Half-Joint Assessment Report - Underbarrow

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Westmorland & Furness Council
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Risk Assessment and Structural Assessment of Post-Tensioned and Half-Joint Bridges SL240 Brigsteer and SL221 Underbarrow
11 June 2024

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Appendices

Executive Summary

Jacobs was commissioned by Westmorland & Furness Council to undertake a structural assessment of the half-joints of Underbarrow. The purpose of this report is to detail the results from this assessment.

An assessment report dated February 1995 produced by Cumbria County Council concluded that the structure has a capacity for 40T Assessment Live Loading and a HB capacity of 22.5 units as stated on the signed certification (dated 14th February 1995). However, a note on the results summary sheet states that the suspended span and the top slab of the hollow parts of the cantilever will carry 30 units HB loading, but if the HB vehicle travels within 150mm of the kerb, allowing associated HA loading, then the capacity reduces to 14 HB units, limited by the lower nib of the half-joints. SLS checks concluded that the actual crack width is greater than twice the allowable width. The cracking was attributed to poor detailing of reinforcement as opposed to overloading.

This structural assessment of the half-joints has been based on the condition of the half-joints as identified by an August 2022 Special Inspection. The half-joints were found to be in a fair condition with cracks noted at the re-entrant corners of the upper and lower nibs. A condition factor of 0.9 has been used for assessment purposes.

The findings of the half-joint inspection found inconsistencies between the available design and assessment information (calculations, drawings etc.) and the actual size of the half-joints as-constructed, and ferro-scanning of the half-joints determined that the reinforcement was more aligned in size to the arrangement shown within the design calculations. The original design calculations were much more conservative than the as-built records and the 1994 assessment calculations. There are no records of intrusive works to verify the assumptions used throughout the 1994 Assessment (only the as-built drawings which have been found to contain inaccuracies). As a result of the inconsistencies in available information, the Approval in Principle dated 12th January 2023 sets out conservative assumptions, utilising the confirmed geometry of the half-joints and reinforcement arrangement indicated within the design calculations.

Based on the results of this assessment, the half-joints have been found to be inadequate for dead loads at ULS and SLS. It is recommended that investigative works are carried out to ascertain the true construction details and material strengths. In the interim, the structure is considered to be sub-standard as a result of this assessment, a CS470 should be carried out to confirm this until further assessment is undertaken to consider the results of material testing. It is recommended that the structure is monitored (visual inspection and non-destructive testing).

Summary of Results

The half joints have been assessed to CS 454 and the results are summarised in the table below:

Structural Element	Loading	Capacity
Half Joint (Upper Nib)	Dead load (ULS)	Inadequate
Half Joint (Lower Nib)	Dead load (ULS)	Inadequate
Half Joint (Upper Nib)	Dead load (SLS)	Adequate
Half Joint (Lower Nib)	Dead load (SLS)	Inadequate

1. Introduction

1.1 Description

Underbarrow, constructed in 1970, carries the C5048 single carriageway Underbarrow Road east - west over the A591, Kendal Bypass County Road, west of Kendal at OS Grid Reference SD 499 924.

The superstructure is a single span made up of in-situ concrete cantilevers and a precast concrete beam suspended span. The cantilevers are of post-tensioned voided construction, integral with the abutments. The suspended span comprises 17No. prestressed pre-tensioned concrete beams and an in-situ reinforced concrete deck slab. The inner beams are inverted T-beams and are transversely post-tensioned. The edge beams are box beams, connected to the rest of the deck by reinforcement protruding from the inner side of each beam. The suspended span is supported by half-joints at the ends of the cantilevers.

The top of the structure comprises hardened verges to the north and south, 1.4m and 2m wide respectively. The carriageway between verges is 6.2m. Edge protection is provided by painted metallic parapets comprising posts, two rails and vertical infill railings. The posts are mounted and countersunk into the parapet plinths using holding down bolts. The parapet plinth/ edge beam is 0.45m wide.

The A591 below is a dual carriageway with a grassed central reserve and grassed verges. There are "limestone pitching" revetments in front of both abutments.

Records state that asphaltic plug type movement joints have been installed above both half-joints. However, one of the joints appears to have been surfaced over and the surfacing has cracked.

The suspended square span measures 18.288m (60' 0") between centrelines of half-joint bearings.

1.2 Structural Type

The deck is a single span comprising in-situ concrete cantilevers, post-tensioned longitudinally, cast integral with the abutments, and a suspended span comprising 17No. longitudinally pre-tensioned concrete beams and an in-situ reinforced concrete deck slab. The inner beams are inverted T-beams and are transversely post-tensioned. The edge beams are box beams.

The west cantilever and integral abutment contains 26 No. post-tensioned cables which are typically at 457.2mm centres. The cables are located within the upper areas of the voided construction, to resist tension due to hogging bending moments, and taper down at either end of the element. The cables which are situated directly above the vertical walls of the voided construction terminate within the walls and do not extend to the half-joints. All the anchorages appear to be recessed into the concrete; although no details are given regarding any capping, it is expected that the recesses were capped following tensioning. At the half-joint the tendons are anchored in the upper area of the deck and do not provide any strength to the lower nib of the half-joint. The strength of the lower nib therefore comes predominantly from the reinforced concrete detailing only and acts in a similar manner to a corbel.

The east cantilever and integral abutment contains 26 No. post-tensioned cables which are typically at 457.2mm centres. The cables are located within the upper areas of the voided construction and taper down at either end of the element. The cables which are situated directly above the vertical walls of the voided construction terminate within the walls and do not extend to the half-joints. All the anchorages appear to be recessed into the concrete; although no details are given regarding any capping, it is expected that the recesses were capped following tensioning. At the half-joint the tendons are anchored in the upper area of the deck and do not provide any strength to the lower nib of the half-joint. The strength of the lower nib therefore comes from the reinforced concrete detailing only and acts in a similar manner to a corbel.

1.3 Foundation Type

The available records show that the abutments are founded on a 230mm thick layer of concrete blinding. Local to the toe and heel, the substrate was excavated and replaced with class E3/4 mass concrete infill (equivalent to modern-day 50 N/mm^2 concrete).

1.4 Span Arrangements

The clear span between abutments is 48.763m; the suspended span between centrelines of half-joint bearings is 18.288m and the length of the integral cantilevers and abutments from the centreline of the half-joint bearings to the back of abutment is 18.2m and 18.1m for the east and west respectively.

The overall width of the structure is 10.5m.

1.5 Articulation Arrangements

Historical drawings marked 'record drawing' detail 17.No elastomeric Dunlop Metalastik bearings. Record drawings detail the following for the same type of bearings; 285.75mm x 146mm x 78.13mm thick. The bearings are presumably centred under each of the 17 No. precast beams. Fixity is provided at the east half-joint by 14 No. horizontal bars at 609mm centres between internal beams.

1.6 Parapets

The parapets comprise posts, two rails and vertical infill railings. There is concern that the parapets do not meet current containment standards.

A VRS, supported on timber posts, is in place at each corner of the structure.

1.7 Scope of Assessment

Only the half-joints have been assessed as part of this commission, in accordance with the AIP dated 12th January 2023.

The assessment processes and basis of assessment for the half-joints follows the requirements of CS 454 and CS 455 supplemented by the additional requirements of CS 466 (section 6).

An assessment report dated February 1995 produced by Cumbria County Council concludes that the structure has a capacity for 40T Assessment Live Loading and a HB capacity of 22.5 units as stated on the signed certification (dated 14th February 1995). However, a note on the results summary sheet states that the suspended span and the top slab of the hollow parts of the cantilever will carry 30 units HB loading, but if the HB vehicle travels within 150mm of the kerb, allowing associated HA loading, then the capacity reduces to 14 HB units, limited by the lower nib of the half-joints. SLS checks concluded that the actual crack width is greater than twice the allowable width. The cracking was attributed to poor detailing of reinforcement (lack of diagonal reinforcement within the lower nib) as opposed to overloading.

1.8 Historical Information

Details of historical information can be found in the Structural Review Report (ref. BCU00015-JAC-SBR-6330-RP-SL221-CB-006).

1.9 Inspection for Assessment

Refer to the Half-Joint Inspection Report – Underbarrow (ref. BCU00015-JAC-SBR-6330-RP-SL221-CB-004).

2. Assessment Parameters

2.1 Assumptions

The assessment process includes a consideration of the condition of the structure as confirmed during the Jacobs Inspection for Assessment, dated 24th August 2022.

The inspection of the half-joints concluded the following:

- Several cracks were found to the upper (2No) and lower nibs (4No) on the east half-joint.
- The half-joints are typically in fair condition despite the number of cracks.

One of the objectives of the half-joint inspection was to confirm that dimensions on site match those shown on record drawings and hence confidence could be taken that the record drawings are a true representation of the structure as-constructed. However, the upper and lower nibs of the half-joints appear to have different depths to those shown on the record drawings, and so it has to be concluded that the record drawings aren't wholly reliable.

For assessment purposes, the size of the upper and lower nib is taken as physically measured.

As there has been no confirmation of the reinforcement detail by breakout and inspection, the reinforcement layout as shown on record drawings has been used for assessment since this seems relatively consistent with that indicated by scanning techniques on site.

2.2 Condition Factors

Previous inspection reports have raised concerns regarding the cracking to the re-entrant corners of the lower nib. By further inspection, it is concluded that the existing cracks do not appear to have grown noticeably.

Recommended condition factor for assessment = 0.9.

2.3 Material Properties

The material properties are assumed in accordance with the values shown on the record drawings.

Concrete Strength

Abutments/ Cantilevers: fcu = 41.4 N/mm2

Precast Beams: fcu = 51.7 N/mm2

Deck Slab: fcu = 41.4 N/mm2

Mild Steel Strength

All Elements: fy = 250 N/mm2 (BS4449:1969)

Refer to section 3.10.1 of the Approval in Principle for further information.

2.4 Method of Analysis

The suspended span has been analysed using a 2-D computer grillage model, assuming original design deck articulation, in order to obtain bearing reactions at the half-joints.

The internal beams have been modelled with torsionless properties. The edge beams (box beams) retain their properties relevant to torsion.

The upper and lower nibs are assessed using the most onerous load effects. Idealised "strut and tie models" as recommended in CS 466 shall be used for assessment of half-joints at ULS taking account of the proposed condition factor outlined above.

The SLS assessment of crack widths has been carried out in accordance with the methodology outlined in Appendix D of CS 466.

2.5 Checking Procedure

The structure is a Category 3 structure in accordance with CG 300. As such, an independent assessment team from a separate organisation [redacted] have carried out an assessment check in accordance with the signed Approval in Principle document.

3. Assessment Results

The assessment has concluded that the half-joints are inadequate for dead load.

At ULS, the ties within each of the applicable strut and tie models are noted to be the critical elements.

At SLS, the lower nib's crack width fails by a significant margin. This is due to the poor detailing of the lower nibs which do not appear to contain any inclined reinforcement.

At SLS, the upper nib's cracking is controlled by the inclined reinforcement shown on 'as built' drawings.

A breakdown of the assessment results showing the worst-case strut, tie or node for each half joint model (as per Appendix E of CS 466) is detailed below in the following tables. The full set of calculations used to derive the results can be found in Appendix A of this report.

	Figure (App. E, CS 466)	Member (Strut / Tie)	Assessment Load Effects				Assessment Resistance			Adequacy			
			Dead + Superimposed Dead + HA Loads	Dead + Superimposed Dead Loads	Type HA Vehicle Loading (40T)	SV Vehicles	Resistance	Condition Factor	Assessment Resistance	Reserve Factor (DL + SIDL)	Critical Element	Reserve Factor (DL + SIDL + HA)	Critical Element
			S^*_A	S^*_D	S^*_{HA}	S^*_{SV}			R^*_A	R^*_A / S^*_D		R^*_A / S^*_A	
Lower Nib	E.16	Strut(s)	11.22	6.15	5.07	N/A – Structure inadequate for Dead LoadLoad	11.75	0.9	10.57	1.72	FS1	0.94	FS1
		Ties(s)	779	426.9	352.1		217.4		195.56	0.46	FT1	0.25	FT1
		Node(s)	11.22	6.15	5.07		16.64		14.97	2.43	Node A	1.33	Node A
	E.3	Strut(s)	13.39	7.34	6.02		11.75		10.57	1.44	FS1	0.79	FS1
		Ties(s)	994.4	477.94	394.3		217.4		195.56	0.41	FT5	0.20	FT5
		Node(s)	13.39	7.34	6.02		16.64		14.97	2.04	Node A	1.12	Node A
	E.9	Strut(s)	13.62	7.46	6.16		11.75		10.57	1.24	FS1	0.78	FS1
		Ties(s)	562.4	270.3	223		217.4		195.56	0.61	FT1	0.35	FT1
		Node(s)	13.62	7.46	6.16		16.64		14.97	2.0	Node A	1.10	Node A

Note: Calculations for SV Vehicles have not been undertaken as certain members within the half joint were found inadequate for Normal Traffic and Dead Loads. Should the half joints be found adequate for Normal Traffic following a re-assessment, taking into account findings from an intrusive investigation, further analysis for SV Vehicles should be undertaken.

	Figure (App. E, CS 466)	Member (Strut / Tie)	Assessment Load Effects				Assessment Resistance			Adequacy			
			Dead + Superimposed Dead + HA Loads	Dead + Superimposed Dead Loads	Type HA Vehicle Loading (40T)	SV Vehicles	Resistance	Condition Factor	Assessment Resistance	Reserve Factor (DL + SIDL)	Critical Element	Reserve Factor (DL + SIDL + HA)	Critical Element
			S^*_A	S^*_D	S^*_{HA}	S^*_{SV}			R^*_A	R^*_A / S^*_D		R^*_A / S^*_A	
Upper Nib	E.16	Strut(s)	7.7	4.2	3.5	N/A – Structure inadequate for Dead Load	13.94	0.9	12.55	2.98	FS2	1.62	FS2
		Ties(s)	513.8	281.5	232.3		217.4		195.56	0.70	FT2	0.38	FT2
		Node(s)	7.7	3.53	3.5		19.75		17.8	5.01	Node A	2.31	Node A
	E.3	Strut(s)	10.37	5.68	4.69		13.94		12.55	2.21	FS2	1.21	FS2
		Ties(s)	1051.9	576.39	475.5		217.4		195.56	0.34	FT5	0.19	FT5
		Node(s)	10.37	5.68	4.69		19.75		17.8	3.13	Node A	1.72	Node A
	E.15	Strut(s)	6.27	3.43	2.89		13.94		12.55	3.66	FS1	2.01	FS1
		Ties(s)	329.82	180.73	149.09		217.4		195.56	1.08	FT2	0.6	FT2
		Node(s)	6.27	3.43	2.89		19.75		17.8	5.2	Node A	2.84	Node A
	E.9	Strut(s)	14.62	8.01	6.61		13.94		12.55	1.57	FS1	0.86	FS1
		Ties(s)	826.18	452.71	373.5		217.4		195.56	0.43	FT1	0.24	FT1
		Node(s)	14.62	8.01	6.61		19.75		17.8	2.2	Node A	1.22	Node A

Note: Calculations for SV Vehicles have not been undertaken as certain members within the half joint were found inadequate for Normal Traffic and Dead Loads. Should the half joints be found adequate for Normal Traffic following a re-assessment, taking into account findings from an intrusive investigation, further analysis for SV Vehicles should be undertaken.

3.1 Sensitivity Analysis

Sensitivity analysis shows that, if intrusive works can confirm that material properties are significantly better than assumed thus far (i.e. if the tensile strength of the reinforcement = 460N/mm2); and that the size of reinforcement is universally 19.05mm diameter as suggested in the 1994 assessment, then the half-joints have capacity to carry some live load, however the capacity will likely remain less than 40T.

Post-tensioning within the cantilevers was not included within the scope of the assessment and AiP on the basis that it terminates within the upper portion of the cantilevers and does not directly provide strength to the lower nib. However, it is further considered that the post-tensioning force may relieve some of the tensile force in the upper tie of the cantilever strut and tie analysis as shown below. The tie components of the models local to the nib will see no increase.

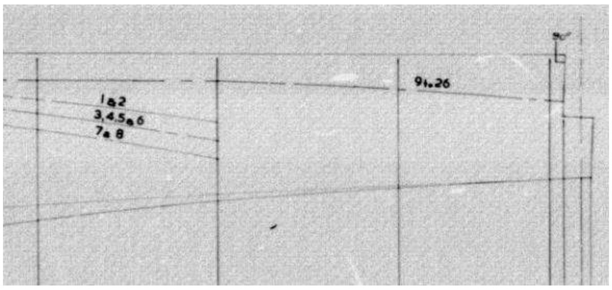

Figure 3 – Showing section through cantilever and location of post tensioning

Figure E.16 Illustrative example of a strut-and-tie model for a system with vertical bars

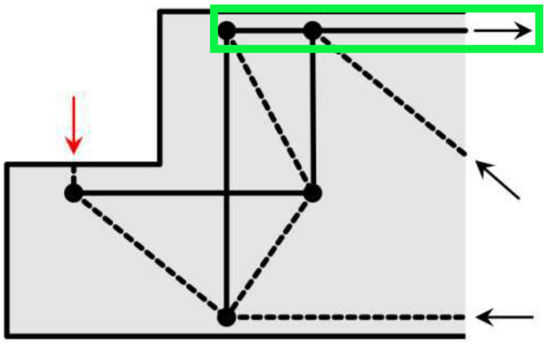

Figure 4 – Showing Analysis model E.16 in accordance with CS466.

GREEN indicates strengthened tie(s) to lower nib model (if considering post-tensioning).

Inclusion of the post-tensioning alone <u>will not</u> see an increase in the global capacity of the half-joints. Inclusion of the post-tensioning in combination with an increase in material strengths may provide an increase in their capacity. To include the post-tensioning in further assessment, their details and condition would need to be established through PTSI Site Investigation.

No modifications have been made to the condition factor for the purpose of sensitivity checks.

4. Conclusions and Recommendations

4.1 Conclusions

The assessment concludes that the half-joints are inadequate for dead load.

The inspection for assessment concluded that the half-joints are in fair condition and they exhibit cracking to the re-entrant corners of the lower nib. The condition factor for assessment is 0.9.

The half-joint inspections found irregularities between the design, assessment, construction records, and as-constructed elements:

- Physical size of half-joint differs to the design dimensions, assessment dimensions & construction record dimensions.
- The scanned reinforcement size and layout conflicted with the assessment & construction records, with a much closer resemblance of the reinforcement detailed in the design calculations.
- The irregularities raise concerns that other construction details may be significantly different to those shown on the record drawings i.e. the post-tensioning.

The material properties have not been confirmed by testing and have been assumed in accordance with the material properties shown on the construction record drawings, as agreed in the AIP.

There is no feasible method of remediating the relatively minor defects of note to the half-joints. Given the critical details in the structure (post-tensioning and half-joints), any investigative work must be carefully considered and carried out in strict accordance with approved method statements. In order to achieve a load rating for the half-joints (< 40T), material testing and concrete breakout is essential to confirm larger diameter bars (ideally 19mm > 12.7mm) than anticipated and a higher tensile strength of reinforcement (ideally ~460N/mm2 > 250 N/mm2) than anticipated. Any investigations impose a risk of allowing for a route for water/ atmospheric conditions to deteriorate the post-tensioning and half-joints which are critical elements.

The half-joints have been found to be inadequate for dead loads at ULS and SLS, however the half-joint elements are not regarded to be in poor condition and the cracks emanating from the re-entrant corners are do not appear to have increased in width since the previous inspection. It is recommended that investigative works are carried out to ascertain the true construction details and material strengths. In the interim, the structure is considered to be sub-standard as a result of this assessment, a CS470 should be carried out to confirm this until further assessment is undertaken to consider the results of material testing. It is recommended that the structure is monitored (visual inspection and non-destructive testing).

As the assessment finds the half-joints inadequate for dead loading, the structure may be considered an immediate risk under CS 470.

However, as the findings of the half-joint inspection conflict with the available design, assessment and construction record information, this suggests that the available information may not be wholly reliable and therefore some details and material properties used in the assessment may not accurately represent the as-built structure. As far as could be seen at the Inspection for Assessment, there is also a lack of ongoing deterioration to the half-joints which are regularly trafficked, presumably to full assessment live loading as certified by the previous assessment (1995).

A CS470 review should therefore be carried out to ascertain whether the structure is of immediate risk or otherwise. The review should consider whether the structure is monitoring-appropriate and, if so, make recommendation for a proposed regime of monitoring interim measures for agreement with the TAA.

4.2 Recommendations

It is recommended that:

1. The structure is managed under CS 470 as 'sub-standard' with an associated monitoring regime established for the half-joints (visual inspection and non-destructive testing)

2. Investigative works are carried out to ascertain the true construction details and material strengths.

3. Consideration be given to establishing the details and condition of the post-tensioning system through PTSI Site Investigation.

4. A reassessment of the half-joints is carried out using the parameters obtained by the above investigations.

The necessary maintenance/upgrade works to prevent further deterioration and to prolong the usable life of the bridge are listed below:

Element	Defect	Recommendation	Cost	Priority
Carriageway	Poor condition of surfacing, cracking etc.	Resurface carriageway.	£40k	High
Verge(s)	Poor condition of surfacing, cracking, light vegetation etc.	Resurface both verges.	£30k	High
Expansion Joints	Expansion joints in poor condition, surfaced over / poor installation.	Replace expansion joints. Type 1 (buried) over east half-joint. Type 2 (asphaltic plug joint) over west half-joint.	£20k	High

Note: Priority Classifications are as follows:

High: Work should be completed within 1-2 years of this report being issued to ensure safety of the public or safeguard structural integrity or avoid a high cost penalty.

Medium: Work should be completed within 3-5 years of this report being issued to ensure safety of the public or safeguard structural integrity or avoid a high cost penalty.

Low: Work should be completed within 5+ years of this report being issued to ensure safety of the public or safeguard structural integrity or avoid a high cost penalty.

Appendix A. Assessment Calculations

REF	CALCULATION	OUTPUT

INTRODUCTION

- These calculations are for Underbarrow Bridge, owned by Cumbria County Council.
- The structure has been assessed in accordance with the AiP, BCU00015-JAC-SBR-6330-RP-SL221-CB-008 P02, agreed and signed 12 January 2023.
- The assessment is limited to the half joints only, considering the upper and lower nibs as corbels.
- The assessment will be level 1, CS454 Table 2.20.1 i.e. Simple structural analysis methods, conservative assumptions for material properties + supplementary values derived from testing material samples where possible.
- It is considered that, globally, there will be minimal transfer of load to the half-joints from a parapet impact event. Therefore, for the purpose of this assessment of the half-joints, parapet impact shall not be considered.
- Deck impact loading will not be considered as part of this assessment of the half-joints. Transverse horizontal or uplift forces from deck impact are not considered to be detrimental to the performance of the half-joints in the longitudinal direction.
- The bridge deck shall be analysed using a 2-D computer grillage model (such as MIDAS) assuming original design deck articulation.
- The internal beams shall be modelled with torsionless properties. The edge beams (box beams) shall retain their properties relevant to torsion.
- For global effects, the derived limiting vertical live loads combined with local effects shall then be used to assess deck elements in accordance with CS 455 and other relevant standards as appropriate.
- The upper & lower nibs be assessed using the most onerous load effects from the global analysis and combined with local effects (under wheel or axle loads) as appropriate. Idealised “strut and tie models” as recommended in CS 466 shall be used for assessment of half-joints at SLS and ULS taking account of proposed condition factor outlined above.

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REF	CALCULATION	OUTPUT

Structure Description

Underbarrow, constructed in 1970, carries the C5048 single carriageway Underbarrow Road east and west over the A591, Kendal Bypass County Road, west of Kendal. The carriageway over the structure is approximately 6.2m wide with hardened verges measuring 1.1m and 2.4m side north and south respectively.

The superstructure is a single span made up of in-situ concrete cantilevers and a precast concrete beam suspended span. The cantilevers are of post-tensioned voided construction, integral with voided abutments. The suspended span comprises 17No. prestressed pre-tensioned concrete beams and an in-situ reinforced concrete deck slab. The inner beams are inverted T-beams and are transversely post-tensioned. The edge beams are box beams. The suspended span is supported by half-joints at the ends of the cantilevers.

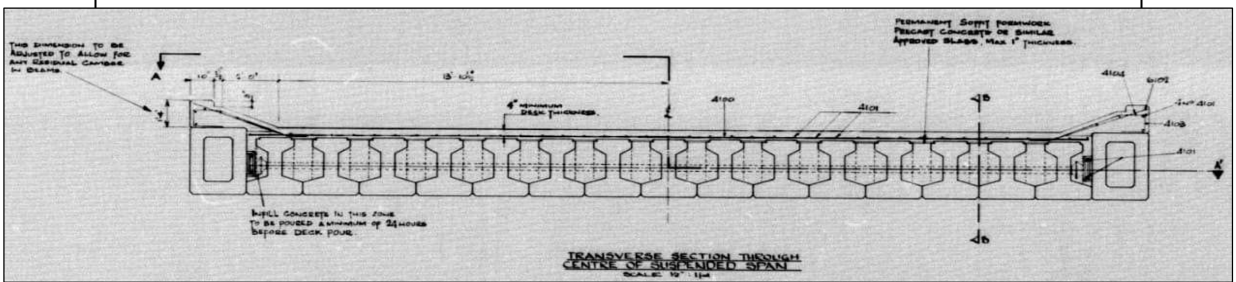
The A591 below is a dual carriageway with a grassed central reserve and grassed verges. There are “limestone pitching” revetments in front/above both abutments.

The half joint form is described as ‘solid or box slab with no access to the bearing shelf’ and is classified as ‘Type A’ in accordance with CS 466 (Figure C.3 and Table C.10).

The suspended square span is 18.288m (60’ 0”) between centrelines of bearings.

The length of each element are as follows:
West Abutment / Cantilever = 18.1mback of abutment to centreline of half-joint.
Suspended Span =18.3mbetween centrelines of half-joints.
East Abutment / Cantilever = 18.2mback of abutment to centreline of half-joint.

Historical drawings marked ‘record drawing’ detail 17.No elastomeric Dunlop Metalastik bearings. Record drawings detail the following for the same type of bearings; 285.75mm x 146mm x 78.13mm thick. The bearings are presumably centred under each of the 17 No. precast beams. Fixity is provided at the east half-joint by 14 No. horizontal bars at 609mm centres between internal beams.

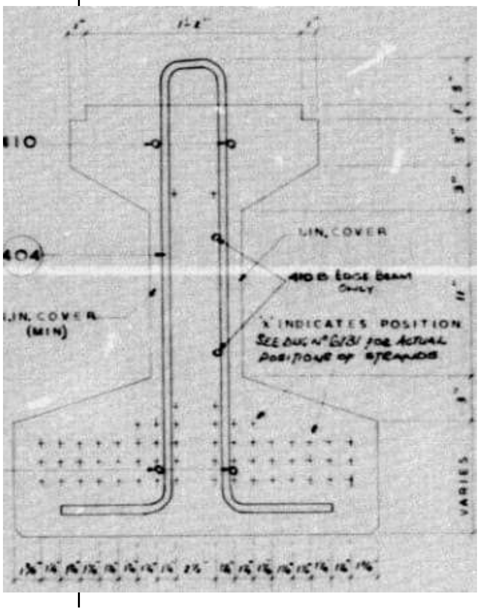


Section through centre of suspended span

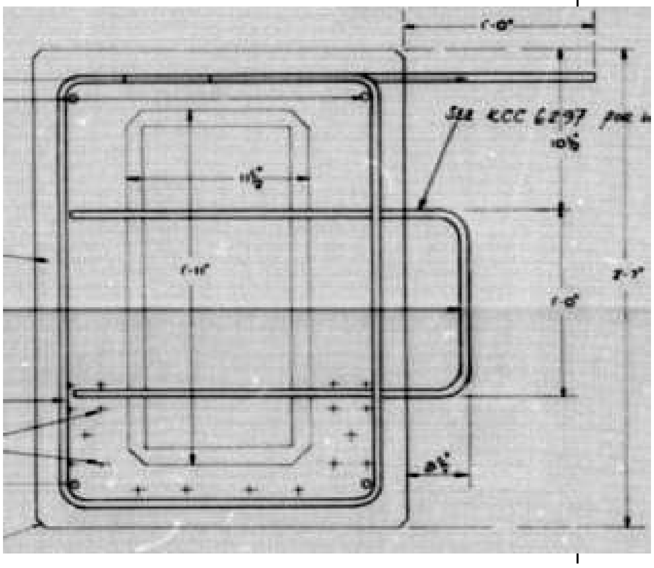
South Elevation on Structure

JACOBS					CALCULATION SHEET
OFFICE	[redacted] Structures Team	PAGE No.	CHK 3	CONT'N PAGE No.	CHK 4
JOB No. & TITLE	BCU00015 Brigsteer & Underbarrow	ORIGINATOR	[redacted]	DATE	26/02/2023
SECTION	Introduction Structure description	CHECKER		DATE	

REF	CALCULATION	OUTPUT



Profile of internal beam used in Model



Profile of external box beam used in Model

REF | CALCULATION | OUTPUT

Partial Factors

Partial Factors on Actions (CS 454)

CS 454 Tab A.1

Permanent Actions		Combination (ULS) All combinations (1-4)	γ_{f3}	Revd. Factor	Combination (SLS) All combinations (1-4)	γ_{f3}	Revd. Factor
Concrete (Mass /Reinforced)	γ_G	1.15	1.1	1.27	1.0	1.0	1.00
Surfacing Superimposed Dead Load		1.75		1.93	1.2		1.20

CS 454 Tab A.1

Variable Actions		Combinations (ULS) 1	2	3	4	γ_{f3}	Revd. Combinations (ULS) 1	2	3	4
Actions for normal / restricted traffic	γ_Q	1.50	1.25	1.25	1.25	1.1	1.65	1.38	1.38	1.38
Footway and cycle track loading		1.50	1.25	1.25	0.00		1.65	1.38	1.38	0.00
Longitudinal load (normal)		0.00	0.00	0.00	1.25		0.00	0.00	0.00	1.38
Actions for HB / assoc. normal traffic		1.30	1.10	1.10	1.10		1.43	1.21	1.21	1.21
Longitudinal load (HB model)		0.00	0.00	0.00	1.10		0.00	0.00	0.00	1.21

CS 454 Tab A.1

Variable Actions		Combinations (SLS) 1	2	3	4	γ_{f3}
Actions for normal / restricted traffic	γ_Q	1.20	1.00	1.00	1.00	1.0
Footway and cycle track loading		1.00	1.00	1.00	0.00	
Longitudinal load (normal)		0.00	0.00	0.00	1.00	
Actions for HB / assoc. normal traffic		1.10	1.00	1.00	1.00	
Longitudinal load (HB model)		0.00	0.00	0.00	1.00	

Partial Factors (ULS) on Materials (CS 455)

	Application	For use with Characteristic strength	For use with worst credible strength
γ_{ms}	Reinforcement and prestressing tendons	1.15	1.10
	Concrete	1.5	1.20
γ_{mv}	Shear in Concrete	1.25	1.15

Note: the higher factor used for worst credible strength due to the uncertainty regarding the 'record' drawings.

Partial Factors (SLS) on Materials (CS 455)

	Application	For use with Characteristic strength	For use with worst credible strength
γ_{mc}	Compression due to bending in the concrete	1	1.00
	Compression due to axial loads in concrete	1.33	1.20
γ_{mv}	Tension and Compression in Reinforcement	1	1.00

Condition Factor

AiP 3.10 — Condition Factor = 0.9

REF | CALCULATION | OUTPUT

Material Properties

Unit Weights

Tab 4.1.1a

Material	Unit Weights (kg/m^3)	Unit Weights (kN/m^3)
Reinforced Concrete	2400	24
Mass concrete / Fill concrete	2300	23
Bituminous Macadam	2560	25.6

Where kg to kN = kg x 0.00981

Durability - materials and finishes / material strengths and basis of assumptions

AiP 3.10

Material	Grade	Characteristic Tensile Strength (N/mm2)	Characteristic Compressive Strength (N/mm2)
Reinforced Concrete (HJ nib)	X 3/8	-	51.7
Reinforced Concrete (Cantilever)	Y 3/4	-	41.4
Mild Steel Reinforcement	Unknown	250	-

Note: There is no suggestion that the mild steel reinforcement has ever been tested, nor has the grade/ strength been confirmed on available 'record' drawings. The Characteristic strength is taken in accordance with BS4449:1969.

REF	CALCULATION	OUTPUT

CS 454 Cl5.17+

Variable Loads

For the purposes of applying the combined uniform and knife-edge loading, the carriageway width shall be divided into a number of notional lanes, nn , using Equation 5.18.

nn = nm

but not less than nmin
and not greater than nmax

where:
nn is the number of notional lanes
nm is the number of marked lanes
nmin is the minimum number of notional lanes taken from Table 5.18
nmax is the maximum number of notional lanes taken from Table 5.18

Carriageway width between kerb faces = 6.1 m

nmin = 2.0 lanes
nmax = 2.0 lanes

Lane width = 3.05 m

Loaded length = 18.3 m

UDL = 230 / $L^{0.67}$ = 230 / 7.012 = 32.80 kN/m

KEL = 82 kN

Conservatively apply reduction factor, K, for surface category and traffic flow (high traffic, poor surface): 0.9

Lane Factors

Lane 1 = 1.0
Lane 2 = 1.0

Revised Loading

For UDL: $\frac{32.80 \times 0.9}{3.05}$ = 9.679 kN/m/m width

For KEL: $\frac{82 \times 0.9}{3.05}$ = 24.197 kN/m

Footway Loading

The pedestrian model shall comprise a uniformly distributed load as defined in Table 5.32a, as modified by the pedestrian live load factor and width factor in Table 5.32b.

Loaded length = 18.3 m

Min footway width = 1.63 m

Pedestrian Live Load, P = 5 kN/m^2

Live load factor = 1

Width factor = 1

For UDL = 5 kN/m2

Note, the variable loads shall be applied in the Midas software using the built-in tool for variable loading to CS 454. The above has been carried out for model check purposes.

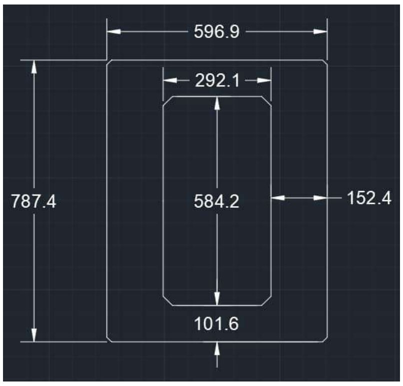
REF	CALCULATION	OUTPUT

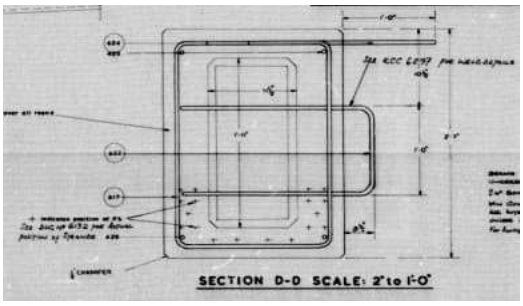
Static Load

Unit Weights

Material	Unit Weights (kg/m^3)	Unit Weights (kN/m^3)
Reinforced Concrete	2400	24
Mass concrete / Fill concrete	2300	23
Bituminous Macadam	2560	25.6

Dead Load of external box beam





Area of internal Box = 169354.5 mm2 constant
0.1694 m2

Area of external box @ midspan = 300322 mm2
0.300322 m2

includes removal of internal box

Section reproduced using Historical Drgs 'Section D-D' for external beams.

Consider length of 18.3m

Total volume = 15.6 m2 x 0.597 = 9.34 m3

volume of void = 6.86 x 0.171 = 1.171 m3 + 1.117 x 0.171 = 2.72 m3

Total volume = 9.34 - 2.72 = 6.61 m3

Total weight per external beam = 6.61 x 24 = 158.74 kN
Total weight per m length = 158.74 / 18.3 = 8.67 kN/m

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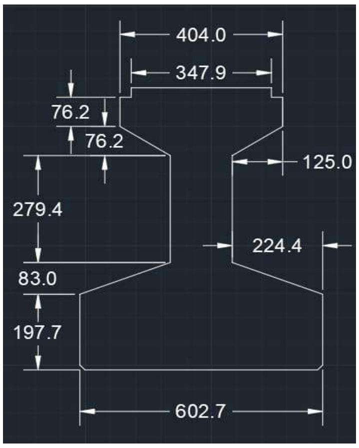
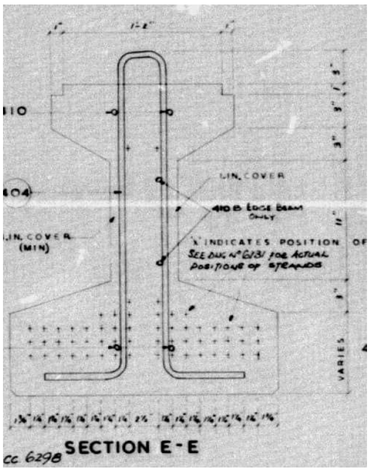
REF	CALCULATION	OUTPUT

Dead load of Internal T Beams





Area of voids considering rectangle of 602.7mm width   190098 mm
0.1901 mm

Section reproduced using Historical Drgs 'Section E-E' for Internal beams.

area of rectangle =   0.429425 m2   0.5726 m2   **0.501** m2   (assumes beam is complete rectangle)

area of beam at mid-span =   0.2442 m2   area immediate to HJ =   0.3873 m2

Average area of beam=   0.3157 m2
3E-07 mm2

Area of void =   0.1853 m2   (Area of beam as complete rectangle - area of actual beam at mid-span)

Volume of void over 18.3m length =   3.39 m3

Area of elevation   =   14.98 m2   (entire suspended span)

Volume of internal beams (concrete) =   5.78 m3

Density = $\frac{134.12 + 3.5753}{18.30}$ = 7.52 kN/m

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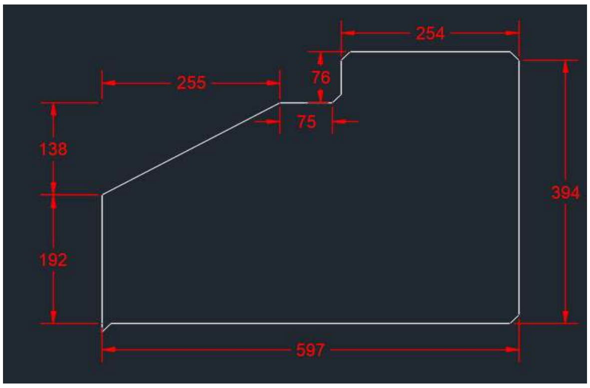
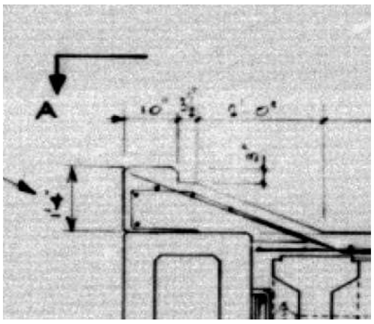
REF	CALCULATION	OUTPUT

Deck Slab Self Weight

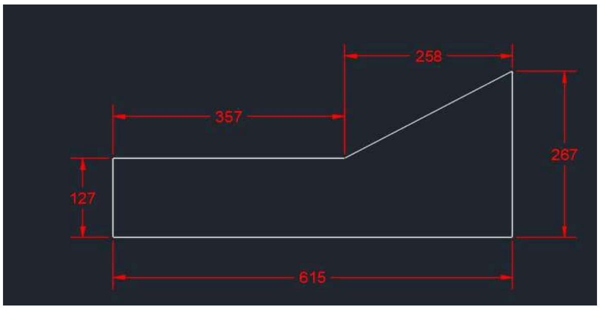
External Beams



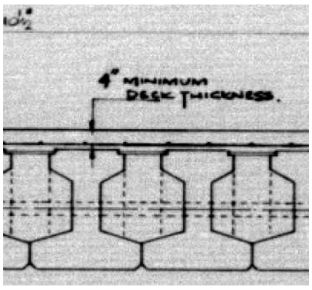
Area =	0.199	m2
Volume per m =	0.199	m3
concrete density =	23	kN/m3
Load to be applied in model =	4.6	kN



Internal beams LHS/RHS

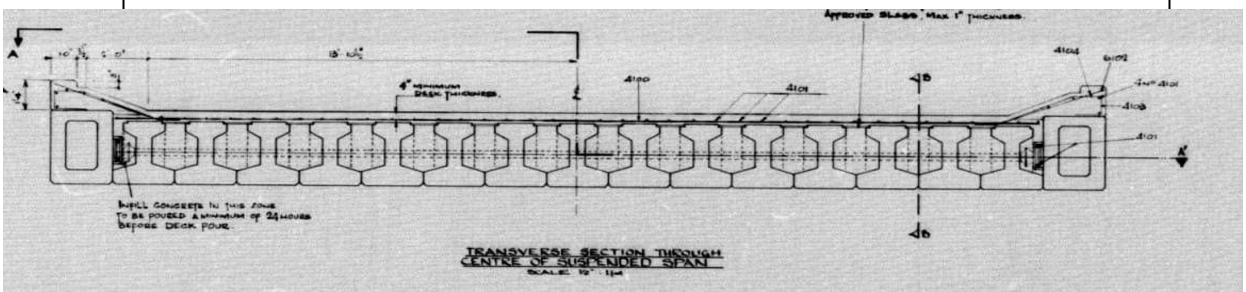


Area =	0.096	m2
Volume per m =	0.096	m3
concrete density =	23	kN/m3
Load to be applied in model =	2.2	kN



Internal beams

Area =	0.078	m2
Volume per m =	0.078	m3
concrete density =	23	kN/m3
Load to be applied in model =	1.8	kN



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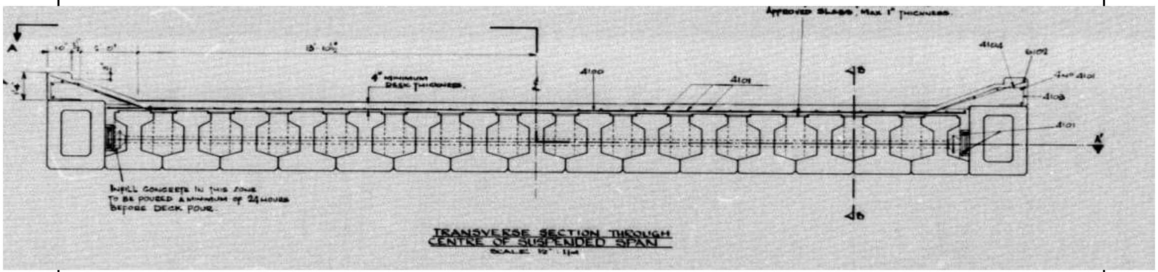
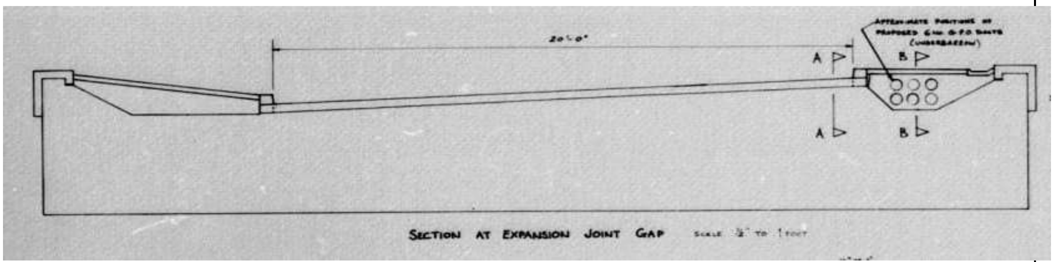
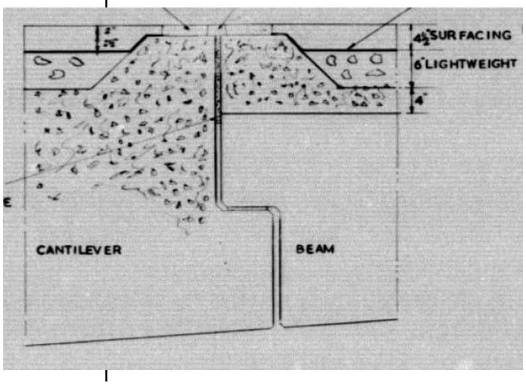
REF	CALCULATION	OUTPUT

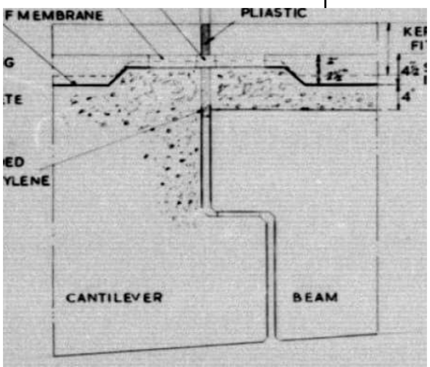
Consider the average profiles for Verge / carriageway densities



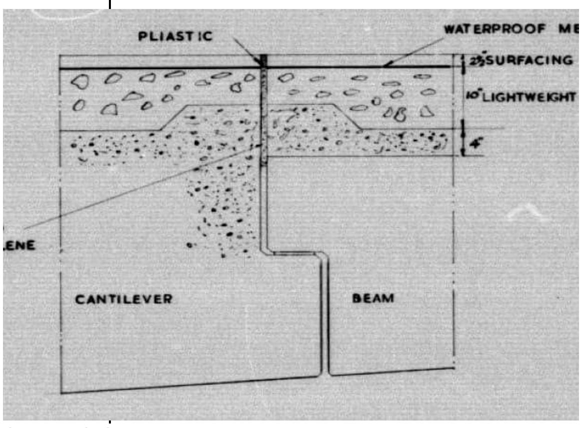




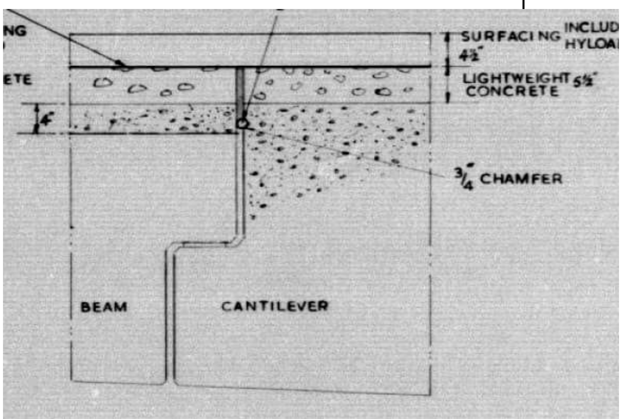
Section A-A



Section B-B



Section C-C



Section D-D

REF	CALCULATION	OUTPUT

Consider the average carriageway profile at Sections A / D.

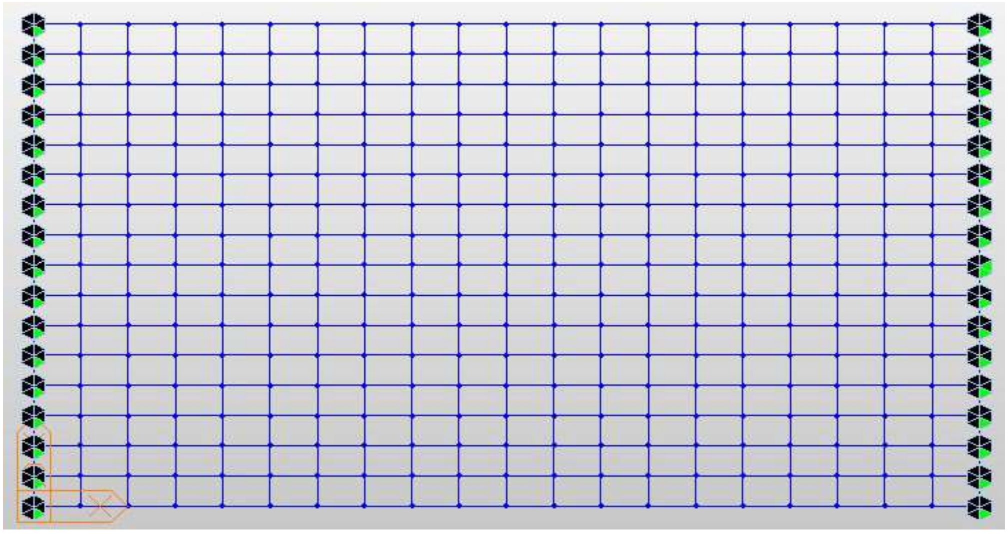
Surfacing thickness =	4	inch	=	101.6	mm		=	1.6	kN
Lightweight Concrete =	6	inch	=	152.4	mm	(conservative)	=	2.1	kN

Consider section C for all beams with verge profile above.

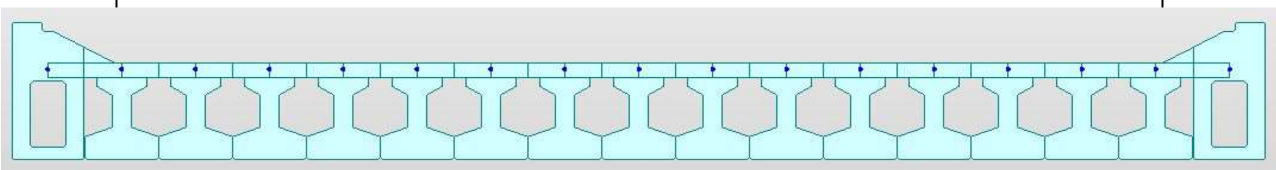
Surfacing thickness =	2.5	inch	=	63.5	mm	=	1.0	kN
Lightweight Concrete =	10	inch	=	254	mm	=	3.6	kN
Lightweight Concrete =	8	inch	=	203.2	mm	=	2.9	kN

REF	CALCULATION	OUTPUT

Model Details



Plan view on Grillage showing support conditions.



Section through grillage showing longitudinal and transverse members

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Analysis results

Considering Axial Loads only for all Combination 1 scenarios (Dead load only).

Node	Load	FZ (kN)
1	SLS dead load	156.7
2	SLS dead load	170.9
3	SLS dead load	108.6
4	SLS dead load	105.3
5	SLS dead load	108.3
6	SLS dead load	108.9
7	SLS dead load	108.9
8	SLS dead load	109.2
9	SLS dead load	111.1
10	SLS dead load	111.3
11	SLS dead load	114.2
12	SLS dead load	114.5
13	SLS dead load	116.3
14	SLS dead load	116.4
15	SLS dead load	118.1
16	SLS dead load	117.9
17	SLS dead load	120.7
18	SLS dead load	121.1
19	SLS dead load	122.0
20	SLS dead load	123.3
21	SLS dead load	122.8
22	SLS dead load	123.3
23	SLS dead load	123.7
24	SLS dead load	124.0
25	SLS dead load	123.4
26	SLS dead load	124.4
27	SLS dead load	124.9
28	SLS dead load	121.3
29	SLS dead load	177.2
30	SLS dead load	194.9
31	SLS dead load	138.8
32	SLS dead load	145.3
71	SLS dead load	125.5
72	SLS dead load	128.7

SUMMATION OF REACTION FORCES

Load	FZ (kN)
SLS dead load	4029.7
Max. Internal	194.9
Max. External	145.3

Node	Load	FZ (kN)
1	ULS dead load C1-4	205.8
2	ULS dead load C1-4	225.5
3	ULS dead load C1-4	142.1
4	ULS dead load C1-4	137.5
5	ULS dead load C1-4	142.0
6	ULS dead load C1-4	142.8
7	ULS dead load C1-4	143.5
8	ULS dead load C1-4	143.9
9	ULS dead load C1-4	146.6
10	ULS dead load C1-4	147.0
11	ULS dead load C1-4	151.0
12	ULS dead load C1-4	151.4
13	ULS dead load C1-4	153.8
14	ULS dead load C1-4	154.0
15	ULS dead load C1-4	156.2
16	ULS dead load C1-4	156.0
17	ULS dead load C1-4	159.6
18	ULS dead load C1-4	160.2
19	ULS dead load C1-4	161.3
20	ULS dead load C1-4	163.1
21	ULS dead load C1-4	162.2
22	ULS dead load C1-4	163.0
23	ULS dead load C1-4	163.3
24	ULS dead load C1-4	163.7
25	ULS dead load C1-4	162.6
26	ULS dead load C1-4	163.9
27	ULS dead load C1-4	163.9
28	ULS dead load C1-4	159.0
29	ULS dead load C1-4	233.3
30	ULS dead load C1-4	257.6
31	ULS dead load C1-4	177.6
32	ULS dead load C1-4	186.1
71	ULS dead load C1-4	159.1
72	ULS dead load C1-4	163.3

SUMMATION OF REACTION FORCES

Load	FZ (kN)
ULS dead load C1-4	4812.2
Max. Internal	257.6
Max.External	186.1

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SECTION	Introduction CS466 Strut & Tie Models	CHECKER		DATE	

REF	CALCULATION	OUTPUT

The following idealised strut and tie models shall be used as outlined in the Approval In Principle (taken from CS 466):

Figure E.3 Illustrative example of strut-and-tie model for a half-joint with long nib reinforcement

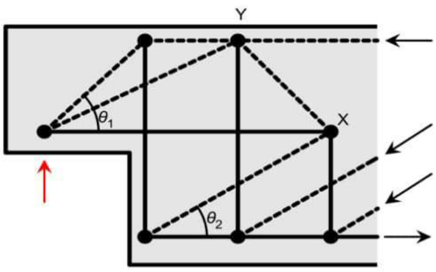

Figure E.3 of CS 466

Figure E.15 Illustrative example of a strut-and-tie model for a system with diagonal bars

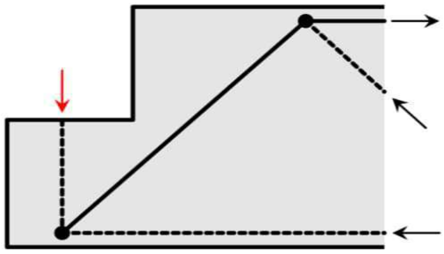

Figure E.15 of CS 466

Figure E.16 Illustrative example of a strut-and-tie model for a system with vertical bars

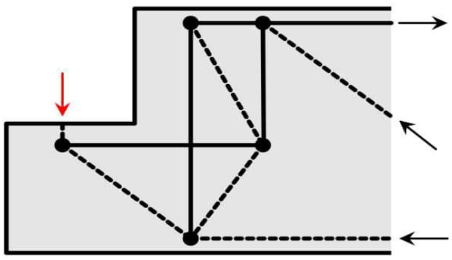

Figure E.16 of CS 466

Figure E.9 Loads applied through discrete bearings - side view

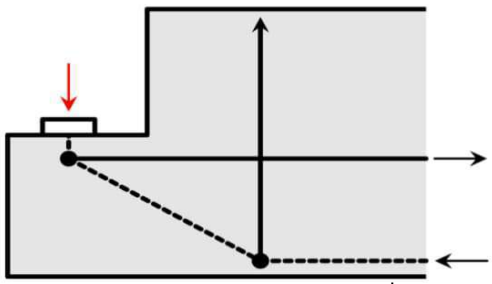

Figure E.9 of CS 466

Figure E.10 Loads applied through discrete bearings - end view

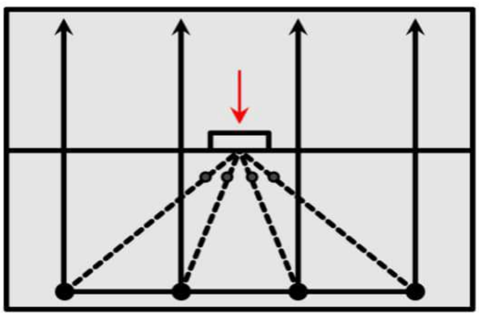

Figure E.10 of CS 466

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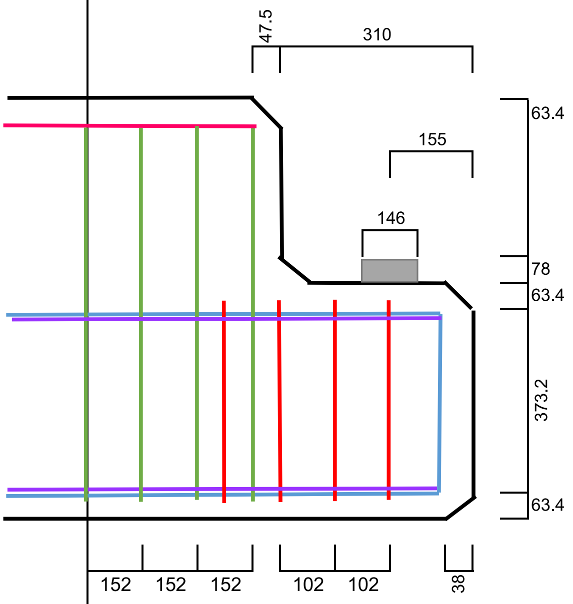
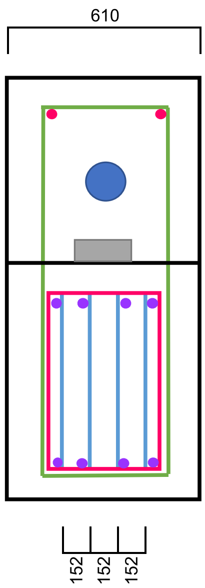
SECTION	CS466 Strut & Tie Models Internal Beams - Lower Nib - Reinforcement Layout	CHECKER		DATE	

REF	CALCULATION	OUTPUT



Long Section on Internal Beam



Cross Section on Internal Beam

Note: The position of reinforcement has been obtained from historical drawings and schedules. It has been assumed that all cover to the outer reinforcement is 38.1mm.

Legend

Reference	Diameter (mm)	Bar Mark (Historical Drg)	Tensile Strength (N/mm2)
Blue	12.7		
RED	19.05		
PURPLE	12.7		250
GREEN	19.05		
PINK	12.7		

conservative

Steel Properties

Mild Steel Reinforcement strength, Fyv = 250 N/mm2
Partial factor for steel, yms = 1.15

Concrete Properties - Lower Nib

Concrete Strength, fcu = 41.4 N/mm2
Partial factor for concrete material, ymc = 1.5
Partial factor for shear in concrete, ymv = 1.25

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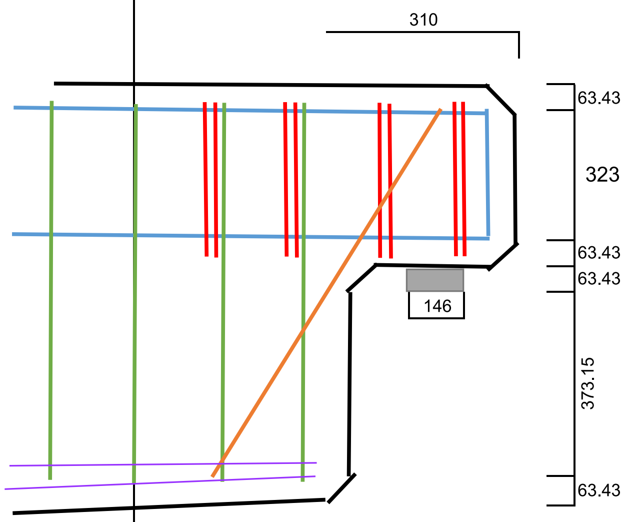
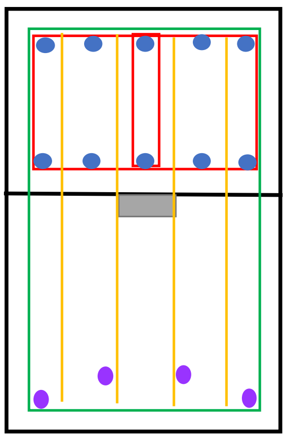
SECTION	CS466 Strut & Tie Models Internal Beams - Upper Nib - Reinforcement Layout	CHECKER		DATE	

REF	CALCULATION	OUTPUT



Long Section on Internal Beam



Cross Section on Internal Beam

Note: The position of reinforcement has been obtained from historical drawings and schedules. It has been assumed that all cover to the outer reinforcement is 38.1mm.

Legend

Reference	Diameter (mm)	Bar Mark (Historical Drg)	Tensile Strength (N/mm2)
Blue	19.05		250
RED	15.9		
PURPLE	19.05		
GREEN	15.9		
Orange	19.05		

Steel Properties

Mild Steel Reinforcement strength, Fyv = 250 N/mm2
Partial factor for steel, yms = 1.15

Concrete Properties - Upper Nib

Concrete Strength, fcu = 51.7 N/mm2
Partial factor for concrete material, ymc = 1.5
Partial factor for shear in concrete, ymv = 1.25

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SECTION	CS466 Strut & Tie Models Internal Beams - Upper Nib - Bearing Stress	CHECKER		DATE	

REF	CALCULATION	OUTPUT
CS455	**Bearing Stress**	
Cl 10.6	Where there are no measures to prevent splitting or spalling of the concrete, such as the provision of well-defined bearing areas or additional binding reinforcement in the ends of the members, the assessment bearing stress in the concrete contact area shall not exceed 0.6fcu/γmc .	*conservative* 18.6 N/mm2 incl. c-factor
Cl 10.7	Where measures have been provided to prevent splitting or spalling of the concrete, such as the provision of well-defined bearing areas or additional binding reinforcement in the ends of the members, the assessment bearing stress in the concrete contact area shall not exceed either of the following:	
	1) The value given in equation 10.7a = 48.7 N/mm2	incl. c-factor
	2) 1.5fcu / γmc = 46.53 N/mm2	incl. c-factor

Equation 10.7a

$$fbc = \frac{3\,(fcu/\gamma mc)}{1 + 2\sqrt{Acon/Asup}} = 54.1 \text{ N/mm2}$$

Where:

Acon is the contact area = 41756 mm2

Asup is the supporting area taken from equation 10.7b = 201375 mm2

Equation 10.7b

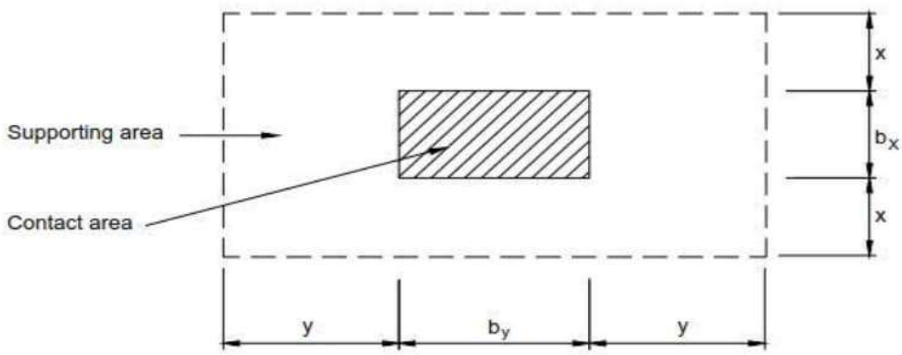
$$Asup = \left(bx + 2x\right)\left(by + 2y\right) = 201375 \text{ mm2}$$

Where:

bx,by are the dimensions of the bearing in the x, y directions respectively

x,y are the dimensions from the boundary of the contact area to the boundary of the support area, as illustrated in Figure 10.7 but limited as below

Figure 10.7 Bearing area for rectangular bearings



bx = 146.0 mm   x = 152.0 mm

by = 286.0 mm   y = 80.8 mm

Compressive stress

Maximum Reaction from model = 257.6 kN

Max. compressive stress = 257637 / 41756 = 6.2 N/mm2

6.2 N/mm2 < 18.6 N/mm2 **OK**

REF	CALCULATION	OUTPUT

CS455 — **Bearing Stress**

Cl 10.6 — Where there are no measures to prevent splitting or spalling of the concrete, such as the provision of well-defined bearing areas or additional binding reinforcement in the ends of the members, the assessment bearing stress in the concrete contact area shall not exceed 0.6fcu/γmc .

Output: *conservative* — 14.9 N/mm2 — incl. c-factor

Cl 10.7 — Where measures have been provided to prevent splitting or spalling of the concrete, such as the provision of well-defined bearing areas or additional binding reinforcement in the ends of the members, the assessment bearing stress in the concrete contact area shall not exceed either of the following:

1) The value given in equation 10.7a = 39.0 N/mm2 — incl. c-factor
2) 1.5fcu / γmc = 37.26 N/mm2 — incl. c-factor

Equation 10.7a

$$f_{bc} = \frac{3\,(f_{cu}/\gamma_{mc})}{1 + 2\sqrt{A_{con}/A_{sup}}} = 43.3 \text{ N/mm2}$$ not incl. condition factor

Where:

Acon is the contact area = 41756 mm2
Asup is the supporting area taken from equation 10.7b = 201375 mm2

Equation 10.7b

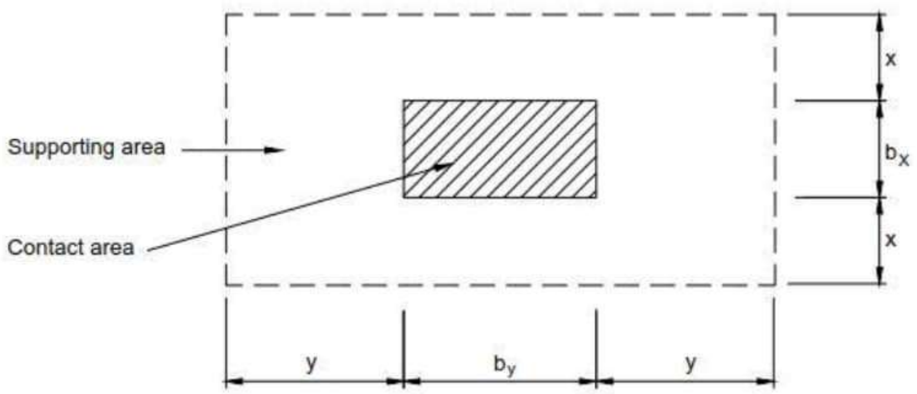
$$A_{sup} = (b_x + 2x)(b_y + 2y) = 201375 \text{ mm2}$$

Where:

bx,by are the dimensions of the bearing in the x, y directions respectively

x,y are the dimensions from the boundary of the contact area to the boundary of the support area, as illustrated in Figure 10.7 but limited as below

Figure 10.7 Bearing area for rectangular bearings



by = 286.0 mm   y = 80.8 mm

Compressive stress

Maximum Reaction from model = 257.6 kN

Max. compressive stress = 257637 / 41756 = 6.2 N/mm2

6.2 N/mm2 < 14.9 N/mm2 — **OK**

REF	CALCULATION	OUTPUT
	Ties - Maximum allowable steel tensile stress	
	σRd,Max = 195.65 N/mm2	*217.39*
	Struts - Maximum allowable concrete compressive stress	
BS EN 1992-1-1-2004 6.5.2(2)	The design strength for concrete struts should be reduced in cracked compression zones and, unless a more rigorous approach is used, may be calculated from:	
BS EN 1992-1-1-2004 (6.56)	$\sigma R_{d,max}$ = $0.6v'f_{cd}$ x F_c (consider as cracked)	
BS EN 1992-1-1-2004 (6.57N)	v' = $1-f_{ck}/250$ = 0.8344	
Drg REF	Characteristic compressive cylinder strength at 28 days (assume $f_{ck,cube} = f_{cu}$) f_{ck} = 41.4 N/mm^2	
	Design value of concrete compressive strength $a_{cc}f_{ck}/y_c$	
	= 0.85 x 41 / 1.5 f_{cd} = 23.46 N/mm^2	
	$\sigma R_{d,max}$ = $0.6v'f_{cd}$ x F_c = 10.57 N/mm2	*11.745*
	Calculate maximum stress at nodes with compression and tension	
BS EN 1992-1-1-2004 6.5.4 (4)(b)	k_2 = 0.85	
	$\sigma R_{d,max}$ (allowable) = $k_2v'f_{cd}$ = 0.85 x 0.83 x 23.46 x 0.9 (F_c) = 14.97 N/mm^2	*16.64*
	Calculate maximum stress at compression nodes only	
BS EN 1992-1-1-2004 6.5.4 (4)(a)	$\sigma R_{d,max}$ (allowable) = $k1v'f_{cd}$ = 1.00 x 0.83 x 23.46 x 0.9 (F_c) = 17.62 N/mm^2	*19.58*
	Calculate maximum stress at tension nodes only	
BS EN 1992-1-1-2004 6.5.4 (4)(c)	$\sigma R_{d,max}$ (allowable) = $k3v'f_{cd}$ = 0.75 x 0.83 x 23.46 x 0.9 (F_c) = 13.21 N/mm^2	*14.68*
	Initial Shear Check	
CS455	Consider Vmax from Cl 5.6.	
	Breadth of beam, b = 610 mm Depth to bottom horizontal reinforcement within half-joint, d0 = 436.6 mm	
	Vu 0.36 $\left(0.7 - \frac{fcu}{250}\right)\left(\frac{fcu}{ymc}\right)$ = 5.31 N/mm2	
	Vubd0 = 1414137 N	
	= 1414 kN	
	Maximum vertical ultimate load, Fv = 257.6 kN	
	257.6 kN < 1414 kN	**OK**

REF	CALCULATION	OUTPUT

Ties - Maximum allowable steel tensile stress

σRd,Max = 195.65 N/mm2 — OUTPUT: *217.39*

Struts - Maximum allowable concrete compressive stress

BS EN 1992-1-1-2004 6.5.2(2): The design strength for concrete struts should be reduced in cracked compression zones and, unless a more rigorous approach is used, may be calculated from:

BS EN 1992-1-1-2004 (6.56): $\sigma R_{d,max}$ = $0.6v'f_{cd}$ x F_c (consider as cracked)

BS EN 1992-1-1-2004 (6.57N): v' = $1-f_{ck}/250$ = 0.7932

Drg REF: Characteristic compressive cylinder strength at 28 days (assume $f_{ck,cube} = f_{cu}$) f_{ck} = 51.7 N/mm^2

Design value of concrete compressive strength $a_{cc}f_{ck}/y_c$

= 0.85 x 52 / 1.5 f_{cd} = 29.30 N/mm^2

$\sigma R_{d,max}$ = $0.6v'f_{cd}$ x F_c = 12.55 N/mm^2 — OUTPUT: *13.943*

Calculate maximum stress at nodes with compression and tension

BS EN 1992-1-1-2004 6.5.4 (4)(b): k_2 = 0.85

$\sigma R_{d,max}$ (allowable) = $k_2v'f_{cd}$ = 0.85 x 0.79 x 29.30 x 0.9 (F_c) = 17.78 N/mm^2

Calculate maximum stress at compression nodes only

BS EN 1992-1-1-2004 6.5.4 (4)(a): $\sigma R_{d,max}$ (allowable) = $k1v'f_{cd}$ = 1.00 x 0.79 x 29.30 x 0.9 (F_c) = 20.91 N/mm^2

Calculate maximum stress at tension nodes only

BS EN 1992-1-1-2004 6.5.4 (4)(c): $\sigma R_{d,max}$ (allowable) = $k3v'f_{cd}$ = 0.75 x 0.79 x 29.30 x 0.9 (F_c) = 15.69 N/mm^2

Initial Shear Check

CS455: Consider Vmax from Cl 5.6.

Breadth of beam, b = 610 mm
Depth to bottom horizontal reinforcement within half-joint, d0 = 386.58 mm

$$Vu \quad 0.36 \left(0.7 - \frac{fcu}{250}\right)\left(\frac{fcu}{ymc}\right) = 6.12 \text{ N/mm2}$$

Vubd = 1443074 N

= 1443 kN

Maximum vertical ultimate load, Fv = 257.6 kN

257.6 kN < 1443 kN — OUTPUT: **OK**

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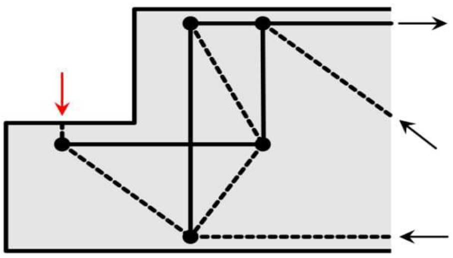
Strut and Tie Checks

The capacity of a half joint may be determined by considering the strut and tie models in Appendix E of CS 466.

Initally conside Strut and Tie model E.16.

Figure E.16 Illustrative example of a strut-and-tie model for a system with vertical bars



A similar model (although inverted) is utilised within Examples for the Design of structural concrete with Strut-and-Tie Models (Karl-Heinz Reineck).

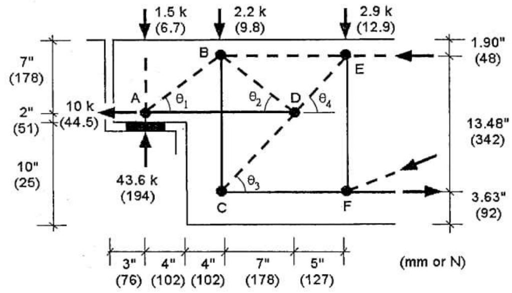


Fig. 2-4: Assumed strut-and-tie model

Considering the method used in the Karl-Heinz Reineck, the following is the approach used to select node locations.

- On the right hand side of the strut and tie model, the strut at the bottom of the section is assumed to be located in the centre of the longitudinal tension reinforcement.
- The tie at the top of the section is assumed to be level with the centre of the longitudinal reinforcement.
- Tie AD is considered to be within the centreline of the top leg of U-bar reinforcement within the lower nib
- at a distance of 38mm +19mm (link dia.) + 6.4mm (0.5 bar dia.) = 63.4mm.
- Tie BC consists of several stirrups and therefore the centroid must be placed away from the end of the beam, in accordance with the sturrup spacings, the Tie is considered to be a distance of 203mm from the
- edge of the beam (second stirrup inwards).
- Tie EF is placed at 2No stirrup spacings further, i.e. 305mm.

See overleaf for proposed strut and tie model.

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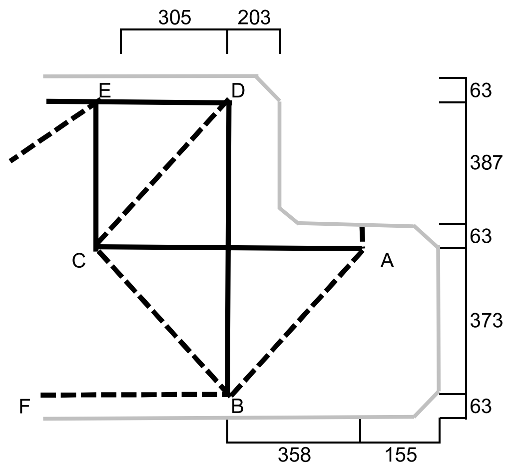
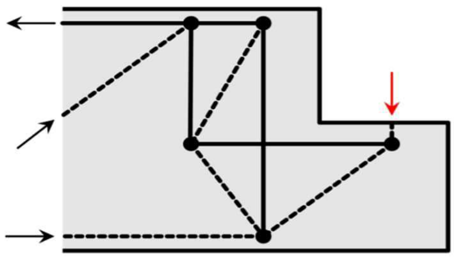
Proposed Strut and Tie Model

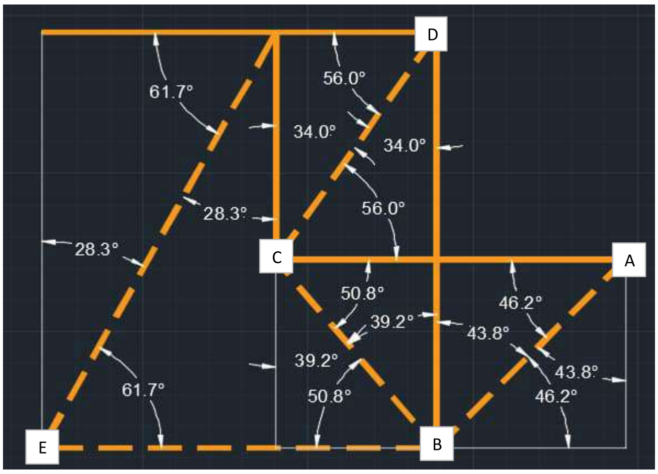


(assuming overall depth = 950mm).

Figure E.16 Illustrative example of a strut-and-tie model for a system with vertical bars



Angles in model:



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REF	CALCULATION	OUTPUT

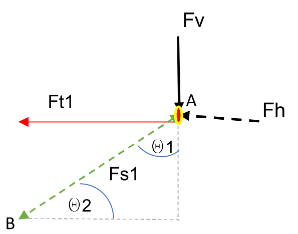
Calculate Strut & Tie Forces

Vertical force, Fv = 257.6 kN

Horizontal force, Fh = 0.0 kN

no horizontal force included due to capacity issues

Consider Node A:



Θ1 = 43.8

Θ2 = 46.2

Fs1 = Fv / CosΘ1 + Fh / sinΘ2

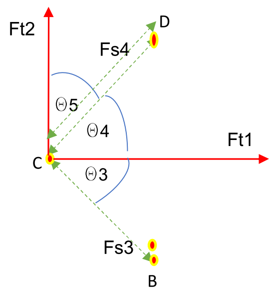
$= \frac{258}{0.72} + \frac{0.0}{0.72}$

= 356.96 + 0 = 356.96 kN — *Fs1*

Ft1 = Fs1 cosΘ2

= 356.96 x 0.69 = 247.06 kN — *Ft1*

Consider Node C:



Θ3 = 50.4

Θ4 = 56.4

Θ5 = 33.6

Ft1 = 247.06 kN = Fs3cosΘ3 + Fs4cosΘ4

∑FH = 0

Fs3 cos (50.4) + Fs4 cos (56.4) = 247.06 kN — Eq1

∑Fv = 0

Fs3 sin (50.4) = Fs4 sin (56.4) — Eq2

Rearrange Eq2 Fs3 = $Fs4 \left(\frac{\sin 56.4}{\sin 50.4}\right)$ — Eq3

Sub Eq3 into Eq 1

$Fs4 \left(\frac{\sin 56.4}{\sin 50.4}\right)$ x cos 50.4 + Fs4 cos (56.4) = 247.06 kN

247 = Fs4 1.39 — Fs4 = 178.2 kN — *Fs4*

Fs3 = $178 \left(\frac{\sin 56.4}{\sin 50.4}\right)$ — Fs3 = 192.65 kN — *Fs3*

Ft2 = Fs4 SinΘ5

= 178.2 x sin 50.4 + Fs3 Sin 50.4 — Ft2 = 285.8 kN — *Ft2*

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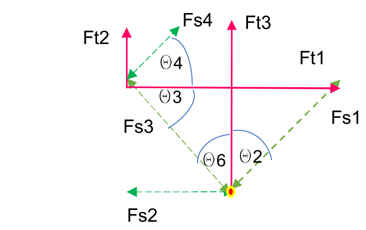
Consider Node B:



Θ2 = 43.8
Θ3 = 50.4
Θ4 = 56.4
Θ6 = 39.6

Fs1 = 356.96 kN
Fs3 = 192.65 kN
Fs4 = 178.2 kN
Ft1 = 247.06 kN

Ft3 = Fs3 cosΘ6 + Fs1 cosΘ2

= 192.65 x cos 39.6 + 356.96 x cos 43.8

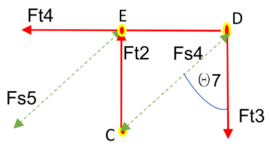
= 148.44 + 257.64 = 406.08 kN — Ft3 = 406.08 kN — *Ft3*

Fs2 = Fs2+Fs3 sinΘ6 = Fs1 sinΘ2

= Fs1 sinΘ2 - Fs3 sinΘ6

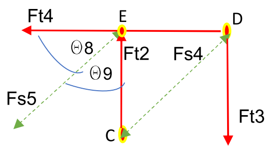
= 356.96 x sin 43.8 - 192.65 x sin 39.6 = 124.26 kN — Fs2 = 124.26 kN — *Fs2*

Consider Node D:



Θ7 = 34
Fs4 = 178.2 kN
Ft4 = Fs4 sin 34 = 99.658 kN — *Ft4i*

Consider Node E:



Θ8 = 61.7
Ft2 = 285.8 kN
Fs5 = Ft2 / sin 61.7 = 324.55 kN — *Fs5*

Θ9 = 28.3
Ft4 = Fs5 sin 28.3 = 153.87 kN + 99.7 = 253.5 kN — *Ft4*

Summary of Strut and Tie forces due to 257.6 kN applied vertically

Force Ref	Force Type	Force (kN)
Fs1	Strut	357.0
Fs2		124.3
Fs3		192.7
Fs4		178.2
Fs5		324.6

Force Ref	Force Type	Force (kN)
Ft1	Tie	247.1
Ft2		285.8
Ft3		406.1
Ft4		253.5

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REF	CALCULATION	OUTPUT

Check member F1 (Strut)

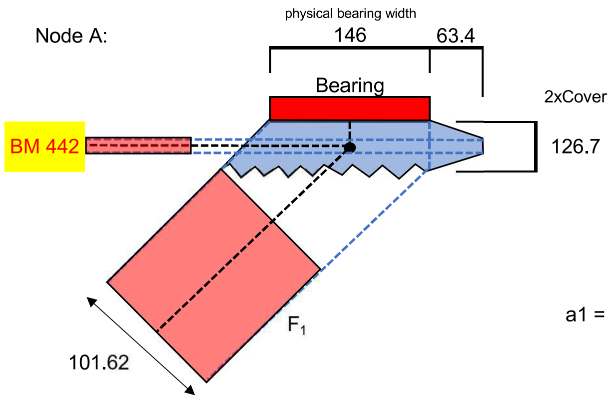
The concrete compressive stress in the strut $\sigma_{c,st}$, can be calculated from:

$$F_{n,st} = \sigma_{c,st}A_{c,st} + \sigma_{s,st}A_{s,st}$$

Where;
- $F_{n,st}$ is the bar force in the strut obtained from the static truss analysis
- $A_{c,st}$ is the effective concrete area of the strut
- $A_{s,st}$ is the area of provided compression reinforcement along the strut
- $\sigma_{s,st}$ is the compressive stress in the reinforcement at the given strut force
- $\sigma_{c,st}$ applied concrete compressive stress in the strut

$A_{c,st}$ is determined by the width of the strut, w, and the depth t of the strut. The depth t can be taken as equal to the thickness of the specimen according to EC2 unless the supports are narrower in which case the width of the strut should be taken to be equal to the width of the support for struts originating at the support.

Node A:



∴ t = 500 mm — *thickness of nib*

∴ w = 101.6 mm

$A_{c,st}$ = 50812 mm^2

$F_{n,st}$ = 356.96 kN

a1 = lb-2so = a1 = 19 mm

F1,max = 10.57 x 50812 = 537111.6 = 537.11 kN

Structures shall be deemed to be capable of carrying the assessment load when the following relationship is satisfied:

$$R_a^* \geq S_a^*$$

537.11 kN ≥ 356.96 kN — **OK**

Structure Adequate

REF	CALCULATION	OUTPUT

Check Tensile Stress in Ft1 (Tie)

Ft1 = 247.1 kN   Bar diameter = 12.7 mm   Number of bars = 4 No.

Area of bar = 126.68 mm2   Total area of rebar = 506.71 mm2

Ft1s Max = 250 x 506.71 / 1.15 x 1000 x 0.9 = 99.138 kN — *487.59*

Structures shall be deemed to be capable of carrying the assessment load when the following relationship is satisfied:

$R_a^* \geq S_a^*$ — **NOT OK**

99.138 < 247.1 kN

Structure Inadequate — 2.49

Check compressive stress in concrete strut Fs3 (Strut)

Fs3 = 192.7 kN

Fs1 strut width = 101.6 mm

Calculate strut width for Fs3 = 2 x Fs1width / 2 / tan∂2 x cosα3 = 105.69 mm   considered conservative value

where α1 = 90 - ∂2 = 46.2   tan∂2 = 0.96
α2 = ∂6 + 46.2 = 85.8   cosα3 = 0.997
α3 = 85.8 - 90 = -4.2

Calculate effective area of concrete strut
thickness of lower nib x width of strut = 500 x 105.69 = 52844 mm2

Calculate stress in concrete stru = 192.7 x 1000 / 52844 = 3.65 N/mm2 < 10.6 N/mm2

Structures shall be deemed to be capable of carrying the assessment load when the following relationship is satisfied:

$R_a^* \geq S_a^*$

10.6 > 3.65 — **OK**

Structure Adequate

Check compressive stress in concrete strut Fs2 (Strut)

Fs3 = 192.7 kN   Bar diameter = 12.7 mm   Number of bars = 4

Area of bar = 126.68 mm2   Area of reinforcement = 506.71 mm2 — *3.79*

Calculate maximum force in concrete strut

width of concrete strut = 126.7 mm limited to 8x bar diameter = 101.6 so max width = 101.6 mm

Fc,max = 10.57 x 50800 / 1.50 x 1000 = 357.99 kN

Structures shall be deemed to be capable of carrying the assessment load when the following relationship is satisfied:

$R_a^* \geq S_a^*$ — **OK**

358.0 > 192.65

Structure Adequate

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REF	CALCULATION	OUTPUT

Check tensile stress in Ft2 & FT3 (Tie)

Ft2+3 max = 691.8 kN   Bar diameter = 19.05 mm

No. legs per link = 2 No.   Number of links within disturbed zone = 6

Area per bar = 285.02 mm2   Total area of reinforcement = 3420.3 mm2   *202.28*

Maximum force in steel = 250 x 3420.3 / 1.15 x 1000 = 743.54 kN
= 669.18 kN incl. condition factor

Structures shall be deemed to be capable of carrying the assessment load when the following relationship is satisfied:

$R_a^* \geq S_a^*$
669.2 > 691.84   **NOT OK**

Structure Inadequate

Check compressive stress in concrete strut Fs4 (Strut)

Fs4 = 178.2 kN

Calculate area of concrete strut

Calculate width of concrete strut = 114.25 mm

Area of concrete strut = 114 x 500 = 57124 mm2

Stress in concrete strut = 3.12 N/mm2

Structures shall be deemed to be capable of carrying the assessment load when the following relationship is satisfied:

$R_a^* \geq S_a^*$
402.56 > 178.2   **OK**
Structure Adequate

Check tensile stress in Ft4

Ft4= 253.5 kN   Bar diameter = 12.7 mm

No. bars = 4 No.

Area per bar = 126.68 mm2   Total area of reinforcement = 506.71 mm2   *500.33*

Maximum force in steel = 250 x 506.71 / 1.15 x 1000 = 110.15 kN
= 99.138 kN incl. condition factor

Structures shall be deemed to be capable of carrying the assessment load when the following relationship is satisfied:

$R_a^* \geq S_a^*$
99.1 > 253.52   **NOT OK**

Check compressive stress in concrete strut Fs5 (Strut)

Fs5 = 324.6 kN

Calculate area of concrete strut
Calculate width of concrete strut = 602.7 mm   width of overall beam

Area of concrete strut = 603 x 152 = 91851 mm2   Stress in concrete strut = 3.53 N/mm2

Structures shall be deemed to be capable of carrying the assessment load when the following relationship is satisfied:

$R_a^* \geq S_a^*$
647.28 > 324.6   **OK**
Structure Adequate

REF	CALCULATION	OUTPUT

Summary of results

Force Ref	Force Type	Force (kN)	Capacity kN	Stress (N/mm2)	Capacity N/mm2	UF
Fs1	Strut	357.0	537.1	7.02	10.6	0.66
Fs2		124.3	358.0	3.79	10.6	0.35
Fs3		192.7	558.6	3.65	10.6	0.34
Fs4		178.2	402.6	3.12	10.6	0.44
Fs5		324.6	647.3	3.53	10.6	0.33

Force Ref	Force Type	Force (kN)	Capacity kN	Stress (N/mm2)	Capacity N/mm2	UF
Ft1	Tie	247.1	99.1	487.6	195.7	2.49
Ft2+3		691.8	669.2	202.3	195.7	1.03
Ft4		253.5	99.1	500.3	195.7	2.56

Force Ref	Force Type	Stress (N/mm2)	Capacity N/mm2	UF
A	Node	7.02	14.97	0.47
B		7.02	14.97	0.47
C		3.65	14.97	0.24
D		3.12	13.21	0.24
E		3.53	13.21	0.27

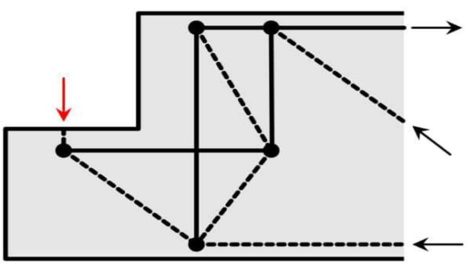
REF	CALCULATION	OUTPUT

Strut and Tie Checks

The capacity of a half joint may be determined by considering the strut and tie models in Appendix E of CS 466.

Initally conside Strut and Tie model E.16.

Figure E.16 Illustrative example of a strut-and-tie model for a system with vertical bars



A similar model (although inverted) is utilised within Examples for the Design of structural concrete with Strut-and-Tie Models (Karl-Heinz Reineck).

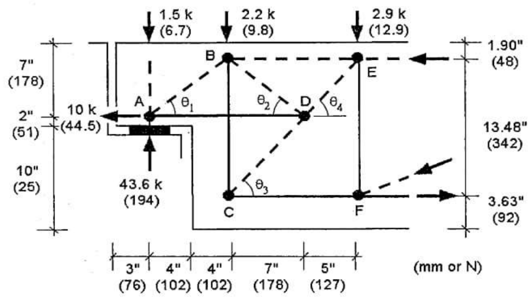


Fig. 2-4: Assumed strut-and-tie model

Considering the method used in the Karl-Heinz Reineck, the following is the approach used to select node locations.

- On the right hand side of the strut and tie model, the strut at the bottom of the section is assumed to be located in the centre of the longitudinal tension reinforcement.
- The tie at the top of the section is assumed to be level with the centre of the longitudinal reinforcement.
- Tie AD is considered to be within the centreline of the top leg of U-bar reinforcement within the lower nib at
- a distance of 38mm +19mm (link dia.) + 6.4mm (0.5 bar dia.) = 63.4mm. 63.4
- Tie BC consists of several stirrups and therefore the centroid must be placed away from the end of the beam, in accordance with the sturrup spacings, the Tie is considered to be a distance of 203mm from the
- edge of the beam (second stirrup inwards).
- Tie DE is placed at 2No stirrup spacings further, i.e. 305mm.

See overleaf for proposed strut and tie model.

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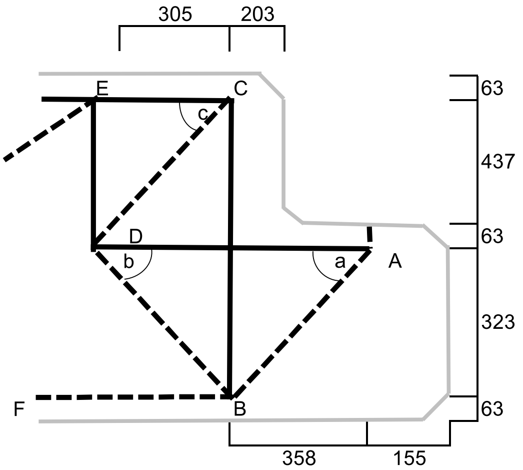
Proposed Strut and Tie Model



(assuming overall depth = 950mm).

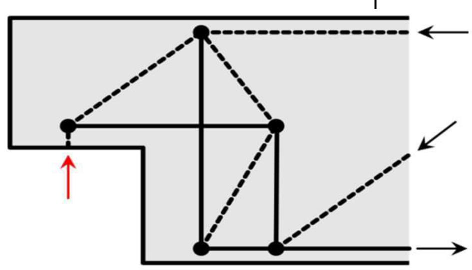
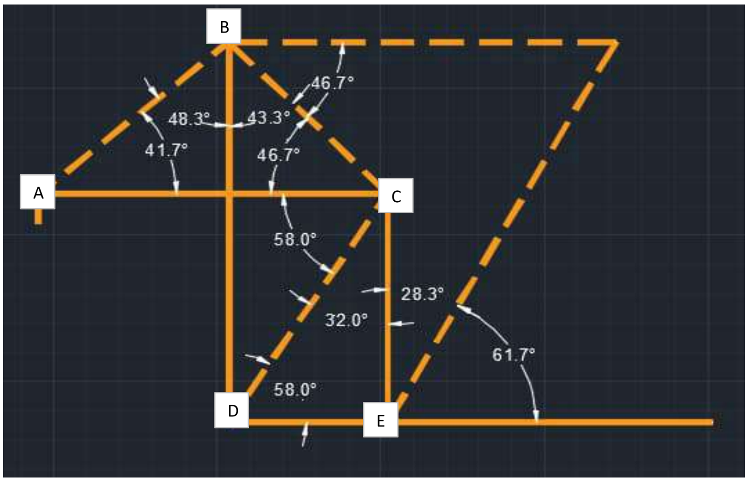


Figure E.16 Illustrative example of a strut-and-tie model for a system with vertical bars

Angles in model:



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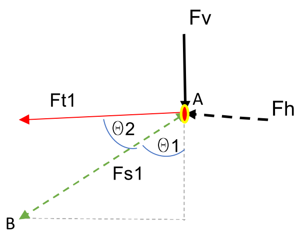
Calculate Strut & Tie Forces

Vertical force, Fv = 257.6 kN   Horizontal force, Fh = 0.0 kN

Consider Node A:



Θ1 = 48.3
Θ2 = 41.7

Fs1 = Fv / CosΘ1 + Fh / sinΘ2

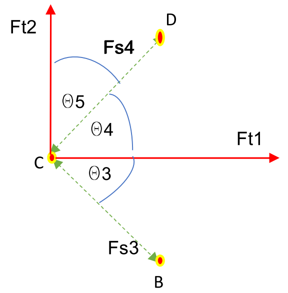
= 258 / 0.67 + 0.0 / 0.67

= 387.29 + 0 = 387.29 kN   *Fs1*

Ft1 = Fs1 cosΘ2

= 387.29 x 0.75 = 289.16 kN   *Ft1*

Consider Node C:



Θ3 = 46.7
Θ4 = 58.0
Θ5 = 32.0

Ft1 = 289.16 kN = Fs3cosΘ3 + Fs4cosΘ4

∑FH = 0

Fs3 cos (46.7) + Fs4 cos (58.0) = 289.16 kN   Eq1

∑Fv = 0

Fs3 sin (46.7) = Fs4 sin (58.0)   Eq2

Rearrange Eq2   Fs3 = Fs4 $\left(\frac{\sin 58.0}{\sin 46.7}\right)$ Eq3

Sub Eq3 into Eq 1

Fs4 $\left(\frac{\sin 58.0}{\sin 46.7}\right)$ x cos 46.7 + Fs4 cos (58.0) = 289.16 kN

289 = Fs4 1.38   Fs4 = 209.8 kN   *Fs4*

Fs3 = 210 $\left(\frac{\sin 58.0}{\sin 46.7}\right)$ Fs3 = 244.53 kN   *Fs3*

Ft2 = Fs4 SinΘ5

= 209.8 x sin 46.7 + Fs3 Sin 46.7   Ft2 = 330.7 kN   *Ft2*

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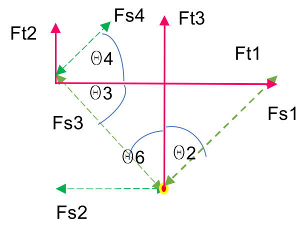
Consider Node B:



Θ2 = 48.3 Fs1 = 387.29 kN
Θ3 = 46.7 Fs3 = 244.53 kN
Θ4 = 58 Fs4 = 209.8 kN
Θ6 = 43.3 Ft1 = 289.16 kN

Ft3 = Fs3 cosΘ6 + Fs1 cosΘ2

= 244.53 x cos 43.3 + 387.29 x cos 48.3

= 177.96 + 257.64 = 435.6 kN Ft3 = 435.6 kN *Ft3*

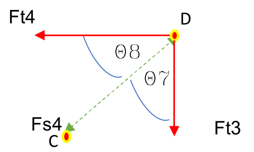
Fs2 = Fs2+Fs3 sinΘ6 = Fs1 sinΘ2

= Fs1 sinΘ2 - Fs3 sinΘ6

= 387.29 x sin 48.3 - 244.53 x sin 43.3 = 121.46 kN Fs2 = 121.46 kN *Fs2*

Consider Node D:

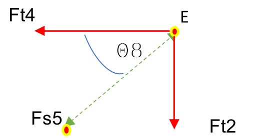
Θ7 = 32.0 Fs4 = 209.8 kN



Ft4 = Fs4 cos 58.0 = 111.2 kN

Consider Node E:

Θ8 = 58.0 Ft2 = 330.7 kN



Ft4 = Fs5 cos 58.0 = 46.5 kN

Fs5 = 330.7 / sin 58.0 = 389.94 kN

Total Ft4 = 157.7 kN

Summary of Forces due to 257.6 kN applied vertically

Force Ref	Force Type	Force (kN)
Fs1	Strut	387.3
Fs2		121.5
Fs3		244.5
Fs4		209.8
Fs5		389.9

Force Ref	Force Type	Force (kN)
Ft1	Tie	289.2
Ft2		330.7
Ft3		435.6
Ft4		157.7

REF	CALCULATION	OUTPUT

Check member F1 (Strut)

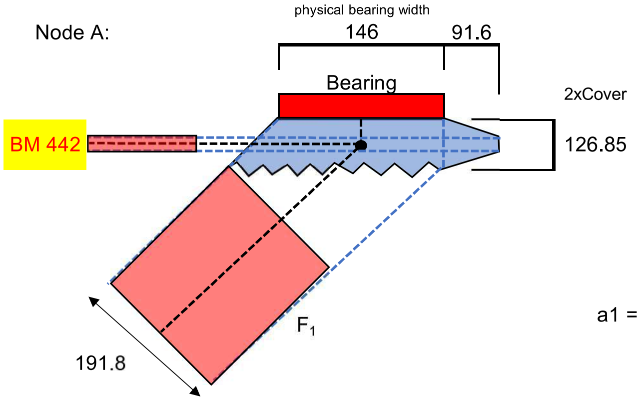
The concrete compressive stress in the strut $\sigma_{c,st}$, can be calculated from:

$$F_{n,st} = \sigma_{c,st}A_{c,st} + \sigma_{s,st}A_{s,st}$$

Where; $F_{n,st}$ is the bar force in the strut obtained from the static truss analysis
$A_{c,st}$ is the effective concrete area of the strut
$A_{s,st}$ is the area of provided compression reinforcement along the strut
$\sigma_{s,st}$ is the compressive stress in the reinforcement at the given strut force
$\sigma_{c,st}$ applied concrete compressive stress in the strut

$A_{c,st}$ is determined by the width of the strut, w, and the depth t of the strut. The depth t can be taken as equal to the thickness of the specimen according to EC2 unless the supports are narrower in which case the width of the strut should be taken to be equal to the width of the support for struts originating at the support.

Node A:



∴ t = 500 mm

∴ w = 191.8 mm

$A_{c,st}$ = 95917 mm^2

$F_{n,st}$ = 387.29 kN | 4.0

a1 = lb-u = a1 = 19 mm

F1,max = 12.549 x 95917 = 1203627 = 1203.6 kN

Structures shall be deemed to be capable of carrying the assessment load when the following relationship is satisfied:

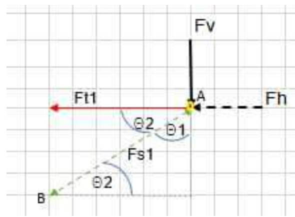
$R_a^* \geq S_a^*$

1203.6 kN ≥ 387.29 kN **OK**

Structure Adequate

REF	CALCULATION	OUTPUT

Check Tensile Stress in Ft1 (Tie)



Ft1 = 289.2 kN

Bar diameter = 19.05 mm   Number of bars = 5 No.

3No in AIP but 5 on drg? BM 607

Area of bar = 285.02 mm2

Total area of rebar = 1425.1 mm2   *202.91*

Ft1 Max = 250 x 1425.1 / 1.15 x 1000 x 0.9 = 278.83 kN

Structures shall be deemed to be capable of carrying the assessment load when the following relationship is satisfied:

$R_a^* \geq S_a^*$

278.8 > 289.2

Structure Inadequate   **NOT OK**

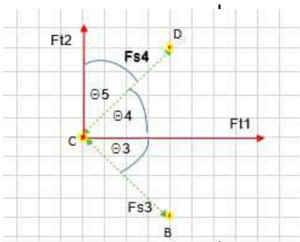
Check compressive stress in concrete strut Fs3 (Strut)

Fs3 = 244.5 kN

Fs1 strut width = 191.8 mm

Calculate strut width for Fs3 = 2 x Fs1width / 2 / tan∂2 x cosα3 = 170.27 mm   considered conservative value

where



$\alpha 1$ = 90 - ∂2 = 41.7

$\alpha 2$ = ∂6 + 41.7 = 85.0   tan∂2 = 1.12

$\alpha 3$ = 85.0 - 90 = -5.0   cos$\alpha 3$ = 0.996

Calculate effective area of concrete strut
thickness of lower nib x width of strut = 500 x 170.27 = 85134 mm2

Calculate stress in concrete strut = 244.5 x 1000 / 85134 = 2.87 N/mm2 < 12.5 N/mm2   **OK**

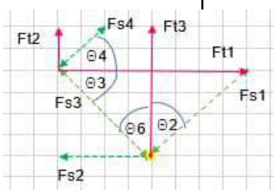
Structures shall be deemed to be capable of carrying the assessment load when the following relationship is satisfied:

$R_a^* \geq S_a^*$

12.5 > 2.9 N/mm2

Structure Adequate

Check compressive stress in concrete strut Fs2 (Strut)



Fs3 = 244.5 kN

Bar diameter = 19.1 mm

Calculate maximum force in concrete strut

width of concrete strut = 126.85 mm limited to 8x bar diameter = 101.6 so max width = 101.6 mm

Fc,max = 12.55 x 50800 / 1.50 x 1000 = 424.98 kN

Stress in concrete strut = 244.5 x 1000 / 50800 = 4.81 N/mm2

Structures shall be deemed to be capable of carrying the assessment load when the following relationship is satisfied:

$R_a^* \geq S_a^*$

12.5 > 4.8 N/mm2   **OK**

Structure Adequate

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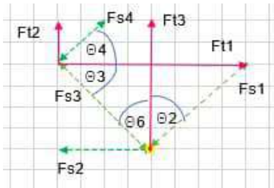
Check tensile stress in Ft2 & FT3 (Tie)



Ft2 + Ft3 766.3 kN

Bar diameter = 15.9 mm
No. legs per link = 2 No.
Number of links within disturbed zone = 6

Area per bar = 198.56 mm2
Total area of reinforcement = 2382.7 mm2 — 321.6

Maximum force in steel = 250 x 2382.7 / 1.15 x 1000 = 517.97 kN
= 466.18 kN incl. condition factor

Structures shall be deemed to be capable of carrying the assessment load when the following relationship is satisfied:

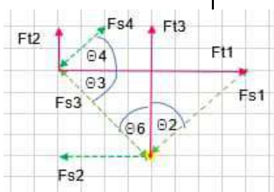
$R_a^* \geq S_a^*$

195.7 > 321.6 N/mm2 — **NOT OK**

Structure Inadequate

Check compressive stress in concrete strut Fs4 (Strut)

Fs4 = 209.8 kN



Calculate area of concrete strut

Calculate width of concrete strut = 198.41 mm

Area of concrete strut = 198 x 500 = 99204 mm2

Stress in concrete strut = 2.12 N/mm2

Capacity of concrete strut = 829.9 kN

Structures shall be deemed to be capable of carrying the assessment load when the following relationship is satisfied:

$R_a^* \geq S_a^*$

12.5 > 2.1 N/mm2 — **OK**

Structure Adequate

Check tensile stress in Ft4

Ft4= 289.2 kN Bar diameter = 19.05 mm

No. bars = 4 No.

Area per bar = 285.02 mm2 Total area of reinforcement = 1140.1 mm2 — 253.63

Maximum force in steel = 250 x 1140.1 / 1.15 x 1000 = 247.85 kN
= 223.06 kN incl. condition factor

Structures shall be deemed to be capable of carrying the assessment load when the following relationship is satisfied:

$R_a^* \geq S_a^*$

223.1 > 289.16 — **NOT OK**

Structure Inadequate

REF	CALCULATION	OUTPUT

Check compressive stress in concrete strut Fs5 (Strut)

Fs5 = 389.9 kN

Calculate area of concrete strut
Calculate width of concrete strut = 602.7 mm width of overall beam

Area of concrete strut = 603 x 152 = 91851 mm2 Stress in concrete strut = 4.25 N/mm2

Structures shall be deemed to be capable of carrying the assessment load when the following relationship is satisfied:

$R_a^* \geq S_a^*$

768.4 > 389.9

Structure Adequate

OK

Summary of results

Force Ref	Force Type	Force (kN)	Capacity kN	Stress (N/mm2)	Capacity N/mm2	UF
Fs1	Strut	387.3	1203.6	4.0	12.5	0.32
Fs2		121.5	425.0	4.81	12.5	0.38
Fs3		244.5	1068.3	2.9	12.5	0.23
Fs4		209.8	829.9	2.1	12.5	0.17
Fs5		389.9	768.4	4.2	12.5	0.34

Force Ref	Force Type	Force (kN)	Capacity kN	Stress (N/mm2)	Capacity N/mm2	UF
Ft1	Tie	289.2	278.8	202.9	195.7	1.04
Ft2/3		766.3	466.2	321.6	195.7	1.64
Ft4		289.2	223.1	253.6	195.7	1.30

Force Ref	Force Type	Stress (N/mm2)	Capacity N/mm2	UF
A	Node	4.04	17.8	0.23
B		4.81	17.8	0.27
C		2.87	17.8	0.16
D		2.12	15.7	0.13
E		4.25	15.7	0.27

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Strut and Tie Checks

The capacity of a half joint may be determined by considering the strut and tie models in Appendix E of CS 466.

Inititally conside Strut and Tie model E.3

Figure E.3 Illustrative example of strut-and-tie model for a half-joint with long nib reinforcement

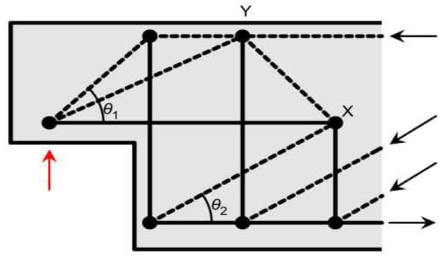
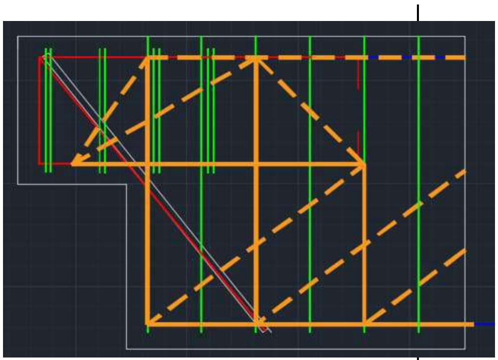
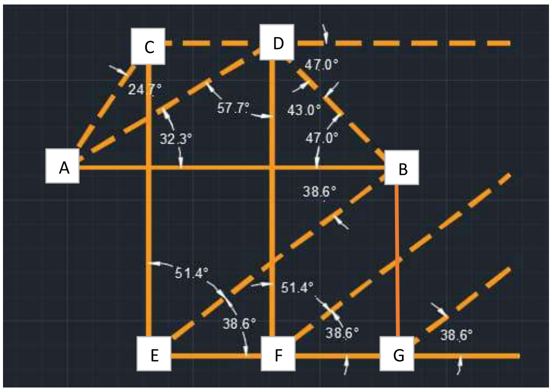


Diagram of model drawn over sketch of nib and reinforcement



Considering the method used in the Karl-Heinz Reineck, the following is the approach used to select node locations.



- The Strut and Tie at the top and bottom of the model are positioned along the centreline of the reinforcement.
- Node A is positioned directly beneath the centre line of the bearing
- The vertical ties, CE, DF and BG are in areas where numeroud stirrups (links) are present and hence these ties are spread evenly throughout the B region. i.e. at 305mm intervals.
- As shown in Figure E.3, the first vertical tie is positioned within the first stirrup.
- Node B is positioned at the bend within the horizontal tie bars which coincides with the placement of the stirrups.

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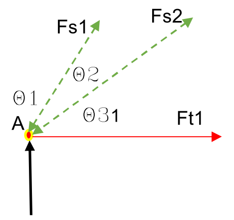
Calculate Strut & Tie Forces

Vertical force, Fv = 257.6 kN   Horizontal force, Fh = 0.0 kN

Consider Node A:



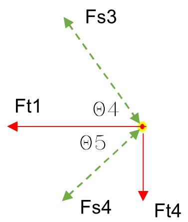
Θ1 = 33   Fv = 257.64 kN
Θ2 = 24.7
Θ3 = 32.3

Fs1 = 257.64 / cos 33 = 307.2 kN — *Fs1*

Fs2 = 257.64 / cos 58 = 482.15 kN — *Fs2*

Ft1 = 307.2 x cos 57 = 167.31 + 482.15 x cos 32.3 = 407.54 kN = 574.85 kN — *Ft1*

Consider Node B:



Θ4 = 47   Θ4α = 43
Θ5 = 38.6   Θ5α = 51.4

Ft1 = 574.85 kN = Fs3 cos 47 + Fs4 cos 38.6 — Eq1

Fs3 sin 47 + Fs4 sin 38.6 — Eq2

Fs3 = $\frac{\text{Fs4 sin } 38.6}{\text{sin } 47}$ — Eq3

Sub eq3 in to Eq1

$\frac{\text{Fs4 cos } 38.6}{\text{sin } 47}$ sin 47 + Fs4 cos 38.6 = 574.85 kN

574.85 = Fs4 1.563

Fs4 = 367.78 kN — *Fs4*

Fs3 = 313.73 kN — *Fs3*

Ft4 = 367.78 sin 51.4 = 287.43 kN — *Ft4*

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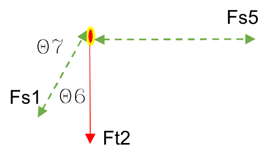
Consider Node C:

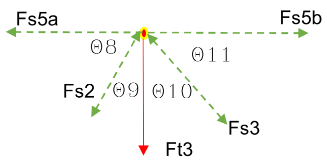


Θ6 = 33
Θ7 = 65

Ft2 = 307.2 sin 33 = 167.31 kN — *Ft2*

Fs5 = 307.2 cos 65 = 128.37 kN — *Fs5*

Consider Node D:



Θ8 =	32.3	Fs2 =	482.15 kN
Θ9 =	57.7	Fs3 =	313.73 kN
Θ10 =	43.0	Fs5a =	128.37 kN
Θ11 =	47.0		

Ft3 = 482.15 sin 57.7 + 313.73 sin 43.0 = 621.51 kN — *Ft3*

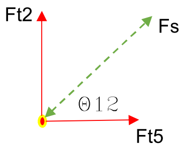
Fs5b = Fs5b + Fs3 sin 47.0 = 128.37 + 482.15 sin 32.3

Fs5b + 229.4 = 386

Fs5b = 156.6 kN

Fs5 tot = 284.92 kN — *Fs5*

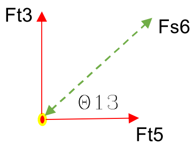
Consider Node E:



Θ12 = 38.6 Fs4 = 367.78 kN

Ft5 = 367.78 cos 38.6 = 287.43 kN

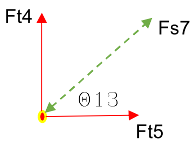
Consider Node F:



Ft5 = Ft3 - Fs6cos 38.6 = 233.76 kN

Fs6 = 621.51 / tan 51.4 = 496.14 kN — *Fs6*

Consider Node G:



Ft5 = Ft4 - Fs7cos 38.6 = 108.11 kN Ft5 total = 229.45 + 233.76 + 287.43 = 750.64 kN — *Ft5*

Fs7 = 287.43 / tan 51.4 = 229.45 kN — *Fs7*

REF	CALCULATION	OUTPUT

Summary of Forces due to 257.6 kN applied vertically

Force Ref	Force Type	Force (kN)
Fs1	Strut	307.2
Fs2		482.1
Fs3		313.7
Fs4		367.8
Fs5		284.9
Fs6		496.1
Fs7		229.4

Force Ref	Force Type	Force (kN)
Ft1	Tie	574.9
Ft2		167.3
Ft3		621.5
Ft4		287.4
Ft5		750.6

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REF	CALCULATION	OUTPUT

Check strut Fs1

Bearing width, lb = 146.00 mm

2So = 38 mm   lb-2So = 108.00   so 0.5*lb-So = 54 mm

U = 2 x cover to centreline of tensile bar = 127 mm

Fs1 strut width = 159.75 mm   Fs2 strut width = 165.06 mm

Maximum force in Ft1 = 902.06 kN   where maximum stress = 12.55 N/mm2

Fs1 = 307.2 kN   stress in Fs1 = 4.27 N/mm2

Structures shall be deemed to be capable of carrying the assessment load when the following relationship is satisfied:

$R_a^* \geq S_a^*$

12.5 > 4.27   **OK**

Structure Adequate

Check strut Fs2

Fs2 strut widith = 165.06 mm

Maximum force in Fs2 = 932.06 kN   where maximum stress = 12.55 N/mm2

Fs2 = 482.1 kN   Stress in Fs2 = 6.49 N/mm2

Structures shall be deemed to be capable of carrying the assessment load when the following relationship is satisfied:

$R_a^* \geq S_a^*$

12.5 > 6.49   **OK**

Structure Adequate

Check tie Ft1

Bar diameter = 19.05 mm   Area of bar = 285.02 mm2

Number of bars = 5   Total area of reinforcement = 1425.1 mm2   *403.4*

Ft1 max = 278.83 kN   Ft1 = 574.9 kN

Structures shall be deemed to be capable of carrying the assessment load when the following relationship is satisfied:

$R_a^* \geq S_a^*$

278.8 > 574.85   **NOT OK**

Structure Inadequate

Check Fs5

width of concrete strut = 127 mm or limited to 8 x bar diameter = 152.4 mm = 127 mm

Fc max = 478.1 kN   Fs5 = 284.9 kN   Maximum stress in concrete strut = 12.55 N/mm2

Stress in Fs5 = 4.99 N/mm2

Structures shall be deemed to be capable of carrying the assessment load when the following relationship is satisfied:

$R_a^* \geq S_a^*$

478.1 > 284.92   **OK**

Structure Adequate

REF	CALCULATION	OUTPUT

Check tie Ft2

Ft2 = 167.3 kN Bar diameter = 15.9 mm Area of bar = 198.56 mm2

Number of bars in tie = 2.0 total area of reinforcement = 397.11 mm2 — *421.32*

Ft2 max = 77.696 kN

Structures shall be deemed to be capable of carrying the assessment load when the following relationship is satisfied:

$R_a^* \geq S_a^*$

77.7 > 167.31

Structure Inadequate — **NOT OK**

Check tie Ft3

Ft3 = 621.5 kN Bar diameter = 15.9 mm Area of bar = 198.56 mm2

Number of bars in tie = 2.0 total area of reinforcement = 397.11 mm2 — *1565.1*

Ft2 max = 77.696 kN

Structures shall be deemed to be capable of carrying the assessment load when the following relationship is satisfied:

$R_a^* \geq S_a^*$

77.7 > 621.51

Structure Inadequate — **NOT OK**

Check tie Ft4

Ft4 = 287.4 kN Bar diameter = 15.9 mm

Number of bars in tie = 2.0 total area of reinforcement = 397.11 mm2 — *723.79*

Ft4 max = 77.696 kN

Structures shall be deemed to be capable of carrying the assessment load when the following relationship is satisfied:

$R_a^* \geq S_a^*$

77.7 > 287.4

Structure Inadequate — **NOT OK**

Check Ties 2,3 & 4 considering all vertical reinforcement in zone

Total Ft load = 1076.2 kN Bar diameter = 15.9 mm Area of bar = 198.56 mm2

Number of bars in tie = 12.0 total area of reinforcement = 2382.7 mm2 — *451.69*

Ft2-4 max = 466.18 kN

Structures shall be deemed to be capable of carrying the assessment load when the following relationship is satisfied:

$R_a^* \geq S_a^*$

466.2 > 1076.2

Structure Inadequate — **NOT OK**

Check strut Fs3

Fs2 strut width = 165.06 mm Fs3 = 313.7 kN

Calculate strut width for Fs3 = 2 x Fs1width / 2 / tan∂2 x cosα3 = 171.21 mm considered conservative value

where

α1 = 90 - ∂1 = 32.3 α2 = ∂2 + 32.3 = 75.3 tan∂2 = 0.93

α3 = 75 - 90 = -14.7 cosα3 = 0.967

Calculate effective area of concrete strut

thickness of lower nib x width of strut = 450 x 171.21 = 77044 mm2

Calculate stress in strut = 313.7 x 1000 / 77044 = 4.07 N/mm2 < 12.5 N/mm2

Structures shall be deemed to be capable of carrying the assessment load when the following relationship is satisfied:

$R_a^* \geq S_a^*$

12.5 > 4.07 — **OK**

Structure Adequate

REF	CALCULATION	OUTPUT

Check strut Fs4

Fs4 strut width = 146.05 mm   Fs4 = 367.8 kN

Calculate effective area of concrete strut

thickness of lower nib x width of strut = 450 x 146.05 = 65722 mm2

Calculate stress in strut = 367.8 x 1000 / 65722 = 5.60 N/mm2 < 12.5 N/mm2

Structures shall be deemed to be capable of carrying the assessment load when the following relationship is satisfied:

$R_a^* \geq S_a^*$

12.5 > 5.60

Structure Adequate

OK

Check tie Ft5

Ft5 = 750.6 kN   Bar diameter = 19.05 mm   Area of bar = 285.02 mm2

Number of bars in tie = 4.0   total area of reinforcement = 1140.1 mm2   *658.40*

Ft4 max = 223.06 kN

Structures shall be deemed to be capable of carrying the assessment load when the following relationship is satisfied:

$R_a^* \geq S_a^*$

223.1 > 750.64

Structure Inadequate

NOT OK

Force Ref	Force Type	Stress (N/mm2)	Capacity N/mm2	UF
Fs1	Strut	4.27	12.5	0.34
Fs2		6.49	12.5	0.52
Fs3		4.07	12.5	0.32
Fs4		5.60	12.5	0.45
Fs5		4.99	12.5	0.40

Force Ref	Force Type	Force (kN)	Capacity kN	Stress (N/mm2)	Capacity N/mm2	UF
Ft1	Tie	574.9	278.8	403.37	195.7	2.06
ft2		167.3	77.7	421.32	195.7	2.15
ft3		621.5	77.7	1565.06	195.7	8.00
ft4		287.4	77.7	723.79	195.7	3.70
ft5		750.6	223.1	658.40	195.7	3.37
ft2-4		1076.2	466.2	451.69	195.7	2.31

Force Ref	Force Type	Stress (N/mm2)	Capacity N/mm2	UF
A	Node	6.49	17.8	0.37
B		5.60	17.8	0.31
C		4.99	17.8	0.28
D		6.49	17.8	0.37
E		5.60	15.7	0.36

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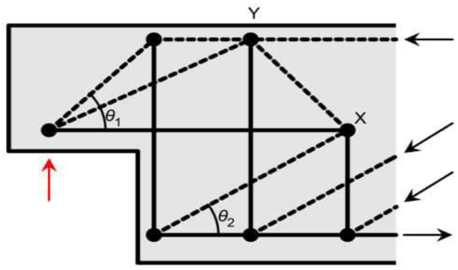
REF	CALCULATION	OUTPUT

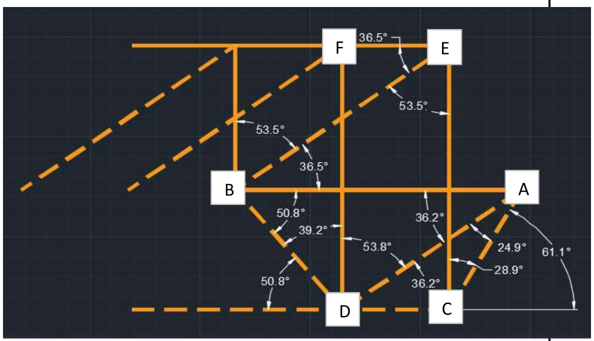
Strut and Tie Checks

The capacity of a half joint may be determined by considering the strut and tie models in Appendix E of CS 466.

Initially conside Strut and Tie model E.16.

Figure E.3 Illustrative example of strut-and-tie model for a half-joint with long nib reinforcement





Considering the method used in the Karl-Heinz Reineck, the following is the approach used to select node locations.

- The Strut and Tie at the top and bottom of the model are positioned along the centreline of the reinforcement.
- Node A is positioned directly beneath the centre line of the bearing
- The vertical ties, CE, DF and BG are in areas where numeroud stirrups (links) are present and hence these ties are spread evenly throughout the B region. i.e. at 305mm intervals.
- As shown in Figure E.3, the first vertical tie is positioned within the first stirrup.
- Node B is positioned at the bend within the horizontal tie bars which coincides with the placement of the stirrups.

See overleaf for proposed strut and tie model.

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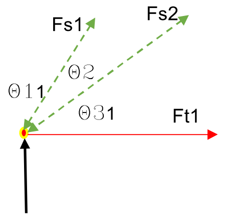
REF	CALCULATION	OUTPUT

<u>Calculate Strut & Tie Forces</u>

Vertical force, Fv = 257.6 kN   Horizontal force, Fh = 0.0 kN

Consider Node A:



Θ1 = 28.9
Θ2 = 24.9
Θ3 = 36.2

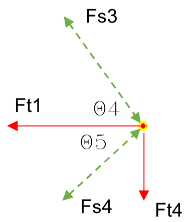
Fv = 257.64 kN

Fs1 = 257.64 / cos 29 = 294.29 kN

Fs2 = 257.64 / cos 54 = 436.22 kN

Ft1 = 294.29 x cos 61.1 = 142.22 + 436.22 x cos 36.2 = 352.02 kN = 494.24 kN

Consider Node B:



Θ4 = 50.8   Θ4α = 39
Θ5 = 36.2   Θ5α = 53.8

Ft1 = 494.24 kN = Fs3 cos 51 + Fs4 cos 36.2   Eq1

Fs3 sin 51 + Fs4 sin 36.2   Eq2

Fs3= Fs4 sin 36.2 / sin 51   Eq3

Sub eq3 in to Eq1

Fs4 cos 36.2 sin 51 / sin 51 + Fs4 cos 36.2 = 494.24 kN

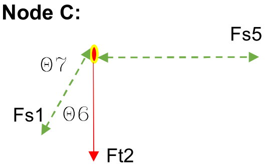
494.24 = Fs4 1.6139

Fs4 = 306.23 kN

Fs3 = 233.39 kN

Ft4 = 306.23 sin 53.8 = 247.12 kN

Consider Node C:



Θ6 = 29
Θ7 = 61

Ft2 = 294.29 sin 29 = 142.22 kN

Fs5 = 294.29 cos 61 = 142.22 kN

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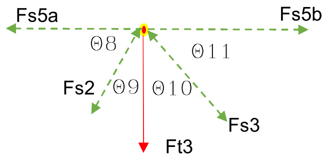
Consider Node D:



Θ8 = 36.2 Fs2 = 436.22 kN

Θ9 = 53.8 Fs3 = 233.39 kN

Θ10 = 39.2 Fs5a = 142.22 kN

Θ11 = 50.8

Ft3 = 436.22 sin 53.8 + 233.39 sin 39.2 = 499.52 kN

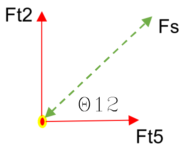
Fs5b = Fs5b + Fs3 sin 50.8 = 142.22 + 436.22 sin 36.2

Fs5b + 180.9 = 399.86 kN

Fs5b = 219.0 kN

Fs5 tot = 361.22 kN

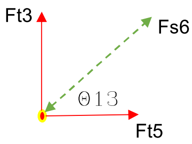
Consider Node E:



Θ12 = 36.5 Fs4 = 306.23 kN

Ft5 = 306.23 cos 36.5 = 246.17 kN

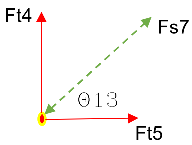
Consider Node F:



Ft5 = Ft3 - Fs6cos 36.5 = 178.97 kN

Fs6 = 499.52 / tan 51.4 = 398.77 kN

Consider Node G:



Ft5 = Ft4 - Fs7cos 36.5 = 88.54 kN Ft5 total = 197.27 + 178.97 + 246.17 = 622.42 kN

Fs7 = 247.12 / tan 51.4 = 197.27 kN

Summary of Forces due to 257.6 kN applied vertically

Force Ref	Force Type	Force (kN)
Fs1	Strut	294.3
Fs2		436.2
Fs3		233.4
Fs4		306.2
Fs5		361.2
Fs6		398.8
Fs7		197.3

Force Ref	Force Type	Force (kN)
Ft1	Tie	494.2
Ft2		142.2
Ft3		499.5
Ft4		247.1
Ft5		622.4

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REF	CALCULATION	OUTPUT

Check strut Fs1

Bearing width, lb = 146.00 mm

2So = 127 mm   lb-2So = 19.00 mm   so 0.5*lb-So = 9.5 mm

U = 2 x cover to centreline of tensile bar = 127 mm

Fs1 strut width = 78.011 mm   Fs2 strut width = 113.71 mm

Maximum force in Ft1 = 1002.3 kN   where maximum stress = 12.55 N/mm2

Fs1 = 294.3 kN   stress in Fs1 = 8.38 N/mm2

Structures shall be deemed to be capable of carrying the assessment load when the following relationship is satisfied:

$R_a^* \geq S_a^*$

12.5 > 8.38

Structure Adequate — **OK**

Check strut Fs2

Fs2 strut width = 113.71 mm   Maximum force in Fs2 = 713.42 kN   Fs2 = 436.2 kN

Stress in Fs2 = 7.67 N/mm2

Structures shall be deemed to be capable of carrying the assessment load when the following relationship is satisfied:

$R_a^* \geq S_a^*$

713.4 > 436.22

Structure Adequate — **OK**

Check tie Ft1

Bar diameter = 19.05 mm   Area of bar = 285.02 mm2

Number of bars = 4   Total area of reinforcement = 1140.1 mm2   *433.51*

Ft1 max = 278.83 kN   Ft1 = 494.2 kN

Structures shall be deemed to be capable of carrying the assessment load when the following relationship is satisfied:

$R_a^* \geq S_a^*$

278.8 > 494.24

Structure Inadequate — **NOT OK**

Check Fs5

Width of concrete strut = 127 mm or limited to 8 x bar diameter = 152.4 mm = 127 mm

Fc max = 531.22 kN   Fs5 = 361.2 kN

Maximum stress in concrete strut = 12.55 N/mm2   Stress in Fs5 = 6.32 N/mm2

Structures shall be deemed to be capable of carrying the assessment load when the following relationship is satisfied:

$R_a^* \geq S_a^*$

12.5 > 6.32

Structure Adequate — **OK**

REF	CALCULATION	OUTPUT

Check tie Ft2

Ft2 = 142.2 kN Bar diameter = 19.05 mm Area of bar = 285.02 mm2

Number of bars in tie = 2.0 total area of reinforcement = 570.05 mm2 — *249.49*

Ft2 max = 111.53 kN *main links only, i.e not incl. local to nib*

Structures shall be deemed to be capable of carrying the assessment load when the following relationship is satisfied:

$R_a^* \geq S_a^*$

111.5 > 142.22

Structure Inadequate — **NOT OK**

Check tie Ft3

Ft3 = 499.5 kN Bar diameter = 19.05 mm Area of bar = 285.02 mm2

Number of bars in tie = 2.0 total area of reinforcement = 570.05 mm2 — *876.29*

Ft2 max = 77.696 kN

Structures shall be deemed to be capable of carrying the assessment load when the following relationship is satisfied:

$R_a^* \geq S_a^*$

77.7 > 499.52

Structure Inadequate — **NOT OK**

Check tie Ft4

Ft4 = 247.1 kN Bar diameter = 19.05 mm Area of bar = 285.02 mm2

Number of bars in tie = 2.0 total area of reinforcement = 570.05 mm2

Ft4 max = 77.696 kN — *433.51*

Structures shall be deemed to be capable of carrying the assessment load when the following relationship is satisfied:

$R_a^* \geq S_a^*$

77.7 > 247.12

Structure Inadequate — **NOT OK**

Check Ties 2,3 & 4 considering all vertical reinforcement in zone

Total Ft load = 888.9 kN Bar diameter = 19.05 mm Area of bar = 285.02 mm2

Number of bars in tie = 12.0 total area of reinforcement = 3420.3 mm2 — *259.88*

Ft2-4 max = 669.18 kN

Structures shall be deemed to be capable of carrying the assessment load when the following relationship is satisfied:

$R_a^* \geq S_a^*$

669.2 > 888.87

Structure Inadequate — **NOT OK**

Check strut Fs3

Fs2 strut width = 113.71 mm Fs3 = 233.4 kN

Calculate strut width for Fs3 = 2 x Fs1width / 2 / tan∂2 x cosα3 = 117.94 mm considered conservative value

where

α1 = 90 - ∂1 = 32.3 tan∂2 = 0.93

α2 = ∂2 + 32.3 = 75.3 cosα3 = 0.967

α3 = 75 - 90 = -14.7

Calculate effective area of concrete strut

thickness of lower nib x width of strut = 500 x 117.94 = 58972 mm2

Calculate stress in concrete strut = 233.4 x 1000 / 58972 = 3.96 N/mm2 < 12.5 N/mm2

Structures shall be deemed to be capable of carrying the assessment load when the following relationship is satisfied:

$R_a^* \geq S_a^*$

12.5 > 3.96

Structure Adequate — **OK**

REF	CALCULATION	OUTPUT

Check strut Fs4

Fs4 strut width = 89.887 mm   Fs4 = 306.2 kN

Calculate effective area of concrete strut

thickness of lower nib x width of strut = 500 x 89.887 = 44944 mm2

Calculate stress in strut = 306.2 x 1000 / 44944 = 6.81 N/mm2 < 12.5 N/mm2

Structures shall be deemed to be capable of carrying the assessment load when the following relationship is satisfied:

$R_a^* \geq S_a^*$

12.5 > 6.81

Structure Adequate — **OK**

Check tie Ft5

Ft5 = 622.4 kN

Bar diameter = 19.05 mm   Area of bar = 285.02 mm2

Number of bars in tie = 4.0   total area of reinforcement = 1140.1 mm2   *545.94*

Ft4 max = 223.06 kN

Structures shall be deemed to be capable of carrying the assessment load when the following relationship is satisfied:

$R_a^* \geq S_a^*$

223.1 > 622.42

Structure Inadequate — **NOT OK**

Force Ref	Force Type	Stress (N/mm2)	Capacity N/mm2	UF
Fs1	Strut	8.38	10.57	0.79
Fs2		7.67	10.57	0.73
Fs3		3.96	10.57	0.37
Fs4		6.81	10.57	0.64
Fs5		6.32	10.57	0.60

Force Ref	Force Type	Force (kN)	Capacity kN	Stress (N/mm2)	Capacity N/mm2	UF
Ft1	Tie	494.2	278.8	433.51	195.65	1.77
ft2		142.2	111.5	249.49	195.65	1.28
ft3		499.5	77.7	876.29	195.65	6.43
ft4		247.1	77.7	433.51	195.65	3.18
ft5		622.4	223.1	545.94	195.65	2.79
ft2-4		888.9	669.2	259.88	195.65	1.33

Force Ref	Force Type	Stress (N/mm2)	Capacity N/mm2	UF
A	Node	8.38	14.97	0.56
B		6.81	14.97	0.46
C		8.38	14.97	0.56
D		7.67	14.97	0.51
E		6.81	13.21	0.52

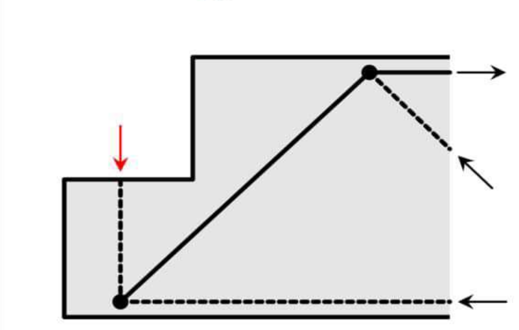
REF	CALCULATION	OUTPUT

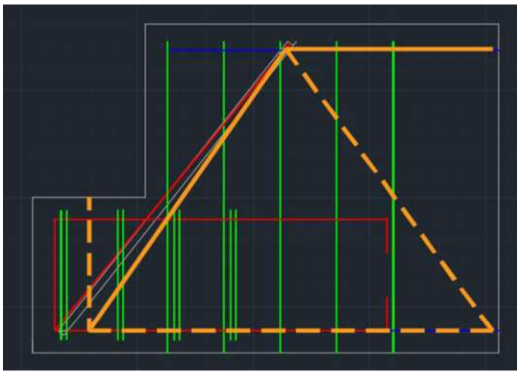
Strut and Tie Checks

The capacity of a half joint may be determined by considering the strut and tie models in Appendix E of CS 466.

Inititally conside Strut and Tie model E.15.

Figure E.15 Illustrative example of a strut-and-tie model for a system with diagonal bars





The following is the approach used to select node locations.

- The centreline of the bearing is considered to be the centreline of the top nib.
- The tie at the top of the section is assumed to be positioned centrally within the longitudinal reinforcement.
- The tie representing the diagonal reinforcement intersects the node (out of alignment) with strut from bearing and top strut.
- Thestrut at the bottom of the section intersects the diagonal tie at the centreline of the longitudinal reinforcement.

See overleaf for proposed strut and tie model.

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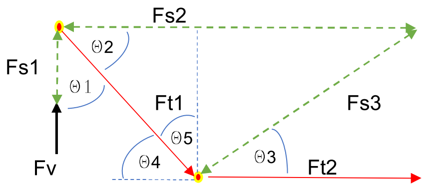
Calculate Strut & Tie Forces

Vertical force, Fv = 257.6 kN

Horizontal force, Fh = 0.0 kN



Θ1 = 33.0
Θ2 = 57.0
Θ3 = 55.7
Θ4 = 57.0
Θ5 = 33.0

Fv = 257.64 kN

Calculate strut & Tie forces

Fs1 = 257.64 kN

Ft1 = 257.64 x 0.84 = 216.07 kN

Fs2 = 216.07 x 0.54 = 117.68 kN

Fs3 = $\frac{216.07 \times 0.54}{0.56}$ = 208.83 kN

Ft2 = 216.07 x 0.5446 + 208.83 x 0.5635 = 235.36 kN

Check stresses

Check compressive stress in concrete strut Fs1 (Strut)

Fn,st = 257.64 kN

Thickness of upper nib = 450 mm   Width of concrete strut = 146 mm width of bearing (conservative)

Area of concrete strut = 65700 mm2

Stress in concrete strut = 257.64 x 1000 / 65700 = 3.92 N/mm2

Maximum allowable stress = 12.55 N/mm2
Structures shall be deemed to be capable of carrying the assessment load when the following relationship is satisfied:

$R_a^* \geq S_a^*$

12.55 > 3.92 **OK**

Structure Adequate

Check compressive stress in strut Fs2 (Strut)

Fn,st = 117.68 kN   Bar diameter = 19.1 mm   Area of bar = 285.02 mm

Number of bars = 5   Total area of reinforcement = 1425.1 mm2

Maximum allowable stress in reinforcement = 250 x 1425.1 / 1.15 x 1000 = 309.81 kN
Considering condition factor = 278.83 kN

Structures shall be deemed to be capable of carrying the assessment load when the following relationship is satisfied:

$R_a^* \geq S_a^*$

278.83 > 117.68 **OK**

Structure Adequate

REF	CALCULATION	OUTPUT

Check concrete strut Fs2

Width of strut is limited to 8x diameter = 8 x 19.1 = 152.4 mm

depth to centreline of strut = 63.5 mm .: width of strut = 127 mm

stress in concrete strut = 117.7 x 1000 / 57150 = 2.06 N/mm2

maximum force in concrete strut = 12.55 x 57150 / 1.50 x 1000 = 478.10 kN
= 430.29 kN incl condition factor

Structures shall be deemed to be capable of carrying the assessment load when the following relationship is satisfied:

$R_a^* \geq S_a^*$

430.29 > 117.68 — **OK**

Structure Adequate

Check tenile stress in tie Ft1 (Tie)

Bar diameter = 19.1 mm   Area of bar = 285.02 mm2

Number of bars = 4   Total area of reinforcement = 1140.1 mm2   *189.52*

Maximum tensile force in steel = 223.06 kN   Ft1 = 216.07 kN

Structures shall be deemed to be capable of carrying the assessment load when the following relationship is satisfied:

$R_a^* \geq S_a^*$

223.06 > 216.07 — **OK**

Structure Adequate

Check tenile stress in tie Ft2 (Tie)

Bar diameter = 19.1 mm   Area of bar = 285.02 mm2

Number of bars = 4   Total area of reinforcement = 1140.1 N/mm2   *206.44*

Maximum tensile force in steel = 223.06 kN   Ft2 = 235.36

Structures shall be deemed to be capable of carrying the assessment load when the following relationship is satisfied:

$R_a^* \geq S_a^*$

223.06 > 235.36 — **NOT OK**

Structure Inadequate

Check concrete strut Fs3

Width of strut is limited to 8x diameter = 8 x 19 = 152 mm

Thickness of beam = 950 mm   Area of concrete = 144400 mm2

Fs3 = 208.83 kN   Stress in concrete strut = 1.45 N/mm2

Structures shall be deemed to be capable of carrying the assessment load when the following relationship is satisfied:

$R_a^* \geq S_a^*$

12.55 > 1.45 — **OK**

Structure Adequate

JACOBS	CALCULATION SHEET				
OFFICE	[redacted] Structures Team	PAGE No.	CHK 53	CONT'N PAGE No.	CHK 54
JOB No. & TITLE	BCU00015 Brigsteer & Underbarrow	ORIGINATOR	[redacted]	DATE	26/02/2023
SECTION	Strut & Tie Checks Upper Nib - Figure E.15	CHECKER		DATE	

REF	CALCULATION	OUTPUT

Force Ref	Force Type	Stress (N/mm2)	Capacity N/mm2	UF
Fs1	Strut	3.92	12.5	0.31
Fs2		2.06	12.5	0.16
Fs3		1.45	12.5	0.12

Force Ref	Force Type	Force (kN)	Capacity kN	Stress (N/mm2)	Capacity N/mm2	UF
Ft1	Tie	216.1	0.0	189.52	195.7	0.97
ft2		235.4	0.0	206.44	195.7	1.06

Force Ref	Force Type	Stress (N/mm2)	Capacity N/mm2	UF
A	Node	3.92	17.8	0.22
B		1.45	15.7	0.09
C		2.06	20.9	0.10

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SECTION	Strut & Tie Checks Lower Nib - Figure E.9	CHECKER		DATE	

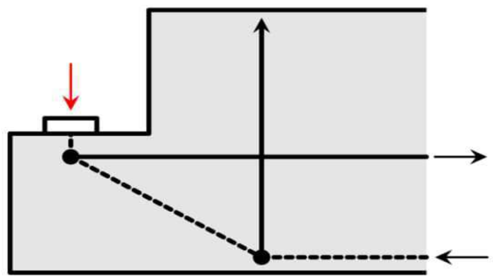
REF	CALCULATION	OUTPUT

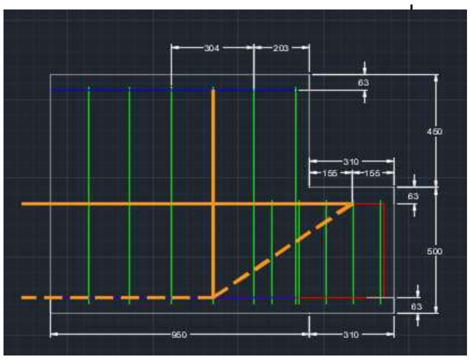
Strut and Tie Checks

The capacity of a half joint may be determined by considering the strut and tie models in Appendix E of CS 466.

Initially conside Strut and Tie model E.16.

Figure E.9 Loads applied through discrete bearings - side view





Considering the method used in the Karl-Heinz Reineck, the following is the approach used to select node locations.

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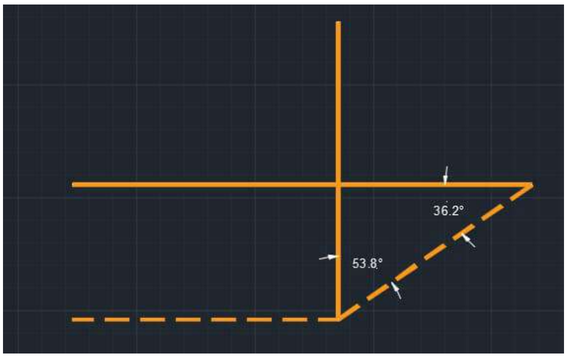
See overleaf for proposed strut and tie model.

REF	CALCULATION	OUTPUT



Calculate Strut & Tie Forces

Vertical force, Fv = 257.6 kN   Horizontal force, Fh = kN

Consider Node A:

∑Fv = 0   Fs1 = 257.6 / cos (53.8) + 0 / sin 36.4 = 436.22 kN   (Strut)

∑FH = 0   Ft1 = 436.22 cos (36.2) = 352.02 kN   (Tie)

Consider Node B:

∑FH = 0   Fs2 = F1 cos (36.2) = 352.02 kN   (Strut)

∑Fv = 0   Ft2 = F1 sin (53.8) = 352.02 kN   (Tie)

Force Ref	Force Type	Force kN
Fs1	Strut	436.22
Fs2		352.02

Force Ref	Force Type	Force kN
Ft1	Tie	352.02
Ft2		352.02

Check strut Fs1

Bearing width, lb = 146.00 mm

2So = 127 mm   lb-2So = 19.00 mm   so 0.5*lb-So = 9.5 mm

U = 2 x cover to centreline of tensile bar = 127 mm

Fs1 strut width = 113.71 mm

Maximum force in Ft1 = 713.42 kN   where maximum stress = 12.55 N/mm2

Fs1 = 436.2 kN   stress in Fs1 = 8.53 N/mm2

Structures shall be deemed to be capable of carrying the assessment load when the following relationship is satisfied:

$R_a^* \geq S_a^*$

12.55 > 8.53

Structure Adequate   **OK**

Check strut Fs2

Fs1 strut width = 113.71 mm Fs2 strut width = 101.6 mm

Maximum force in Ft1 = 637.47 kN where maximum stress = 12.55 N/mm2

Fs2 = 352.0 kN stress in Fs1 = 6.88 N/mm2

Structures shall be deemed to be capable of carrying the assessment load when the following relationship is satisfied:

$R_a^* \geq S_a^*$

12.55 > 6.88 **OK**

Structure Adequate

Check tie Ft1

Bar diameter = 19.05 mm Area of bar = 285.02 mm2

Number of bars = 4 Total area of reinforcement = 1140.1 mm2 *308.76*

Ft1 max = 223.06 kN Ft1 = 352.0 kN

Structures shall be deemed to be capable of carrying the assessment load when the following relationship is satisfied:

$R_a^* \geq S_a^*$

223.06 > 352.02 **NOT OK**

Structure Inadequate

Check tie Ft2

Bar diameter = 19.05 mm Area of bar = 285.02 mm2

Number of bars = 12 Total area of reinforcement = 3420.3 mm2 *102.92*

Ft2 max = 669.18 kN *considers 6no links in section (2 legs per link)*

Ft2 = 352.0 kN

Structures shall be deemed to be capable of carrying the assessment load when the following relationship is satisfied:

$R_a^* \geq S_a^*$

669.18 > 352.02 **OK**

Structure Adequate

Force Ref	Force Type	Stress (N/mm2)	Capacity N/mm2	UF
Fs1	Strut	8.53	10.57	0.81
Fs2		6.88	10.57	0.65

Force Ref	Force Type	Force kN	Capacity kN	Stress (N/mm2)	Capacity N/mm2	UF
Ft1	Tie	352.02	223.1	308.76	195.7	1.58
Ft2		352.02	669.2	102.92	195.7	0.53

Force Ref	Force Type	Stress (N/mm2)	Capacity N/mm2	UF
A	Node	8.53	14.97	0.57
B		8.53	14.97	0.57

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SECTION	Strut & Tie Checks Upper Nib - Figure E.9	CHECKER		DATE	

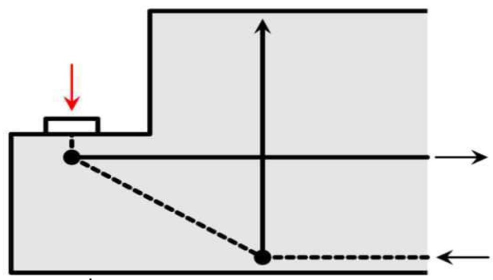
REF	CALCULATION	OUTPUT

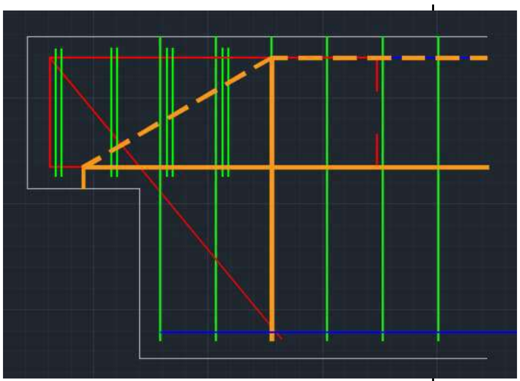
Strut and Tie Checks

The capacity of a half joint may be determined by considering the strut and tie models in Appendix E of CS 466.

Inititally conside Strut and Tie model E.9

Figure E.9 Loads applied through discrete bearings - side view





Considering the method used in the Karl-Heinz Reineck, the following is the approach used to select node locations.

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See overleaf for proposed strut and tie model.

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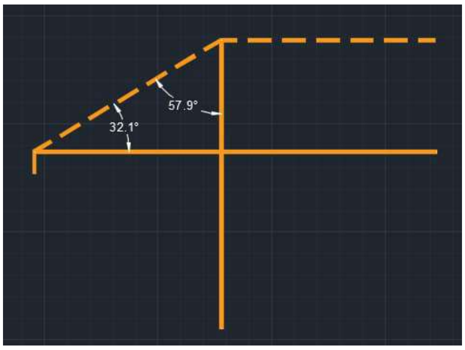
JOB No. & TITLE	BCU00015 Brigsteer & Underbarrow	ORIGINATOR	[redacted]	DATE	26/02/2023
SECTION	Strut & Tie Checks Upper Nib - Figure E.9	CHECKER		DATE	

REF	CALCULATION	OUTPUT



Calculate Strut & Tie Forces

Vertical force, Fv = 257.6 kN   Horizontal force, Fh = kN

Consider Node A:

∑Fv = 0   Fs1 = 257.6 / cos (57.9) + 0 / sin 36.4 = 484.83 kN   (Strut)

∑FH = 0   Ft1 = 484.83 cos (32.1) = 410.71 kN   (Tie)

Consider Node B:

∑FH = 0   Fs2 = F1 cos (32.1) = 410.71 kN   (Strut)

∑Fv = 0   Ft2 = F1 sin (57.9) = 410.71 kN   (Tie)

Force Ref	Force Type	Force kN
Fs1	Strut	484.83
Fs2		410.71

Force Ref	Force Type	Force kN
Ft1	Tie	410.71
Ft2		410.71

Check strut Fs1

Bearing width, lb = 146.00 mm

2So = 127.05 mm   lb-2So = 18.95 mm   so 0.5*lb-So = 9.475 mm

U = 2 x cover to centreline of tensile bar = 127.05 mm

Fs1 strut width = 117.7 mm   Fs1 = 484.8 kN   stress in Fs1 = 9.15 N/mm2

Maximum force in Ft1 = 738.46 kN   where maximum stress = 12.55 N/mm2

Structures shall be deemed to be capable of carrying the assessment load when the following relationship is satisfied:

$R_a^* \geq S_a^*$

12.55 > 9.15   **OK**

Structure Adequate

JACOBS					CALCULATION SHEET
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SECTION	Strut & Tie Checks Upper Nib - Figure E.9	CHECKER		DATE	

Check strut Fs2

Fs1 strut width = 117.7 mm   Fs2 strut width = 152.4 mm

Maximum force in Ft1 = 956.2 kN   where maximum stress = 12.55 N/mm2

Fs2 = 410.7 kN   stress in Fs1 = 7.75 N/mm2

Structures shall be deemed to be capable of carrying the assessment load when the following relationship is satisfied:

$R_a^* \geq S_a^*$

12.55 > 7.75

Structure Adequate   **OK**

Check tie Ft1

Bar diameter = 15.9 mm   Area of bar = 198.56 mm2

Number of bars = 4   Total area of reinforcement = 794.23 mm2   *517.12*

Ft1 max = 155.39 kN   Ft1 = 410.7 kN

Structures shall be deemed to be capable of carrying the assessment load when the following relationship is satisfied:

$R_a^* \geq S_a^*$

155.39 > 410.71   **NOT OK**

Structure Inadequate

Check tie Ft2

Bar diameter = 19.05 mm   Area of bar = 285.02 mm2

Number of bars = 12   Total area of reinforcement = 3420.3 mm2   *120.08*

Ft1 max = 669.18 kN   *considers 6no links in section*

Ft1 = 410.7 kN

Structures shall be deemed to be capable of carrying the assessment load when the following relationship is satisfied:

$R_a^* \geq S_a^*$

669.18 > 410.71

Structure Adequate   **OK**

Force Ref	Force Type	Stress (N/mm2)	Capacity N/mm2	UF
Fs1	Strut	9.15	12.55	0.73
Fs2		7.75	12.55	0.62

Force Ref	Force Type	Force kN	Capacity kN	Stress (N/mm2)	Capacity N/mm2	UF
Ft1	Tie	410.71	155.4	517.12	195.7	2.64
Ft2		410.71	669.2	120.08	195.7	0.61

Force Ref	Force Type	Stress (N/mm2)	Capacity N/mm2	UF
A	Node	9.15	17.78	0.51
B		9.15	17.78	0.51

Cumbria CC
Underbarrow Bridge
SLS half joint

Note: BA 39/93 has been superseded by CS 466, however its application within SCALE software remains applicable for SLS analysis of the half-joints. No further calculations required.

Office: [redacted]

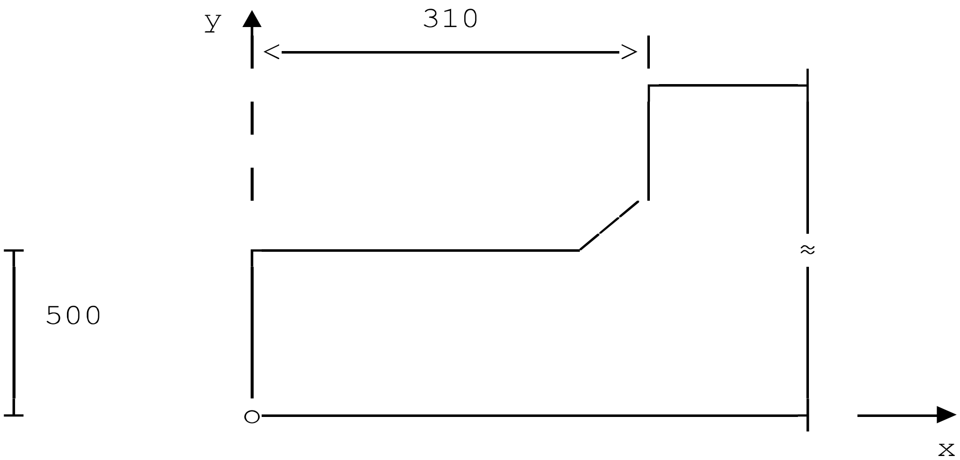
Location: Ex1 -Example from Appendix B BA 39/93

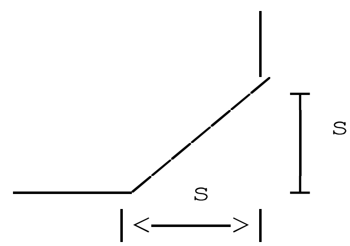
Assessment of Half-Joints at Serviceability Limit State

to DoT Advice Note BA 39/93

Analysis is for lower half-joint. Geometry of half-joint is:

Breadth of half-joint	b=0609 mm
Depth of half-joint	h=500 mm
Length of half-joint	k=310 mm
Splay dimensions	s=038 mm





Splay Dimensions

s=38 mm

Breadth of half-joint 609 mm

Details of crack line (coordinates of tip of crack):

x coordinate	xc=k-s/2=310-38/2=291 mm
y coordinate	yc=h+s/2=500+38/2=519 mm
Gradient of crack	mc=TAN(RAD(315))=-1

Details of reinforcement groups:

Young's modulus of reinforcement	Es=200000 N/mm^2
Number of reinforcement groups	nog=2

Reinforcement group 1 :

Anti-clockwise angle from x axis ang(1)=00°

y coordinate of a point in group

x coordinate	x(1)=0 mm
y coordinate	y(1)=437.00 mm
Area of reinforcement	As(1)=506.7 mm^2
Diameter of bars in group	d(1)=12.7 mm
Spacing of bars in group	s(1)=152 mm

Reinforcement group horizontal.

Coordinates of intersection of group with crack line.

x coordinate xi(1)=(-mc*xc-y(i)+yc)/-mc
=(--1*291-437+519)/--1
=373 mm

y coordinate yi(1)=y(i)=437 mm

Effective area of reinforcement
group normal to crack line　　Ae(1)=As(i)*(COS(PI/4-RAD(ang(i))))^2
=506.7*(COS(3.1416/4-RAD(0)))^2
=253.35 mm²

Distance to intersection from
crack tip　　dc(1)=SQR((xc-xi(i))^2+(yc-yi(i))^2)
=SQR((291-373)^2+(519-437)^2)
=115.97 mm

Effective depth of r'ment group　　437 mm

Reinforcement group 2 :
Anti-clockwise angle from x axis　　ang(2)=90°
x coordinate of a point in group
x coordinate　　x(2)=361 mm
y coordinate　　y(2)=0 mm
Area of reinforcement　　As(2)=570.04 mm²
Diameter of bars in group　　d(2)=19.05 mm
Spacing of bars in group　　s(2)=152 mm
Reinforcement group vertical.
Coordinates of intersection of group with crack line.
x coordinate　　xi(2)=x(i)=361 mm
y coordinate　　yi(2)=(xi(i)-xc)*mc+yc
=(361-291)*-1+519
=449 mm

Effective area of reinforcement
group normal to crack line　　Ae(2)=As(i)*(COS(PI/4-RAD(ang(i))))^2
=570.04*(COS(3.1416/4-RAD(90)))^2
=285.02 mm²

Distance to intersection from
crack tip　　dc(2)=SQR((xc-xi(i))^2+(yc-yi(i))^2)
=SQR((291-361)^2+(519-449)^2)
=98.995 mm

Effective depth of r'ment group　　449 mm
Inclined or vertical r'ment group 2 is nearest to tip of crack.

Concrete properties:
Concrete cube strength　　fcu=41.4 N/mm²
Modulus of rupture　　ft=0.556*SQR(fcu)=0.556*SQR(41.4)
=3.5775 N/mm²
Young's modulus　　Ec=35400 N/mm²

Vertical applied loading:
Load　　FAV(1)=-194.9 kN
x coordinate　　xR(1)=155 mm
Dimension "a" BA 39/93 Figure 2.2　　a=k-xR(i)=310-155=155 mm
Horizontal applied loading
Number of applied horiz. loads　　noh=0

Intersection of Neutral Axis and crack line:
y coordinate　　yn=XVAL=59.574 mm
x coordinate　　xn=xc+yc-yn=291+519-59.574
=750.43 mm
Concrete compressive strain　　ec=XVALA=0.20806E-3

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Reinforcement group 1 :

Strain normal to crack at depth	437 mm
Strain	ei(1)=SQR(2)*ec*(yi(i)-yn)/yn =SQR(2)*0.20806E-3*(437-59.574) /59.574 =0.0018641
Strain in steel direction	es(1)=ei(i)*COS(PI/4-RAD(ang(i))) =0.0018641*COS(3.1416/4-RAD(0)) =0.0013181
Stress in steel	fs(1)=es(i)*Es=0.0013181*200000 =263.62 N/mm²
Force in steel	Fs(1)=fs(i)*As(i)/1000 =263.62*506.7/1000 =133.58 kN
Horizontal force component	Fsh(1)=Fs(i)*COS(RAD(ang(i))) =133.58*COS(RAD(0)) =133.58 kN
Vertical force component	Fsv(1)=0 kN
Moments about Neutral Axis:	
Horizontal force component	Msh(1)=Fsh(i)*(yi(i)-yn)/1000 =133.58*(437-59.574)/1000 =50.416 kNm
Vertical force component	Msv(1)=0 kNm

Reinforcement group 2 :

Strain normal to crack at depth	449 mm
Strain	ei(2)=SQR(2)*ec*(yi(i)-yn)/yn =SQR(2)*0.20806E-3*(449-59.574) /59.574 =0.0019234
Strain in steel direction	es(2)=ei(i)*COS(PI/4-RAD(ang(i))) =0.0019234*COS(3.1416/4-RAD(90)) =0.00136
Stress in steel	fs(2)=es(i)*Es=0.00136*200000 =272.01 N/mm²
Force in steel	Fs(2)=fs(i)*As(i)/1000 =272.01*570.04/1000 =155.05 kN
Horizontal force component	Fsh(2)=0 kN
Vertical force component	Fsv(2)=Fs(i)*SIN(RAD(ang(i))) =155.05*SIN(RAD(90)) =155.05 kN
Moments about Neutral Axis:	
Horizontal force component	Msh(2)=0 kNm
Vertical force component	Msv(2)=Fsv(i)*(xn-xi(i))/1000 =155.05*(750.43-361)/1000 =60.382 kNm

Concrete force FCH=-ec*Ec*b*yn/2000
=-0.20806E-3*35400*609*59.574/2000
=-133.61 kN

Concrete moment MCH=FCH*2*yn/3000
=-133.61*2*59.574/3000
=-5.3064 kNm

Applied loads

1. Vertical direction

Load FAV=FAV(i)=-194.9 kN

Moment about Neutral Axis MAV=MAV+FAV(i)*(xn-xR(i))/1000
=0+-194.9*(750.43-155)/1000
=-116.05 kNm

2. Horizontal direction

Load FAH=0 kN

Moment about Neutral Axis MAH=0 kNm

Equilibrium of forces and moments:

Force equilibrium RHF=FAH+FSH+FCH=0+133.58+-133.61
=-0.029636 kN

Moment equilibrium RM=MAH+MAV+MSV+MSH-MCH
=0+-116.05+60.382+50.416--5.3064
=0.055711 kNm

Reinforcement group 2 is outermost layer and controls crack width.

Bar diameter 19.05 mm Spacing of bars 152 mm
Group is vertical Slippage factor K1 (Clause 2.4) K1=3.5

Tension strains:

Normal to crack at tip e1=ec*(yc-yn)*SQR(2)/yn
=0.20806E-3*(519-59.574)*SQR(2)
/59.574
=0.0022691

Normal to crack in outermost reinforcement group 0.0019234.

Effective area of all reinforcement groups in tension zone
measured normal to crack.

Reinforcement group 1 :

Effective area Asn(1)=As(i)*COS(PI/4-RAD(ang(i)))^2
=506.7*COS(3.1416/4-RAD(0))^2
=253.35 mm²

Reinforcement group 2 :

Effective area Asn(2)=As(i)*COS(PI/4-RAD(ang(i)))^2
=570.04*COS(3.1416/4-RAD(90))^2
=285.02 mm²

Total effective area As=As=538.37 mm²

Partial safety factor for material strength at Serviceability
Limit State gamma m gm=1

Factor K2 (Clause 2.4) K2=0.0003

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Reduction in strain for tension stiffening

```
Reduction                         re=K2*b*h*ft/(Es*ei(ii)*As*gm)
                                    =0.3E-3*609*500*3.5775/(200000
                                     *0.0019234*538.37*1)
                                    =0.001578

Modified strain at tip            e'=K1*e1-re=3.5*0.0022691-0.001578
                                    =0.0063638
```

The crack width is determined from the lesser of the two expressions as per Clause 2.5.

```
Expression 1 crack width          w1=SQR(2)*(a-0.5*s)*e'
                                    =SQR(2)*(155-0.5*38)*0.0063638
                                    =1.224 mm

Distance from outermost group to tip of crack
measured normal to group          dcnb=dc(ii)*COS(PI/4-RAD(ang(ii)))
                                      =98.995*COS(3.1416/4-RAD(90))
                                      =70 mm
Distance from bar to tip of crack acr=SQR((s(ii)/2)^2+dcnb^2)-d(ii)/2
                                     =SQR((152/2)^2+70^2)-19.05/2
                                     =93.8 mm

Expression 2 crack width          w2=3*acr*e'=3*93.8*0.0063638
                                    =1.7908 mm
```

Crack width is 1.224 mm from Expression 1.

Crack width should be less than the permissible value from Table 1 of BS5400:Part 4:1990.
If the crack width exceeds the permissible value, inspection of the half-joint should be undertaken to confirm the condition of the joint.

SUMMARY

Concrete compressive strain	0.20806E-3
Crack width (from Expression 1)	1.224 mm

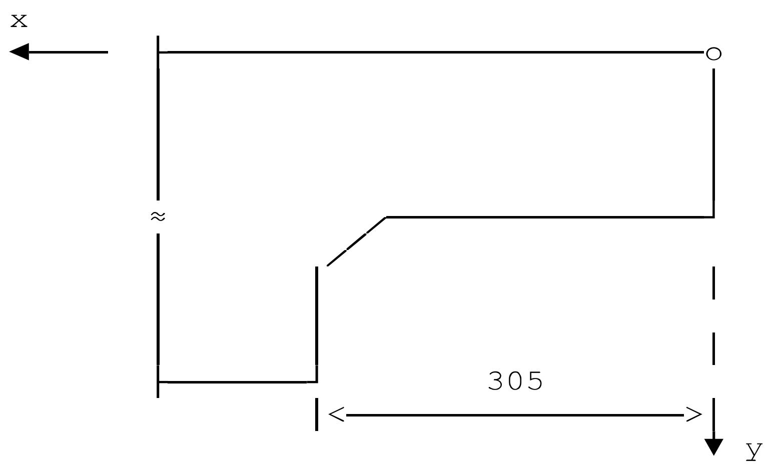
Location: Ex1 -Example from Appendix B BA 39/93

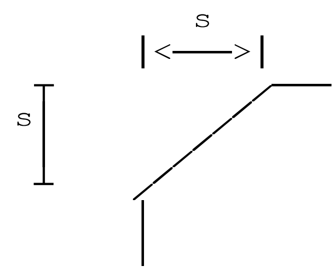
Assessment of Half-Joints at Serviceability Limit State

to DoT Advice Note BA 39/93

Analysis is for upper half-joint. Geometry of half-joint is:

Breadth of half-joint	b=0610 mm
Depth of half-joint	h=425 mm
Length of half-joint	k=305 mm
Splay dimensions	s=038 mm





Splay Dimensions

s=38 mm

Breadth of half-joint 610 mm

Details of crack line (coordinates of tip of crack):

x coordinate	xc=k-s/2=305-38/2=286 mm
y coordinate	yc=h+s/2=425+38/2=444 mm
Gradient of crack	mc=TAN(RAD(315))=-1

Details of reinforcement groups:

Young's modulus of reinforcement	Es=200000 N/mm^2
Number of reinforcement groups	nog=3

Reinforcement group 1 :

Anti-clockwise angle from x axis	ang(1)=00°
y coordinate of a point in group	
x coordinate	x(1)=0 mm
y coordinate	y(1)=314.00 mm
Area of reinforcement	As(1)=1425 mm^2
Diameter of bars in group	d(1)=19.05 mm
Spacing of bars in group	s(1)=150 mm

Reinforcement group horizontal.
Coordinates of intersection of group with crack line.

x coordinate	xi(1)=(-mc*xc-y(i)+yc)/-mc
	=(--1*286-314+444)/--1
	=416 mm
y coordinate	yi(1)=y(i)=314 mm

Office: [redacted]

```
Effective area of reinforcement
group normal to crack line         Ae(1)=As(i)*(COS(PI/4-RAD(ang(i))))^2
                                        =1425*(COS(3.1416/4-RAD(0)))^2
                                        =712.5 mm²
Distance to intersection from
crack tip                          dc(1)=SQR((xc-xi(i))^2+(yc-yi(i))^2)
                                        =SQR((286-416)^2+(444-314)^2)
                                        =183.85 mm
Effective depth of r'ment group    314 mm

Reinforcement group 2 :
Anti-clockwise angle from x axis   ang(2)=90°
x coordinate of a point in group
x coordinate                       x(2)=374 mm
y coordinate                       y(2)=0 mm
Area of reinforcement              As(2)=0397 mm²
Diameter of bars in group          d(2)=15.9 mm
Spacing of bars in group           s(2)=150 mm
Reinforcement group vertical.
Coordinates of intersection of group with crack line.
x coordinate                       xi(2)=x(i)=374 mm
y coordinate                       yi(2)=(xi(i)-xc)*mc+yc
                                        =(374-286)*-1+444
                                        =356 mm
Effective area of reinforcement
group normal to crack line         Ae(2)=As(i)*(COS(PI/4-RAD(ang(i))))^2
                                        =397*(COS(3.1416/4-RAD(90)))^2
                                        =198.5 mm²
Distance to intersection from
crack tip                          dc(2)=SQR((xc-xi(i))^2+(yc-yi(i))^2)
                                        =SQR((286-374)^2+(444-356)^2)
                                        =124.45 mm
Effective depth of r'ment group    356 mm

Reinforcement group 3 :
Anti-clockwise angle from x axis   ang(3)=49°
Coordinates x,y of a point in group:
x coordinate                       x(3)=360 mm
y coordinate                       y(3)=370 mm
Area of reinforcement              As(3)=4560 mm²
Diameter of bars in group          d(3)=19.05 mm
Spacing of bars in group           s(3)=152 mm
Gradient of reinforcement group    m(3)=TAN(RAD(ang(i)))=1.1504
Coordinates of intersection of group with crack line.
x coordinate                       xi(3)=(m(i)*x(i)-mc*xc-y(i)+yc)
                                         /(m(i)-mc)
                                        =(1.1504*360--1*286-370+444)
                                         /(1.1504--1)
                                        =360 mm
y coordinate                       yi(3)=(xi(i)-xc)*mc+yc
                                        =(360-286)*-1+444
                                        =370 mm
Effective area of reinforcement
group normal to crack line         Ae(3)=As(i)*(COS(PI/4-RAD(ang(i))))^2
                                        =4560*(COS(3.1416/4-RAD(49)))^2
                                        =4537.8 mm²
```

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Distance to intersection from crack tip	dc(3)=SQR((xc-xi(i))^2+(yc-yi(i))^2) =SQR((286-360)^2+(444-370)^2) =104.65 mm
Effective depth of r'ment group	370 mm
Inclined or vertical r'ment group	3 is nearest to tip of crack.
Concrete properties:	
Concrete cube strength	fcu=51.7 N/mm²
Modulus of rupture	ft=0.556*SQR(fcu)=0.556*SQR(51.7) =3.9978 N/mm²
Young's modulus	Ec=37600 N/mm²
Vertical applied loading:	
Load	FAV(1)=-257.6 kN
x coordinate	xR(1)=152.5 mm
Dimension "a" BA 39/93 Figure 2.2	a=k-xR(i)=305-152.5=152.5 mm
Horizontal applied loading	
Number of applied horiz. loads	noh=0
Intersection of Neutral Axis and crack line:	
y coordinate	yn=XVAL=144.33 mm
x coordinate	xn=xc+yc-yn=286+444-144.33 =585.67 mm
Concrete compressive strain	ec=XVALA=0.12665E-3
Reinforcement group 1 :	
Strain normal to crack at depth	314 mm
Strain	ei(1)=SQR(2)*ec*(yi(i)-yn)/yn =SQR(2)*0.12665E-3*(314-144.33) /144.33 =0.21056E-3
Strain in steel direction	es(1)=ei(i)*COS(PI/4-RAD(ang(i))) =0.21056E-3*COS(3.1416/4-RAD(0)) =0.14889E-3
Stress in steel	fs(1)=es(i)*Es=0.14889E-3*200000 =29.778 N/mm²
Force in steel	Fs(1)=fs(i)*As(i)/1000 =29.778*1425/1000 =42.433 kN
Horizontal force component	Fsh(1)=Fs(i)*COS(RAD(ang(i))) =42.433*COS(RAD(0)) =42.433 kN
Vertical force component	Fsv(1)=0 kN
Moments about Neutral Axis:	
Horizontal force component	Msh(1)=Fsh(i)*(yi(i)-yn)/1000 =42.433*(314-144.33)/1000 =7.1997 kNm
Vertical force component	Msv(1)=0 kNm

Reinforcement group 2 :

```
Strain normal to crack at depth   356 mm
Strain                            ei(2)=SQR(2)*ec*(yi(i)-yn)/yn
                                       =SQR(2)*0.12665E-3*(356-144.33)
                                        /144.33
                                       =0.26268E-3
Strain in steel direction         es(2)=ei(i)*COS(PI/4-RAD(ang(i)))
                                       =0.26268E-3*COS(3.1416/4-RAD(90))
                                       =0.18575E-3
Stress in steel                   fs(2)=es(i)*Es=0.18575E-3*200000
                                       =37.149 N/mm²
Force in steel                    Fs(2)=fs(i)*As(i)/1000
                                       =37.149*397/1000
                                       =14.748 kN
Horizontal force component        Fsh(2)=0 kN
Vertical force component          Fsv(2)=Fs(i)*SIN(RAD(ang(i)))
                                        =14.748*SIN(RAD(90))
                                        =14.748 kN
Moments about Neutral Axis:
Horizontal force component        Msh(2)=0 kNm
Vertical force component          Msv(2)=Fsv(i)*(xn-xi(i))/1000
                                        =14.748*(585.67-374)/1000
                                        =3.1218 kNm
```

Reinforcement group 3 :

```
Strain normal to crack at depth   370 mm
Strain                            ei(3)=SQR(2)*ec*(yi(i)-yn)/yn
                                       =SQR(2)*0.12665E-3*(370-144.33)
                                        /144.33
                                       =0.28006E-3
Strain in steel direction         es(3)=ei(i)*COS(PI/4-RAD(ang(i)))
                                       =0.28006E-3*COS(3.1416/4-RAD(49))
                                       =0.27938E-3
Stress in steel                   fs(3)=es(i)*Es=0.27938E-3*200000
                                       =55.875 N/mm²
Force in steel                    Fs(3)=fs(i)*As(i)/1000
                                       =55.875*4560/1000
                                       =254.79 kN
Horizontal force component        Fsh(3)=Fs(i)*COS(RAD(ang(i)))
                                        =254.79*COS(RAD(49))
                                        =167.16 kN
Vertical force component          Fsv(3)=Fs(i)*SIN(RAD(ang(i)))
                                        =254.79*SIN(RAD(49))
                                        =192.29 kN
Moments about Neutral Axis:
Horizontal force component        Msh(3)=Fsh(i)*(yi(i)-yn)/1000
                                        =167.16*(370-144.33)/1000
                                        =37.722 kNm
Vertical force component          Msv(3)=Fsv(i)*(xn-xi(i))/1000
                                        =192.29*(585.67-360)/1000
                                        =43.395 kNm
```

```
Concrete force                      FCH=-ec*Ec*b*yn/2000
                                       =-0.12665E-3*37600*610*144.33/2000
                                       =-209.63 kN
Concrete moment                     MCH=FCH*2*yn/3000
                                       =-209.63*2*144.33/3000
                                       =-20.171 kNm
```

Applied loads

```
1. Vertical direction
Load                                FAV=FAV(i)=-257.6 kN
Moment about Neutral Axis           MAV=MAV+FAV(i)*(xn-xR(i))/1000
                                       =0+-257.6*(585.67-152.5)/1000
                                       =-111.58 kNm

2. Horizontal direction
Load                                FAH=0 kN
Moment about Neutral Axis           MAH=0 kNm


Equilibrium of forces and moments:
Force equilibrium                   RHF=FAH+FSH+FCH=0+209.59+-209.63
                                       =-0.040123 kN
Moment equilibrium                  RM=MAH+MAV+MSV+MSH-MCH
                                      =0+-111.58+46.516+44.922--20.171
                                      =0.024573 kNm
Reinforcement group 3 is outermost layer and controls crack width.

Bar diameter 19.05 mm     Spacing of bars  152 mm
Inclination 49°          Slippage factor K1 ( Clause 2.4 ) K1=2.3

Tension strains:
Normal to crack at tip              e1=ec*(yc-yn)*SQR(2)/yn
                                      =0.12665E-3*(444-144.33)*SQR(2)
                                       /144.33
                                      =0.37189E-3
Normal to crack in outermost reinforcement group 0.28006E-3.

Effective area of all reinforcement groups in tension zone
measured normal to crack.

Reinforcement group 1 :
Effective area                      Asn(1)=As(i)*COS(PI/4-RAD(ang(i)))^2
                                          =1425*COS(3.1416/4-RAD(0))^2
                                          =712.5 mm²
Reinforcement group 2 :
Effective area                      Asn(2)=As(i)*COS(PI/4-RAD(ang(i)))^2
                                          =397*COS(3.1416/4-RAD(90))^2
                                          =198.5 mm²
Reinforcement group 3 :
Effective area                      Asn(3)=As(i)*COS(PI/4-RAD(ang(i)))^2
                                          =4560*COS(3.1416/4-RAD(49))^2
                                          =4537.8 mm²
Total effective area                As=As=5448.8 mm²
```

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```
Partial safety factor for material strength at Serviceability
Limit State  gamma m                gm=1
Factor K2 ( Clause 2.4 )            K2=0.0003

Reduction in strain for tension stiffening
Reduction                           re=K2*b*h*ft/(Es*ei(ii)*As*gm)
                                      =0.3E-3*610*425*3.9978/(200000
                                       *0.28006E-3*5448.8*1)
                                      =0.0010188

Modified strain at tip              e'=K1*e1-re=2.3*0.37189E-3-0.0010188
                                      =-0.16343E-3

Modified strain at tip is compressive
Crack width is zero
```

SUMMARY

```
Concrete compressive strain         0.12665E-3
Modified strain at tip is compressive
Crack width is zero
```

No125

Appendix B. Assessment Check Calculations (CAT3)

ISSUE	TOTAL SHEETS	AUTHOR	DATE	CHECKED BY	DATE	APPROVED BY	DATE	COMMENTS
1	[redacted]							
2								
3								
4								
5								

SUPERSEDES DOC No

DATE

DESIGN BASIS STATEMENT (Inc. sources of info/data, assumptions made, standards, etc.)

Introduction

This calculation contains the category 3 assessment of Underbarrow half joint bridge. Dead loads have been determined in accordance with historic drawings and CS 454. Live loads have been determined in accordance with CS 454. Material properties have been determined in accordance with CS 454 and CS 455. The structural analysis of the bridge has been executed by strut-and-tie analysis in accordance with CS 466.

Assumptions

1) Failure of the bridge has been assumed to occur through inadequate capacity of the reinforcement as opposed to failure of the concrete therefore sensitive analysis of the concrete struts within the strut-and-tie models has been omitted. Struts are assumed to have a width of 80mm and depth equal to the width of each beam.

2) The condition factor of 0.9 has been applied to the material resistance values of both the concrete and reinforcing steel.

3) The reinforcement profile applied for analysis is modelled in accordance with that stated in AiP.

References

Ref. 1: CS 454 Assessment of highway bridges and structures

Ref. 2 CS 455 The asessment of concrete highway structures

Ref. 3 CS 466 Risk management and structural assessment of concrete half-joint deck structures

Ref. 4 Strut-and-tie Models How to design concrete members using strut-and-tie models in accordance with Eurocode 2

Ref. 5 BCU00015-JAC-SBR-6330-RP-SL221-CB-008 P02 Approval in Principle (Half Joint Assessment) - Underbarrow

Ref. 6 BCU00015-JAC-SBR-6330-RP-SL221-CB-004 Half Joint Inspection Report - Underbarrow

SUBJECT	CALCULATIONS			OUTPUT

Dead Loads

Input Parameters

Ref.	Parameter	Value	Unit
Ref. 1 Table 4.1.1a	RC density =	2400	kg/m^3
Ref. 1 Table 4.1.1a	Bituminous macadam density =	2400	kg/m^3
	Acceleration due to gravity =	9.81	m/s^2
	RC unit weight =	23.544	kN/m^3
	Bituminous macadam unit weight =	23.544	kN/m^3
	Bridge length =	18.3	m

RC beams

Edge Beams

Ref.	Parameter	Value	Unit
Figure 4 & Figure 5	End cross-sectional area =	0.571	m^2
Figure 3	Mid-span cross-sectional area =	0.495	m^2
	No. =	2	
	Load per m =	25.10	kN/m

Internal Beams

Ref.	Parameter	Value	Unit
Figure 2	End cross-sectional area =	0.422	m^2
Figure 1	Mid-span cross-sectional area =	0.286	m^2
	No. =	15	
	Load per m =	125.0	kN/m

Concrete Plinth

Applied as line loads of varying magnitude to the internal beams

Load applied to beam:	Value	Unit	Output
2 =	4.8	kN/m	Figure 9
3 =	4.8	kN/m	Figure 9
4 =	0.3	kN/m	Figure 9
5 =	0.9	kN/m	Figure 9
6 =	1.6	kN/m	Figure 9
7 =	2.2	kN/m	Figure 9
8 =	2.8	kN/m	Figure 9
9 =	3.5	kN/m	Figure 9
10 =	4.1	kN/m	Figure 9
11 =	5.4	kN/m	Figure 9
12 =	6.0	kN/m	Figure 9
13 =	6.6	kN/m	Figure 9
14 =	14	kN/m	Figure 9
15 =	14	kN/m	Figure 9

Concrete plinth load per m = 71 kN/m

Parapets

Applied as a 1.0kN/m line to either edge beam.

Parameter	Value	Unit	Output
Parapet load per m =	2	kN/m	Figure 10

SUBJECT	CALCULATIONS	OUTPUT

Road surfacing

Applied as 1.3kN/m line load to the central 11 beams.

Ref				Output
Figure 6	surfacing thickness=	101.6	mm	
Figure 6	carriageway width =	6096	mm	
	Surfacing load per m =	14.6	kN/m	
	No beams applied to =	11		
	Surfacing load per m per beam =	1.33	kN/m	Figure 11
Ref. 1 Table A.1	Partial factor for surfacing superimposed dead load =	1.20		

Upper nib

The upper nib of the drop-in-span beams is modelled as a point load applied to the end of each beam defined by the geometry described in the AiP.

			Output
Width =	305	mm	
Breadth =	596	mm	
Height =	450	mm	
Vol =	0.082	m^3	
No =	34		
Load =	65.5	kN	Figure 12
Total SLS super-imposed dead load per m =	240.6	kN/m	
Total SLS super-imposed dead load =	4468.5	kN	

Check against model output

			Output
Total SLS load from model =	4394.1	kN	
Perecentage difference =	1.7	%	OK

Live Loads

Pedestrian ALL

Ref				Output
Ref. 1 Table 5.32a	Pedestrian load =	5	kN/m^2	
Ref. 1 Table 5.32b	Pedestrian LL factor =	0.8		

Ref		*North*	*South*		Output
	Footway width =	1.8	2.2	m	
Ref. 1 Table 5.32c	Width factor =	1.0	1.0		
	Pedestrian ALL =	7.0	8.8	kN/m	Figure 13
	Pedestrian ALL applied individually to 2 beams =	3.5	4.4	kN/m	

SUBJECT	CALCULATIONS	OUTPUT

Loading Figures

RC Beams

Edge Beams

Figure 1: Edge beam cross-section at mid-span.

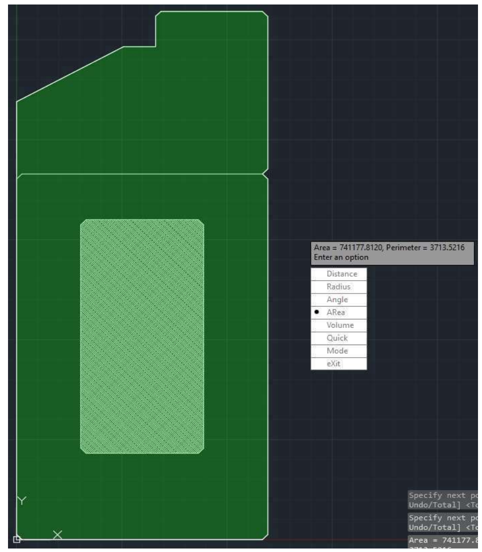

Figure 2: Edge beam cross-section at end.

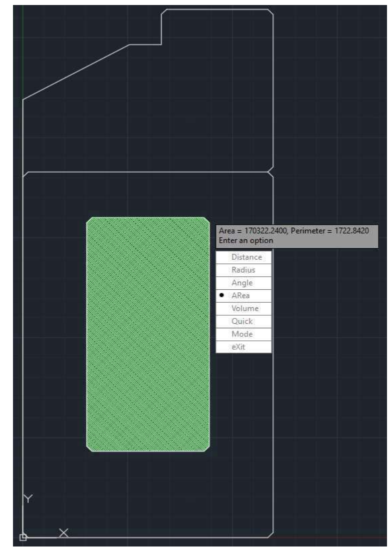

Figure 3: Edge beam void area.

Internal Beams

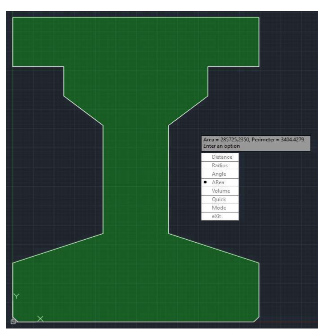

Figure 4: Internal beam cross-section at mid-span.

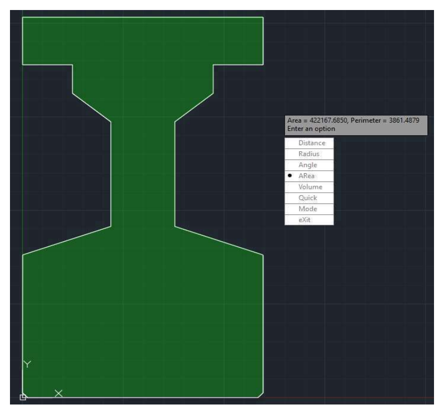

Figure 5: Internal beam cross-section at mid-span

[redacted]	DOCUMENT NO.	SHEET 5 OF 26

SUBJECT

SUBJECT	CALCULATIONS	OUTPUT

Superimposed Dead Loads

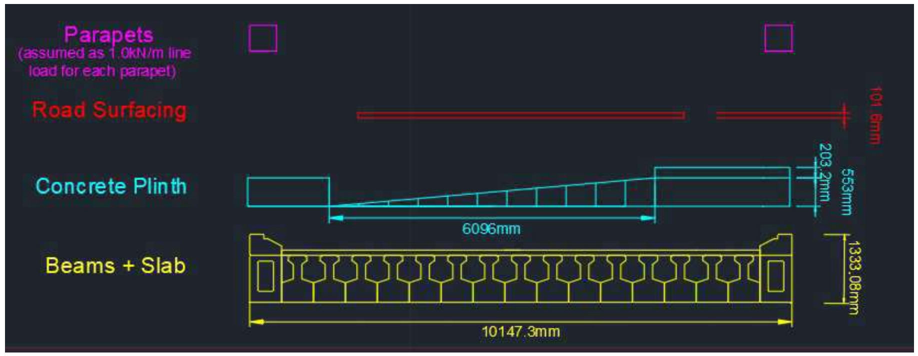

Figure 6: Breakdown of superimposed dead load.

Historical Drawings

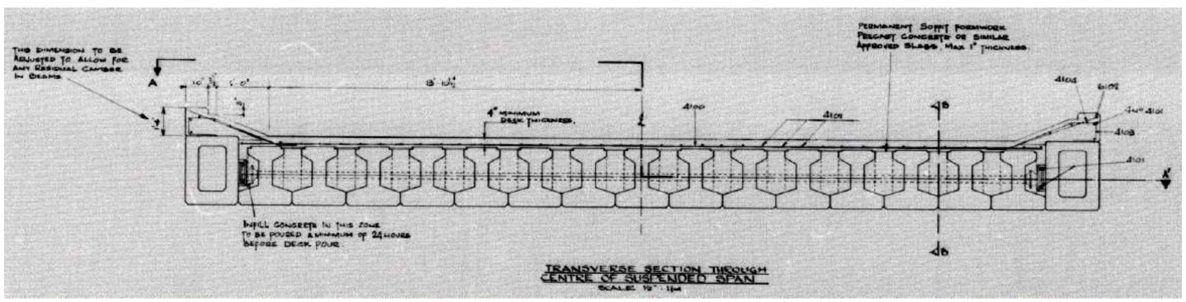

Figure 7: Cross-section of beams and slab at mid-span from historical drawing.

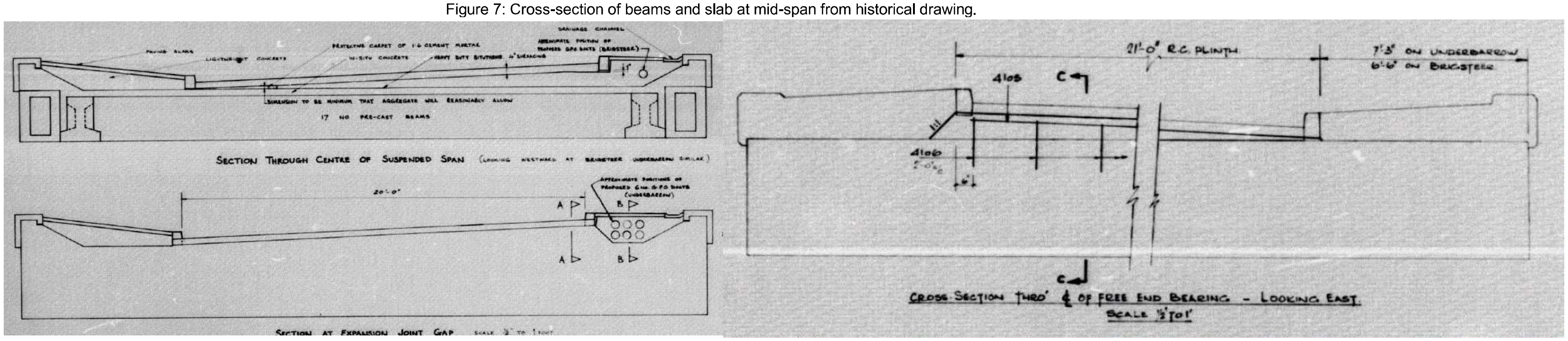

Figure 8: Cross-section of bridge from historical drawing.

SUBJECT	CALCULATIONS	OUTPUT

Model Load Application

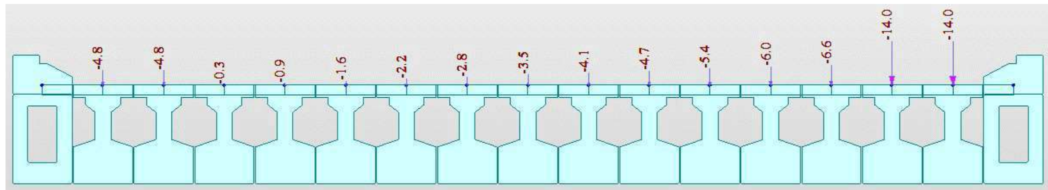

Figure 9: Concrete plinth load application in MIDAS model.

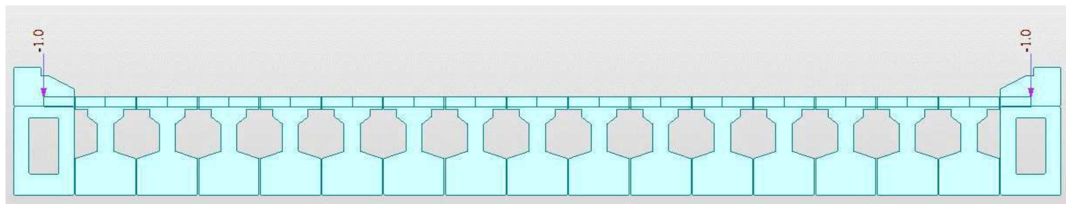

Figure 10: Parapet load application in MIDAS model.

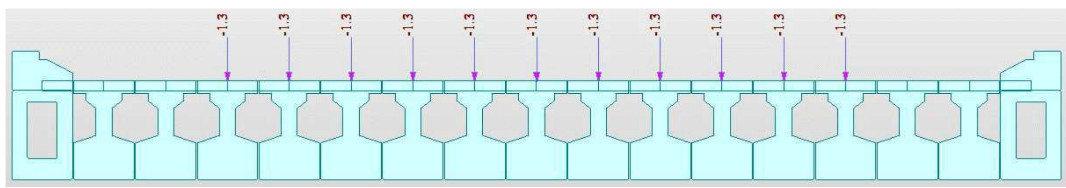

Figure 11: Road surfacing load application in MIDAS model.

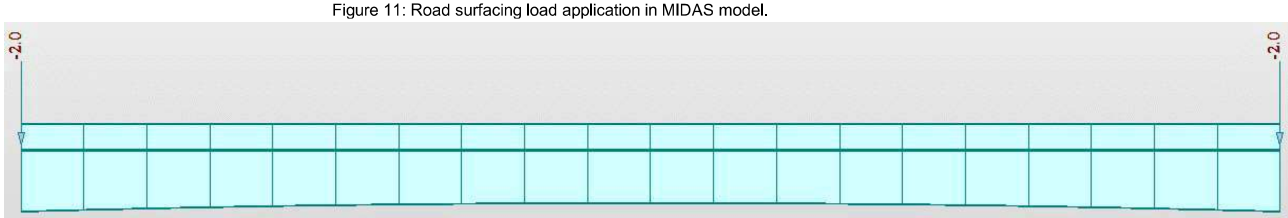

Figure 12: Upper nib load application in MIDAS model.

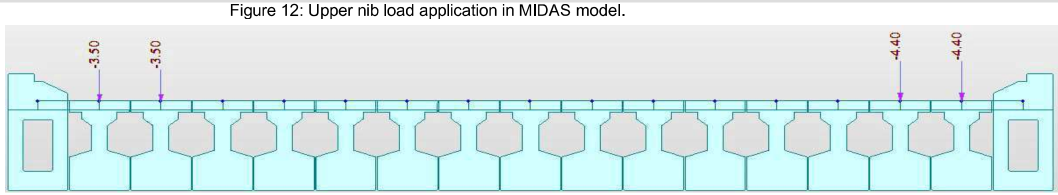

Figure 13: Pedestrian ALL application in MIDAS model.

[redacted]	DOCUMENT NO.	SHEET 7 OF 26

SUBJECT

SUBJECT	CALCULATIONS	OUTPUT

Introduction

This worksheet contains the output of the grillage of Underbarrow modelled in Midas. The vertical reaction at each node subject to serviceability and ultimate loading states is given.

Node	SLS [kN[	ULS SDL [kN]	ULS CS 454 3t ALL Model 2 [kN]			ULS CS 454 7.5t ALL Model 2 [kN]			ULS CS 454 18t ALL Model 2 [kN]			ULS CS 454 26t ALL Model 2 [kN]			ULS CS 454 40t ALL Model 2 [kN]		
			C1	C2 + C3	C4	C1	C2 + C3	C4	C1	C2 + C3	C4	C1	C2 + C3	C4	C1	C2 + C3	C4
1	226.0	288.4	377.2	358.0	274.7	381.6	361.7	278.4	400.0	377.0	293.7	411.5	386.6	303.3	416.3	390.6	307.3
21	225.5	287.9	374.5	355.7	274.3	379.0	359.4	278.0	397.9	375.2	293.8	409.7	385.0	303.6	414.7	389.2	307.8
22	159.0	202.9	258.6	246.2	192.8	261.5	248.6	195.3	273.8	258.9	205.5	281.4	265.3	211.9	284.7	268.0	214.6
42	158.5	202.4	256.7	244.5	192.3	259.7	247.1	194.8	272.3	257.6	205.4	280.3	264.2	212.0	283.6	267.0	214.8
43	147.8	189.0	236.5	225.7	180.8	239.6	228.3	183.5	252.9	239.4	194.5	261.2	246.3	201.4	264.7	249.3	204.4
63	147.5	188.7	235.7	225.0	180.4	238.9	227.7	183.1	252.5	239.0	194.4	261.0	246.1	201.5	264.6	249.1	204.5
64	135.2	174.0	209.4	200.9	175.1	215.5	205.9	180.2	241.0	227.2	201.4	256.9	240.5	214.7	263.7	246.1	220.3
84	135.0	173.8	209.4	200.8	174.2	215.5	205.9	179.2	241.1	227.2	200.6	257.1	240.6	213.9	263.9	246.2	219.6
85	130.7	168.6	192.6	186.1	166.4	197.1	189.8	170.1	215.9	205.4	185.7	227.6	215.2	195.5	232.6	219.4	199.7
105	130.6	168.5	193.2	186.5	165.1	197.6	190.2	168.8	216.3	205.8	184.4	228.0	215.6	194.2	233.0	219.7	198.3
106	127.2	164.4	189.4	182.8	167.3	195.5	187.8	172.4	221.1	209.2	193.8	237.1	222.5	207.1	243.9	228.2	212.8
126	127.2	164.4	190.4	183.6	166.0	196.5	188.6	171.1	221.9	209.9	192.3	237.9	223.1	205.6	244.6	228.8	211.2
127	123.6	160.0	183.5	177.2	164.9	190.0	182.6	170.2	217.1	205.1	192.8	234.0	219.3	207.0	241.2	225.3	213.0
147	123.7	160.1	184.7	178.2	163.2	191.1	183.5	168.6	218.0	205.9	190.9	234.7	219.9	204.9	241.9	225.8	210.9
148	119.9	155.5	174.8	169.2	158.9	180.3	173.8	163.5	203.6	193.2	182.9	218.2	205.4	195.0	224.4	210.5	200.2
168	120.2	155.7	176.1	170.3	156.3	181.5	174.9	160.8	204.4	193.9	179.8	218.7	205.8	191.7	224.7	210.9	196.8
169	116.0	150.6	184.7	176.7	187.1	194.9	185.3	195.6	237.9	221.1	231.4	264.7	243.4	253.8	276.1	252.9	263.3
189	116.3	151.0	186.1	177.9	164.8	196.2	186.4	173.2	238.7	221.8	208.7	265.3	243.9	230.8	276.6	253.3	240.2
190	111.9	145.4	166.0	160.3	149.7	171.5	165.0	154.3	194.8	184.4	173.7	209.4	196.5	185.9	215.6	201.7	191.0
210	112.3	145.8	167.4	161.6	147.2	172.9	166.1	151.8	195.7	185.2	170.8	210.0	197.1	182.7	216.1	202.2	187.8
211	108.0	140.3	166.3	159.8	146.9	172.8	165.2	152.3	199.9	187.8	174.9	216.8	201.9	189.0	224.0	207.9	195.0
231	108.6	141.0	168.0	161.4	145.8	174.4	166.7	151.2	201.3	189.1	173.5	218.1	203.1	187.5	225.2	209.0	193.5
232	104.3	135.5	164.2	157.4	141.1	170.3	162.5	146.1	195.9	183.8	167.5	211.9	197.1	180.8	218.7	202.8	186.5
252	105.0	136.3	166.0	159.0	140.5	172.1	164.1	145.6	197.6	185.3	166.8	213.5	198.6	180.1	220.2	204.2	185.7
253	101.2	131.3	160.2	153.4	132.4	164.6	157.1	136.2	183.4	172.7	151.8	195.1	182.5	161.6	200.1	186.7	165.7
273	102.0	132.3	161.7	154.8	132.2	166.2	158.5	135.9	184.9	174.1	151.5	196.6	183.9	161.3	201.6	188.0	165.4
274	99.7	129.1	170.8	161.9	134.3	176.8	166.9	139.3	202.3	188.1	160.6	218.2	201.4	173.8	225.0	207.0	179.5
294	100.5	130.1	171.7	162.8	134.5	177.8	167.9	139.6	203.5	189.3	160.9	219.5	202.6	174.3	226.3	208.3	179.9
295	104.8	134.6	194.3	182.3	131.4	197.4	184.9	134.1	210.7	196.0	145.1	219.0	202.9	152.0	222.5	205.8	154.9
315	105.4	135.4	194.4	182.5	132.0	197.6	185.2	134.7	211.2	196.5	146.0	219.7	203.6	153.1	223.3	206.6	156.1
316	111.9	143.4	205.2	192.8	138.7	208.2	195.2	141.1	220.4	205.4	151.3	228.1	211.8	157.7	231.4	214.5	160.4
336	111.4	142.8	203.5	191.2	138.2	206.5	193.8	140.7	219.2	204.3	151.3	227.1	210.9	157.9	230.5	213.7	160.7
337	169.8	217.3	313.8	294.4	210.2	318.2	298.1	213.8	336.6	313.5	229.2	348.1	323.1	238.8	353.0	327.1	242.8
357	167.3	214.3	308.5	289.6	207.3	313.0	293.3	211.1	331.9	309.0	226.8	343.7	318.9	236.6	348.7	323.0	240.8
Max Vertical Reaction [kN] =	226.0	288.4	377.2	358.0	274.7	381.6	361.7	278.4	400.0	377.0	293.8	411.5	386.6	303.6	416.3	390.6	307.8

Total bridge load =	4394.1	kN

SUBJECT	CALCULATIONS	OUTPUT

Introdcution

This sheet contains the calculation of crack width limits of cracks at the re-entrant corner of the lower nib. The SLS assessment of crack widths has been carried out in accordance with the methodology outlined in Appendix D of CS 466.

Lower Nib

Input Parameters

Steel Modulus of Elasticity	E_s =	200	Gpa
Concrete Modulus of Elasticity	E_c =	35	GPa
Modular Ratio		5.71	
Diameter of lower nib bending reinforcement	Ø =	19.05	mm
No bars elevation	n =	3	
Depth to reinforcement centreline	$d_{reinforcement\ c.l.}$ =	414.0	mm
Width of section	$w_{section}$ =	596.2	mm

Strain distribution calculation

Hooke's Law

SLS tension in steel	T =	225954.909	N
Stress in steel	σ_{steel} =	264.3	N/mm^2
Strain in steel	ϵ_s =	0.00132	

Strain in concrete by equivalent area

	"a" =	298.1
	"b" =	4886.1
	"c" =	-33632567.6
	"y"=	327.8
Strain in concrete	ϵ_c =	-0.00502

Ref. 3 Equation D.1

Equation D.1 Crack width 1

w = 5.45 mm

Ref. 3 Equation D.2

Equation D.2 Crack width 2

w = 3.17 mm

where:

a =	152.5	mm
y =	13.5	mm
a_{cr} =	40	mm
em	-0.02643	

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SUBJECT

SUBJECT	CALCULATIONS			OUTPUT
Ref. 3 Equation D.3	**Equation D.3 Mean strain**			
		ϵ_m =	-0.02643	
	where:			
		K_1 =	2.3	
		ϵ_1 =	-0.00265	
		K_2 =	0.003	
		b =	596.2 mm	
		h =	450 mm	
		f_{ctm} =	2 N/mm^2	
		E_s =	200000 N/mm^2	
		ϵ_s =	0.00132	
		A_s =	299.48 mm^2	
Ref. 3 Equation D.4	**Equation D.4 Effective area of steel**			
		A_s =	299.48 mm^2	
	where:			
		A_{si} =	285.02 mm^2	
		β_i =	0 °	
	SLS crack width limit	w =	3.17 mm	
Ref. 6 pg. 6	Measure crack width	w_m =	1.5 mm	PASS

SUBJECT	CALCULATIONS				OUTPUT
	Introduction				
	This sheet contains the calculation of the required anchorage length for bending reinforcement in both the upper and lower nib. The anchorage length is calculated based on the yield stress of the reinforcement therefore giving a conservative value for anchorage.				
	Input Parameters				
	Steel yield stress	f_y =	250	N/mm^2	
	Concrete cube strength	f_{cu} =	41.4	N/mm^2	
	Condition factor	C =	0.9		
	Upper Nib				
Ref. 2 Equation 9.1a	Anchorage resistance required before yield	F_{ub} =	64130.165	N	
Ref. 2 Equation 9.1b	Average anchorage bond strength over effective le	f_{ub} =	1.7	N/mm^2	
	where:				
Ref. 2 Equation 9.1b		k =	1		
Ref. 2 Table 9.1		β =	0.39		
		f_{cu} =	37.26	N/mm^2	
Ref. 2 Table 2.13a		γ_{mb} =	1.4		
Ref. 2 Equation 9.1b		k_{cov} =	1		
Ref. 2 Equation 9.1b		a_{con} =	0.4		
		c =	76.2		
Ref. 5 pg. 10		ϕ =	19.1	mm	
		L_a =	210.1	mm	
	Length of upper nib bending reinforcement		880	mm	Max length usable for tie = 669.9mm
	Max. length usable for tie		669.9	mm	
	Lower Nib				
Ref. 2 Equation 9.1a	Anchorage resistance required before yield	F_{ub} =	28502.296	N	
Ref. 2 Equation 9.1b	Average anchorage bond strength over effective le	f_{ub} =	1.7	N/mm^2	
	where:				
Ref. 2 Equation 9.1b		k =	1		
Ref. 2 Table 9.1		β =	0.39		
		$f_{cu,factored}$ =	37.26	N/mm^2	
Ref. 2 Table 2.13a		γ_{mb} =	1.4		
Ref. 2 Equation 9.1b		k_{cov} =	1		
Ref. 2 Equation 9.1b		a_{con} =	0.4		
		c =	76.2		
Ref. 5 pg. 10		ϕ =	12.7	mm	Max length usable for tie = 945.0mm
		L_a =	105.02877	mm	
	Length of upper nib bending reinforcement		1050	mm	
	Max. length usable for tie		**945.0**	mm	

SUBJECT	CALCULATIONS	OUTPUT

Introduction

This sheet assesses the upper nib of Underbarrow in accordance with strut-and-tie model E.3 of CS 466.

Figure E.3 Illustrative example of strut-and-tie model for a half-joint with long nib reinforcement

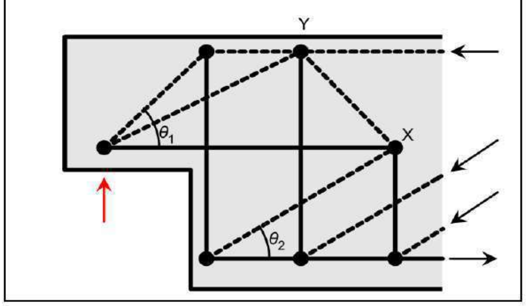

Figure 1: STM layout E.3 in accordance with CS 466.

Input Parameters

Material Strengths

Ref 5. pg. 4

Concrete cube strength	f_{cu} =	41.4	N/mm^2
Steel yield stress	f_y =	250	N/mm^2
Condition Factor	C =	0.9	

Half Joint Dimensions

Ref. 5 pg. 9

	Breadth [mm]	Horizontal [mm]	Vertical [mm]
Lower nib	596.2	310.0	500.0
Upper nib (external)	596.2	305.0	450.0
Upper nib (internal)	596.2	305.0	450.0

Bearing Dimensions

Ref. 5 pg. 3

Width =	146	mm
Length =	285.8	mm
Height =	78.1	mm
Centreline distance from concrete face =	155	mm

Reinforcement

Ref. 5 pg. 10

	Bar diameter [mm]	Cover [mm]	No. of bars (elevation)	Spacing (elevation) [mm]
Upper Nib				
Shear	20	40	3	
Bending	20	40	3	152.4
In deck cantilever				
Shear	19	80	3	
Bending	20	50	4	152.4
Lower Nib				
Shear	18	70		101.6
Bending	12	80		152.4
Top of drop-in span:				
Shear	20	35	3	

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SUBJECT

SUBJECT	CALCULATIONS	OUTPUT

STM Element Summary

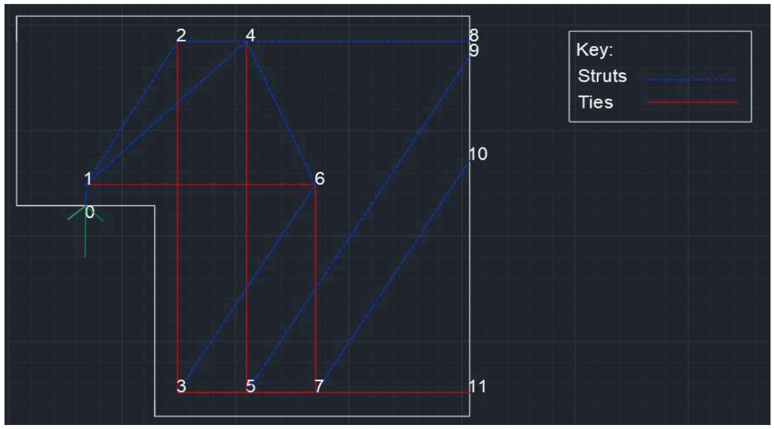

Figure 2: STM layout and node numbering for STM E.3.

Element	Horizontal Length [mm]	Vertical Length [mm]	Absolute Length [mm]	Inclination [◦]	Unit Force [kN]
Strut 0 - 1	0.0	40.0	40.0	90.0	1.00
Strut 1 - 2	202.5	340.0	395.7	59.2	0.97
Strut 1 - 4	354.9	340.0	491.5	43.8	0.24
Tie 1 - 6	507.3	0.0	507.3	0.0	0.67
Tie 2 - 3	0.0	833.5	833.5	90.0	0.83
Strut 2 - 4	152.4	0.0	152.4	0.0	0.50
Strut 3 - 6	304.8	493.5	580.0	58.3	0.98
Tie 3 - 5	152.4	0.0	152.4	0.0	0.52
Tie 4 - 5	0.0	833.5	833.5	90.0	0.51
Strut 4 - 6	152.4	340.0	372.6	65.9	0.38
Strut 4 - 8	492.6	0.0	492.6	0.0	0.52
Tie 5 - 7	152.4	0.0	152.4	0.0	0.83
Strut 5 - 9	492.6	797.6	937.4	58.3	0.60
Tie 6 - 7	0.0	493.5	493.5	90.0	0.49
Strut 7 - 10	340.2	550.8	647.4	58.3	0.57
Tie 7 - 11	340.2	0.0	340.2	0.0	1.13

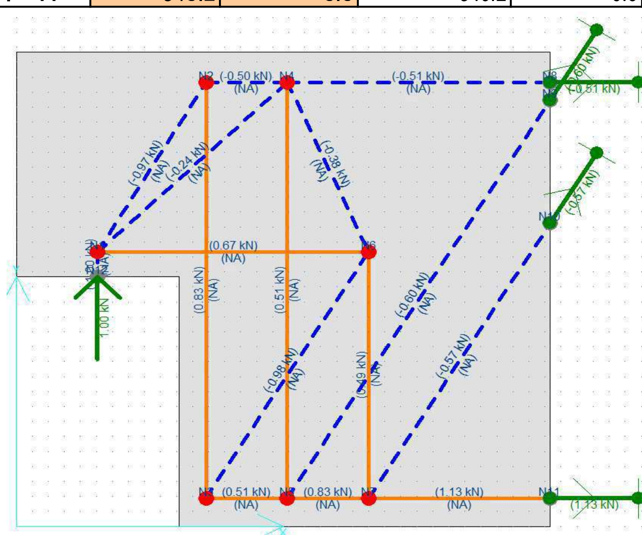

Figure 3: STM load distribution from unit force application using CAST software.

SUBJECT	CALCULATIONS	OUTPUT

STM Element Resistances

(NOTE: The width of concrete struts has been assigned as 80mm and assumed to act across the width of one beam in elevation. The use of 80mm wide struts satisfies cover requirements of the half joint. No further sensitivity checks of struts has been executed as failure is assumed and has been proven to occur within the ties of the STM model.

Strut 0 -1

Width =	80.0	mm
Area =	47696.0	mm^2
Compressive strength =	1777.2	kN

Strut 1 - 2

Width =	80.0	mm
Area =	47696.0	mm^2
Compressive strength =	1777.2	kN

Strut 1 - 4

Width =	80.0	mm
Area =	47696.0	mm^2
Compressive strength =	1777.2	kN

Tie 1 - 6

No bars plan =	1	
No bars elevation =	3	
Total Area Steel =	942.5	mm^2
Tensile strength =	212.1	kN

Tie 2 - 3

No bars plan =	1	
No bars elevation =	3	
Total Area Steel =	942.5	mm^2
Tensile strength =	212.1	kN

Strut 2 - 4

Width =	80.0	mm
Area =	47696.0	mm^2
Compressive strength =	1777.2	kN

Strut 3 - 6

Width =	80.0	mm
Area =	47696.0	mm^2
Compressive strength =	1777.2	kN

Tie 3 - 5

No bars plan =	1	
No bars elevation =	4	
Total Area Steel =	1256.6	mm^2
Tensile strength =	282.7	kN

Tie 4 - 5

No bars plan =	1	
No bars elevation =	3	
Total Area Steel =	850.6	mm^2
Tensile strength =	191.4	kN

Strut 4 - 6

Width =	80.0	mm
Area =	47696.0	mm^2
Compressive strength =	1777.2	kN

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SUBJECT

SUBJECT	CALCULATIONS	OUTPUT

Strut 4 - 8

Width =	80.0	mm
Area =	47696.0	mm^2
Compressive strength =	1777.2	kN

Tie 5 - 7

No bars plan =	1	
No bars elevation =	4	
Total Area Steel =	1256.6	mm^2
Tensile strength =	282.7	kN

Strut 5 - 9

Width =	80.0	mm
Area =	47696.0	mm^2
Compressive strength =	1777.2	kN

Tie 6 - 7

No bars plan =	1	
No bars elevation =	3	
Total Area Steel =	850.6	mm^2
Tensile strength =	191.4	kN

Strut 7 - 10

Width =	80.0	mm
Area =	47696.0	mm^2
Compressive strength =	1777.2	kN

Tie 7 - 11

No bars plan =	1	
No bars elevation =	3	
Total Area Steel =	942.5	mm^2
Tensile strength =	212.1	kN

Stress at nodes

Reference			
Ref. 4 Exp (6.56)	v' =	0.85096	
Ref. 4 Exp (3.15)	f_{cd} =	21.114	N/mm^2
Ref. 4 3.1.6 (1) & NA Table 2.1N	a_{cc} =	0.85	
	γ =	1.5	

Node	Type	Design Compressive Stress Resistance [N/mm^2]	Unit Compressive force [N/mm^2]
1	CCT	15.3	0.046
2	CCT	15.3	0.031
3	CTT	13.5	0.021
4	CCT	15.3	0.034
5	CTT	13.5	0.013
6	CTT	13.5	0.029
7	CTT	13.5	0.012

SUBJECT	CALCULATIONS	OUTPUT

Introduction

This sheet assesses the upper nib of Underbarrow in accordance with strut-and-tie model E.15 of CS 466.

Figure E.15 Illustrative example of a strut-and-tie model for a system with diagonal bars

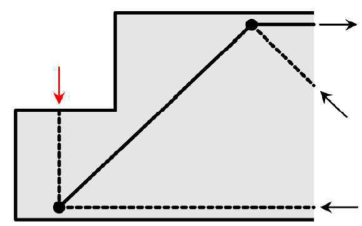


Figure 1: STM layout E.15 in accordance with CS 466.

Input Parameters

Material Strengths

Ref 5. pg. 4

Concrete cube strength	f_{cu} =	41.4	N/mm^2
Steel yield stress	f_y =	250	N/mm^2
Condition factor	C =	0.9	

Half Joint Dimensions

Ref. 5 pg. 9

	Breadth [mm]	Horizontal [mm]	Vertical [mm]
Lower nib	596.2	310	500
Upper nib (external)	596.2	305	450
Upper nib (internal)	596.2	305	450

Bearing Dimensions

Ref. 5 pg. 3

Width =	146	mm
Length =	285.8	mm
Height =	78.1	mm
Centreline distance from concrete face =	155	mm

Reinforcement

Ref. 5 pg. 10

	Bar diameter [mm]	Cover [mm]	No. bars	Spacing (elevation) [mm]
Upper Nib				
Shear	20	40	3	
Bending	20	40	3	152.4
In deck cantilever				
Shear	19	80	3	
Bending	20	50	4	152.4
Diagonal	19.05		4	
Lower Nib				
Shear	18	70		101.6
Bending	12	80		152.4
Top of drop-in span:				
Shear	20	35	3	

SUBJECT	CALCULATIONS	OUTPUT

STM Element Summary

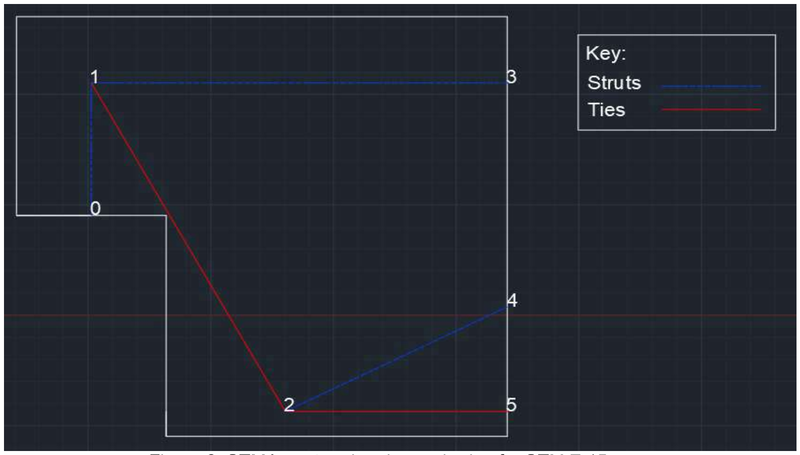


Figure 2: STM layout and node numbering for STM E.15.

Element	Horizontal Lengt	Vertical Length	Absolute Length	Inclination	Unit force
Strut 0 - 1	0	300	300	90	1
Tie 1 - 2	395.5	743.5	842.1475524	61.98954528	1.133
Strut 1 - 3	847.5	0	847.5	0	0.532
Strut 2 - 4	451.0234	237.17	509.579941	27.73757275	2.155
Tie 2 - 5	452	0	452	0	2.441

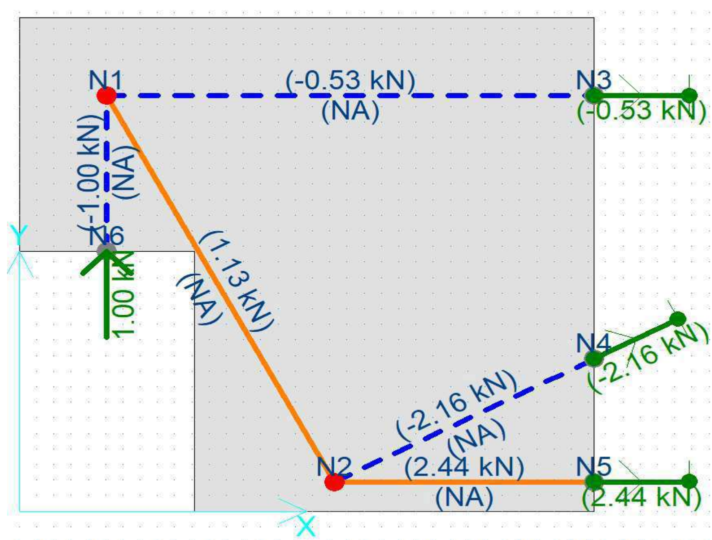


Figure 3: STM load distribution from unit force application using CAST software.

SUBJECT	CALCULATIONS	OUTPUT

STM Element Resistances

(NOTE: The width of concrete struts has been assigned as 80mm and assumed to act across the width of one beam in elevation. The use of 80mm wide struts satisfies cover requirements of the half joint. No further sensitivity checks of struts has been executed as failure is assumed and has been proven to occur within the ties of the STM model.

Strut 0 -1

Width =	80.0	mm
Area =	47696.0	mm^2
Compressive strength =	1777.2	kN

Tie 1 - 2

No bars plan =	1	
No bars elevation =	4	
Total Area Steel =	1140.1	mm^2
Tensile strength =	256.5	kN

Strut 1 - 3

Width =	80.0	mm
Area =	47696.0	mm^2
Compressive strength =	1777.2	kN

Strut 2 - 4

Width =	80.0	mm
Area =	47696.0	mm^2
Compressive strength =	1777.2	kN

Tie 2 - 5

No bars plan =	1	
No bars elevation =	4	
Total Area Steel =	1256.6	mm^2
Tensile strength =	282.7	kN

Stress at nodes

Ref.			
Ref. 4 Exp (6.56)	v' =	0.85096	
Ref. 4 Exp (3.15)	f_{cd} =	21.114	N/mm^2
Ref. 4 3.1.6 (1) & NA	a_{cc} =	0.85	
	γ =	1.5	

Node	Type	Design Compressive Stress Resistance [N/mm^2]	Unit Compressive force [N/mm^2]
1	CCT	15.3	0.032
2	CTT	13.5	0.045

SUBJECT	CALCULATIONS	OUTPUT

Introduction

This sheet assesses the lower nib of Underbarrow in accordance with strut-and-tie model E.16 of CS 466.

Figure E.16 Illustrative example of a strut-and-tie model for a system with vertical bars

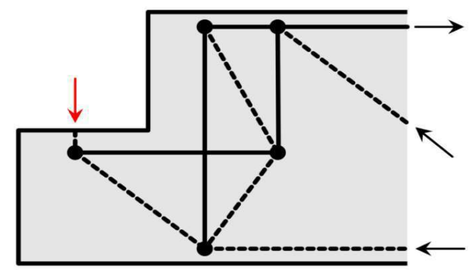


Figure 1: STM layout E.16 in accordance with CS 466.

Input Parameters

Material Strengths

Ref 5. pg. 4

Concrete cube strength	f_{cu} =	41.4	N/mm^2
Steel yield stress	f_y =	250	N/mm^2
Condition factor	C =	0.9	

Half Joint Dimensions

Ref. 5 pg. 9

	Breadth [mm]	Horizontal [mm]	Vertical [mm]
Lower nib	596.2	310	500
Upper nib (external)	596.2	305	450
Upper nib (internal)	596.2	305	450

Bearing Dimensions

Ref. 5 pg. 3

Width =	146	mm
Length =	285.8	mm
Height =	78.1	mm
Centreline distance from concrete face =	155	mm

Reinforcement

Ref. 5 pg. 10

	Bar diameter [mm]	Cover [mm]	No bars elevation	Spacing (elevation) [mm]
Upper Nib				
Shear	20	40	3	
Bending	20	40	3	152.4
In deck cantilever				
Shear	19	80	3	
Bending	20	50	4	152.4
Lower Nib				
Shear	18	70		101.6
Bending	12	80		152.4
Top of drop-in span:				
Shear	20	35	3	

SUBJECT	CALCULATIONS	OUTPUT

STM Element Summary

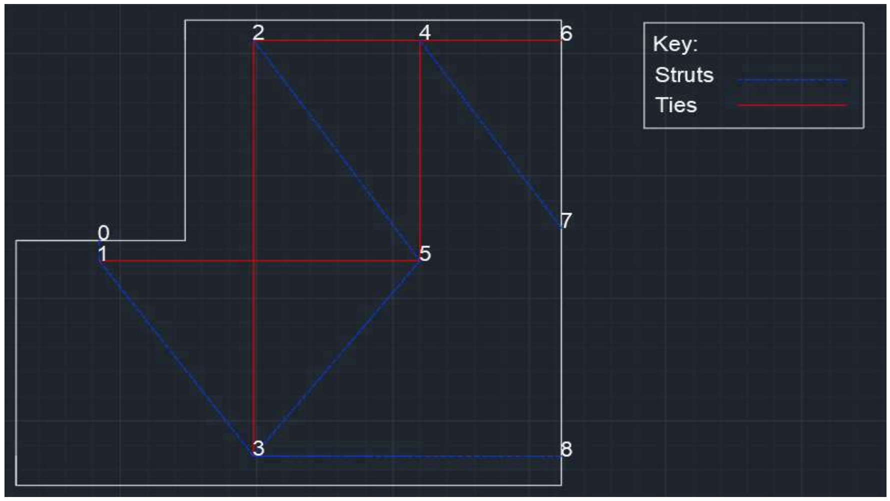


Figure 2: STM layout and node numbering for STM E.16.

Element	Horizontal Length [mm]	Vertical Length [mm]	Absolute Length [mm]	Inclination [◦]	Unit Force [kN]
Strut 0 - 1	0	41.35	41.35	90	1
Strut 1 - 3	283.225	398.65	489.0176102	54.60771759	1.223
Tie 1 - 5	588.0251	0	588.0251	0	0.704
Tie 2 - 3	0	848.65	848.65	90	1.019
Tie 2 - 4	304.8	0	304.8	0	0.69
Strut 2 - 5	304.8	449.8772	543.4082582	55.88164445	1.23
Strut 3 - 5	304.8	398.65	501.8215445	52.59917818	0.023
Strut 3 - 8	564.275	0	564.275	0	0.69
Tie 4 - 5	0	450	450	90	1
Tie 4 - 6	259.4758	0	259.4758	0	1.367
Strut 4 - 7	259.4758	382.4488	462.163148	55.84471953	1.208

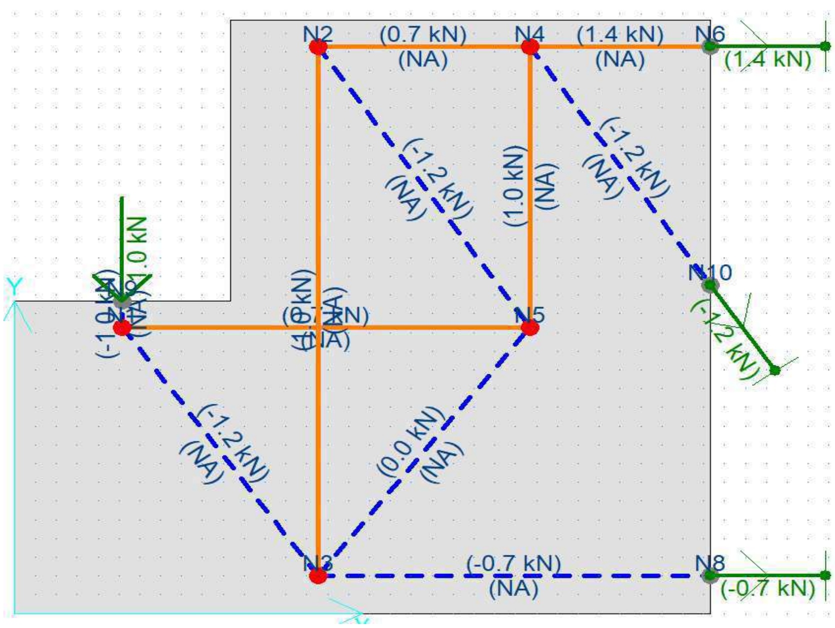


Figure 3: STM load distribution from unit force application using CAST software.

SUBJECT	CALCULATIONS	OUTPUT

STM Element Resistances

(NOTE: The width of concrete struts has been assigned as 80mm and assumed to act across the width of one beam in elevation. The use of 80mm wide struts satisfies cover requirements of the half joint. No further sensitivity checks of struts has been executed as failure is assumed and has been proven to occur within the ties of the STM model.

Strut 0 -1

Width =	80	mm
Area =	47696	mm^2
Compressive strength =	1777.2	kN

Strut 1 - 3

Width =	80.0	mm
Area =	47696	mm^2
Compressive strength =	1777.2	kN

Tie 1 - 5

No bars plan =	1	
No bars elevation =	4	
Total Area Steel =	452	mm^2
Tensile strength =	101.8	kN

Tie 2 - 3

No bars plan =	2	
No bars elevation =	6	
Total Area Steel =	3054	mm^2
Tensile strength =	687.1	kN

Tie 2 - 4

No bars plan =	1	
No bars elevation =	4	
Total Area Steel =	452	mm^2
Tensile strength =	101.8	kN

Strut 2 - 5

Width =	80	mm
Area =	47696	mm^2
Compressive strength =	1777.2	kN

Strut 3 - 5

Width =	80	mm
Area =	47696	mm^2
Compressive strength =	1777.2	kN

Strut 3 - 8

Width =	80	mm
Area =	47696	mm^2
Compressive strength =	1777.2	kN

Tie 4 - 5

No bars plan =	2	
No bars elevation =	6	
Total Area Steel =	3054	mm^2
Tensile strength =	687.1	kN

Tie 4 - 6

No bars plan =	1	
No bars elevation =	4	
Total Area Steel =	452	mm^2
Tensile strength =	101.8	kN

SUBJECT	CALCULATIONS	OUTPUT

Strut 4 - 7

Width = 80 mm
Area = 47696 mm^2
Compressive strength = 1777.2 kN

Stress at nodes

Ref. 4 Exp (6.56) — v' = 0.85
Ref. 4 Exp (3.15) — f_{cd} = 21.1 N/mm^2
Ref. 4 3.1.6 (1) & NA — a_{cc} = 0.85
γ = 1.5

Node	Type	Design Compressive Stress Resistance [N/mm^2]	Unit Compressive force [N/mm^2]
1	CCT	15.3	0.047
2	CCT	15.3	0.026
3	CTT	13.5	0.041
4	CCT	15.3	0.025
5	CTT	13.5	0.026

SUBJECT	CALCULATIONS	OUTPUT

Introduction

This sheet assesses the lower nib of Underbarrow in accordance with strut-and-tie model E.9 of CS 466.

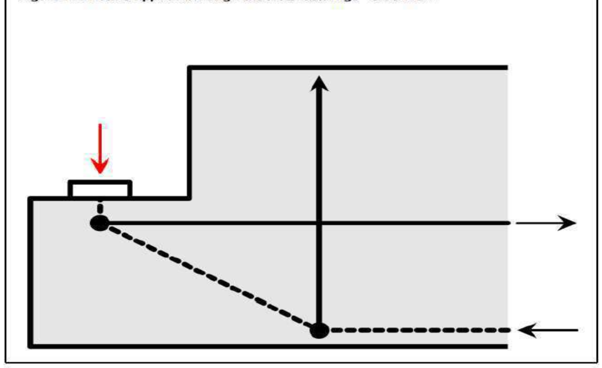


Figure E.9 Loads applied through discrete bearings - side view

Figure 1: STM layout E.9 in accordance with CS 466.

Input Parameters

Material Strengths

Ref 5. pg. 4

Concrete cube strength	f_{cu} =	41.4	N/mm^2
Steel yield stress	f_y =	250	N/mm^2
Condition factor	C =	0.9	

Half Joint Dimensions

Ref. 5 pg. 9

	Breadth [mm]	Horizontal [mm]	Vertical [mm]
Lower nib	596.2	310	500
Upper nib (external)	596.2	305	450
Upper nib (internal)	596.2	305	450

Bearing Dimensions

Ref. 5 pg. 3

Width	$w_{bearing}$ =	146	mm
Length	$l_{bearing}$ =	285.8	mm
Height	$h_{bearing}$ =	78.1	mm
Centreline distance from concrete face		155	mm

Reinforcement

Ref. 5 pg. 10

	Bar diameter [mm]	Cover [mm]	Spacing (plan) [mm]	Spacing (elevation) [mm]
Upper Nib				
Shear	20	40	3	
Bending	20	40	3	152.4
In deck cantilever				
Shear	19	80	3	
Bending	20	50	4	152.4
Lower Nib				
Shear	18	70		101.6
Bending	12	80	4	152.4
Top of drop-in span:				
Shear	20	35	3	

SUBJECT	CALCULATIONS	OUTPUT

STM Element Summary

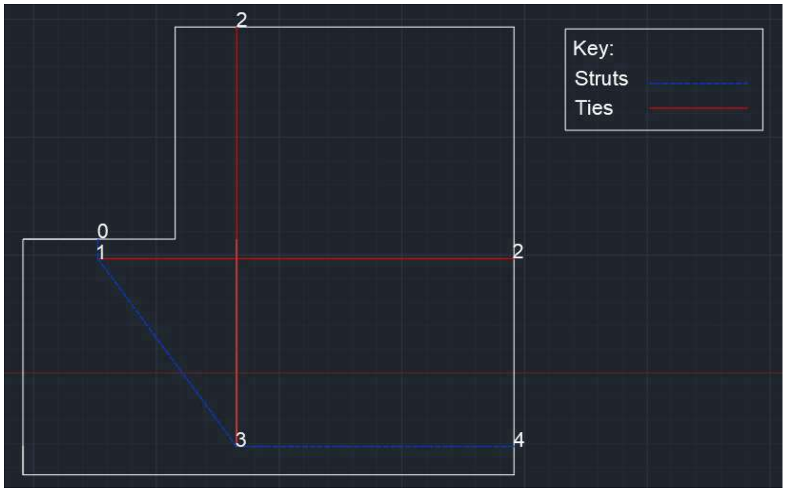


Figure 2: STM layout and node numbering for STM E.9.

Element	Horizontal Length [mm]	Vertical Length [mm]	Absolute Length [mm]	Inclination [◦]	Unit Force [kN]
Strut 0 - 1	0	41.35	41.35	90	1
Tie 1 - 2	890	0	890	0	0.7
Strut 1 - 3	847.5	398.65	936.5778518	25.19151829	1.22
Tie 2 - 3	0	890	890	90	1
Strut 3 - 4	564.8	0	564.8	0	0.7

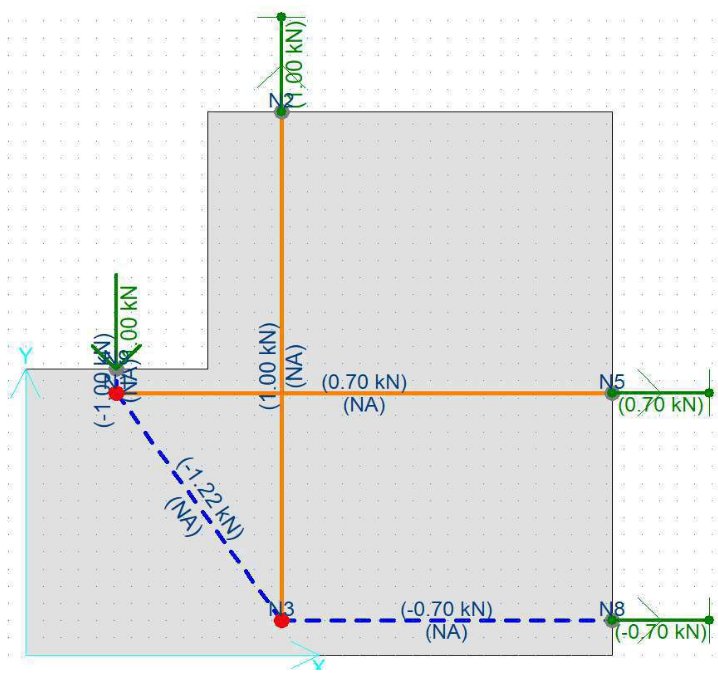


Figure 3: STM load distribution from unit force application using CAST software.

STM Element Resistances

(NOTE: The width of concrete struts has been assigned as 80mm and assumed to act across the width of one beam in elevation. The use of 80mm wide struts satisfies cover requirements of the half joint. No further sensitivity checks of struts has been executed as failure is assumed and has been proven to occur within the ties of the STM model.

Strut 0 -1

Width	w_{0-1} =	80.0	mm
Area	A_{0-1} =	47696.0	mm^2
Compressive strength	C_{Rd0-1} =	1777.2	kN

Tie 1 - 2

No bars plan	1	
No bars elevation	4	
Total Area Steel	452.4	mm^2
Tensile strength	101.8	kN

Strut 1 - 3

Width	w_{0-1} =	80.0	mm
Area	A_{0-1} =	47696.0	mm^2
Compressive strength	C_{Rd0-1} =	1777.2	kN

Tie 2 - 3

No bars plan	1	
No bars elevation	3	
Total Area Steel	942.5	mm^2
Tensile strength	212.1	kN

Strut 3 - 4

Width	w_{0-1} =	80.0	mm
Area	A_{0-1} =	47696.0	mm^2
Compressive strength	C_{Rd0-1} =	1777.2	kN

Stress at nodes

Reference			
Ref. 4 Exp (6.56)	v' =	0.85096	
Ref. 4 Exp (3.15)	f_{cd} =	21.114	N/mm^2
Ref. 4 3.1.6 (1) & NA	a_{cc} =	0.85	
	γ =	1.5	

Node	Type	Design Compressive Stress Resistance [N/mm^2]	Unit Compressive force [N/mm^2]
1	CCT	15.3	0.0
3	CTT	13.5	0.0

E.3

STM Member Summary

Member	Resistance	SLS SDL Member Force	SLS SDL Utilisation	ULS SDL Member Force	ULS SDL Utilisation	ULS + CS 454 3t ALL Model 2 Member Force	ULS + CS 454 3t ALL Model 2 Utilisation	ULS + CS 454 7.5t ALL Model 2 Member Force	ULS + CS 454 7.5t ALL Model 2 Utilisation	ULS + CS 454 18t ALL Model 2 Member Force	ULS + CS 454 18t ALL Model 2 Utilisation	ULS + CS 454 26t ALL Model 2 Member Force	ULS + CS 454 26t ALL Model 2 Utilisation	ULS + CS 454 40t ALL Model 2 Member Force	ULS + CS 454 40t ALL Model 2 Utilisation
Strut 0 - 1	1777.2	226.0	0.1	288.4	0.2	377.2	0.2	381.6	0.2	400.0	0.2	411.5	0.2	416.3	0.2
Strut 1 - 2	1777.2	219.2	0.1	279.7	0.2	365.9	0.2	370.1	0.2	388.0	0.2	399.1	0.2	403.8	0.2
Strut 1 - 4	1777.2	54.5	0.0	69.5	0.0	90.9	0.1	92.0	0.1	96.4	0.1	99.2	0.1	100.3	0.1
Tie 1 - 6	212.1	151.4	0.7	193.2	0.9	252.7	1.2	255.7	1.2	268.0	1.3	275.7	1.3	278.9	1.3
Tie 2 - 3	212.1	188.2	0.9	240.2	1.1	314.2	1.5	317.9	1.5	333.2	1.6	342.7	1.6	346.8	1.6
Strut 2 - 4	1777.2	112.1	0.1	143.0	0.1	187.1	0.1	189.3	0.1	198.4	0.1	204.1	0.1	206.5	0.1
Strut 3 - 6	1777.2	221.2	0.1	282.3	0.2	369.3	0.2	373.6	0.2	391.6	0.2	402.8	0.2	407.6	0.2
Tie 3 - 5	282.7	116.4	0.4	148.5	0.5	194.3	0.7	196.5	0.7	206.0	0.7	211.9	0.7	214.4	0.8
Tie 4 - 5	191.4	116.1	0.6	148.2	0.8	193.9	1.0	196.1	1.0	205.6	1.1	211.5	1.1	214.0	1.1
Strut 4 - 6	1777.2	86.1	0.0	109.9	0.1	143.7	0.1	145.4	0.1	152.4	0.1	156.8	0.1	158.6	0.1
Strut 4 - 8	1777.2	116.4	0.1	148.5	0.1	194.3	0.1	196.5	0.1	206.0	0.1	211.9	0.1	214.4	0.1
Tie 5 - 7	282.7	188.0	0.7	239.9	0.8	313.8	1.1	317.5	1.1	332.8	1.2	342.3	1.2	346.4	1.2
Strut 5 - 9	1777.2	136.5	0.1	174.2	0.1	227.8	0.1	230.5	0.1	241.6	0.1	248.5	0.1	251.5	0.1
Tie 6 - 7	191.4	109.8	0.6	140.2	0.7	183.3	1.0	185.4	1.0	194.4	1.0	200.0	1.0	202.3	1.1
Strut 7 - 10	1777.2	129.0	0.1	164.7	0.1	215.4	0.1	217.9	0.1	228.4	0.1	234.9	0.1	237.7	0.1
Tie 7 - 11	212.1	255.8	1.2	326.5	1.5	427.0	2.0	432.0	2.0	452.8	2.1	465.8	2.2	471.3	2.2

STM Node Summary

Node	Compressive Resistance	Compressive stress	Utilisation	Compressive stress	Utilisation	Compressive stress	Utilisation	Compressive stress	Utilisation	Compressive stress	Utilisation	Compressive stress	Utilisation	Compressive stress	Utilisation
1	15.3	10.5	0.7	13.4	0.9	17.5	1.1	17.7	1.2	18.5	1.2	19.1	1.2	19.3	1.3
2	15.3	6.9	0.5	8.9	0.6	11.6	0.8	11.7	0.8	12.3	0.8	12.6	0.8	12.8	0.8
3	13.5	4.6	0.3	5.9	0.4	7.7	0.6	7.8	0.6	8.2	0.6	8.4	0.6	8.5	0.6
4	15.3	7.7	0.5	9.9	0.6	12.9	0.8	13.1	0.9	13.7	0.9	14.1	0.9	14.3	0.9
5	13.5	2.9	0.2	3.7	0.3	4.8	0.4	4.8	0.4	5.1	0.4	5.2	0.4	5.3	0.4
6	13.5	6.4	0.5	8.2	0.6	10.8	0.8	10.9	0.8	11.4	0.8	11.7	0.9	11.9	0.9
7	13.5	2.7	0.2	3.5	0.3	4.5	0.3	4.6	0.3	4.8	0.4	4.9	0.4	5.0	0.4

E.15

STM Member Summary

Member	Resistance	SLS SDL Member Force	SLS SDL Utilisation	ULS SDL Member Force	ULS SDL Utilisation	ULS + CS 454 3t ALL Model 2 Member Force	ULS + CS 454 3t ALL Model 2 Utilisation	ULS + CS 454 7.5t ALL Model 2 Member Force	ULS + CS 454 7.5t ALL Model 2 Utilisation	ULS + CS 454 18t ALL Model 2 Member Force	ULS + CS 454 18t ALL Model 2 Utilisation	ULS + CS 454 26t ALL Model 2 Member Force	ULS + CS 454 26t ALL Model 2 Utilisation	ULS + CS 454 40t ALL Model 2 Member Force	ULS + CS 454 40t ALL Model 2 Utilisation
Strut 0 - 1	1777.2	226.0	0.1	288.4	0.2	377.2	0.2	381.6	0.2	400.0	0.2	411.5	0.2	416.3	0.2
Tie 1 - 2	256.5	256.0	1.0	326.8	1.3	427.4	1.7	432.3	1.7	453.2	1.8	466.2	1.8	471.7	1.8
Strut 1 - 3	1777.2	120.2	0.1	153.4	0.1	200.7	0.1	203.0	0.1	212.8	0.1	218.9	0.1	221.5	0.1
Strut 2 - 4	1777.2	486.9	0.3	621.5	0.3	812.9	0.5	822.3	0.5	861.9	0.5	886.7	0.5	897.2	0.5
Tie 2 - 5	282.7	551.6	2.0	704.0	2.5	920.8	3.3	931.4	3.3	976.3	3.5	1004.4	3.6	1016.3	3.6

STM Node Summary

Node	Compressive Resistance	Compressive stress	Utilisation	Compressive stress	Utilisation	Compressive stress	Utilisation	Compressive stress	Utilisation	Compressive stress	Utilisation	Compressive stress	Utilisation	Compressive stress	Utilisation
1	15.3	7.3	0.5	9.3	0.6	12.1	0.8	12.3	0.8	12.8	0.8	13.2	0.9	13.4	0.9

SUBJECT | CALCULATIONS | OUTPUT

STM Node Summary

Node	Compressive Resistance	Compressive stress	Utilisation	Compressive stress	Utilisation	Compressive stress	Utilisation	Compressive stress	Utilisation	Compressive stress	Utilisation	Compressive stress	Utilisation	Compressive stress	Utilisation
1	15.3	10.5	0.7	13.4	0.9	17.6	1.2	17.8	1.2	18.6	1.2	19.2	1.3	19.4	1.3
2	15.3	5.8	0.4	7.4	0.5	9.7	0.6	9.8	0.6	10.3	0.7	10.6	0.7	10.7	0.7
3	13.5	9.2	0.7	11.7	0.9	15.3	1.1	15.5	1.1	16.2	1.2	16.7	1.2	16.9	1.3
4	15.3	5.7	0.4	7.3	0.5	9.6	0.6	9.7	0.6	10.1	0.7	10.4	0.7	10.5	0.7
5	13.5	5.9	0.4	7.6	0.6	9.9	0.7	10.0	0.7	10.5	0.8	10.8	0.8	10.9	0.8

E.9

STM Member Summary

		SLS SDL		ULS SDL		ULS + CS 454 3t ALL Model 2		ULS + CS 454 7.5t ALL Model 2		ULS + CS 454 18t ALL Model 2		ULS + CS 454 26t ALL Model 2		ULS + CS 454 40t ALL Model 2	
Member	**Resistance**	**Member Force**	**Utilisation**	**Member Force**	**Utilisation**	**Member Force**	**Utilisation**	**Member Force**	**Utilisation**	**Member Force**	**Utilisation**	**Member Force**	**Utilisation**	**Member Force**	**Utilisation**
Strut 0 - 1	1777.2	226.0	0.1	288.4	0.2	377.2	0.2	381.6	0.2	400.0	0.2	411.5	0.2	416.3	0.2
Tie 1 - 2	101.8	158.2	1.6	201.9	2.0	264.0	2.6	267.1	2.6	280.0	2.8	288.0	2.8	291.4	2.9
Strut 1 - 3	1777.2	275.7	0.2	351.8	0.2	460.2	0.3	465.5	0.3	488.0	0.3	502.0	0.3	507.9	0.3
Tie 2 - 3	212.1	226.0	1.1	288.4	1.4	377.2	1.8	381.6	1.8	400.0	1.9	411.5	1.9	416.3	2.0
Strut 3 - 4	1777.2	158.2	0.1	201.9	0.1	264.0	0.1	267.1	0.2	280.0	0.2	288.0	0.2	291.4	0.2

STM Node Summary

Node	Compressive Resistance	Compressive stress	Utilisation	Compressive stress	Utilisation	Compressive stress	Utilisation	Compressive stress	Utilisation	Compressive stress	Utilisation	Compressive stress	Utilisation	Compressive stress	Utilisation
1	15.3	10.5	0.7	13.4	0.9	17.6	1.1	17.8	1.2	18.6	1.2	19.2	1.3	19.4	1.3
3	13.5	9.1	0.7	11.6	0.9	15.2	1.1	15.4	1.1	16.1	1.2	16.6	1.2	16.8	1.2

Conclusion

Underbarrow half joint has failed the check when assessed at SLS and ULS using the strut-and-tie models in accordance with CS 466. The ties emulating the bending reinforcement of the lower nib have failed in both models E.16 (tie 1 - 5) and E.9 (tie 1 - 2) due to the conservative nature of the assessment it is possible that the bending reinforcement has diameter 19.05mm as stated in the historical drawings as opposed to 12.7mm diameter bars used for assessment, therefore giving the joint greater capacity than has been determined.

For the upper nib, failure occurs in model E.3 at Ties 5 - 7 and 7 - 11 and E.15 in Tie 2 - 5. Failure of these ties in the STM model is not necessarily representative of failure of the half joint as the pre-stressed tendons provide the majority of the tensile reistance of the drop-in span rather than bending reinforcement itself.

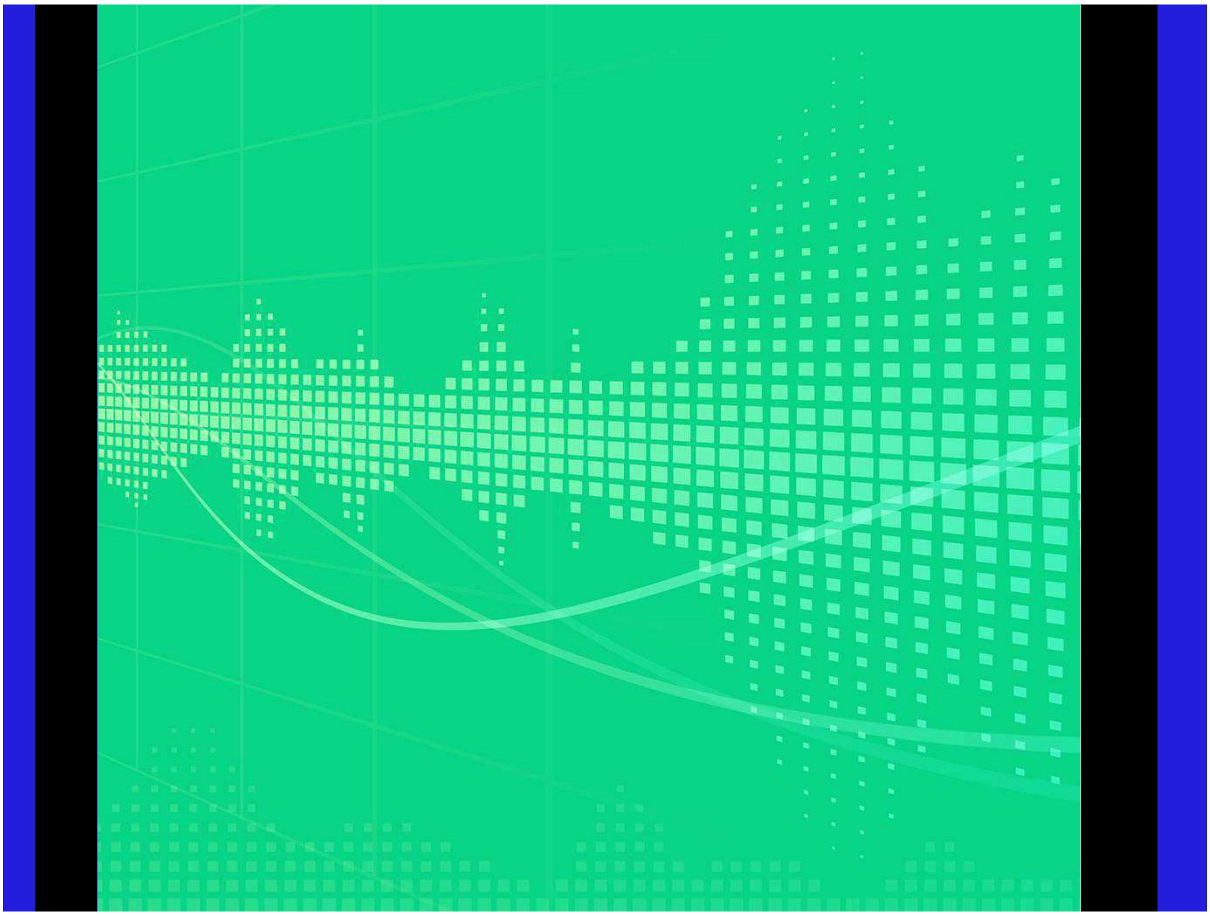
Appendix C. Approval In Principle

Approval In Principle (Half Joint Assessment) – Underbarrow

Document no: BCU00015-JAC-SBR-6330-RP-SL221-CB-008
Revision no: P02

Cumbria County Council
6330

Risk Assessment and Structural Assessment of Post Tensioned and Half Joint Bridges SL240 Brigsteer and SL221 Underbarrow
6 January 2023



Approval In Principle (Half Joint Assessment) – Underbarrow

Client name: Cumbria County Council

Project name: Risk Assessment and Structural Assessment of Post Tensioned and Half Joint Bridges SL240 Brigsteer and SL221 Underbarrow

Client reference: 6330

Project no: BCU00015

Document no: BCU00015-JAC-SBR-6330-RP-SL221-CB-008

Project Manager: [redacted]

Revision no: P02

Prepared by: [redacted]

Date: 6 January 2023

File name: BCU00015-JAC-SBR-6330-RP-SL221-CB-008

Doc status: Revised Following Client Comments

Document history and status

Revision	Date	Description	Author	Checked	Reviewed	Approved
P01	04/10/2022	First Issue	[redacted]			
P02	06/01/2023	Amended Following Client Comments				

Distribution of copies

Revision	Issue approved	Date issued	Issued to	Comments
P01	[redacted]	04/10/2022	[redacted]	Issue to Cumbria County Council
P02		09/01/2023		Issue to Cumbria County Council

[redacted]

Contents

Appendices

Project Details:

Name of project: Risk Assessment and Structural Assessment of Post-Tensioned and Half Joint Bridges SL240 Brigsteer and SL221 Underbarrow.

Name of bridge or structure: Underbarrow

Structure reference no. SL221

Summary: This Approval in Principle covers the assessment methodology for SL221 Underbarrow.

1. HIGHWAY DETAILS

1.1 Type of Highway

Over – Underbarrow Rd (Local road).

Under – A591 Kendal Bypass.

1.2 Design Traffic Speed

Over - 60 mph.

Under – 70 mph.

1.3 Existing Restrictions

There are no signed restrictions.

2. SITE DETAILS

2.1 Obstacles Crossed

A591, Kendal Bypass.

3. PROPOSED STRUCTURE

3.1 Description of Structure and Design Working Life

Underbarrow, constructed in 1970, carries the C5048 single carriageway Underbarrow Road east and west over the A591, Kendal Bypass County Road, west of Kendal. The carriageway over the structure is approximately 6.2m wide with hardened verges measuring 1.1m and 2.4m side north and south respectively.

The superstructure is a single span made up of in-situ concrete cantilevers and a precast concrete beam suspended span. The cantilevers are of post-tensioned voided construction, integral with voided abutments. The suspended span comprises 17No. prestressed pre-tensioned concrete beams and an in-situ reinforced concrete deck slab. The inner beams are inverted T-beams and are transversely post-tensioned. The edge beams are box beams. The suspended span is supported by half-joints at the ends of the cantilevers.

The A591 below is a dual carriageway with a grassed central reserve and grassed verges. There are "limestone pitching" revetments in front/above both abutments.

The half joint form is described as 'solid or box slab with no access to the bearing shelf' and is classified as 'Type A' in accordance with CS 466 (Figure C.3 and Table C.10).

The suspended square span is 18.288m (60' 0") between centrelines of bearings.

The length of each element are as follows:

West Abutment / Cantilever =	18.1m	back of abutment to centreline of half-joint.
Suspended Span =	18.3m	between centrelines of half-joints.
East Abutment / Cantilever =	18.2m	back of abutment to centreline of half-joint.

This AIP seeks approval for the following:

- Quantitative assessment/check, limited to the half-joints only, in accordance with CS 454, CS 455, CS 466 and all relevant documents referenced in the TAS schedule included in Appendix A.

3.2 Structural Type

Single span superstructure comprising in-situ concrete cantilevers, integral with large abutments, and a precast concrete beam suspended span supported on half-joints. The cantilevers are longitudinally post-tensioned and integral with the abutments; both cantilevers and abutment are voided. The suspended span comprises 17 No. prestressed pre-tensioned concrete beams and an in-situ reinforced concrete deck slab that is considered as acting compositely. The inner beams are inverted T-beams and are transversely post-tensioned. The edge beams are box beams, connected to the rest of the deck by reinforcement protruding from the inner side of each beam. The suspended span is supported by half-joints at the ends of the cantilevers.

The west cantilever and integral abutment contains 26 No. post-tensioned cables which are typically at 457.2mm centres. The cables are located within the upper areas of the voided construction, to resist tension due to hogging bending moments, and taper down at either end of the element. The cables which are situated directly above the vertical walls of the voided construction terminate within the walls and do not extend to the half-joints. All the anchorages appear to be recessed into the concrete; although no details are given regarding any capping, it is expected that the recesses were capped following tensioning. At the half-joint the tendons are anchored in the upper area of the deck and do not provide any strength to the lower nib of the half-joint. The strength of the lower nib therefore comes from the reinforced concrete detailing only and acts in a similar manner to a corbel.

The east cantilever and integral abutment contains 26 No. post-tensioned cables which are typically at 457.2mm centres. The cables are located within the upper areas of the voided construction and taper down at either end of the element. The cables which are situated directly above the vertical walls of the voided construction terminate within the walls and do not extend to the half-joints. All the anchorages appear to be recessed into the concrete; although no details are given regarding any capping, it is expected that the recesses were capped following tensioning. At the half-joint the tendons are anchored in the upper area of the deck and do not provide any strength to the lower nib of the half-joint. The strength of the lower nib therefore comes from the reinforced concrete detailing only and acts in a similar manner to a corbel.

3.3 Foundation Type

The available records show that the integral cantilever / abutments are founded on a 230mm thick layer of concrete blinding. Local to the toe and heel, the substrate has been excavated and replaced with class E3/4 mass concrete infill (equivalent to modern-day 50 N/mm^2 concrete).

3.4 Span Arrangements

The clear span between abutments is 48.763m, the suspended span between centrelines of bearings is 18.288m and the length of the integral cantilevers and abutments from the centreline of the half-joint bearings to the back of abutment is 18.2m and 18.1m for the east and west respectively. The overall width of the structure is 10.5m.

3.5 Articulation Arrangements

Historical drawings marked 'record drawing' detail 17.No elastomeric Dunlop Metalastik bearings. Record drawings detail the following for the same type of bearings; 285.75mm x 146mm x 78.13mm thick. The bearings are presumably centred under each of the 17 No. precast beams. Fixity is provided at the east half-joint by 14 No. horizontal bars at 609mm centres between internal beams.

3.6 Road Restraint Systems Requirements

The parapets comprise post and vertical infill railings. There is concern that the parapets do not meet current containment standards.

A VRS, supported on timber posts, is in place at each corner of the structure.

3.7 Proposals for Water Management

The original waterproofing is shown to be heavy duty bitumen, thickness of the waterproofing is not stated. There are no records available to show that the original waterproofing has ever been replaced.

3.8 Proposed arrangements for future maintenance and inspection / inspection for assessment:

3.8.1 Traffic Management

The topside of the structure can be safely inspected without the need of special access equipment or traffic management.

Future maintenance and inspection activities on top of the structure may require traffic management. Depending on the nature of maintenance or inspection work, a single lane closure may be sufficient. If a full closure is required, the diversion route is approximately 8.6 miles which would cause significant disruption to the public (during day-time hours).

There is no safety barrier within the central reserve of the A591 which is simply level, kerbed and grassed. In the event that any maintenance work or inspection of the deck soffit, half joints and substructure are required, there are a number of traffic management options for consideration:

- A closure of the A591 in both directions.
- Lane closures with reduced speed restriction for the carriageway being worked in, TVCBs to provide a temporary barrier between northbound/ southbound carriageways whilst works are undertaken.

Note, in the event of a closure of the A591, the only viable diversion route is through Kendal Town centre and presumably this may be limited to overnight working.

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

In order to access the soffit, half joints and substructure, a Mobile Elevation Work Platform (MEWP) is a necessity in combination with either of the above traffic management options.

3.8.3 Intrusive or further investigations proposed

The July 2022 Half Joint Inspection (see report in appendix B) has revealed that there are concerns as to the accuracy of record drawings as a result it has been recommended by Jacobs that:

- The Client undertakes investigations to confirm the presence, type and details of the post-tensioning and its condition. The recommended investigations shall be outlined in PTSI Risk Management Plan Report, BCU00015-JAC-SBR-6330-RP-SL221-CB-009.
- If, based on the conservative assumptions of tensile strength, the structure fails assessment for Normal Traffic (40/44T) ALL and 45HB Units, intrusive works will be required to verify the material properties and confirm the size / layout of reinforcement.

3.9 Environment and Sustainability

There are currently no proposals for works which will have any significant impact on the environment.

3.10 Durability - materials and finishes/materials strengths assumed and basis of assumptions

3.10.1 Material Strengths

It is noted that there are considerable variations between available design and 'record' information. Variations between the design and construction cannot be clarified as there are no available investigation works that have been undertaken to confirm existing arrangements. However, discrepancies have been confirmed regarding the size of the half-joints (by physical on-site measurements) and the local reinforcement (by on-site ferro-scanning).

Drawings show a concrete class of 'Y ¾' for the in-situ concrete in the east and west integral abutments and cantilevers. Historical material information *(Ministry of Transport, Specification for Road & Bridge works 3rd Edition 1963, Tables A & B)* states that this class of concrete represents a 28-day compressive cube strength of 6000psi (41.4N/mm^2) and maximum aggregate size of 0.75 inches (19mm). Drawings indicate that the classes of concrete used in the suspended span are 'X $^3/_8$' for the precast beams (6000psi psi or 41.4N/mm^2 at transfer and 7500psi or 51.7N/mm^2 at 28 days and max. aggregate size of 9.5mm), 'Y ¾' for the deck (6000psi or 41.4N/mm^2 and max. aggregate size of 19mm).

The historical assessment, carried out 1991-1994, clearly outlines the material assumptions as follows (matching 'record' drawings):

Concrete Strength

Abutments/ Cantilevers:	f_{cu} =	41.4 N/mm^2
Precast Beams:	f_{cu} =	51.7 N/mm^2
Deck Slab:	f_{cu} =	41.4 N/mm^2

Mild Steel Strength

All Elements:	f_y =	250 N/mm^2	(BS4449:1969)

Note, there is no suggestion that these values (for concrete and mild steel) have been verified as a result of material testing.

3.10.2 Condition Factor

Taking account of the cracking noted to the upper and lower nibs within the half joint, and the inability to determine the significance of the cracks without access on to the bearing shelf, it is considered that for assessment purposes, the condition factor should be reduced from unity.

Recommended condition factor = 0.9

In the event that the half joints are determined to be under capacity, the cracks should be considered for further investigation by non-destructive means where possible.

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

Not applicable.

3.12 Resilience and security

Not applicable.

3.13 Year of construction

The structure file states that the year of construction is 1970, which correlates with the drawings and letter correspondence.

3.14 Reason for Assessment

As part of this commission, Jacobs has undertaken Risk Reviews and Risk Assessments to CS465 (Management of post-tensioned concrete bridges) and CS466 (Risk Management and Structural Assessment of Concrete Half-joint Deck Structures).

The Risk Rating for Underbarrow in accordance with the processes laid out in CS466 was concluded to be very high due to the secondary consequential risk and half-joint form meaning it is difficult to access for inspection and maintenance.

CS466 requires that, following the risk assessment for structural assessment, the structure shall be reviewed in accordance with CS451 to determine if a structural assessment is necessary. A structural review has been carried out (RSRF dated 8th November 2022) and this recommended an assessment of the half-joints be carried out.

3.15 Part of structure to be assessed

Only the half-joints are to be assessed as part of this commission.

The assessment processes and basis of assessment for the half joints shall follow the requirements of CS 454 and CS 455 supplemented by the additional requirements of CS 466 (section 6).

An assessment report dated January 1994 produced by Cumbria County Council concludes that the structure has a capacity for 40T Assessment Live Loading and a HB capacity of 30 units as stated on the signed certification (dated 14th February 1995). However, a note on the results summary sheet states that the suspended span and the top slab of the hollow parts of the cantilever will carry 30 units HB loading, but if the HB vehicle travels within 150mm of the kerb, allowing associated HA loading, then the capacity reduces to 14 HB units, limited by the lower nib of the half-joints. SLS checks concluded that the actual crack width is greater than twice the allowable width. The cracking was attributed to poor detailing of reinforcement (lack of diagonal reinforcement within the lower nib) as opposed to overloading.

4. ASSESSMENT CRITERIA

4.1 Actions

4.1.1 Permanent Actions

Dead load and superimposed dead loads in accordance with CS454 appropriate to relevant limit state considered.

The concrete slab is indicated to be constructed from lightweight concrete on record information but this has not been proven. It shall be considered conservatively to have a density in accordance with mass concrete from CS454. The bituminous surfacing shall be considered conservatively to have a density in accordance with bituminous macadam from CS454. In the event that the structure fails by a small margin, sensitivity analysis will be carried out using reduced density values for the lightweight concrete slab and the bituminous surfacing. Material investigations and surfacing thickness cores may then be recommended to confirm the actual density and gauge its effect on the assessment rating.

The permanent loads shall be calculated using the layout of the deck and surfacing shown on record drawings, with the exclusion of the half joints for which the permanent load shall be calculated based on the measured geometry from the inspection, see 5.2.1.

4.1.2 Snow, Wind and Thermal Actions

Snow and wind loading will be ignored as this is not considered to have a governing effect on the assessment.

The effects of temperature difference are not applicable to assessment at ULS.

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

Actions relating to normal traffic shall be considered at ULS & SLS.

Primary variable loads shall be considered together with appropriate permanent loads in accordance with CS454. In addition, secondary variable loads shall be considered together with appropriate primary live loads. Secondary variable loads shall be considered separately from one-another and are not to be combined.

Accidental Wheel Loading shall not be considered acting with other primary live loads.

Values of Assessment Live Loading shall be obtained from Figure 5.19c K-factor for low traffic flow, poor surface, assumed conservatively to account for future deterioration of the surfacing. In the absence of accurate traffic flow data, the traffic flow is considered low on the basis of typical traffic flow witnessed at the various site visits and based on judgement of the traffic flow categories in CS454.

Considering clause 6.4.1 of CS466, longitudinal load from skidding vehicles, clause 5.35 of CS454, shall be included within the assessment of the half-joints.

4.1.4 Actions relating to General Order traffic under STGO regulations

An SV rating shall be determined using the load models outlined in clause 3.6 of CS458.

4.1.5 Footway or footbridge variable actions

Footway loading in accordance with section 5.29 of CS454.

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 applicable.

4.1.7 Accidental actions

Accidental wheel loads will be checked on the verge in accordance with clause 5.27 of CS 454.

Quantitative assessment of the parapets will not be undertaken.

No superstructure or substructure impact loading will be considered in the assessment.

4.1.8 Actions during construction

Not applicable.

4.1.9 Any special action not covered above

Not applicable.

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

Not applicable.

4.3 Minimum headroom provided

Approximately 5.2m.

4.4 Authorities consulted and any special conditions required

Not applicable.

4.5 Standards and documents listed in the Technical Approval Schedule

Refer to Appendix A Technical Approval Schedule (TAS).

4.6 Proposed departures from standards listed in 4.5

Not applicable.

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

Not applicable.

4.8 Proposals for assessment of safety critical fixings.

Not applicable.

5. STRUCTURAL ANALYSIS

5.1 Methods of analysis proposed for superstructure, substructure and foundations

Superstructure:

The half-joints shall be assessed at ULS and SLS, and in accordance with the requirements of CS454, CS455, CS466 and CS458.

A condition factor shall be applied = 0.9.

ALL Model 2 shall be used in accordance with Clause 5.5.2 of CS454.

The effects of accidental wheel loading shall be considered in accordance with 5.27 of CS 454.

The assessment will be level 1, CS454 Table 2.20.1 i.e. Simple structural analysis methods, conservative assumptions for material properties + supplementary values derived from testing material samples where possible.

It is considered that, globally, there will be minimal transfer of load to the half-joints from a parapet impact event. Therefore, for the purpose of this assessment of the half-joints, parapet impact shall not be considered.

Deck impact loading will not be considered as part of this assessment of the half-joints. Transverse horizontal or uplift forces from deck impact are not considered to be detrimental to the performance of the half-joints in the longitudinal direction.

The bridge deck shall be analysed using a 2-D computer grillage model (such as MIDAS) assuming original design deck articulation.

The internal beams shall be modelled with torsionless properties. The edge beams (box beams) shall retain their properties relevant to torsion.

For global effects, the derived limiting vertical live loads combined with local effects shall then be used to assess deck elements in accordance with CS 455 and other relevant standards as appropriate.

The lower nibs shall be assessed using the most onerous load effects from the global analysis and combined with local effects (under wheel or axle loads) as appropriate. Idealised "strut and tie models" as recommended in CS 466 shall be used for assessment of half-joints at SLS and ULS taking account of proposed condition factor outlined above.

The upper nibs shall be assessed using the most onerous load effects from the global analysis and combined with local effects (under wheel or axle loads) as appropriate. Idealised "strut and tie models" as recommended in CS 466 shall be used for assessment of half-joints at SLS and ULS taking account of proposed condition factor outlined above.

Refer to Appendix C for the appropriate "strut and tie" models.

The SLS assessment of crack widths shall be carried out in accordance with the methodology outlined in Appendix D of CS466.

Substructure:

Assessment not required under this commission.

Foundations:

Assessment not required under this commission.

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

See 5.1 and diagrams contained within Appendix C.

5.2.1 Justification for Proposed Idealised Structure

Available design calculations and previous assessment calculations differ significantly in terms of the physical size of the upper and lower nibs of the half joints but also in the size of reinforcement used for design / assessment.

Size of Half-Joint Nibs

A site inspection, carried out in July 2022, confirmed that the half joints are in fact much larger than shown in the design calculations and significantly deeper than shown on available 'record' drawings. As such the available historical information is **not** considered wholly reliable.

	Design Calculations		Record Drawings		Inspection Measurements	
	(ft / in)	(mm)	(ft / in)	(mm)	(ft / in)	(mm)
Lower nib	5 1/2" x 17 3/8"	140mm x 440mm	12" x 1'5"	305mm x 430mm	-	310mm x 500mm
Upper nib (external)	9" x 20"	228mm x 508mm	1' x 1'8"	305mm x 508mm	-	*305mm x 450mm
Upper nib (internal)	9" x 16"	228mm x 406mm	1' x 1'4"	305mm x 405mm	-	-

*Note: The parapet upstand may mask the vertical extent (450mm / 508mm) of the element.

On this basis, it is recommended that the following sizes are utilised for assessment of the upper and lower nibs:

Lower Nib = 310mm x 500mm (W x D).

Upper Nib (external) = 305 x 450mm (W x D)

Upper Nib (internal) = 305 x 405mm (W x D)

Reinforcement

As part of the July 2022 inspection, both upper and lower nibs were ferro-scanned to indicate the arrangement of the reinforcement and check whether it conforms with that shown within the design calculations or record drawings.

Whilst not 100% accurate, the scanning broadly conforms with the reinforcement sizes and spacings shown within the design calculations.

	Design Calculations		Record Drawings		Inspection Ferro-Scanning	
	Diameter (mm)	Spacing (mm)	Diameter (mm)	Spacing (mm)	Diameter (mm)	Spacing (mm)
Lower Nib: Shear	19.05	101.6	19.05	152	19	N/A
Lower Nib: Bending	12.7	152.4	19.05	152	11	N/A
Upper Nib: Shear	15.9	3No	19.05	152	8*	N/A
Upper Nib: Bending	19.05	3No	19.05	5No	19	N/A

this scan is noted to be an anomaly due to the presence of surrounding reinforcement which was picked up by the scan and reduces the median size of reinforcement measured.

The diagonal bars, shown on 'record' drawings to be present, within the upper nibs could not be found by the ferroscan due to reinforcement congestion. It is probable that they are present but this has not been confirmed. Similarly, it is not possible to confirm that there are no diagonal bars in the lower nibs, as the drawings suggest. For the purpose of assessment, the bars shown on the drawings will be assumed to be present.

On this basis, the following shall be adopted for assessment:

Lower Nib =	Shear:	19.05mm bars @ 101.6mm spacing.
	Bending:	12.7mm bars @ 152.4mm spacing.
	Diagonal Reinforcement:	N/A.
Upper Nib =	Shear:	3No x 15.9mm bars.
	Bending:	3No x 19.05mm bars.
	Diagonal Reinforcement:	4No x 19.05mm bars.

5.3 Assumptions intended for calculation of structural element stiffness

Loss of section established from the inspection will be used where appropriate including the implementation of condition factors.

The effective span used in the calculations will be as per the requirements of clause 6.6 of CS 454.

The modulus of elasticity value shall be calculated in accordance with clause 3.5 of CS455.

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 to be used in the assessment and reasons for any proposed changes

Not applicable.

6.2 Summary of design for highway structure in ground investigation report

Not applicable.

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

Differential settlement shall not be considered.

6.4 If the ground investigation report is not yet available, state when the results are expected and list the sources of information used to justify the preliminary choice of foundations.

Not applicable.

7. CHECK

7.1 Proposed category

Category III

7.2 If category 3, name of proposed independent Checker

[redacted]

7.3 Erection proposals or temporary works for which types S and P proposals will be required, listing structural parts of the permanent structure affected with reasons

Not applicable.

8. DRAWINGS AND DOCUMENTS

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

See 8.2 for record drawings and historical calculations.

See Appendix B for the Half Joint Inspection Report, 2022.

See Appendix C for Idealised Diagrams for use in the Assessment of the Half Joints.

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

586/16/3/6/A – Details of Suspended Span Edge Beam for Overbridges.

586/16/3/5/A – Details of Suspended Span Internal Beam for Overbridges.

586/16/3/10 – Wing Wall Details.

586/16/3/9B – Pre-cast Slabs.

586/16/3/14 – Underbarrow Abutment Cable Profiles.

586/16/3/4 – Underbarrow Abutment.

586/16/3/1A – Plan and Elevation.

586/16/3/16 – Revised Parapet Railing Detail.

586/16/3/2 – General Layout.

586/16/3/3 – Kendal Abutment.

KB/22-24 – Various Wing-Wall Drawings (John Laing Construction Ltd.).

586/16/3 – Kendal Abutment Order of Prestressing.

586/16/3/12B – Underbarrow Abutment Order of Prestressing.

586/16/3/15A – Kendal Abutment Cable Profiles.

A591 – Underbarrow Abutment / South Elevation – Scarf Joint.E 06509 Underbarrow and Brigsteer -design calcs.

E 06511 Underbarrow and Brigsteer – Assessment.

E 06510 Brigsteer - design calcs. *Note, includes Brigsteer & Underbarrow.*

E 06509 - Underbarrow and brigsteer design calcs.

Note: Brigsteer and Underbarrow are of similar construction, as such the calculations above typically refer to both bridges.

8.3 List of pile driving or other construction records

Not applicable.

8.4 List of previous inspection and assessment reports

SL221_UNDERBARROW PBI 2018

E 06511 Underbarrow and Brigsteer – Assessment.

BCU00015-JAC-SBR-6330-RP-SL221-CB-004 – Half Joint Inspection for Assessment.

9. THE ABOVE IS SUBMITTED FOR ACCEPTANCE

Signed	[redacted]
Name	[redacted] Assessment Team Leader
Engineering Qualifications	CEng MICE
Name of Organisation	Jacobs UK Ltd
Date	9th January 2023

Signed	[redacted]
Name	[redacted] Check Team Leader
Engineering Qualifications	CEng FICE PGCert
Name of Organisation	[redacted]
Date	9th January 2023

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

Signed	[redacted]
Name	[redacted]
Position Held	[redacted]
Engineering Qualifications	BEng(Hons) CEng MICE
TAA	Cumbria County Council
Date	12th January 2023

Appendix A. Technical Approval Schedule (TAS)

Schedule of Documents Relating to Design of Highway Bridges and Structures

(All documents are taken to include revisions current as of 04 July 2022)

The standards listed are typically required for a highway structure.

Additional standards needed for a particular design should be added to the section at the bottom of the TAS.

The Designer is responsible for ensuring that the standards and references given in the schedule are correct and up to date.

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.~~
~~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 1991-1-1:2002~~	~~Eurocode 1: Actions on structures. General Actions. Densities, self-weight, imposed load for buildings~~	~~Corrigenda December 2004 and March 2009~~	
~~NA to BS EN 1991-1-1:2002~~	~~UK National Annex to Eurocode 1: Actions on structures. General Actions. Densities, self-weight, imposed load for buildings~~	~~Corrigenda July 2019~~	
~~BS EN 1991-1-3:2003 +A1:2015~~	~~Eurocode 1: Actions on structures. General Actions. Snow loads~~	~~+A1:2015 Incorporating corrigenda December 2004 and March 2009~~	
~~NA + A2:18 to BS EN 1991-1-3:2003+A1:2015~~	~~UK National Annex to Eurocode 1: Actions on structures. General Actions. Snow loads~~	~~+A2:2018 Incorporating corrigenda June 2007, December 2015 and October 2018~~	
~~BS EN 1991-1-4:2005 +A1:2010~~	~~Eurocode 1: Actions on structures. General Actions. Wind actions~~	~~+A1:2010 Corrigenda July 2009 and January 2010~~	
~~NA to BS EN 1991-1-4:2005 + A1:2010~~	~~UK National Annex to Eurocode 1: Actions on structures. General Actions. Wind actions~~	~~National Amendment No.1~~	
~~BS EN 1991-1-5:2003~~	~~Eurocode 1: Actions on structures. General Actions. Thermal actions~~	~~Corrigenda December 2004 and March 2009~~	

Eurocodes and associated UK National Annexes			
Eurocode part	**Title**	**Amendment / Corrigenda**	***Notes***
~~NA to BS EN 1991-1-5:2003~~	~~UK National Annex to Eurocode 1: Actions on structures. General Actions. Thermal actions~~	-	
~~BS EN 1991-1-6:2005~~ —	~~Eurocode 1: Actions on structures. General Actions. Actions during execution~~	~~Corrigenda July 2008, November 2012 and February 2013~~	
~~NA to BS EN 1991-1-6:2005~~	~~UK National Annex to Eurocode 1: Actions on structures. General Actions. Actions during execution~~	-	
~~BS EN 1991-1-7:2006 +A1:2014~~	~~Eurocode 1: Actions on structures. General Actions. Accidental actions~~	~~+A1: 2014 Corrigendum February 2010~~	
~~NA+A1 to BS EN 1991-1-7:2006+A1:2014~~	~~UK National Annex to Eurocode 1: Actions on structures. Part 1-7 : Accidental actions~~	~~+A1:2014 Incorporating corrigenda August 2014 and November 2015~~	~~See CD 350 for additional guidance.~~
~~BS EN 1991-2:2003~~	~~Eurocode 1: Actions on structures. Traffic loads on bridges~~	~~Corrigenda December 2004 and February 2010~~	~~See CD 350 section 7 for additional guidance.~~
~~NA +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~~	
~~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~~		
~~BS EN 1992-2:2005~~	~~Eurocode 2: Design of concrete structures – Part 2: Concrete bridges – Design and detailing rules~~	~~Corrigendum July 2008~~	
~~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~~	-	
~~BS EN 1992-3:2006~~	~~Eurocode 2: Design of concrete structures – Part 3: Liquid retaining and containment structures~~	-	
~~NA to BS EN 1992-3:2006~~	~~UK National Annex to Eurocode 2: Design of concrete structures – Part 3: Liquid retaining and containment structures~~	-	
~~BS EN 1992-4:2018~~	~~Eurocode 2: Design of concrete structures – Part 4: Design of fastenings for use in concrete~~		

Eurocodes and associated UK National Annexes			
Eurocode part	**Title**	**Amendment / Corrigenda**	***Notes***
~~NA to BS EN 1992-4:2018~~	~~UK National Annex to Eurocode 2: Design of concrete structures – Part 4: Design of fastenings for use in concrete~~		
~~**Eurocode 3**~~	~~**Design of steel structures**~~		
~~BS EN 1993-1-1:2005 + A1:2014~~	~~Eurocode 3: Design of steel structures – Part 1-1 General rules and rules for buildings~~	~~Corrigenda February 2006 and April 2009~~	
~~NA + A1:2014 to BS EN 1993-1-1:2005 + A1:2014~~	~~UK National Annex to Eurocode 3: Design of steel structures – Part 1-1 General rules and rules for buildings~~	~~-~~	
~~BS EN 1993-1-3:2006~~	~~Eurocode 3: Design of steel structures – Part 1-3 General rules – Supplementary rules for cold-formed members and sheeting~~	~~Corrigendum November 2009~~	
~~NA to BS EN 1993-1-3:2006~~	~~UK National Annex to Eurocode 3: Design of steel structures – Part 1-3 Supplementary rules for cold-formed members and sheeting~~	~~-~~	
~~BS EN 1993-1-4:2006 + A2:2020~~	~~Eurocode 3: Design of steel structures – Part 1-4 General rules – Supplementary rules for stainless steels~~	~~+ A1:2015 Amendment No. 1 + A2:2020 Amendment No. 2~~	~~Supersedes BS EN 1993-1-4:2006 + A1:2015~~
~~NA+A1:15 to BS EN 1993-1-4:2006+A1:2015~~	~~UK National Annex to Eurocode 3: Design of steel structures – Part 1-4 Supplementary rules for stainless steels~~	~~+ A1:2015 Amendment No. 1~~	
~~BS EN 1993-1-5:2006+A2:2019~~	~~Eurocode 3: Design of steel structures – Part 1-5 Plated structural elements~~	~~Corrigendum April 2009, +A1:2017 Amendment No. 2, +A2:2019~~	
~~NA+A1:2016 to BS EN 1993-1-5:2006~~	~~UK National Annex to Eurocode 3: Design of steel structures – Part 1-5 Plated structural elements~~	~~+ A1:2016 Amendment No. 1~~	
~~BS EN 1993-1-6:2007+ A1:2017~~	~~Eurocode 3: Design of steel structures – Part 1-6 Strength and stability of shell structures~~	~~+ A1:2017 Amendment No. 1~~	
~~BS EN 1993-1-7:2007~~	~~Eurocode 3: Design of steel structures – Part 1-7 Plated structures subject to out of plane loading~~	~~Corrigendum April 2009~~	
~~BS EN 1993-1-8:2005~~	~~Eurocode 3: Design of steel structures – Part 1-8 Design of joints~~	~~Corrigenda December 2005, September 2006, July 2009 and August 2010~~	
~~NA to BS EN 1993-1-8:2005~~	~~UK National Annex to Eurocode 3: Design of steel structures – Part 1-8 Design of joints~~	~~-~~	

Eurocodes and associated UK National Annexes			
Eurocode part	**Title**	**Amendment / Corrigenda**	***Notes***
~~BS EN 1993-1-9:2005~~	~~Eurocode 3: Design of steel structures – Part 1-9 Fatigue~~	~~Corrigenda December 2005, September 2006 and April 2009~~	
~~NA to BS EN 1993-1-9:2005~~	~~UK National Annex to Eurocode 3: Design of steel structures – Part 1-9 Fatigue~~	-	
~~BS EN 1993-1-10:2005~~	~~Eurocode 3: Design of steel structures – Part 1-10 Material toughness and through-thickness properties~~	~~Corrigenda December 2005, September 2006 and March 2009~~	
~~NA to BS EN 1993-1-10:2005~~	~~UK National Annex to Eurocode 3: Design of steel structures – Part 1-10 Material toughness and through thickness properties~~	-	
~~BS EN 1993-1-11:2006~~	~~Eurocode 3: Design of steel structures – Part 1-11 Design of structures with tension components~~	~~Corrigendum April 2009~~	
~~NA to BS EN 1993-1-11:2006~~	~~UK National Annex to Eurocode 3: Design of steel structures – Part 1-11 Design of structures with tension components~~	-	
~~BS EN 1993-1-12:2007~~	~~Eurocode 3: Design of steel structures – Part 1-12 Additional rules for the extension of EN 1993 up to steel grades S 700~~	~~Corrigendum April 2009~~	
~~NA to BS EN 1993-1-12:2007~~	~~UK National Annex to Eurocode 3: Design of steel structures – Part 1-12 Additional rules for the extension of EN 1993 up to steel grades S 700~~	-	
~~BS EN 1993-2:2006~~	~~Eurocode 3: Design of steel structures – Part 2 Steel bridges~~	~~Corrigendum July 2009~~	
~~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~~	
~~BS EN 1993-5:2007~~	~~Eurocode 3: Design of steel structures – Part 5 Piling~~	~~Corrigendum May 2009~~	
~~NA + A1:2012 to BS EN 1993-5:2007~~	~~UK National Annex to Eurocode 3: Design of steel structures – Part 5 Piling~~	~~+ A1:2012~~	
Eurocode 4	**Design of composite steel and concrete structures**		
~~BS EN 1994-1-1:2004~~	~~Eurocode 4: Design of composite steel and concrete structures – Part 1-1 General rules and rules for buildings~~	~~Corrigendum April 2009~~	
~~NA to BS EN 1994-1-1:2004~~	~~UK National Annex to Eurocode 4: Design of composite steel and concrete structures – Part 1-1 General rules and rules for buildings~~	-	
~~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~~	

Eurocodes and associated UK National Annexes			
Eurocode part	**Title**	**Amendment / Corrigenda**	***Notes***
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 5	**Design of timber structures**		
BS EN 1995-1-1:2004 + A2:2014	Eurocode 5: Design of timber structures – Part 1-1 General – common rules and rules for buildings	+ A2:2014 Incorporating corrigendum June 2006	
NA to BS EN 1995-1-1:2004 + A2:2014	UK National Annex to Eurocode 5: Design of timber structures – Part 1-1 General – common rules and rules for buildings	+ A2:2014	
BS EN 1995-2:2004	Eurocode 5: Design of timber structures – Part 2 Bridges	-	
NA to BS EN 1995-2:2004	UK National Annex to Eurocode 5: Design of timber structures – Part 2 Bridges	-	
Eurocode 6	**Design of masonry structures**		
BS EN 1996-1-1:2005+A1:2012	Eurocode 6: Design of masonry structures – Part 1-1 General rules for reinforced and unreinforced masonry structures	+A1:2012 Corrigenda February 2006 and July 2009	
NA to BS EN 1996-1-1:2005 +A1:2012	UK National Annex to Eurocode 6: Design of masonry structures – Part 1-1 General rules for reinforced and unreinforced masonry structures	+A1:2012	
BS EN 1996-2:2006	Eurocode 6: Design of masonry structures – Part 2 Design considerations, selection of materials and execution of masonry	Corrigendum September 2009	
NA to BS EN 1996-2:2006	UK National Annex to Eurocode 6: Design of masonry structures – Part 2 Design considerations, selection of materials and execution of masonry	Corrigendum No.1	
BS EN 1996-3:2006	Eurocode 6: Design of masonry structures – Part 3 Simplified calculation methods for unreinforced masonry structures	Corrigendum October 2009	
NA +A1:2014 to BS EN 1996-3:2006	UK National Annex to Eurocode 6: Design of masonry structures – Part 3 Simplified calculation methods for unreinforced masonry structures	+A1:2014	
Eurocode 7	**Geotechnical design**		
BS EN 1997-1:2004+A1:2013	Eurocode 7: Geotechnical design – Part 1 General rules	+A1:2013 Corrigendum February 2009	
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

Eurocodes and associated UK National Annexes			
Eurocode part	**Title**	**Amendment / Corrigenda**	***Notes***
~~BS EN 1997-2:2007~~	~~Eurocode 7: Geotechnical design – Part 2 Ground investigation and testing~~	~~Corrigendum June 2010~~	
~~NA to BS EN 1997-2:2007~~	~~UK National Annex to Eurocode 7: Geotechnical design – Part 2 Ground investigation and testing~~	-	
Eurocode 8	**Design of structures for earthquake resistance**		
~~BS EN 1998-1:2004 + A1:2013~~	~~Eurocode 8: Design of structures for earthquake resistance – Part 1 General rules, seismic actions and rules for buildings~~	~~Corrigendum June 2009, January 2011 and March 2013~~	
~~NA to BS EN 1998-1:2004~~	~~UK National Annex to Eurocode 8: Design of structures for earthquake resistance – Part 1 General rules, seismic actions and rules for buildings~~	-	
~~BS EN 1998-2:2005+A2:2011~~	~~Eurocode 8: Design of structures for earthquake resistance – Part 2 Bridges~~	~~Corrigenda February 2010 and February 2012~~	
~~NA to BS EN 1998-2:2005~~	~~UK National Annex to Eurocode 8: Design of structures for earthquake resistance – Part 2 Bridges~~	-	
~~BS EN 1998-5:2004~~	~~Eurocode 8: Design of structures for earthquake resistance – Part 5 Foundations, retaining structures and geotechnical aspects~~	-	
~~NA to BS EN 1998-5:2004~~	~~UK National Annex to Eurocode 8: Design of structures for earthquake resistance – Part 5 Foundations, retaining structures and geotechnical aspects~~	-	
Eurocode 9	**Design of aluminium structures**		
~~BS EN 1999-1-1:2007 + A2:2013~~	~~Eurocode 9: Design of aluminium structures– Part 1-1 General structural rules~~	~~+ A2:2013 Incorporating corrigendum March 2014~~	
~~NA to BS EN 1999-1-1:2007 + A1:2009~~	~~UK National Annex to Eurocode 9: Design of aluminium structures – Part 1-1 General structural rules~~	~~National Amendment No.1 Corrigendum No.1~~	
~~BS EN 1999-1-3:2007 + A1:2011~~	~~Eurocode 9: Design of aluminium structures – Part 1-3 Structures susceptible to fatigue~~	~~+ A1:2011~~	
~~NA to BS EN 1999-1-3:2007 + A1:2011~~	~~UK National Annex to Eurocode 9: Design of aluminium structures – Part 1-3 Structures susceptible to fatigue~~	~~+ A1:2011~~	
~~BS EN 1999-1-4:2007 +A1:2011~~	~~Eurocode 9: Design of aluminium structures – Part 1-4 Cold formed structural sheeting~~	~~+ A1:2011 Corrigendum November 2009~~	
~~NA to BS EN 1999-1-4:2007~~	~~UK National Annex to Eurocode 9: Design of aluminium structures – Part 1-4 Cold formed structural sheeting~~	-	

Eurocodes and associated UK National Annexes			
Eurocode part	**Title**	**Amendment / Corrigenda**	***Notes***

Bsi Published Documents

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

Published Document reference	**Title**	**Notes**
~~PD 6687-1:2020~~	~~Background paper to the UK National Annexes to BS EN 1992-1 and BS EN 1992-3~~	~~Supersedes PD 6687-1:2010~~ ~~See CD 350 clauses 3.6, 4.1, 4.2 and Appendix A for additional guidance.~~ ~~Clause 3.6 in CD 350 refers to clause 2.5 in PD 6687-1, this is now clause 4.5 in PD 6687-1~~ ~~Clause 4.2 in CD 350 refers to clause 2.22 in PD 6687-1, this is now clause 4.21.4 in PD 6687-1~~
~~PD 6687-2:2008~~	~~Recommendations for the design of structures to BS EN 1992-2:2005~~	~~See CD 350 clauses 4.1, 4.2 and Appendix A for additional guidance.~~
~~PD 6688-1-1:2011~~	~~Recommendations for the design of structures to BS EN 1991-1-1~~	~~See CD 350 Appendix A for additional guidance.~~
~~PD 6688-1-4:2015~~	~~Background paper to the UK National Annex to BS EN 1991-1-4~~	~~See CD 350 Appendix A for additional guidance.~~
~~PD 6688-1-7:2009 +A1:2014~~	~~Recommendations for the design of structures to BS EN 1991-1-7~~	~~See CD350 clause 3.7 and Appendix B for additional guidance.~~
~~PD 6688-2:2011~~	~~Recommendations for the design of structures to BS EN 1991-2~~	~~See CD 350 Appendix A for additional guidance.~~
~~PD 6694-1:2011 + A1:2020~~	~~Recommendations for the design of structures subject to traffic loading to BS EN 1997-1~~	~~Incorporating Corrigendum January 2022~~ ~~See CD 350 Appendix A for additional guidance.~~
~~PD 6695-1-9:2008~~	~~Recommendations for the design of structures to BS EN 1993-1-9~~	~~See CD 350 Appendix A for additional guidance.~~
~~PD 6695-1-10:2009~~	~~Recommendations for the design of structures to BS EN 1993-1-10~~	~~See CD 350 Appendix A for additional guidance.~~
~~PD 6695-2:2008 + A1:2012 Incorporating Corrigendum No.1~~	~~Recommendation for the design of bridges to BS EN 1993~~	~~See CD 350 Appendix A for additional guidance.~~
~~PD 6696-2:2007 + A1:2012~~	~~Background paper to BS EN 1994-2 and the UK National Annex to BS EN 1994-2~~	~~See CD 350 Appendix A for additional guidance.~~
~~PD 6698:2009~~	~~Recommendations for the design of structures for earthquake resistance to BS EN 1998~~	~~See CD 350 section 7 for additional guidance.~~
~~PD 6702-1:2009+A1:2019~~	~~Structural use of aluminium. Recommendations for the design of aluminium structures to BS EN 1999~~	~~Amended 31 May 2019~~
~~PD 6703:2009~~	~~Structural bearings – Guidance on the use of structural bearings~~	
~~PD 6705-2:2020~~	~~Structural use of steel and aluminium. Execution of steel bridges conforming to BS EN 1090-2. Guide~~	~~Replaces PD 6705-2:2010 + A1:2013~~
~~PD 6705-3:2009~~	~~Recommendations on the execution of aluminium structures to BS EN 1090-3~~	

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

Product Standards referenced in British Standards or Eurocodes		
Product Standard reference	**Title**	**Notes**
BS EN 206:2013+A2:2021	Concrete – Specification, performance, production and conformity	Supersedes BS EN 206:2013+A1:2016
BS EN 1317-1:2010	Road Restraint Systems – Part 1 – Terminology and general criteria for test methods	
BS EN 1317-2:2010	Road Restraint Systems – Part 2 – Performance classes, impact test acceptance criteria and test methods for safety barriers.	
BS EN 1317-3:2010	Road Restraint Systems – Part 3 – Performance classes, impact test acceptance criteria and test methods for crash cushions.	
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*
BS EN 1317-5:2007+A2:2012	Road Restraint Systems – Part 5 - Product requirements and evaluation of conformity for vehicle restraint systems	Incorporating corrigendum August 2012 *Draft prEN 1317-5 for public comment published in December 2013*

Product Standards referenced in British Standards or Eurocodes		
Product Standard reference	**Title**	**Notes**
~~PD CEN/TR 16949:2016~~	~~Road Restraint System – Pedestrian restraint system - Pedestrian parapets~~	~~*Bsi Published Document / CEN Technical Report published in July 2016*~~ ~~*(This document should not be used. The requirements of BS 7818:1995 apply.)*~~
~~Draft prEN 1317-7~~	~~Road restraint systems - Part 7: Performance classes, impact test acceptance criteria and test methods for terminals of safety barriers~~	~~*Draft prEN 1317-7 for public comment published in June 2012*~~ ~~*(This document should not be used. All terminals should continue to be in accordance with ENV1317-4.)*~~
~~PD CEN/TS 17342:2019~~	~~Road restraint systems - Motorcycle road restraint systems which reduce the impact severity of motorcyclist collisions with safety barriers~~	~~*Replaces* PD CEN/TS 1317-8:2012~~ ~~*(This document should not be used.)*~~
~~PD CEN/TR 17081:2018~~	~~Design of fastenings for use in concrete – Plastic design of fastenings with headed and post-installed fasteners~~	
~~BS EN 1337-1:2000~~	~~Structural bearings – Part 1: General Design Rules~~	
~~BS EN 1337-2:2004~~	~~Structural bearings – Part 2: Sliding elements~~	
~~BS EN 1337-3:2005~~	~~Structural bearings – Part 3: Elastomeric bearings~~	
~~BS EN 1337-4:2004~~	~~Structural bearings – Part 4: Roller bearings~~	~~Corrigendum No.1 March 2007~~
~~BS EN 1337-5:2005~~	~~Structural bearings – Part 5: Pot bearings~~	
~~BS EN 1337-6:2004~~	~~Structural bearings – Part 6: Rocker bearings~~	
~~BS EN 1337-7:2004~~	~~Structural bearings – Part 7: Spherical and cylindrical PTFE bearings~~	
~~BS EN 1337-8:2007~~	~~Structural bearings – Part 8: Guide bearings and restraint bearings~~	
~~BS EN 1337-9:1998~~	~~Structural bearings – Part 9: Protection~~	
~~BS EN 1337-10:2003~~	~~Structural bearings – Part 10: Inspection and maintenance~~	~~Corrigendum No.1 November 2003~~
~~BS EN 1337-11:1998~~	~~Structural bearings – Part 11: Transport, Storage and Installation.~~	
~~BS EN 10025-1:2004~~	~~Hot rolled products of structural steels Part 1: General technical delivery conditions.~~	
~~BS EN 10025-2:2019~~	~~Hot rolled products of structural steels Part 2: Technical delivery conditions for non-alloy structural steels.~~	~~Supersedes BS EN 10025-1:2004~~

Product Standards referenced in British Standards or Eurocodes

Product Standard reference	Title	Notes
~~BS EN 10025-3:2019~~	~~Hot rolled products of structural steels Part 3: Technical delivery conditions for normalized/normalized rolled weldable fine grain structural steels.~~	~~Supersedes BS EN 10025-3:2004~~
~~BS EN 10025-4:2019~~	~~Hot rolled products of structural steels Part 4: Technical delivery conditions for thermomechanical rolled weldable fine grain structural steels.~~	~~Supersedes BS EN 10025-4:2004~~
~~BS EN 10025-5:2019~~	~~Hot rolled products of structural steels – Part 5: Technical delivery conditions for structural steels with improved atmospheric corrosion resistance~~	~~Supersedes BS EN 10025-5:2004~~
~~BS EN 10025-6:2019~~	~~Hot rolled products of structural steels – Part 6: Technical delivery conditions for flat products of high yield strength structural steels in the quenched and tempered condition.~~	~~Supersedes BS EN 10025-6:2004+A1:2009~~
~~BS EN 10080:2005~~	~~Steel for the reinforcement of concrete – Weldable reinforcing steel - General~~	
~~BS EN 10210-1:2006~~	~~Hot finished structural hollow sections of non-alloy and fine grain steels – Part 1: Technical delivery conditions~~	
~~BS EN 10210-2:2019~~	~~Hot finished structural hollow sections – Part 2: Tolerances, dimensions and sectional properties~~	~~Supersedes BS EN 10210-2:2006~~
~~BS EN 10248-1:1996~~	~~Hot rolled sheet piling of non alloy steels.~~ ~~Technical delivery conditions~~	
~~BS EN 10248-2:1996~~	~~Hot rolled sheet piling of non alloy steels.~~ ~~Tolerances on shape and dimensions~~	
~~BS EN 12063:1999~~	~~Execution of special geotechnical work. Sheet pile walls.~~	
~~BS EN 14388:2005~~	~~Road traffic noise reducing devices~~	~~There is a 2015 version, however the 2015 version is not harmonised.~~
~~BS EN 15050:2007 + A1:2012~~	~~Precast concrete products – Bridge elements~~	~~See CD 350 clause 3.8.1 for additional guidance.~~
~~BS EN 15258:2008~~	~~Precast concrete products - Retaining wall elements~~	

British Standards		
British Standard reference	**Title**	**Notes**
~~BS 4449:2005+A3:2016~~	~~Steel for the reinforcement of concrete~~	~~No longer covers plain round bar. (See BS4482 up to 12mm dia, see BS EN 10025-1 for larger sizes and dowels. See BS EN 13877-3 for dowel bars in concrete pavements.)~~
~~BS 5896:2012~~	~~Specification for high tensile steel wire and strand for the prestressing of concrete~~	
~~BS 7818:1995~~	~~Specification for pedestrian restraint systems in metal~~	~~Incorporating Corrigendum No.1 May 2004 and Corrigendum No.2 September 2006~~ ~~Currently the requirements of BS 7818:1995 are to be used instead of PD CEN/TR 16949:2016~~
~~BS 8002:2015~~	~~Code of practice for earth retaining structures~~	
~~BS 8004:2015 +A1 2020~~	~~Code of practice for foundations~~	~~Amendment +A1:2020~~
~~BS 8006-1:2010+A1:2016~~	~~Code of practice for strengthened/reinforced soils and other fills~~	
~~BS 8500-1:2015+A2:2019~~	~~Concrete – Complementary British Standard to BS EN 206: Method of specifying and guidance for the specifier.~~	~~Incorporating Corrigendum No.1 and Corrigendum No.2 June 2020~~ ~~Amendment +A2:2019~~
~~BS 8500-2:2015+A2:2019~~	~~Concrete – Complementary British Standard to BS EN 206 : Specification for constituent materials and concrete.~~	~~Amendment +A2:2019~~
~~BS 8666:2020~~	~~Scheduling, dimensioning, bending and cutting of steel reinforcement for concrete~~	~~Supersedes BS 8666:2005~~

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

MCHW Volume 3: February 2017	Highway Construction Details	

The Design Manual for Roads and Bridges (DMRB)		
DMRB reference	**Title**	**Notes**
GG 101 Revision 0.1.0	Introduction to the Design Manual for Roads and Bridges	Replaces GG 101 Revision 0
GG 102 Revision 0	Quality Management Systems for Highway Design	Replaces GD 02/16
GG 103 Revision 0	Introduction and general requirements for sustainable development and design	
GG 104 Revision 0	Requirements for Safety Risk Assessment	Replaces GD04/12 and IAN 191/16
GG 184 Revision 0	Specification for the use of Computer Aided Design	Replaces IAN 184/16
CG 300 Revision 0.1.0	Technical approval of highway structures	Supersedes BD 2/12
CG 302 Revision 0	As-built, operational and maintenance records for highway structures	Supersedes BD 62/07
CG 303 Revision 0	Quality assurance scheme for paints and similar protective coatings	Supersedes BD 35/14
CG 305 Revision 0	Identification marking of highway structures	Supersedes BD 45/93
CG 501 Revision 2	Design of highway drainage systems	Supersedes HD 33/16, TA 80/99
CD 127 Revision 1.0.1	Cross-sections and headrooms	Replaces TD 27/05 and TD 70/08
CD 350 Revision 0	The design of highway structures	Supersedes BD 100/16, BA 57/01, BD 57/01 and IAN 124/11
CD 351 Revision 0	The design and appearance of highway structures	Supersedes BA 41/98
CD 352 Revision 0	Design of road tunnels	Supersedes BD 78/99
CD 353 Revision 0	Design criteria for footbridges	Supersedes BD 29/17
CD 354 Revision 1.1.0	Design of minor structures	Supersedes CD 354 Revision 1
CD 355 Revision 0	Application of whole-life costs for design and maintenance of highway structures	Replaces BD 36/92 and BA 28/92
CD 356 Revision 1	Design of highway structures for hydraulic action	Supersedes BA 59/94
CD 357 Revision 1	Bridge expansion joints	Replaces BD 33/94, BA 26/94, IAN 168/12 and IAN 169/12
CD 358 Revision 2.4.0	Waterproofing and surfacing of concrete bridge decks	Supersedes CD 358 Revision 2.3.0
CD 359 Revision 0	Design requirements for permanent soffit formwork	Supersedes BA 36/90 and IAN 131/11
CD 360 Revision 2	Use of compressive membrane action in bridge decks	Supersedes BD 81/02
CD 361 Revision 0	Weathering steel for highway structures	Supersedes BD 7/01
CD 362	Enclosure of bridges	Replaces BD 67/96 and BA 67/96

The Design Manual for Roads and Bridges (DMRB)		
DMRB reference	**Title**	**Notes**
~~Revision 1~~		
~~CD 363 Revision 0~~	~~Design rules for aerodynamic effects on bridges~~	~~Replaces BD 49/01~~
~~CD 364 Revision 0~~	~~Formation of continuity joints in bridge decks~~	~~Replaces BA 82/00~~
~~CD 365 Revision 1~~	~~Portal and cantilever signs/signals gantries~~	~~Replaces BD 51/14, IAN 193/16, BE 7/04~~
~~CD 366 Revision 0~~	~~Design criteria for collision protection beams~~	~~Replaces BD 65/14~~
~~CD 367 Revision 0~~	~~Treatment of existing structures on highways widening schemes~~	~~Replaces BD 95/07~~
~~CD 368 Revision 0~~	~~Design of fibre reinforced polymer bridges and highway structures~~	~~Replaces BD 90/05~~
~~CD 369 Revision 0~~	~~Surface protection for concrete highway structures~~	~~Replaces BA 85/04~~
~~CD 371 Revision 0~~	~~Strengthening highway structures using fibre-reinforced polymers and externally bonded steel plates~~	~~Replaces BD 85/08, BD 84/02~~
~~CD 372 Revision 0~~	~~Design of post-installed anchors and reinforcing bar connections in concrete~~	~~Supersedes IAN 104/15~~
~~CD 373 Revision 0~~	~~Impregnation of reinforced and prestressed concrete highway structures using hydrophobic pore-lining impregnants~~	~~Supersedes BD 43/03~~
~~CD 374 Revision 0~~	~~The use of recycled aggregates in structural concrete~~	~~Supersedes BA 92/07~~
~~CD 375 Revision 1~~	~~Design of corrugated steel buried structures~~	~~Supersedes BD 12/01~~
~~CD 376 Revision 0~~	~~Unreinforced masonry arch bridges~~	~~Replaces BD 91/04~~
~~CD 377 Revision 4~~	~~Requirements for road restraint systems~~	~~Supersedes TD 19/06~~
~~CD 622 Revision 1~~	~~Managing geotechnical risk~~	~~Replaces HD 22/08, BD 10/97 and HA 120/08~~
~~CS 461 Revision 0~~	~~Assessment and upgrading of in-service parapets~~	~~Supersedes BA 37/92 and IAN 97/07~~
~~GD 304 Revision 2~~	~~Designing health and safety into maintenance~~	~~Replaces IAN 69/15~~
~~LA 104 Revision 1~~	~~Environmental assessment and monitoring~~	~~Supersedes HA 205/08, HD 48/08, IAN 125/15, and IAN 133/10~~
~~LA 106 Revision 1~~	~~Cultural heritage assessment~~	~~Supersedes HA 208/07, HA 60/92, HA 75/01~~
~~LA 110 Revision 0~~	~~Material assets and waste~~	~~Supersedes IAN 153/11~~
~~LA 113 Revision 1~~	~~Road drainage and the water environment~~	~~Supersedes HD 45/09~~
~~LD 119 Revision 0~~	~~Roadside environmental mitigation and enhancement~~	~~Formerly LA 119, which superseded HA 65/94 and HA 66/95~~

Interim Advice Notes		
IAN reference	**Title**	**Notes**

The Design Manual for Roads and Bridges (DMRB)		
DMRB reference	**Title**	**Notes**
~~IAN 105/08~~	~~Implementation of construction (design and management) 2007 and the withdrawal of SD 10 and SD 11~~	

Miscellaneous		
Standard reference	**Title**	**Notes**
~~CIRIA C543~~	~~Bridge Detailing Guide~~	
~~CIRIA C686~~	~~Safe Access for Maintenance and Repair~~	
~~CIRIA C760~~	~~Guidance on embedded retaining wall design~~	
~~CIRIA C766~~	~~Control of cracking caused by restrained deformation in concrete~~	~~Supersedes C660~~

Additional Standards **Additional standards needed for a particular design should be listed here.**		
Reference	**Title**	**Notes**
CS 454	Assessment of highway bridges and structures	
CS 455	The Assessment of concrete highway bridges and structures	
CS 466	Risk management and structural assessment of concrete half-joint deck structures	

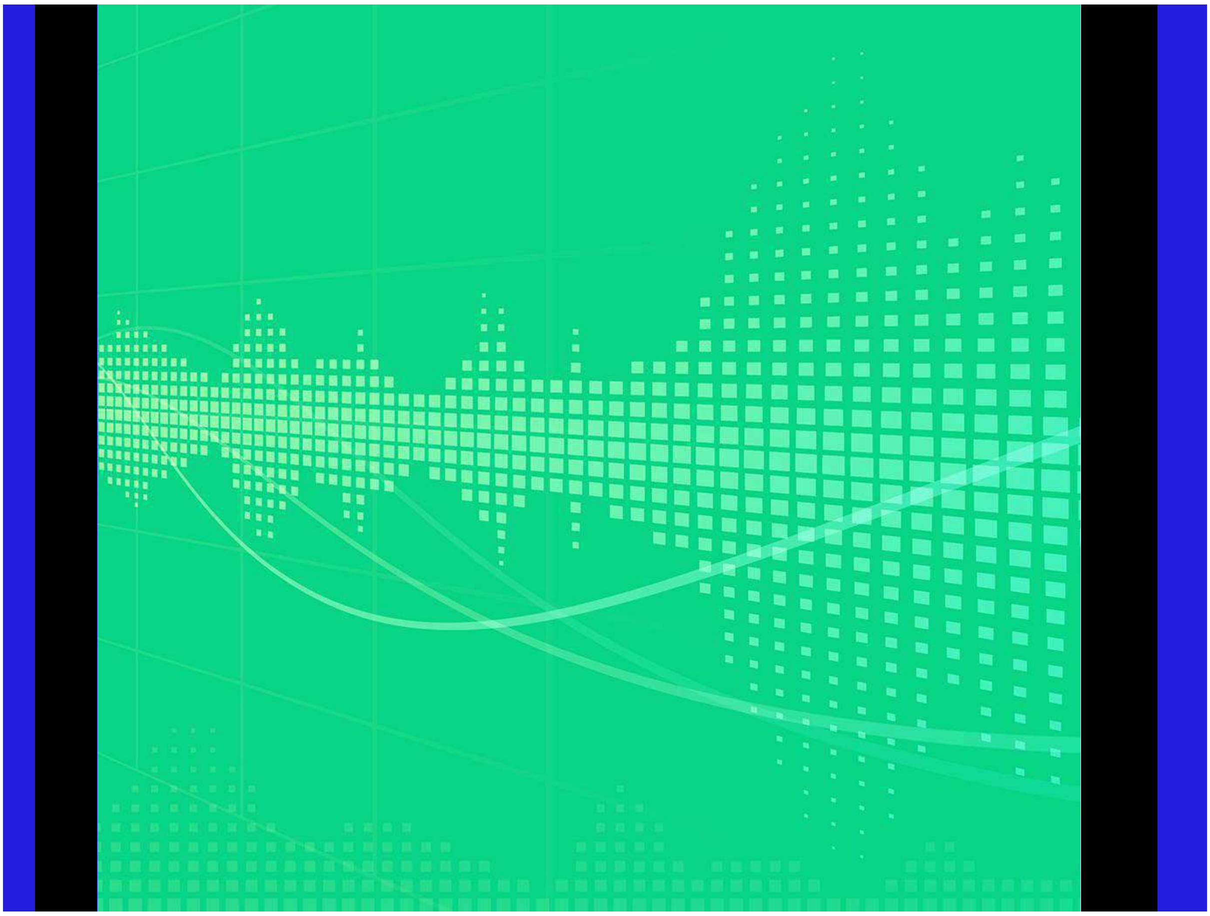
Appendix B. Half Joint Inspection Report

Half Joint Inspection Report - Underbarrow

Document no: BCU00015-JAC-SBR-6330-RP-SL221-CB-004
Revision no: P01

Cumbria County Council
6330

Risk Assessment and Structural Assessment of Post Tensioned and Half Joint Bridges SL240 Brigsteer and SL221 Underbarrow
24 August 2022



Half Joint Inspection Report - Underbarrow

Client name:	Cumbria County Council		
Project name:	Risk Assessment and Structural Assessment of Post Tensioned and Half Joint Bridges SL240 Brigsteer and SL221 Underbarrow		
Client reference:	6330	**Project no:**	BCU00015
Document no:	BCU00015-JAC-SBR-6330-RP-SL221-CB-004	**Project manager:**	[redacted]
Revision no:	P01	**Prepared by:**	[redacted]
Date:	24 August 2022	**File name:**	BCU00015-JAC-SBR-6330-RP-SL221-CB-004
Doc status:	Suitable for Issue		

Document history and status

Revision	Date	Description	Author	Checked	Reviewed	Approved
P01	24/08/2022	Half Joint Inspection	[redacted]			

Distribution of copies

Revision	Issue approved	Date issued	Issued to	Comments
P01	[redacted]	24/08/2022	[redacted]	Issue to Cumbria County Council

[redacted]

Contents

Appendices

1. Introduction and General Details

1.1 Introduction

Jacobs UK Ltd was commissioned by Cumbria County Council to carry out a risk assessment and structural assessment of post tensioned half joints to SL221 Underbarrow, Kendal.

This report covers the inspection of the half joints for assessment purposes.

Where the inspection of the half-joints was limited by obstructions or restricted access, details of limitations have been identified and discussed within the report text under the appropriate headings. The inspection was undertaken such that negative impact on the environment was mitigated; no flora or fauna were disturbed. All materials brought to site were removed at the end of the inspection.

This report describes the findings of the inspection and provides recommendations for condition factors.

Record information, including historical inspections reports, maintenance records and drawings, were obtained from Essex County Council Highways. An Initial Review has been carried out in advance of this inspection, see BCU00015-JAC-SBR-6330-RP-SL221-CB-001.

The assessment of this structure will be reported in a subsequent Assessment Report.

1.2 Description

Underbarrow, constructed in 1970, carries the C5048 single carriageway Underbarrow Road east and west over the A591, Kendal Bypass County Road, west of Kendal.

The superstructure is a single span made up of in-situ concrete cantilevers and a precast concrete beam suspended span. The cantilevers are of post-tensioned voided construction, integral with voided abutments. The suspended span comprises 17No. prestressed pre-tensioned concrete beams and an in-situ reinforced concrete deck slab. The inner beams are inverted T-beams and are transversely post-tensioned. The edge beams are box beams. The suspended span is supported by half-joints at the ends of the cantilevers.

The A591 below is a dual carriageway with a grassed central reserve and grassed verges. There are “limestone pitching” revetments in front/above both abutments.

The half joint form is described as ‘solid or box slab with no access to the bearing shelf’ and is classified as ‘Type A’ in accordance with CS 466 (Figure C.3 and Table C.10).

The suspended square span is 18.288m (60’ 0”) between centrelines of bearings.

The bridge is located at OS Grid Ref. SD 499 924.

1.3 Half Joint Details

The half joint form is described as 'solid or box slab with no access to the bearing shelf' and is classified as 'Type A' in accordance with CS 466 (Figure C.3 and Table C.10).

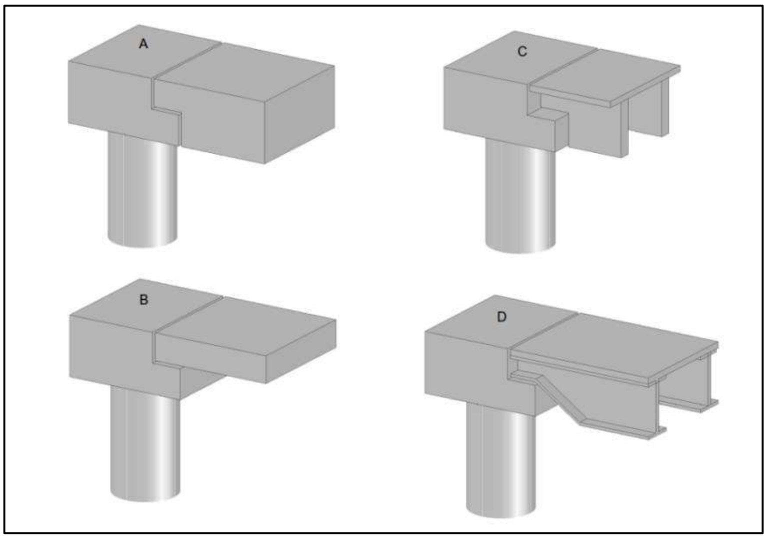


Figure 1 – Visualisation of Half-joint types (CS 466, Figure C.3)

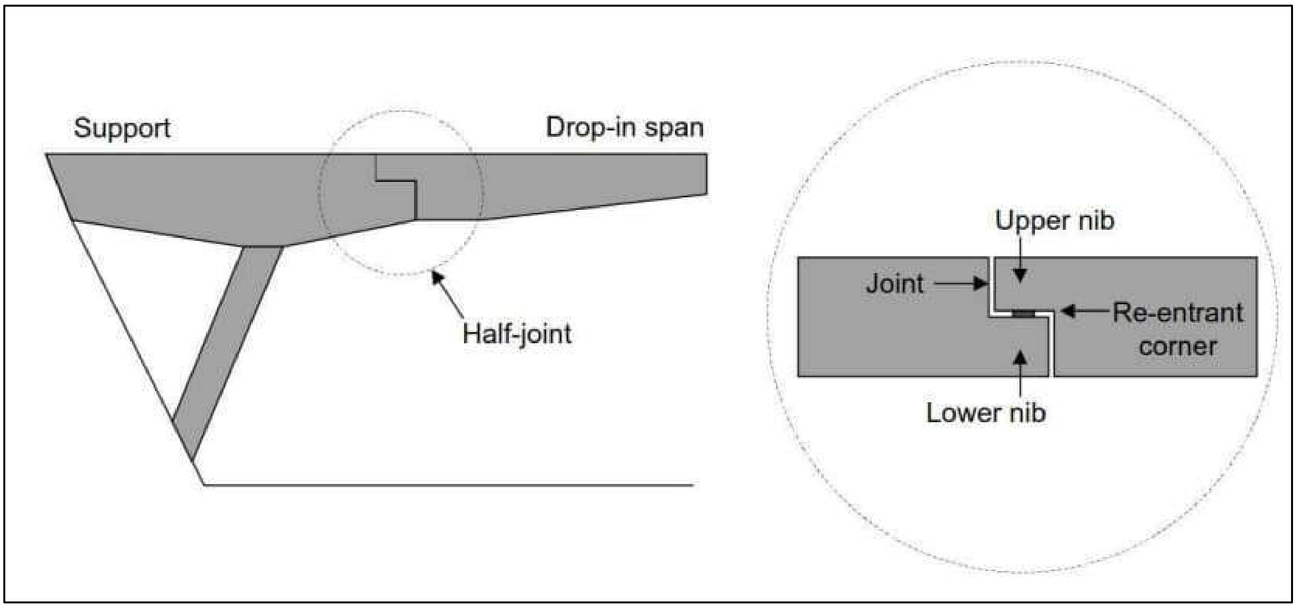


Figure 2 – Terminology used to describe Half joint elements (CS 466, Figure A.1)

2. Maintenance and Inspection History

2.1 Details of Previous Inspections and Assessments

Assessment 1991-94, Cumbria County Council

An assessment report dated January 1994 produced by Cumbria County Council concludes that the structure has a capacity for 40T Assessment Live Loading, full HA Loading and a HB capacity of 30 units as stated on the signed certification (dated 14th February 1995). However, a note on the results summary sheet states that the suspended span and the top slab of the hollow parts of the cantilever will carry 30 units HB loading, but if the HB vehicle travels within 150mm of the kerb, allowing associated HA loading, then the capacity reduces to 14 HB units, limited by the lower nib of the half-joints. SLS checks concluded that the actual crack width is greater than twice the allowable width. The cracking was attributed to poor detailing of reinforcement as opposed to overloading.

A set of comprehensive assessment calculations are available to BD 21/93 which supplement the assessment report. Since the assessment BD 21/93 has been replaced and the current assessment standard is CS 454.

No Approval in Principle (AIP) is available, and no reference is contained within the assessment report. In accordance with current standard CG 300 the structure is Category 3 and will require an AIP for future assessments of the structure and an independent calculation check from a separate organisation.

Principal Bridge Inspection, 2018, CAPITA

The 2018 Principal Inspection noted cracks extending from the internal corner of the lower nibs of the north-east and south east half-joints. The report noted evidence of old repairs along the horizontal edge of the lower nib, some of which sounded hollow when hammer tested.

2.2 Details of Previous Maintenance

There is evidence within the structure file that the deck infill was excavated to reveal the top face of the deck and half joints during 1974. The extent of works carried out at this time is unclear.

Records state that type 3 – nosing with poured sealant joints were originally installed within the carriageway above the half joints and that the verges were sealed with a 25mm thick strip of rubber bitumen sealant.

Further record drawings dated 1981 state that the type 3 - nosing with poured sealant expansion joints were removed in their entirety, replaced by type 2 – asphaltic plug expansion joints.

The Principal Inspection report dated September 2018 notes that the carriageway has been surfaced dressed, the report also notes evidence of concrete repairs to the abutments and half-joints. No dates are mentioned within the report and no other details can be found regarding this work.

2.3 Records of Intrusive works

The available records do not detail any intrusive works having been carried out previously.

3. Description of the Half Joint Inspection

3.1 General

The half joint inspection was undertaken by Jacobs UK during July 2022. Inspection on top of the structure was undertaken during daylight hours on Monday 4th July, inspection of the underside was undertaken during night-time hours between Tuesday 5th and Wednesday 6th July.

The lead inspecting engineer who is also responsible for overseeing the risk review, risk assessment and risk management process and the post-tensioned special inspection (PTSI) is [redacted] CEng MICE, who has experience of inspection of highway structures including post tensioned bridges. Accompanying [redacted] as a secondary inspector was [redacted] who as experience of inspection of highway structures.

At the time of the inspection the weather was warm with light rain for a short period mid-inspection. The weather preceding the inspection had generally been clear and warm.

3.2 Access Arrangements

General access over the structure was undertaken on foot via the verges, carriageway, embankments and access walkways. No traffic management for inspection over the structure was required. Access beneath the structure was provided by a Mobile Elevated Platform (MEWP) situated on the carriageway beneath the structure within the extents of a full night-time northbound and southbound carriageway closure of the A591. A borescope was utilised to inspect the internal parts of the half joints within the limitations of access and capability of the borescope.

3.3 Intrusive Investigations

There were no intrusive works carried out, however, a ferroscan and GPR were hired and used as part of the inspection in an attempt to confirm or otherwise the size, layout and cover to reinforcement.

Scanning was carried out to the surrounding areas of the half joint, up to approximately 1m either side of the joint centreline.

4. Results of the Half Joint Inspection

4.1 General

Numbered photographs of bridge elements are included in Appendix A.

For general photographs, see photographs 1 to 4.

A summary schedule shall be provided as a separate deliverable.

4.2 East Half Joint

The east half joint has a narrow gap between the upper and lower nibs which, in combination with the narrow gap observed at the east half joint, may infer some historical movement. Generally, the beams do not appear to be spaced or seated evenly (photograph 5).

4.2.1 Top of Deck

The east half joint has no formal expansion joint installed within the carriageway surfacing (photograph 6).

There are 2No areas of potholing; to the centreline and the westbound lane (photographs 7 & 8).

The carriageway is cracking on the eastbound lane (photograph 9).

The verges have joints directly above the half joint and are sealed although there is cracking and vegetation growth (photographs 6 & 9) to both verges (within the joint).

The parapets are showing signs of significant expansion although they have not reached the end of their tolerance (photograph 10).

4.2.2 Upper Nib

Typically, the inspection within the half joint was limited by the presence of formwork, polystyrene and debris (photographs 11 to 13).

By inspection of the elevations, the upper nib of the edge box beams is in good condition (photographs 14 & 15).

There is a shallow spall exposing corroding reinforcement to the upper nib on the south elevation (photograph 15).

Via borescope inspection, 2No cracks were found on the upper nib;

1. at the upper nib of the T-beam third in from the north elevation (photograph 16),
2. at the upper nib of the T-beam second in from the south elevation (photograph 17).

   There are no signs of spalling to the surrounding concrete or rust staining to indicate deterioration of reinforcement.

4.2.3 Lower Nib

On the north elevation there is a hairline crack (1.5mm wide) which is historical and shows no signs of deterioration since the previous inspection report (photographs 18 & 19). The edge of the lower nib has been repaired previously. Tell tales have been installed in the past but no longer remain, so any further deterioration cannot be accurately quantified.

The soffit of the lower nib has leachate staining at the south end (photograph 20).

Via borescope inspection, 4No cracks were found on the lower nib;

1. At the lower nib below the north box beam (photograph 21),

2. At the lower nib below the north box beam (photograph 22) nearby to photograph 21,
3. At the lower nib below the T-beam second from the south elevation (photograph 23),
4. At the lower nib below the south box beam (photograph 24).

Although not relatable to the half joint due to its distance from the lower nib, on the north elevation there is a large area of cracking concrete which continues onto the soffit of the cantilevering deck (photographs 25 & 26). The cracking is unsightly due to its combination with leachate staining. There is no sign of loose concrete.

4.2.4 Bearings

Not visible for inspection due to debris, formwork and access limitations.

4.3 West Half Joint

The west half joint has large gap (maximum 50mm located at the base of the joint) which, in combination with the narrow gap observed at the east half joint, may infer some historical movement (photographs 34 & 35).

4.3.1 Top of Deck

The west half joint is unsightly due to the poor workmanship and installation of the bituminous sealant of the type 2 expansion joint (photographs 27 & 28)

Both hardened verges have vegetation growth and debris within the joint at surface level (photographs 29 & 30).

There is cracking to the sealant within the parapet upstand on the south side (photograph 31).

The surfacing west of the half joint is in poor condition with multiple potholes (photograph 32).

4.3.2 Upper Nib

Typically, the inspection within the half joint was limited by the presence of formwork, polystyrene and debris (photograph 33).

On elevation, the upper nibs to both of the edge box beams are in good condition, free from cracking, spalling and staining (photographs 34 & 35).

There is shrinkage cracking to the upper nib of the northernmost T-beam (photograph 36)

4.3.3 Lower Nib

The lower nib on each elevation appears to have been subjected to concrete repairs (photographs 34 & 35).

The south elevation has multiple small spalls (photograph 34).

The north elevation has localised algal staining at the re-entrant corner (photograph 37). There is extensive leachate staining to the soffit of the deck cantilever (photograph 38).

The chamfer at the back of the lower nib is in good condition (photograph 39).

Between the south beams there is leaked bitumen from the above carriageway expansion joint and corrosion staining to the T-beam (photograph 40).

4.3.4 Bearings

The bearings to the south side appear to have a metallic plate in place, the purpose of which could not be confirmed (photographs 41 & 42). This is not shown on available record drawings.

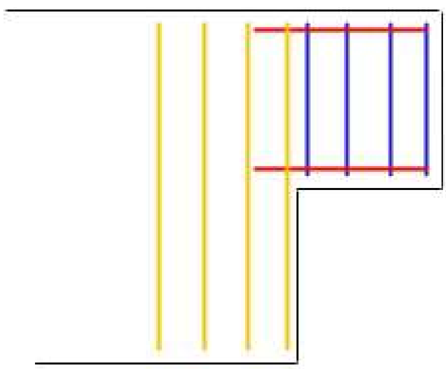
4.3.5 Reinforcement Scanning

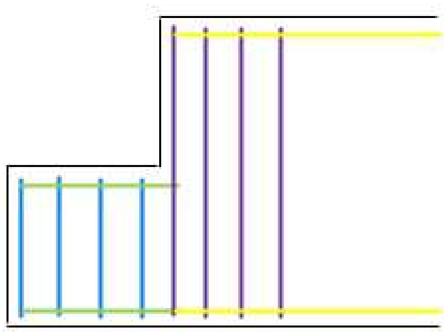
Localised scanning of the half-joints was undertaken using a Ferroscan and GPR. Areas of the half-joint which were scanned included, the elevations of the upper nib and lower nib in the box edge beams and the cantilever soffit. The purpose of the scanning was an attempt to confirm the diameter and spacings of reinforcement shown on available 'record' drawings as to provide confidence in the 'record' drawings.

Note: No intrusive works were commissioned by the Client as part of these works so caution must be taken when using information obtained from the scanning as the details have not been confirmed via concrete breakout. Exact matches in reinforcement details is not expected between 'record' drawings and the scanning due to construction tolerances and accuracy of the scanning equipment and on site conditions. Comparison of data however, will indicate a level of confidence as to how accurate the 'record' drawings are with constructed details.

In general, the spacing of reinforcement observed by scanning does not coincide with the details expected from reviewing record drawings. It is difficult to ascertain the accuracy of the scanned data considering the volume of reinforcement within the half joints. It is therefore suggested that, since the typical size of bar matches those shown on record drawings, the spacing of bars is determined from the record drawings. Should the Client want a more accurate representation of the reinforcement layout, it is recommended that local breakouts are undertaken.

Upper nib:			
Shear reinforcement:	20mm diameter	Cover = 40mm	Photograph 43
Bending reinforcement:	20mm diameter	Cover = 40mm	Photograph 44
In deck cantilever:			
Shear reinforcement:	19mm diameter	Cover = 80mm	Photograph 47
Bending reinforcement:	20mm diameter	Cover = 50mm	Photograph 49
Lower nib:			
Shear reinforcement:	18mm diameter	Cover = 70mm	Photograph 45
Bending reinforcement	12mm diameter	Cover = 80mm	Photograph 46
Top of drop-in span:			
Shear Reinforcement:	20mm diameter	cover = 35mm	Photograph 48

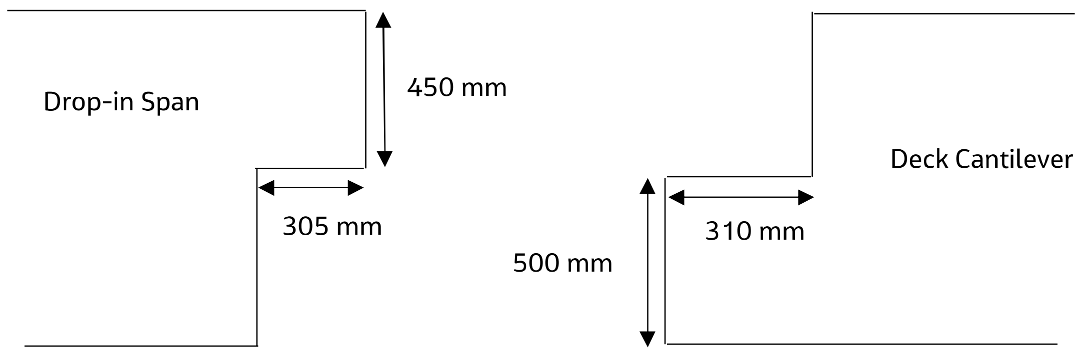




4.3.6 Survey of Half-Joints

	Design Calculations		Record Drawings		Inspection Measurements	
	(ft / in)	(mm)	(ft / in)	(mm)	(ft / in)	(mm)
Lower nib	5 1/2" x 17 3/8"	140mm x 440mm	12" x 1'5"	305mm x 430mm	-	310mm x 500mm
Upper nib (external)	9" x 20"	228mm x 508mm	1' x 1'8"	305mm x 508mm	-	*305mm x 450mm
Upper nib (internal)	9" x 16"	228mm x 406mm	1' x 1' 4"	305mm x 405mm	-	-

*Note: It is noted that the parapet upstand may mask the vertical extent (450mm / 508mm) of the element.



5. Inspection Conclusions and Recommendations

5.1 Conclusions

The expansion joints at carriageway level are in poor condition with cracking and potholing evident in the vicinity of each joint. The cracking, potholing and vegetation growth will contribute to more rapid deterioration of the half-joints and the wider structure as ponding water freeze-thaws and vegetation continues to grow.

A borescope inspection of both nibs at both half joints shows cracking is evident towards the elevations. The cracking appears to be minor and there are no signs of loss of concrete or deteriorating reinforcement. Due to the lack of access into the half joint it is difficult to ascertain the length, width and significance of the cracks.

Typically, there are cracks emanating from the re-entrant corner of the lower nib. Each crack is hairline (< 0.3mm wide), showing no signs of increased movement (considering the findings of historical inspection reports) and are not considered to be of significant concern at present.

One of the objectives of the half joint inspection was to confirm that dimensions on site match those shown on record drawings and hence confidence could be taken that the record drawings are a true representation of the structure. However, the upper and lower nibs of the half joints appear to have different depths to those shown on the record drawings, and so it has to be concluded that the record drawings aren't wholly reliable.

It is suggested that for assessment purposes, the size of the upper and lower nib is taken as physically measured. It is further recommended that, where there is no confirmation of reinforcement detail by breakout and inspection, the reinforcement layout as shown on record drawings is used for assessment since this seems relatively consistent with that noted by scanning techniques.

5.2 Condition Factor for Assessment

Despite the frequency of cracking noted to the upper and lower nibs within the half joint, and the inability to determine the significance of the cracks without access on to the bearing shelf (without a borescope), it is considered that for assessment purposes, the condition factor should be reduced from unity.

Recommended condition factor = 0.9

In the event that the half joints are determined to be under capacity, the cracks should be considered for further investigation by non-destructive means where possible.

5.3 Recommendations

It is recommended that:

- The carriageway and verges are resurfaced,
- The verges are cleared of debris (any saplings should be treated prior to removal),
- Type 2 (asphaltic plug) expansion joints are installed to the carriageway and type 1 installed to the verges.
- The existing cracks on elevation and internally to the half joints are monitored at future inspections.
- The bearings are monitored at future principal inspections (a borescope will be required).

Appendix A. Inspection Photographs

Photograph 1 - View on underbarrow road looking west

Photograph 2 - View on underbarrow road looking east

Photograph 3 - North elevation

Photograph 4 - South Elevation

Photograph 5 – View along east half joint showing poor alignment of deck beams.

Photograph 6 - View north on east half joint

Photograph 7 - Potholing close up on east half joint

Photograph 8 - Potholing to centreline of carriageway above east half joint

Photograph 9 - Cracking to north side of carriageway at east half joint

Photograph 10 – South-east parapet expansion

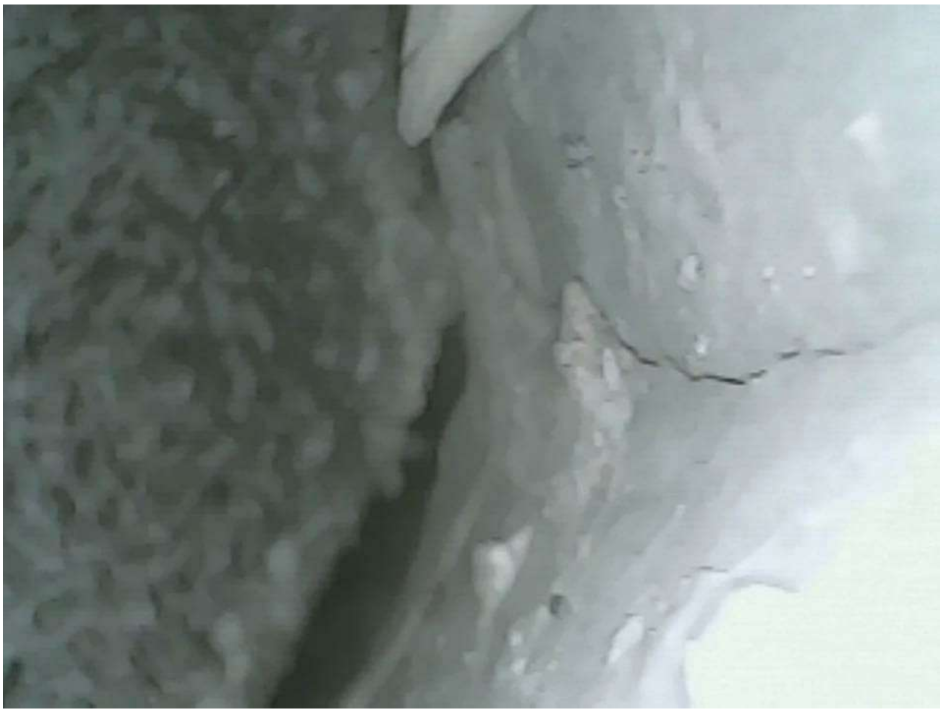
Photograph 11 - Timber formwork left in place between southern beams

Photograph 12 - Polystyrene in joint

Photograph 13 - Timber and polystyrene between edge box beam and internal beam around diaphragm cap.

Photograph 14 – North-east half joint

Photograph 15 – South-east half joint. Note, previous repair and exposed reinforcement.



Photograph 16 - Narrow crack to upper nib of T-beam (3rd from north)

Photograph 17 - Crack to upper nib (2nd beam from south)

Photograph 18 - Cracking and previous repairs to north-east half joint

Photograph 19 - Close up on cracking to north-east half joint

Photograph 20 - Leachate staining to cantilever soffit.

Photograph 21 - Crack to lower nib (north elevation)

Photograph 22 - Second crack to lower nib

Photograph 23 - Crack to lower nib (2nd beam from south)

Photograph 24 - Crack to lower nib of south box beam.

Photograph 25 - Cracking to deck cantilever behind half joint (not associated with half joint)

Photograph 26 - Cracking to deck cantilever soffit behind half joint (not associated with half joint)

Photograph 27 - View north on west half joint

Photograph 28 - View south on west half joint

Photograph 29 - North verge west half joint

Photograph 30 - South verge west half joint

Photograph 31 - Cracking to sealant at west half joint

Photograph 32 - Potholing at west end

Photograph 33 - Timber and polystyrene left in place between beams.

Photograph 34 – South-west half joint. Note, previous repair and minor spalls.

Photograph 35 – North-west half joint

Photograph 36 - Shrinkage cracking to northern T-beam upper nib.

Photograph 37 - Algal staining and previous repair to north-west half joint. Note, the joint is fully sealed.

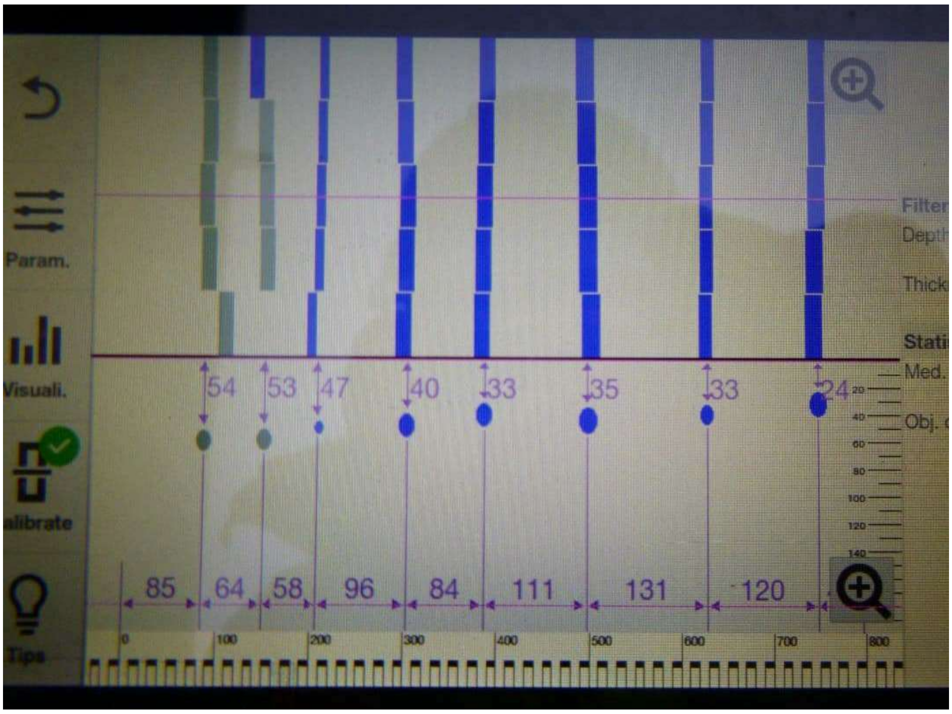
Photograph 38 - Leachate staining to deck soffit.

Photograph 39 - Lower nib in good condition

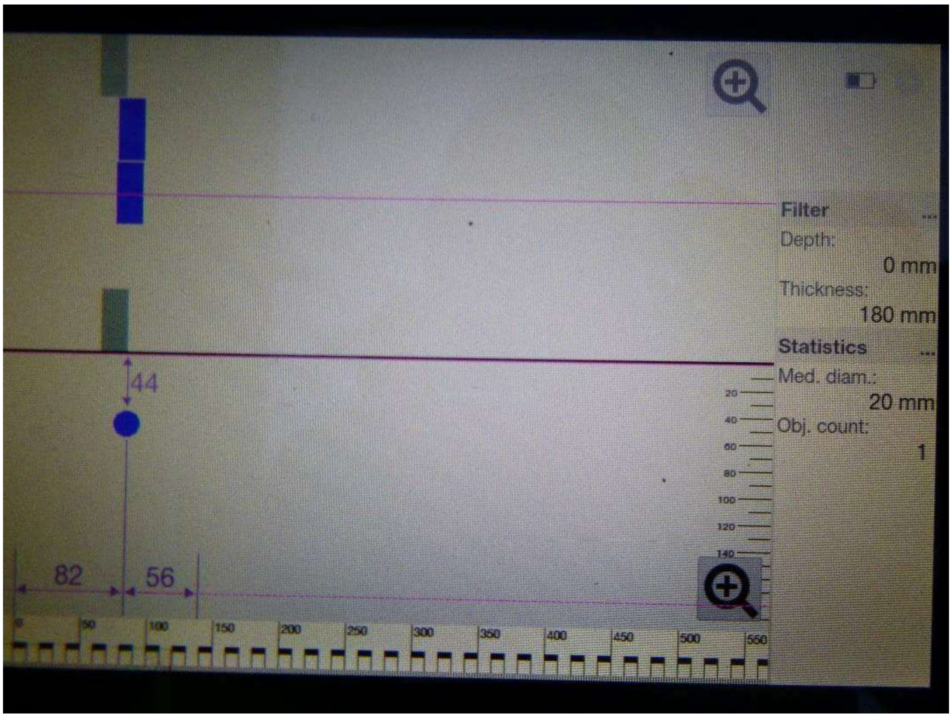
Photograph 40 - Bitumen leak and rust staining between south beams

Photograph 41 - Bearing keep plate to south side of west half joint

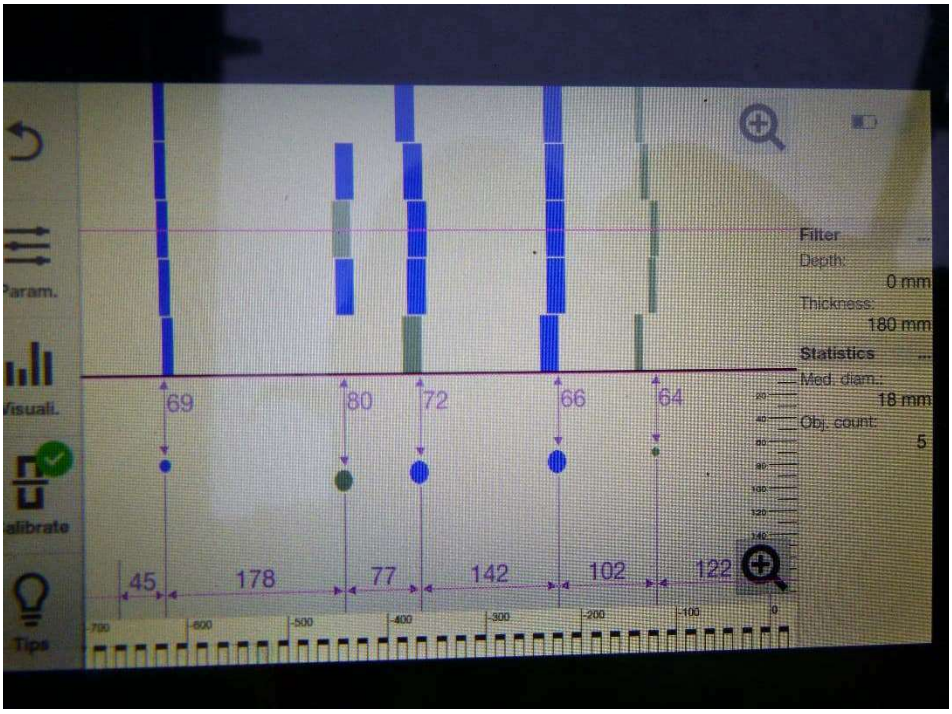
Photograph 42 – Metal plate between beams on the south edge



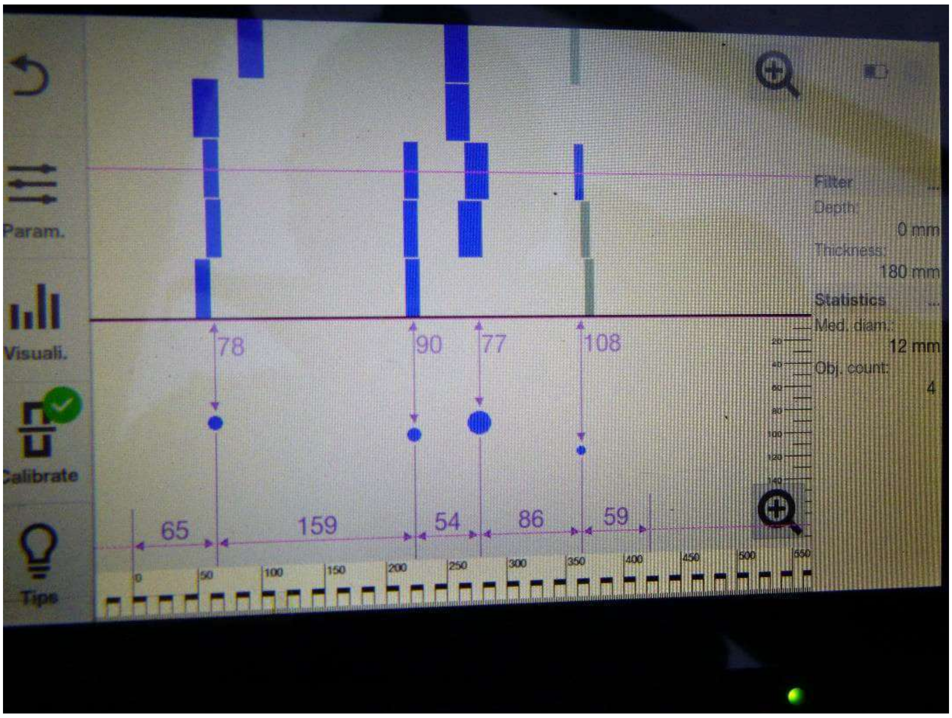
Photograph 43 - Upper nib horizontal scan showing shear links.



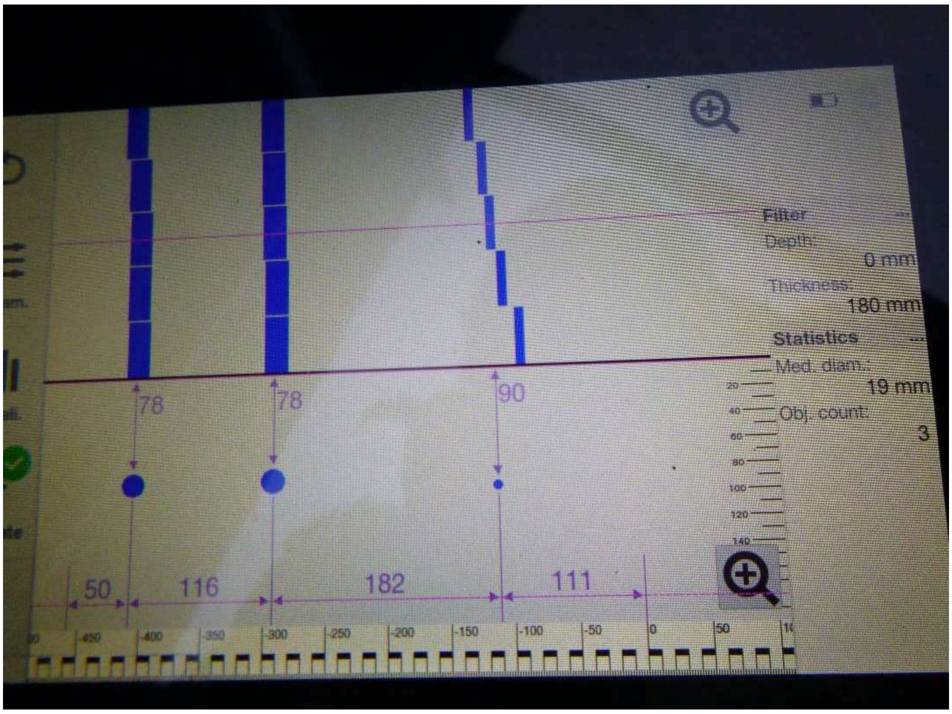
Photograph 44 - Upper nib vertical scan showing bending reinforcement.



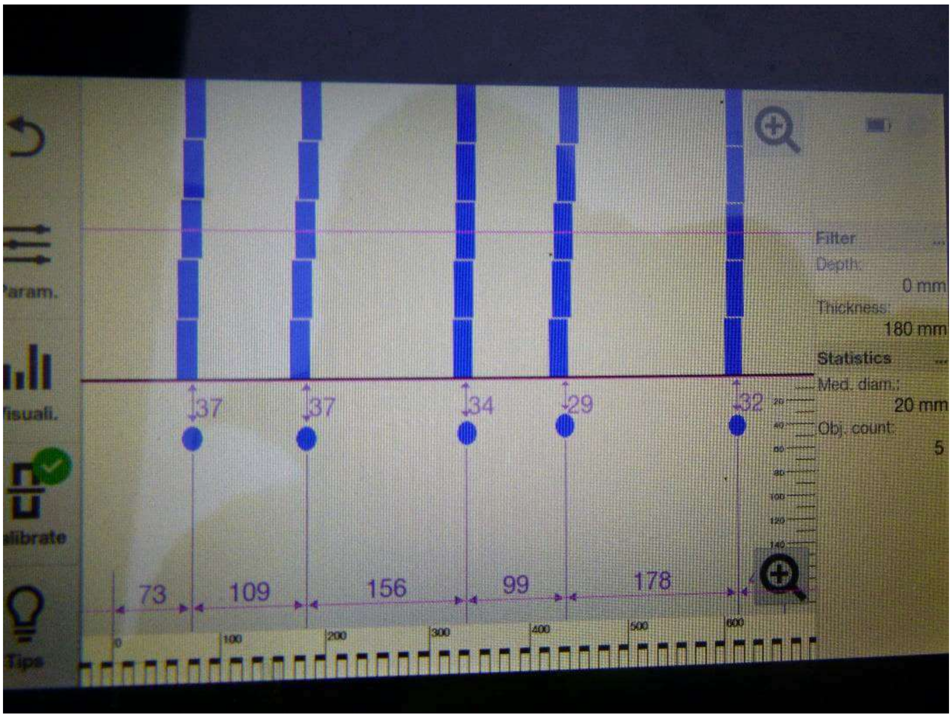
Photograph 45 - Lower nib horizontal scan showing shear links.



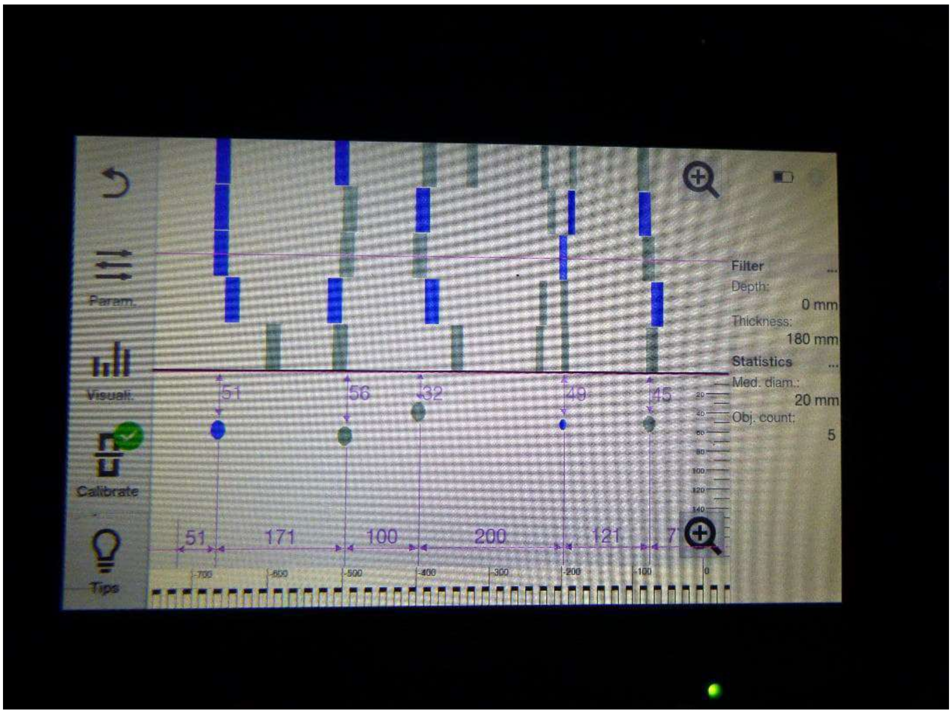
Photograph 46 - Lower nib vertical scan showing bending reinforcement.



Photograph 47 - Horizontal scan on elevation showing shear links

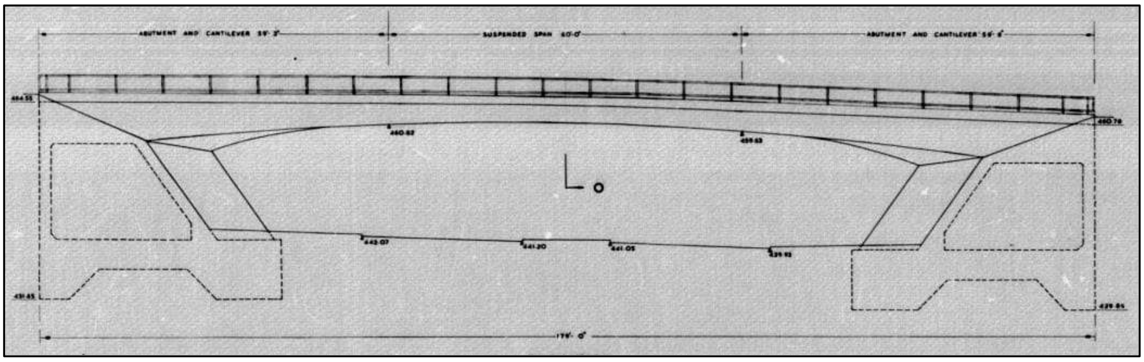


Photograph 48 - Horizontal scan on external beam showing shear links.

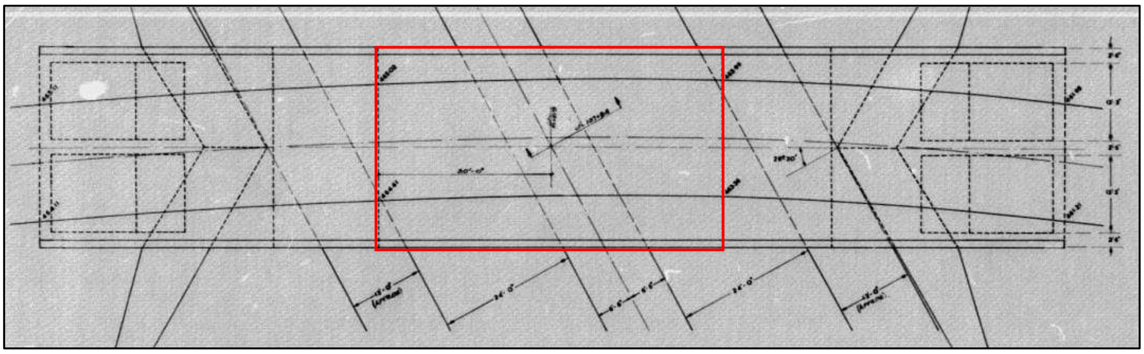


Photograph 49 - Scan on cantilever soffit showing bending reinforcement.

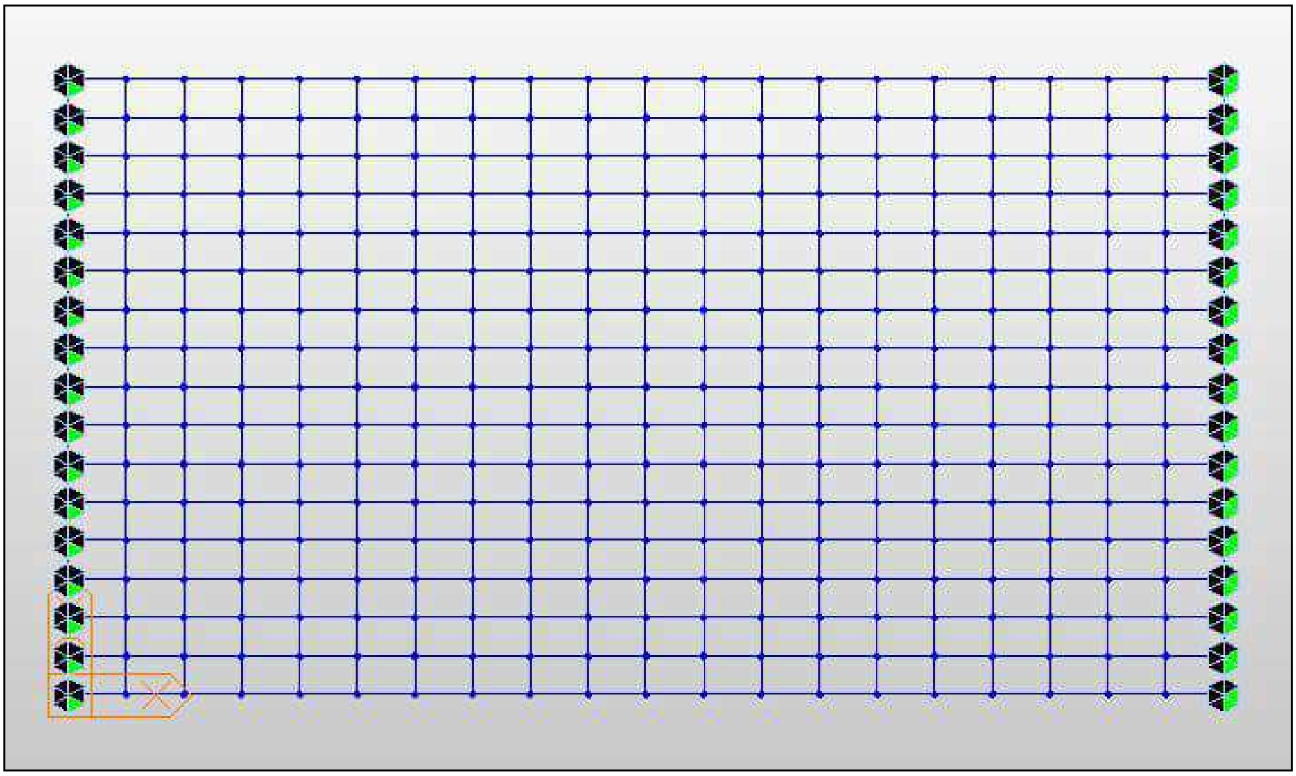
Appendix C. Idealised Diagrams



Elevation from drawing 584/16/3/1A.

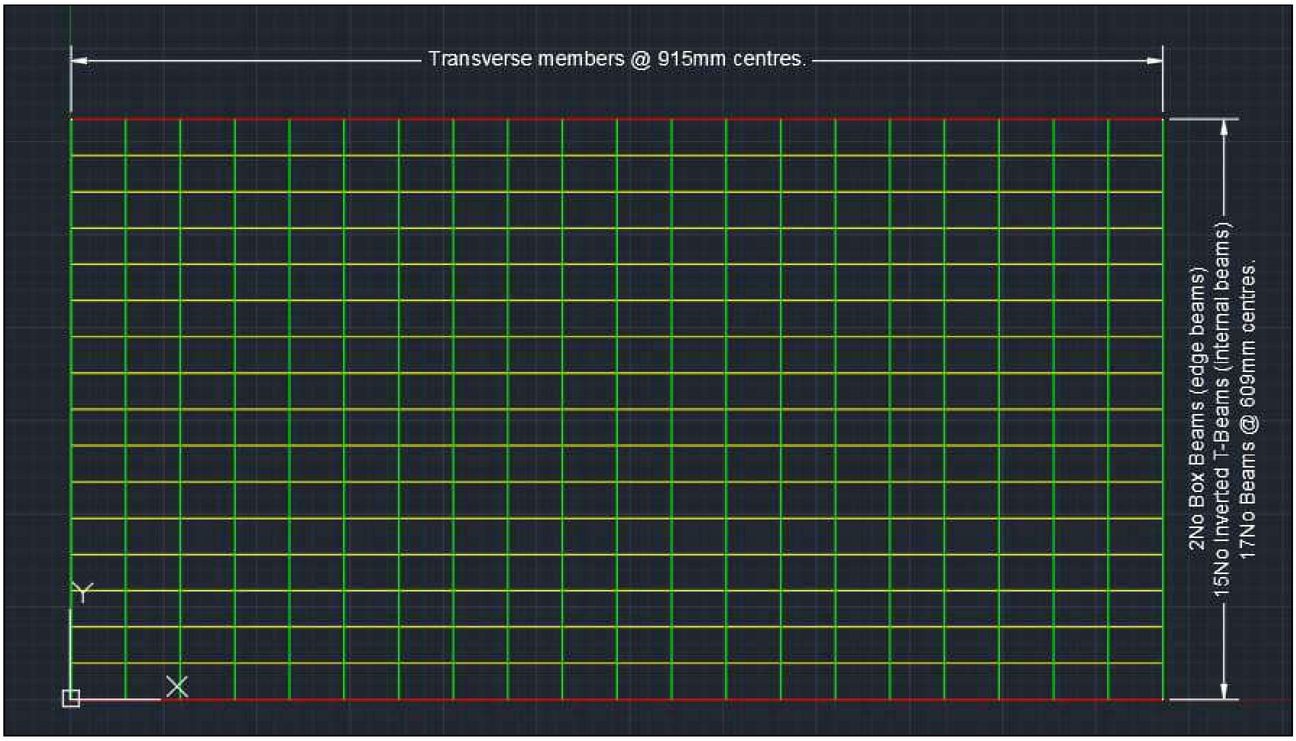


Plan from drawing (Underbarrow Overbridge General Layout) showing suspended span is square.

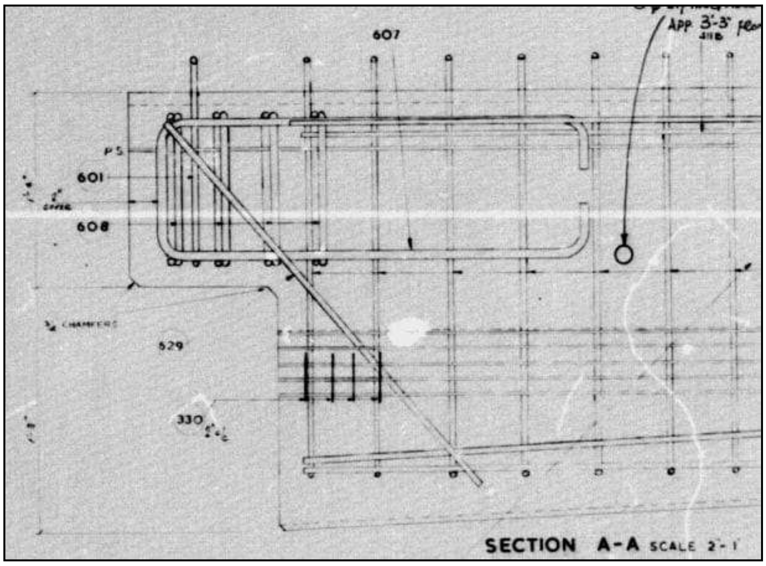


Idealised Diagram for determination of reaction forced on half joints at the ends of the suspended span.

West abutment: fixed in DZ direction only. East abutment: fixed in DZ and DX directions.



AutoCAD sketch of the above grillage showing spacing of grillage members.



Drg 586/16/3/6A showing section through suspended span external beams (internal beam similar).

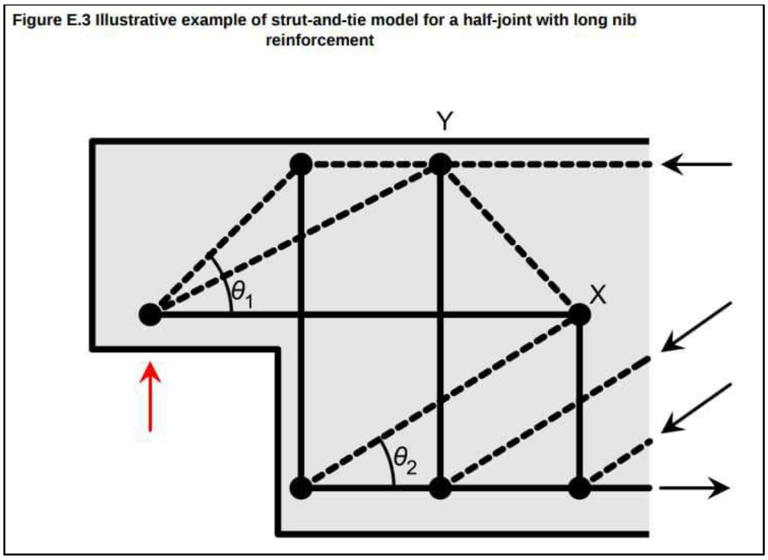


Figure E.3 of CS 466 showing idealised strut and tie model, assuming longitudinal reinforcement is as shown on record drawings).

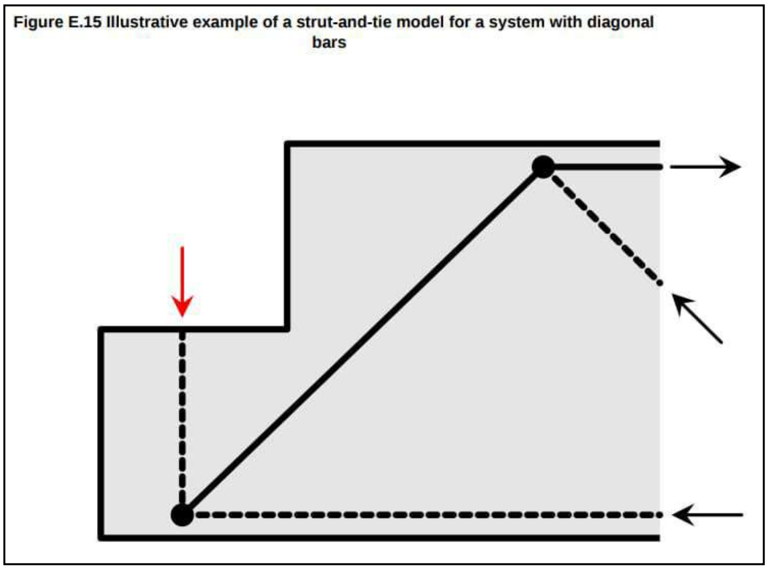
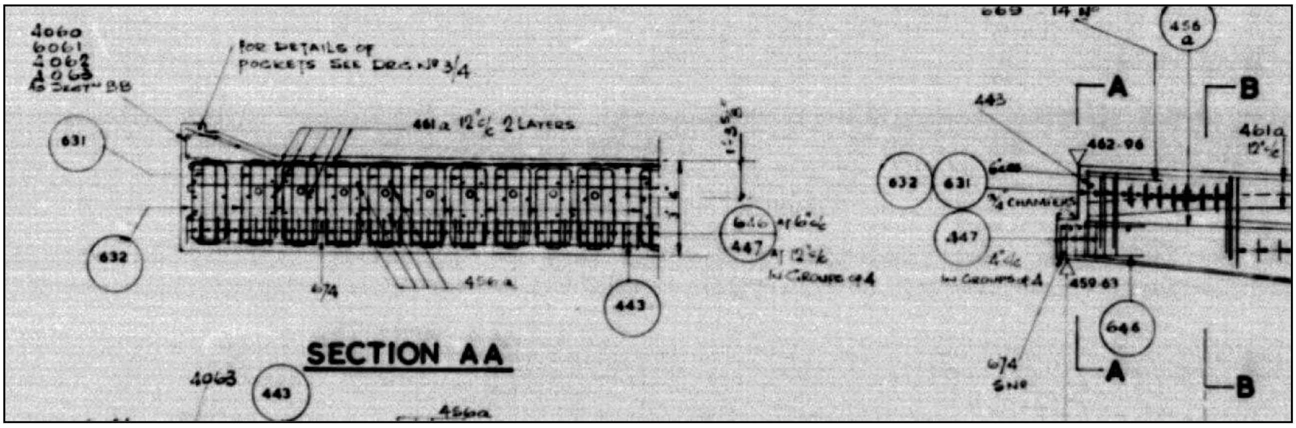


Figure E.15 of CS 466 showing idealised strut and tie model for top nib diagonal reinforcement (joint shown inverted).



586/16/3/3C – Underbarrow Lower nib details (Brigsteer similar).

Figure E.16 Illustrative example of a strut-and-tie model for a system with vertical bars

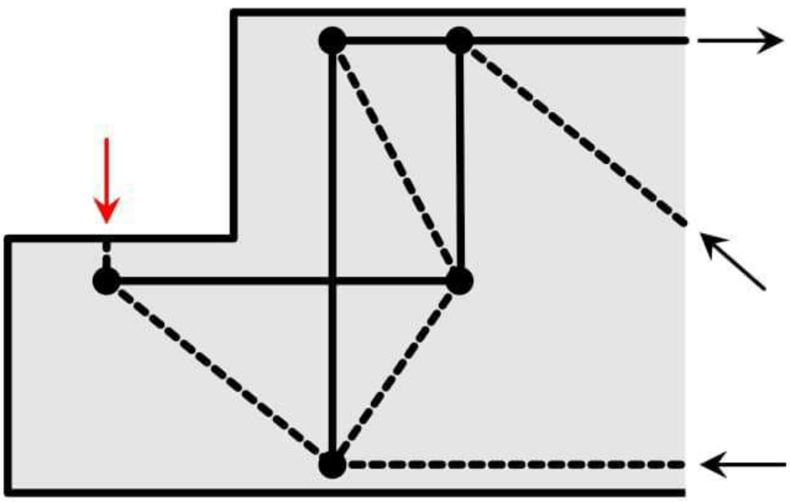


Figure E.16 of CS 466 – Idealised strut and tie model for lower nib considering no diagonal reinforcement.

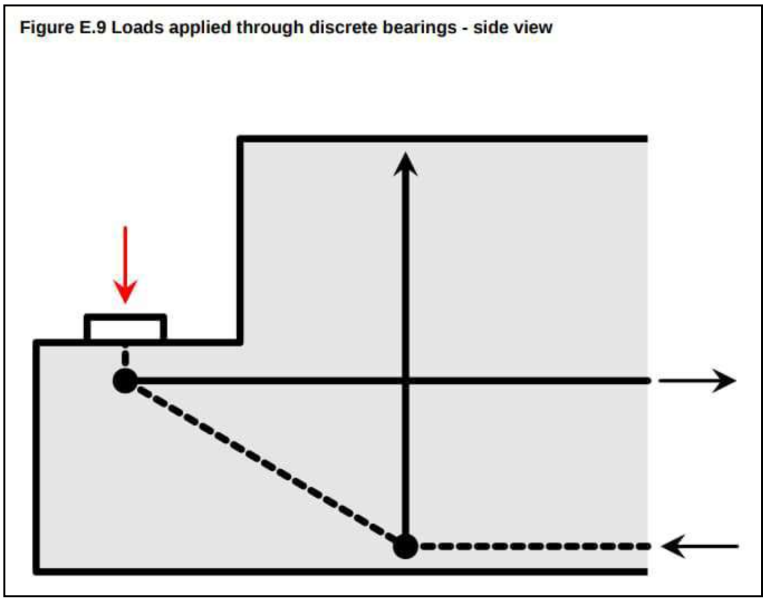


Figure E.9 of CS 466 showing idealised strut and tie model for loads applied through discrete bearings.

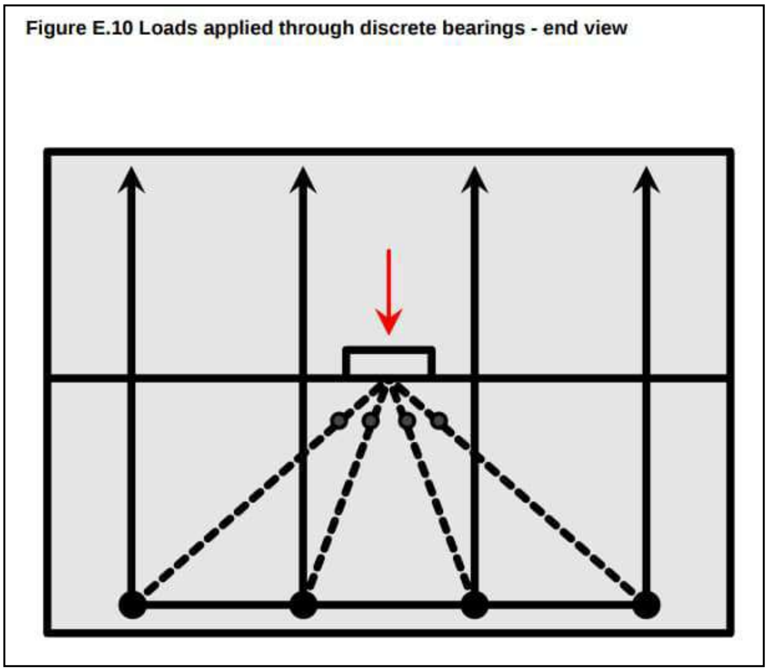


Figure E.9 of CS 466 showing idealised strut and tie model for loads applied through discrete bearings.

Appendix D. Assessment Certificate

Project details:

Name of Project	Risk Assessment and Structural Assessment of Post-Tensioned and Half-Joint Bridges SL240 Brigsteer and SL221 Underbarrow
Name of Bridge or Structure	Underbarrow
Structure No.	SL221

Section 1

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

1) It has been assessed in accordance with

    b. The Approval in Principle Report dated 12th January 2023.

2)

    b. The assessed capacity of the structure, or elements of the structure, is as follows:

    Half-Joints: inadequate for dead load.

3) Not used.

Signed [redacted]

Name [redacted]

Assessment Team leader

Engineering Qualifications CEng MICE

Signed [redacted]

Name [redacted]

Position held [redacted]

Name of Organisation Jacobs UK. Ltd

Date 03/07/2024

Section 2

The certificate is accepted by the TAA

Signed	[redacted]
Name	[redacted]
Position held	[redacted]
Engineering Qualifications	BEng(Hons) CEng MICE
TAA	Westmorland and Furness Council
Date	03/07/2024

Appendix E. Assessment Check Certificate

Project details:

Name of Project	Risk Assessment and Structural Assessment of Post-Tensioned and Half-Joint Bridges SL240 Brigsteer and SL221 Underbarrow
Name of Bridge or Structure	Underbarrow
Structure No.	SL221

Section 1

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

1) It has been checked in accordance with

    b. The Approval in Principle Report dated 12th January 2023.

2)

    b. The assessed capacity of the structure, or elements of the structure, is as follows:

    Half-Joints: Inadequate for dead load.

3) Not used.

Signed [redacted]

Name [redacted]

Check Team leader

Engineering Qualifications BEng MSc CEng MICE

Signed [redacted]

Name [redacted]

Position held [redacted]

Name of Organisation

[redacted]

Date 25/06/2024

Section 2

The certificate is accepted by the TAA

Signed [redacted]

Name [redacted]

Position held [redacted]

Engineering Qualifications BEng(Hons) CEng MICE

TAA Westmorland and Furness Council

Date 03/07/2024