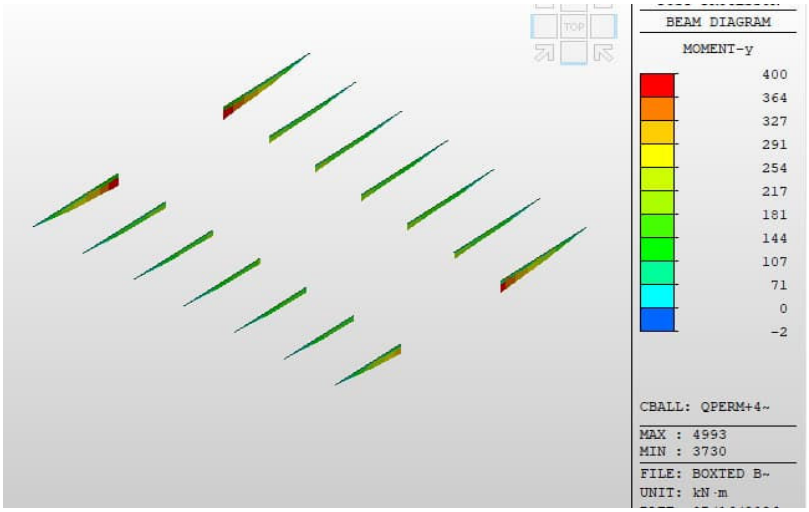
Bending resistance with condition factor

cl. 9.8 The compression flange is restrained by the concrete jack arches hence it is assumed that buckling is prevented and the effective length = 0

cl. 9.9.1.2 $M_{ult} = \frac{0.85}{\gamma_{f3}} \times \frac{M_{pe}}{\gamma_m} =$ 398 kNm

Model rev. D20 Applied bending moment from u-frame action M = 400 kNm Permanent +44t

Utilisation = 1.01 NOT OK



Shear resistance

cl. 9.9.2.2

Depth of web d_w = 356 - 11 - 11 = 334 mm

Thickness of the web

Average web thickness t_w = 12.0 mm

Largest hole or cut-out h_h = 0 mm

Clear depth of web d_{we} = 356 - 11 - 11 - 2 x 76 = 182 mm

Nominal yield stress of web material σ_{yw} = 246 MPa

Nominal yield stress of flange material σ_{yf} = 246 MPa

Clear length of panel a = 6122 mm

Flange thickness t_f = 11 mm

b_{fe} is the smallest of:

a) $10t_f\sqrt{\frac{355}{\sigma_{yf}}}$; or

b) the distance from the mid-plane of the web to the nearer edge of the flange; i.e. taken as zero if there is no flange outstand; or

c) if there are two or more webs, half the clear distance between webs;

a) $10t_f\sqrt{\frac{355}{\sigma_{yf}}}$ = 150 mm

b) 305 / 2 = 152.5 mm

c) N/A

Hence b_{fe} = 150 mm

$$m_{fw} = \frac{\sigma_{yf} b_{fe} t_f^2}{2\sigma_{yw} d_{we}^2 t_w} = 0.0228$$

REFERENCE	CALCULATIONS	OUTPUT

$$\lambda = \frac{d_{we}}{t_w}\sqrt{\frac{\sigma_{yw}}{355}} = 13$$

$$\tau_y = \frac{\sigma_{yw}}{\sqrt{3}} = 142 \text{ MPa}$$

Aspect ratio of the panel $\varphi = \frac{a}{d_{we}} = 33.64$

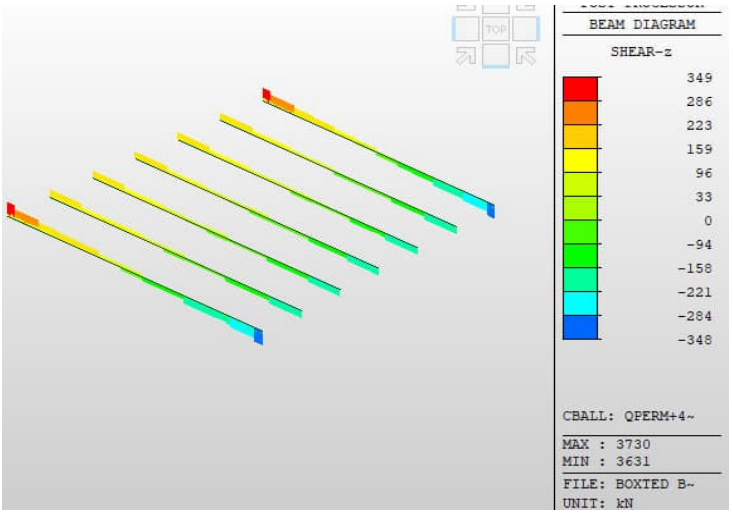
Figure 12 to 18 $\tau_l / \tau_y = 1$

$\tau_l = 141.9$ MPa

$$V_D = \left[\frac{t_w\left(d_w - h_h\right)}{\gamma_m \gamma_{f3}}\right]\tau_\ell = 492 \text{ kN}$$

With condition factor 0.85 x 492 = 418 kN

Model rev. D08 Applied shear force V = 349 kN Utilisation = 0.83 Permanent loads +44t OK



REFERENCE	CALCULATIONS	OUTPUT

Longitudinal RSJs

Take BSB 203x152x52 from British Standard 4 1903.

Historical Steelwork Handbook

Height of section	D =	203.2	mm
Top flange thickness		15.24	mm
Top flange width		152.4	mm
Web thickness		11.2	mm
Bottom flange width		152.4	mm
Bottom flange thickness		15.24	mm

Gross section properties

Properties about major (x-x) axis:

Element	b mm	d mm	A mm^2	y mm
Flange top	152.4	15.2	2.3E+03	7.6
Web	11.2	172.7	1.9E+03	101.6
Flange bottom	152.4	15.2	2.3E+03	195.58

Total **6.6E+03**

Plastic neutral axis y_P = $\frac{\sum A / 2 - \text{(area of top flange and top angles)}}{\text{(thickness of web)}}$ + thickness of top flange

y_P = 101.6 mm

Section classification

Web

Depth of web plate on the compressive side of the plastic NA = y_P - thickness of top flange = 86.36 mm

Depth of the web plate = 172.7 mm

BS 5400-3 cl. 9.3.7.2

Ratio m = 0.50

Nominal yield stress σ_{yw} = 230 MPa

Thickness of web plate t_w = 11.2 mm

Limiting depth $\frac{34t_w}{m}\sqrt{\frac{355}{\sigma_{yw}}}$ when m does not exceed 0.5; and $\frac{374t_w}{(13m-1)}\sqrt{\frac{355}{\sigma_{yw}}}$ when m exceeds 0.5; → 944.16 mm > 172.72 mm

i.e. web is compact

Compression flange

Compression flange outstand b_{f0} = 70.612 mm

Nominal yield stress σ_{yf} = 230 MPa

Mean thickness of outstand

Flange thickness with one additional plate t_{f0} = 15.24 mm

$7t_{fo}\sqrt{\frac{355}{\sigma_{yf}}}$ = 133 mm > 70.612 mm

i.e. flange is compact

Plastic moment of resistance

y'_p = ABS(y_P - y)

REFERENCE	CALCULATIONS	OUTPUT

Properties about major (x-x) axis:

Element	b mm	d mm	A mm^2	y mm	y'p mm	A y'p mm^3
Flange top	152.4	15.2	2.3E+03	8	94.0	2.18E+05
Web top	11.2	86.4	9.7E+02	58	43.2	4.17E+04
Web bottom	11.2	86.4	9.7E+02	145	43.2	4.17E+04
Flange bottom	152.4	15.2	2.3E+03	196	94.0	2.18E+05

totals: 6.6E+03 5.20E+05

Plastic moment of resistance $M_{pe} = \Sigma A\,y'_p \; \sigma_{ys}$ = 120 kNm

cl. 9.7.1 Plastic modulus $Z_{pe} = M_{pe} / \sigma_{ys} = \Sigma A\,y'_p$ = 5.20E+05 mm^3

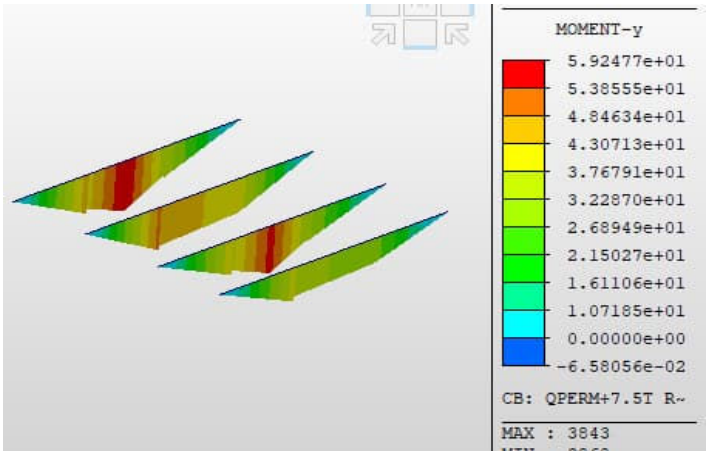
Bending resistance with condition factor

cl. 9.8 The compression flange is restrained by the concrete jack arches hence it is assumed that buckling is prevented and the effective length = 0

cl. 9.9.1.2 $M_{ult} = \frac{0.85}{\gamma_{f3}} \times \frac{M_{pe}}{\gamma_m}$ = 88 kNm

Model rev. D26 Applied bending moment M = 59 kNm Permanent loads + 7.5t

Utilisation = 0.67 OK



Shear resistance

cl. 9.9.2.2 Depth of web d_w = 203.2 - 15.2 - 15.2 = 172.72 mm

Thickness of the web

Average web thickness t_w = 11.2 mm

Largest hole or cut-out h_h = 0 mm

Clear depth of web d_{we} = 203.2 - 15.2 - 15.2 = 172.72 mm

Nominal yield stress of web material σ_{yw} = 230 MPa

Nominal yield stress of flange material σ_{yf} = 230 MPa

Clear length of panel between transverse stiffeners a = 3190 mm

Flange thickness t_f = 15.24 mm

b_{fe} is the smallest of:

a) $10t_f\sqrt{\frac{355}{\sigma_{yf}}}$; or

b) the distance from the mid-plane of the web to the nearer edge of the flange; i.e. taken as zero if there is no flange outstand; or

c) if there are two or more webs, half the clear distance between webs;

REFERENCE	CALCULATIONS	OUTPUT

a) $10t_f\sqrt{\frac{355}{\sigma_{yf}}}$ = 214 mm

b) 152.4 / 2 = 76.2 mm

c) N/A

Hence b_{fe} = 76 mm

$$m_{fw} = \frac{\sigma_{yf} b_{fe} t_f^2}{2\sigma_{yw} d_{we}^2 t_w} = 0.0265$$

$$\lambda = \frac{d_{we}}{t_w}\sqrt{\frac{\sigma_{yw}}{355}} = 12$$

$$\tau_y = \frac{\sigma_{yw}}{\sqrt{3}} = 133 \text{ MPa}$$

Aspect ratio of the panel $\varphi = \frac{a}{d_{we}} = 18.47$

Figure 12 to 18

τ_l / τ_y = 1

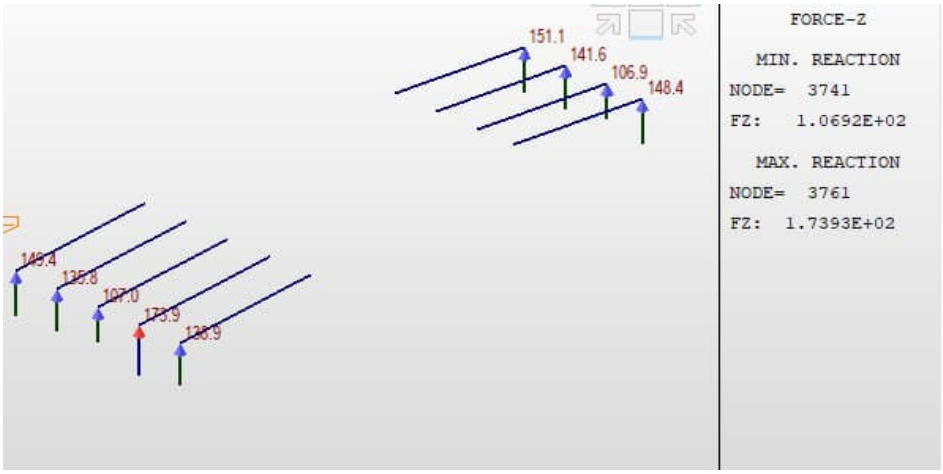
τ_l = 132.8 MPa

$$V_D = \left[\frac{t_w(d_w - h_h)}{\gamma_m \gamma_{f3}}\right]\tau_\ell = 222 \text{ kN}$$

With condition factor 0.85 x 222 = 189 kN

Model rev. D08

Applied shear force V = 174 kN Utilisation = 0.92 Permanent loads +44t OK



Channels

Height of section	D =	254	mm
Top flange thickness		11	mm
Top flange width		76.2	mm
Web thickness		8.1	mm
Bottom flange width		76.2	mm
Bottom flange thickness		11	mm

Gross section properties

Channel with no section loss - about major (x-x) axis:

Element	b mm	d mm	A mm^2	y mm
Flange top	76.2	11.0	8.4E+02	5.5
Web	8.1	232.0	1.9E+03	127
Flange bottom	76.2	11.0	8.4E+02	248.5

Total **3.6E+03**

Plastic neutral axis y_P = ∑A / 2 - (area of top flange and top angles) / (thickness of web) + thickness of top flange

y_P = 127.0 mm

Channel with section loss - about major (x-x) axis:

Element	b mm	d mm	A mm^2	y mm
Flange top	76.2	11.0	8.4E+02	5.5
Web	8.1	232.0	1.9E+03	127
Flange bottom	45.0	11.0	5.0E+02	248.5

IFA photo 30

Total **3.2E+03**

Plastic neutral axis y_P = ∑A / 2 - (area of top flange and top angles) / (thickness of web) + thickness of top flange

y_P = 105.8 mm

Section classification

Web

Depth of web plate on the compressive side of the plastic NA = y_P - thickness of top flange = 116.00 mm

Depth of the web plate = 232.0 mm

BS 5400-3 cl. 9.3.7.2

Ratio m = 0.50

Nominal yield stress σ_{yw} = 230 MPa

Thickness of web plate t_w = 8.1 mm

Limiting depth $\frac{34t_w}{m}\sqrt{\frac{355}{\sigma_{yw}}}$ when m does not exceed 0.5; and $\frac{374t_w}{(13m-1)}\sqrt{\frac{355}{\sigma_{yw}}}$ when m exceeds 0.5; → 684 mm > 232 mm

i.e. web is compact

Compression flange

Compression flange outstand b_{f0} = 34.05 mm

Nominal yield stress σ_{yf} = 230 MPa

Mean thickness of outstand

Flange thickness t_{f0} = 11.0 mm

$7t_{fo}\sqrt{\frac{355}{\sigma_{yf}}}$ = 96 mm > 34 mm

i.e. flange is compact

REFERENCE	CALCULATIONS	OUTPUT

Plastic moment of resistance - Channel with no section loss

$$y'_p = ABS(y_P - y)$$

Channel with no section loss - about major (x-x) axis:

Element	b mm	d mm	A mm^2	y mm	y'_p mm	$A\,y'_p$ mm^3
Flange top	76.2	11.0	8.4E+02	6	121.5	1.02E+05
Web top	8.1	116.0	9.4E+02	69	58.0	5.45E+04
Web bottom	8.1	116.0	9.4E+02	185	58.0	5.45E+04
Flange bottom	76.2	11.0	8.4E+02	249	121.5	1.02E+05
totals:			3.6E+03			3.13E+05

Plastic moment of resistance $M_{pe} = \Sigma A\,y'_p \; \sigma_{ys} = 72$ kNm

cl. 9.7.1 Plastic modulus $Z_{pe} = M_{pe} / \sigma_{ys} = \Sigma A\,y'_p = 3.13E+05$ mm^3

Bending resistance with condition factor

cl. 9.8 The compression flange is restrained by the concrete jack arches hence it is assumed that buckling is prevented and the effective length = 0

cl. 9.9.1.2 $M_{ult} = \frac{0.85}{\gamma_{f3}} \times \frac{M_{pe}}{\gamma_m} = 53$ kNm

Model rev. D08 Applied bending moment $M = 14$ kNm Permanent loads

Utilisation = 0.27 OK

Plastic moment of resistance - Channel with section loss

$$y'_p = ABS(y_P - y)$$

Channel with section loss - about major (x-x) axis:

Element	b mm	d mm	A mm^2	y mm	y'_p mm	$A\,y'_p$ mm^3
Flange top	76.2	11.0	8.4E+02	6	100.3	8.41E+04
IFA ref. photo 30 Web top	8.1	94.8	7.7E+02	58	47.4	3.64E+04
Web bottom	8.1	137.2	1.1E+03	174	68.6	7.62E+04
Flange bottom	45.0	11.0	5.0E+02	249	142.7	7.06E+04
totals:			3.2E+03			2.67E+05

Plastic moment of resistance $M_{pe} = \Sigma A\,y'_p \; \sigma_{ys} = 61$ kNm

cl. 9.7.1 Plastic modulus $Z_{pe} = M_{pe} / \sigma_{ys} = \Sigma A\,y'_p = 2.67E+05$ mm^3

Bending resistance with condition factor

cl. 9.8 The compression flange is restrained by the concrete jack arches hence it is assumed that buckling is prevented and the effective length = 0

cl. 9.9.1.2 $M_{ult} = \frac{0.85}{\gamma_{f3}} \times \frac{M_{pe}}{\gamma_m} = 45$ kNm

Model rev. D08 $M = 64$ kNm Permanent loads + 18t

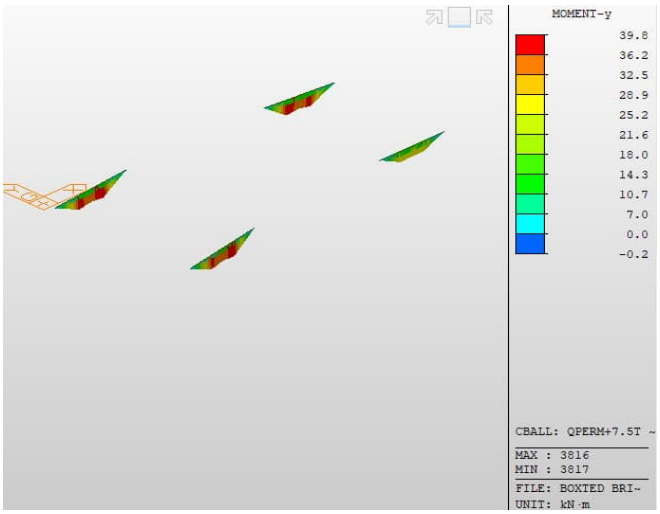
Applied bending moment from global model

Utilisation = 1.41 NOT OK

Model rev. D08 $M = 40$ kNm Permanent loads + 7.5t

Applied bending moment from global model

Utilisation = 0.88 OK



Shear resistance

cl. 9.9.2.2

Depth of web d_w = 254 - 11 - 11 = 232 mm

Thickness of the web

Average web thickness t_w = 8.1 mm

Largest hole or cut-out h_h = 0 mm

Clear depth of web d_{we} = 254 - 11 - 11 = 232 mm

Nominal yield stress of web material σ_{yw} = 230 MPa

Nominal yield stress of flange material σ_{yf} = 230 MPa

Clear length of panel a = 3190 mm

Flange thickness t_f = 11 mm

b_{fe} is the smallest of:

a) $10t_f\sqrt{\frac{355}{\sigma_{yf}}}$; or

b) the distance from the mid-plane of the web to the nearer edge of the flange; i.e. taken as zero if there is no flange outstand; or

c) if there are two or more webs, half the clear distance between webs;

a) $10t_f\sqrt{\frac{355}{\sigma_{yf}}}$ = 155 mm

b) 76.2 / 2 = 38.1 mm

c) N/A

Hence b_{fe} = 38 mm

$$m_{fw} = \frac{\sigma_{yf} b_{fe} t_f^2}{2\sigma_{yw} d_{we}^2 t_w} = 0.0053$$

$$\lambda = \frac{d_{we}}{t_w}\sqrt{\frac{\sigma_{yw}}{355}} = 23$$

$$\tau_y = \frac{\sigma_{yw}}{\sqrt{3}} = 133 \text{ MPa}$$

Aspect ratio of the panel $\varphi = \frac{a}{d_{we}} = 13.75$

REFERENCE	CALCULATIONS	OUTPUT

Figure 12 to 18

τ_l / τ_y = 1

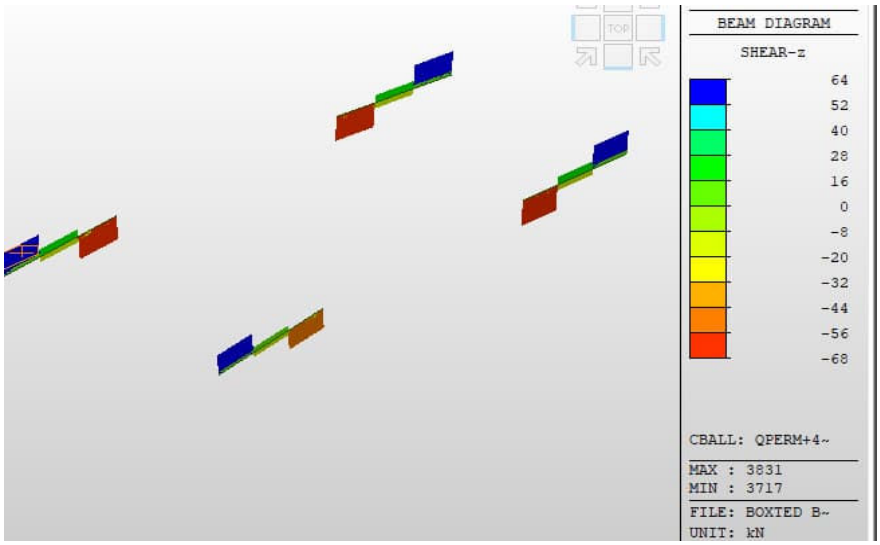
τ_l = 132.8 MPa

$$V_D = \left[\frac{t_w\left(d_w - h_h\right)}{\gamma_m \gamma_{f3}}\right] \tau_\ell = 216 \text{ kN}$$

With condition factor 0.85 x 216 = 184 kN

Model rev. D08

Applied shear force V = 68 kN Utilisation = 0.37 Permanent loads +44t OK



REFERENCE	CALCULATIONS	OUTPUT

Rivets

Shear capacity

Check shear flow at the interface between the flange and the web in the plate girder

Second moment of area of the girder at the support $I_{x,3}$ = 8.20E+09 mm^4

Properties of top flange at the support about major (x-x) axis:

Element	b mm	d mm	A mm^2	y mm	Ay mm^3	y' mm	Ay' mm^3
Flange top	305.0	10.0	3.1E+03	5.0	1.53E+04	637.9	1.95E+06
Flange top vert. x2	20.0	65.0	1.3E+03	42.5	5.53E+04	600.4	7.81E+05
Top angle flange x2	200.0	10.0	2.0E+03	15.0	3.00E+04	627.9	1.26E+06
Top angle web x2	20.0	70.0	1.4E+03	55.0	7.70E+04	587.9	8.23E+05
Total			**7.8E+03**		1.78E+05		4.81E+06

Distance from ENA to comp fibre (whole section) $y_{xc,3}$ = 642.9 mm

Ay' = 4.8E+06 mm^3

Model rev. D08 — Shear force on beam V_{girder} = 551 kN — Permanent loads + 44t

Shear flow $V_{girder} A y' / I_{x,3}$ = 323 kN/m

Drawing BR0059-01-1101A — The rivets are at approx. 102 mm centres

Shear force per rivet V = 323 kN/m x 102 mm = 32.9 kN

Check shear flow at the interface between the flange and the web in the crossbeam

Second moment of area of the crossbeam at the support I_x = 2.67E+08 mm^4

Properties of top flange at the support about major (x-x) axis:

Element	b mm	d mm	A mm^2	y mm	Ay mm^3	y' mm	Ay' mm^3
Flange top	305.0	11.0	3.4E+03	5.5	1.85E+04	132.0	4.43E+05
Total			**3.4E+03**		1.85E+04		4.4E+05

Distance from ENA to comp fibre (whole section) y_{xc} = 137.5 mm

Ay' = 4.4E+05 mm^3

Model rev. D08 — Shear force on beam $V_{crossbeam}$ = 227 kN — Permanent loads + 7.5t

= 156 kN — Permanent loads + 3t

Shear flow $V_{crossbeam} A y' / I_x$ = 376 kN/m — Permanent loads + 7.5t

= 258 kN/m — Permanent loads + 3t

Site measurement — The rivets are at approx. 152.4 mm centres

Shear force per rivet V = 376 kN/m x 152.4 mm = 57.3 kN — Permanent loads + 7.5t

Shear force per rivet V = 258 kN/m x 152.4 mm = 39.3 kN — Permanent loads + 3t

Consider one missing rivet in the crossbeam at IFA defect photo 77

Second moment of area of the crossbeam at this location I_x = 5.78E+08 mm^4

REFERENCE	CALCULATIONS	OUTPUT

Properties of top flange at the support about major (x-x) axis:

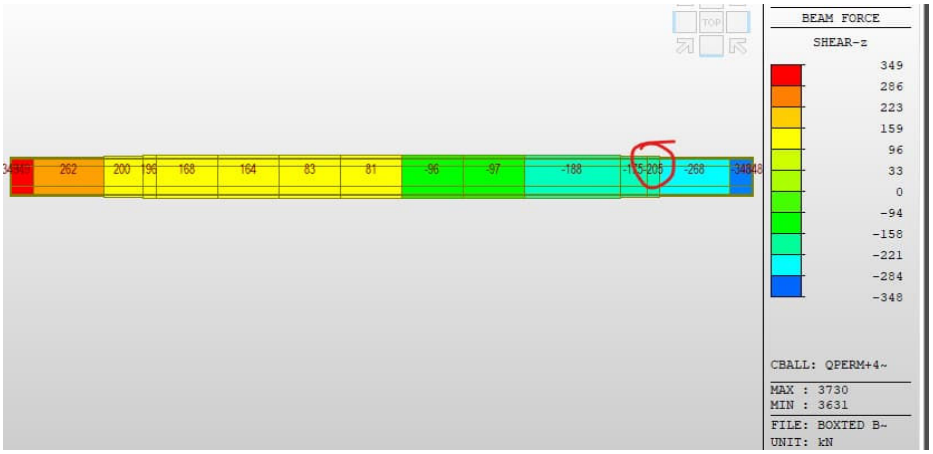
Element	b mm	d mm	A mm^2	y mm	Ay mm^3	y' mm	Ay' mm^3
Flange top	305.0	11.0	3.4E+03	11.5	3.86E+04	166.5	5.59E+05
Total			**3.4E+03**		3.86E+04		5.6E+05

Distance from ENA to comp fibre (whole section) y_{xc} = 178.0 mm

Ay' = 5.6E+05 mm^3

Model rev. D08

Shear force at the defect location $V_{crossbeam}$ = 205 kN — Permanent loads + 44t



Shear flow $V_{crossbeam} A y' / I_x$ = 198 kN/m — Permanent loads + 44t

The rivets are at approx. 304.8 mm centres

Shear force per rivet in the flange

$$V = \frac{198 \text{ kN/m}}{2} \times 152.4 \text{ mm} = 15.1 \text{ kN}$$ — Permanent loads + 44t

15.1 kN < 57 kN

Hence the shear force per rivet at the end of the crossbeam is more onerous than at this location

Verify shear capacity

BS 5400-3 cl. 14.5.3.4

Number of shear planes resisting the applied shear n = 2

Diameter of the fastener 18 mm

Area of the fastener A_{eq} = 254.5 mm^2

Average shear stress $\tau = \frac{V}{nA_{eq}}$ = 112.6 MPa — Permanent loads + 7.5t crossbeam

= 77.3 MPa — Permanent loads + 3t crossbeam

= 64.7 MPa — Permanent loads + 44t plate girder

σ_q = 0.85 x 230 MPa = 195.5 MPa

CS 456 Table 2

γ_m = 1.1 for fasteners in shear

γ_{f3} = 1.1

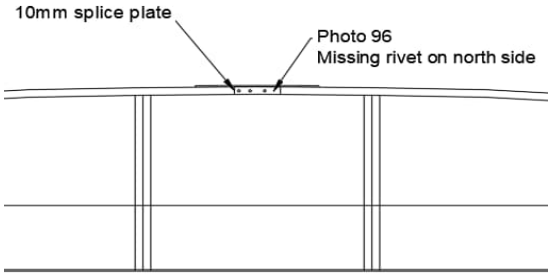
Shear stress capacity $\frac{\sigma_q}{\gamma_m \gamma_{f3} \sqrt{2}}$ = 114 MPa

With condition factor 0.85 x 114 = 97.1 MPa

REFERENCE	CALCULATIONS				OUTPUT
	Utilisation = 112.6 / 97.1 = 1.16		Permanent loads + 7.5t crossbeam		NOT OK
	Utilisation = 77.3 / 97.1 = 0.80		Permanent loads + 3t crossbeam		OK
	Utilisation = 64.7 / 97.1 = 0.67		Permanent loads + 44t plate girder		OK

Check rivets in mid-span vertical splice plate in the plate girder

IFA defect photo 96



Model rev. D08	Bending moment at mid-span	M =	1058	kNm		Permanent loads only	
	Second moment of area at mid-span	I =	1.7E+10	mm^4			
	Distance from elastic neutral axis to top of plate			y =	866	mm	
	Distance from elastic neutral axis to bottom of plate			y =	801	mm	
	Longitudinal stress at top of plate		M y / I	=	52.7	MPa	Permanent loads only
	Longitudinal stress at bottom of plate		M y / I	=	48.8	MPa	
	Average stress				50.8	MPa	
	Area of plate	65.0	x	10 =	650	mm^2	
	Total force in the splice plate to be trasferred by the splice				33	kN	
Model rev. D08	499.5 kN/m x 65 mm = 32 kN						Permanent loads only
	Compare with force in analysis model						

The splice of this vertical downstand comprises 2 rivets either side of the splice. However, one rivet is missing (IFA photo 96).

	Hence	V =	33	kN	
BS 5400-3 cl. 14.5.3.4	Number of shear planes resisting the applied shear		n =	1	
	Average shear stress $\tau = \frac{V}{nA_{eq}}$	=	129.7	MPa	Permanent loads Plate girder splice
	Utilisation = 129.7 / 97.1 = 1.34				NOT OK

Note: the defect occurs at the less critical west plate girder and will have a minor effect on the top flange capacity. The east girder is more critical due to the inadequate end restraint.

Check most onerous bearing pressure in riveted fasteners

k1 =	1	
k2 =	2.5	
k3 =	0.95	
k4 =	1	

	Allowable bearing pressure $\frac{k_1 k_2 k_3 k_4 \sigma_y}{\gamma_m \gamma_{f3}}$	=	451.4	MPa			
	Bearing area	A_{eb} =	18	x 11.0 =	198	mm^2	Permanent loads + 7.5t crossbeam
	Applied bearing pressure	V / A_{eb} =	57.3	/ 198 =	290	MPa	
	Utilisation = 290 / 451.4 = 0.64						OK

REFERENCE	CALCULATIONS	OUTPUT

Bearing area A_{eb} = 18 x 10.0 = 180 mm^2 — Permanent loads + 44t plate girder

Applied bearing pressure V / A_{eb} = 32.9 / 180 = 183 MPa

Utilisation = 183 / 451.4 = 0.41 — OK

Bearing area A_{eb} = 18 x 10 = 180 mm^2 — Permanent loads plate girder splice

Applied bearing pressure V / A_{eb} = 32.5 / 180 = 180 MPa

Utilisation = 180 / 451.4 = 0.40 — OK

Check rivets in bearing stiffener connection

CS 456 App. A cl. 9.14.1

Assessment shear in connection 2.5% x 276 = 6.9 kN

By inspection this will result in smaller shear force per rivet hence less onerous than the cases considered above.

Crossbeam flange curtailment

NR-GN-CIV-025 - Issue 3

v_1 = 1.09

d = 18 mm

BS 5400-3 cl. 9.8

For compact sections, $M_{ult} = M_{pe}$. Also, as the crossbeams are restrained for buckling, $M_R = M_{ult}$

M_R / M_{ult} = 1

Transition between two additional plates and a single additional plate.

Effective cross-sectional area of the flange to which the curtailed flange is attached

Crossbeam with one additional plate

Element	b mm	d mm	A mm^2
Flange bottom	305.0	23.0	7.0E+03
Bot angle flange x2	204.0	9.4	1.9E+03
Bot angle web x2	18.8	66.6	1.3E+03

Total **1.0E+04**

Ae = 1.0E+04 mm^2

Crossbeam with two additional plates

Element	b mm	d mm	A mm^2
Flange bottom	305.0	35.0	1.1E+04
Bot angle flange x2	204.0	9.4	1.9E+03
Bot angle web x2	18.8	66.6	1.3E+03

Total **1.4E+04**

A_{CF} = 1.4E+04 mm^2

$$n = v_1 v_2 \frac{A_{CF}}{d^2}\left(\frac{A_e}{A_e + A_{CF}}\right) = 20.0$$

Curtailment length $\frac{n \times 152.4}{2 \text{ rows}}$ = 1524 mm

Transition between one additional plate and no additional plates

Crossbeam with no additional plates

Element	b mm	d mm	A mm^2
Flange bottom	305.0	11.0	3.4E+03
Bot angle flange x2	204.0	9.4	1.9E+03
Bot angle web x2	18.8	66.6	1.3E+03

Total **6.5E+03**

Ae = 6.5E+03 mm^2

A_{CF} = 1.4E+04 mm^2

$$n = v_1 v_2 \frac{A_{CF}}{d^2}\left(\frac{A_e}{A_e + A_{CF}}\right) = 15.0$$

Curtailment length $\frac{n \times 152.4}{2 \text{ rows}}$ = 1143 mm

CS 461

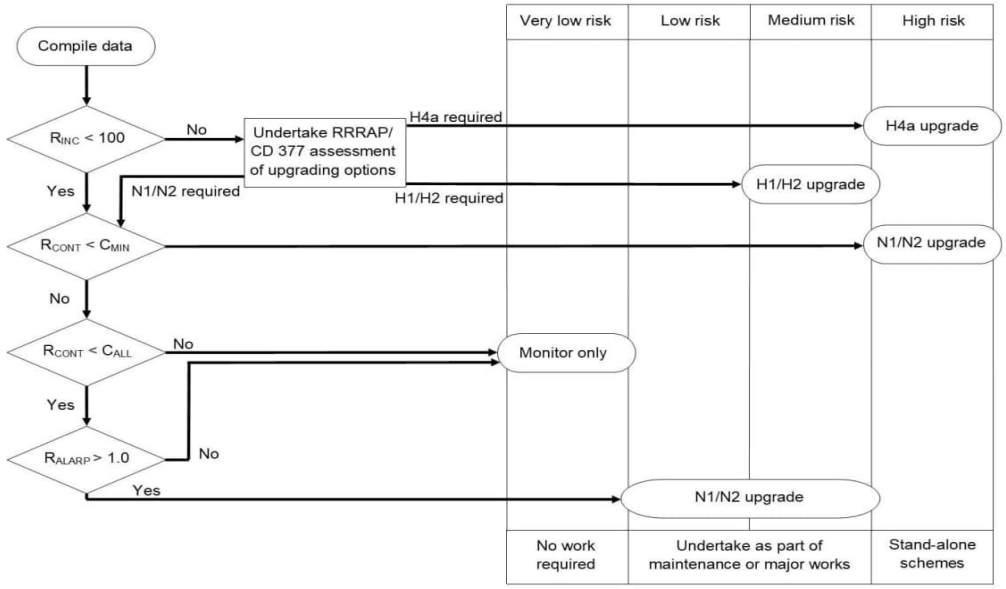
Risk assessment of existing parapet

cl. 3.3

In accordance with CS 461 cl. 3.3, Figure 3.2a may be used for bridges over rivers, without the incursion risk ranking tools.

Figure 3.2a

Figure 3.2a Assessment flowchart - bridges over roads



Assume R_{INC} < 100 for the bridge over river.

CS 461

So far as is reasonably practicable (SFAIRP) principle

Table A.2, Table 3.6

C_{REQ} = 0.73 N2

cl. 4.18

R_{CONT} = 0.00 N2 → 0% of C_{REQ}

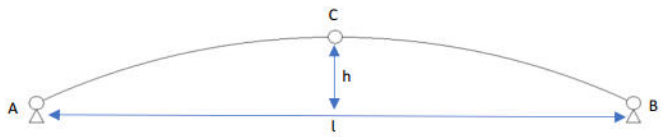
Table 3.8

C_{MIN} = 0.15 N2

R_{CONT} < C_{MIN} → N1/N2 upgrade recommended

Remnant capacity is taken as zero as the calculations show failures for permanent loads.
If the capacity of the girders is restored, the remnant capacity is expected to be less than C_{MIN}

CIS 22 Jack Arch Qualitative Assessment



h = 100 mm
l = 1020 mm

Table 1: Criteria for Empirical Assessment of Jack Arches

Ref	Structure Configuration	Comments	Pass/Fail
1	Maximum clear span of jack arch <2.0m	Maximum jack arch clear span = 715mm	PASS
2	The jack arch springs from the bottom flange of the adjoining beams	Jack arches spring from bottom flanges	PASS
3	Adequate horizontal restraint is provided to resist horizontal forces and to maintain composite action within the deck construction.	For the longitudinal jack arches, a horizontal restraint is provided through the load path from the crossbeams, through the deck, the longitudinal beams and the abutments. For the transverse jack arches, horizontal restraint is provided by the longitudinal channels. At one end, the channel is restrained by the main plate girder. At the other end, the channel is restrained by the abutment and a tie bar.	PASS
4	For any masonry or concrete jack arch, the minimum combined thickness of the jack arch plus the concrete fill above ≥ 220mm	Depth of concrete at the crown = 414mm	PASS
5	There are no structural defects due to deterioration or distress that could affect the capacity of the jack arches (see type no. 6-13 in the table below)	No defects identified	PASS
6	For all bridge decks, 'structural backing' is provided to at least the height of the extrados of the arch, subject to a minimum effective shear depth obtained from Figure 1 of CIS22.	Arch rise = 100mm; Structural fill above crown = 414 mm Minimum Permissible Effective Shear Depth = ~300mm Arch rise + structural fill = 514mm	PASS
7	A maximum gross aspect ratio for a jack arch of 10 (beam spacing / rise of arch)	1020mm spacing / 100mm rise = approx. 10 ≤ 10	PASS

Notes
Adequate horizontal resistance is determined in type no. 2, 3a, 3b in the table below
Structural fill/backing is classified as concrete, gypsum lime or mortared masonry
Effective shear depth = (arch rise + height of structural fill above crown of extrados)

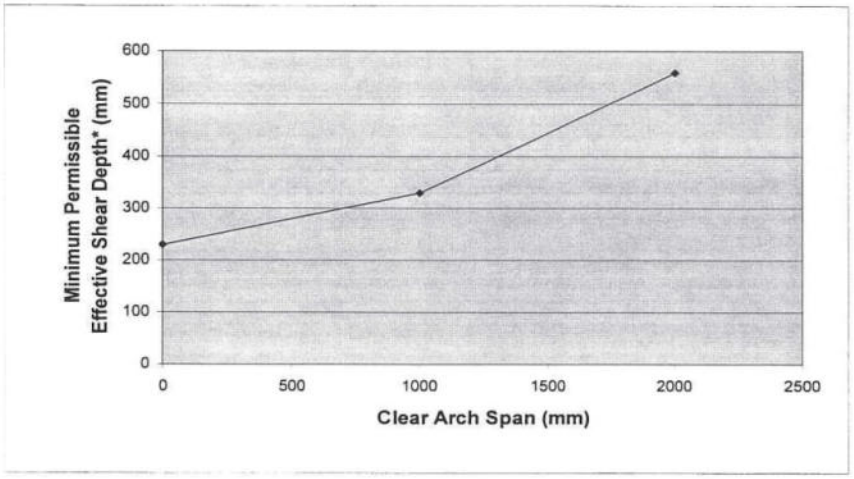
Table 2: Identification of Structural Defects or Deficiencies

Type No.	Deficiency/Defect	Comments for Empirical Assessment	Pass/Fail
	Deficiency		
1	No structural backing to crown level of extrados	Structural backing/fill is present	PASS
2	No ties in edge bay (if a jack arch)	Restraint provided through edge beams and a single tie near each abutment	PASS
3a (CI)	Ties in edge bay (if a jack arch) if < 260mm^2 /m or if >2.5m apart	N/A	PASS
3b (WI/ST)	Ties in edge bay (if a jack arch) if < 260mm^2 /m or if >3.0m apart	The channels are restrained partly by the edge plate girders, abutment connection and a single tie bar hence cannot be directly quantified, however from engineering judgement the restraint appears to be adequate.	PASS
4	Ties located in crown of external arch	Tie bars are above the crown of the hogging plate within the concrete fill (through the crown of the arch). However, the channel is partly restrained by the edge girder and is embedded in the abutment, therefore it is considered to be adequately restrained.	PASS
5	Jack arch adjacent to 'soft' edge service bay area treat as 2,3a or 3b accordingly	No 'soft' connection to edge bay	PASS
	Defect		
6	Rotation of supporting beam	No rotation identified	PASS
7	Horizontal displacement of supporting beam	No horizontal displacement identified	PASS
8	Inadequate support to springings (e.g. corrosion of bottom flange of supporting beam over a significant length, missing bedding mortar)	No defects identified around the support to the jack arch spring	PASS
9	Transversely bowed bottom flange of supporting beam	No bowing to edge beam identified	PASS
10	Cracking at crown of arch owing to spreading of springings (other than 12,13)	No cracking at the jack arch crown identified	PASS

11	Distortion and any associated cracking of jack arch barrel	No distortion to the jack arch barrel identified	PASS
12	Arch crack resulting in substructure crack	No cracks identified in the jack arch	PASS
13	Substructure crack or other distress resulting in crack to jack arch	A verical crack on the east side of each abutment has been identified however no damage to the adjacent jack arch has been identified. The vertical cracks in the abutments are likely due to impact to the plate girders and not related to the jack arches.	PASS

Notes
CI = Cast Iron; WI = Wrought Iron; ST = Steel

Extract of Figure 1 from CIS22



**(= arch rise + barrel thickness + height of structural fill above crown of extrados)*

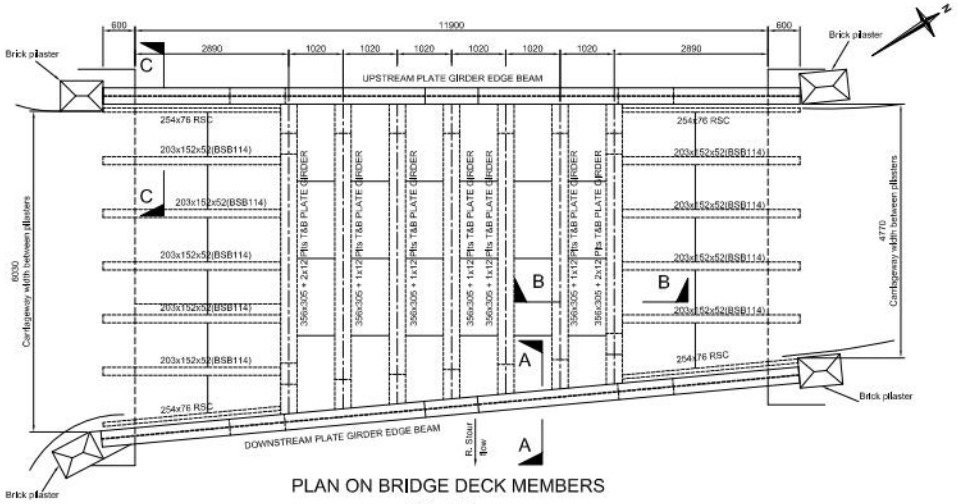
Figure 1

Edge bay restraint:



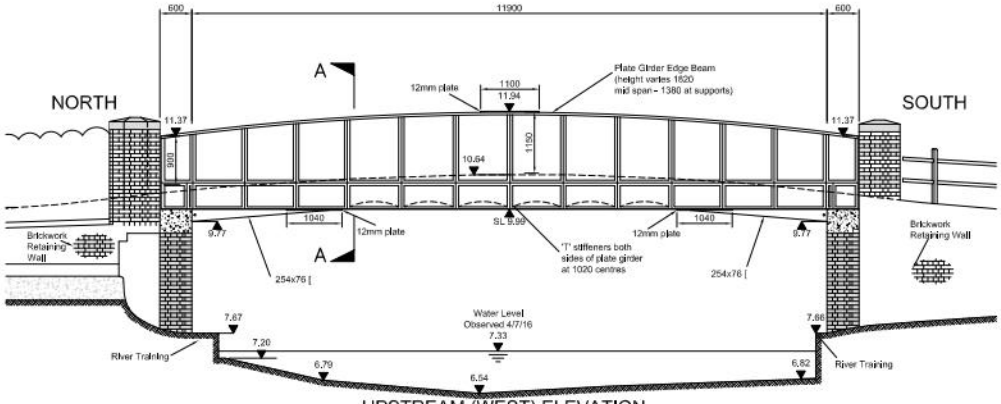
Element	Effect type	Capacity	Load combination	Load effect	Utilisation ratio	Remarks
Plate girder, mid-span	Bending moment (lateral torsional buckling)	1053 kNm	Permanent loads	1059 kNm	1.01	Capacity using slenderness from BS 5400-3 cl. 9.7.5. The end restraint does not meet provisions from BS 5400-3 cl. 9.12.5.2 hence increased slenderness from CS 456 cl. 9.6.1 is applied.
Intermediate transverse stiffener in plate girders	Stress in vertical stiffener	181 MPa	Permanent loads (including forces from u-frame restraints)	480 MPa	2.65	Most onerous location considering section losses from defect ref. photo 85 and 87 in the IFA. Including u-frame force from BS 5400-3 cl. 9.12.3.1.
South-east, south-west and north-west bearing stiffeners	Buckling of the effective stiffener section (ratio)	0.64	Permanent loads (including end-restraint force)	0.33	0.51	Including the end-restraint force from cl. 9.12.5.2
North-east bearing stiffener	Buckling of the effective stiffener section (ratio)	0.64	Permanent loads (including end-restraint force)	1.22	1.89	North-east bearing stiffener, considering section losses. Including the end-restraint force from cl. 9.12.5.2
Plate girder	Shear force	1692 kN	Permanent + 44t	551 kN	0.33	
Crossbeam (2 additional plates)	Bending moment	860 kNm	Permanent + 44t	485 kNm	0.56	
Crossbeam (1 additional plate)	Bending moment	622 kNm	Permanent + 44t	457 kNm	0.73	Considering flange plate curtailment to NR-GN-CIV-025
Crossbeam (no additional plate)	Bending moment	398 kNm	Permanent + 44t	400 kNm	1.01	Considering flange plate curtailment to NR-GN-CIV-025, without section loss
Crossbeam	Shear force	418 kN	Permanent + 44t	349 kN	0.83	
Longitudinal RSJ	Bending moment	88 kNm	Permanent + 18t	95 kNm	1.08	
Longitudinal RSJ	Bending moment	88 kNm	Permanent + 7.5t	59 kNm	0.67	
Longitudinal RSJ	Shear force	189 kN	Permanent + 44t	174 kN	0.92	
Channel	Bending moment	45 kNm	Permanent + 18t	64 kNm	1.41	Channel with section loss, IFA ref. photo 30
Channel	Bending moment	45 kNm	Permanent + 7.5t	40 kNm	0.88	Channel with section loss, IFA ref. photo 30
Channel	Shear force	184 kN	Permanent + 44t	68 kN	0.37	
Rivets in plate girders - general	Shear stress in rivet	97 MPa	Permanent + 44t	65 MPa	0.67	
Rivets in crossbeams	Shear stress in rivet	97 MPa	Permanent + 7.5t	113 MPa	1.16	
Rivets in crossbeams	Shear stress in rivet	97 MPa	Permanent + 3t	77 MPa	0.80	

Appendix D. General Arrangement



PLAN ON BRIDGE DECK MEMBERS

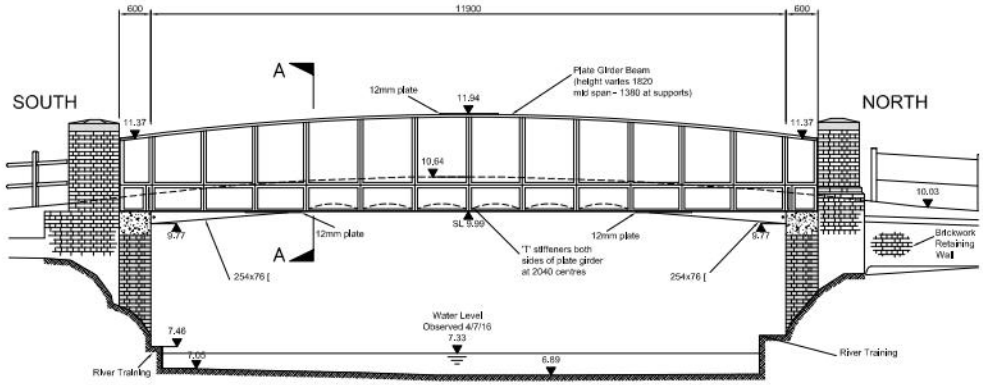
Scale 1:50



UPSTREAM (WEST) ELEVATION

WATERWAY AREA= 37.70 m²

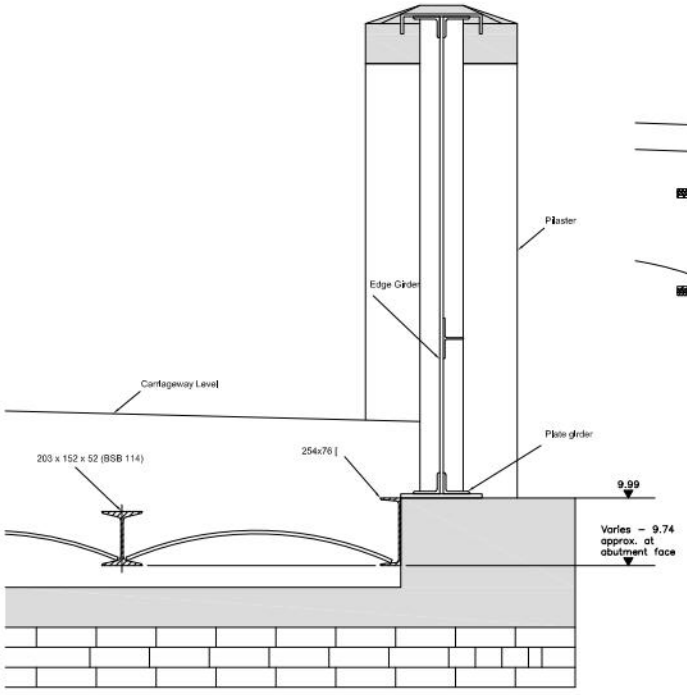
Scale 1:50



DOWNSTREAM (EAST) ELEVATION

WATERWAY AREA= 36.15 m²

Scale 1:50

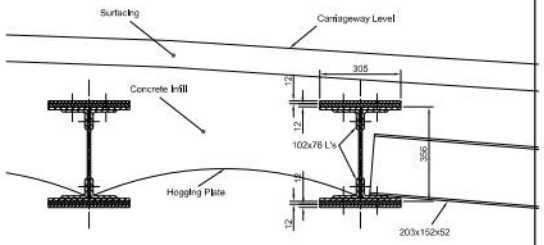


SECTION C-C

Scale 1:10

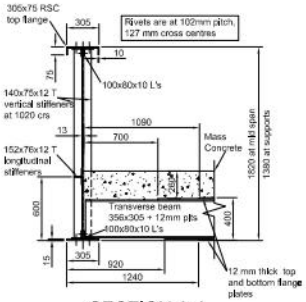
NOTES

1. All dimensions are in millimetres unless noted otherwise.
2. All levels are in metres above Ordnance Datum unless noted otherwise.



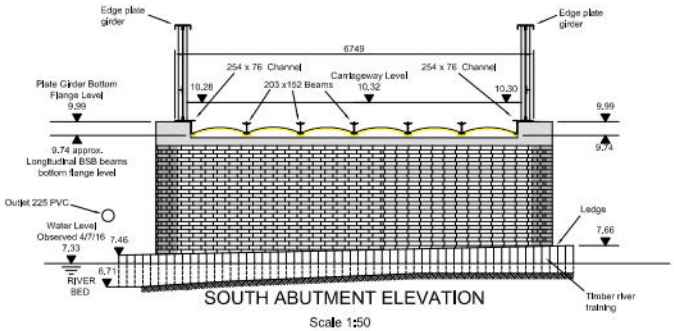
SECTION B-B

Scale 1:10



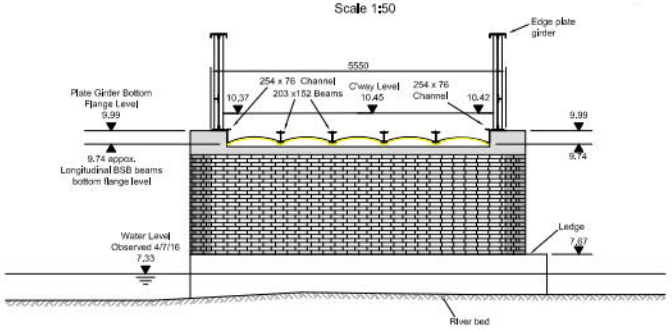
SECTION A-A

Scale 1:25



SOUTH ABUTMENT ELEVATION

Scale 1:50



NORTH ABUTMENT ELEVATION

Scale 1:50

Rev.	Date	Description of revision	Drawn	Checked	Reviewed	Approved
A	08/16	River bed profile amended and other minor amendments				

DRAWING STATUS

INFORMATION

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SCHEME TITLE

BOXTED BRIDGE
ECC BRIDGE No. 059

DRAWING TITLE

EXISTING BRIDGE
GENERAL ARRANGEMENT

DESIGNED	DRAWN	CHECKED	REVIEWED	APPROVED
DATE	DATE JUN14	DATE AUG 16	DATE AUG 16	DATE SEP 16

DRAWING UNITS (UNO)
DIMENSIONS IN MILLIMETRES
LEVELS IN METRES

SCALE AT A1 (841X594mm)
As Shown

DRAWING No.
BR0059-01-1101

REV.
A

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