JACOBS

BRB (Residuary) Ltd
VAR9/830

03/04 BE4 Assessment Programme

ASSESSMENT AND INSPECTION REPORT

BE 4 1967 Assessment

Structure AGB/3

Revised: March 2006

JACOBS

Document control sheet   Form IP180/B

Client BRB (Residuary) Ltd
Project 03/04 BE4 Assessment Programme   Job No J20308B
Title Structure AGB/3 -

	Prepared by	Reviewed by	Approved by
ORIGINAL	NAME [redacted]	NAME [redacted]	NAME [redacted]
DATE 30 July 2004	SIGNATURE	SIGNATURE	SIGNATURE

REVISION 1	NAME [redacted]	NAME [redacted]	NAME [redacted]
DATE 15 March 2006			

REVISION	NAME	NAME	NAME
DATE	SIGNATURE	SIGNATURE	SIGNATURE

REVISION	NAME	NAME	NAME
DATE	SIGNATURE	SIGNATURE	SIGNATURE

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Contents

1 General Description and Structural Details

1.1 Location

Structure AGB/3 is situated on the Gullane to Longniddry disused railway line at Grid Reference NT483797 The structure carries a public road and is named Luffness Mains Bridge

1.2 Construction type

The single span structure comprises of two longitudinal main girders with 10 No cross girders which support metallic deck plates The bridge is built to a skew of 26 5 degrees The abutments, wingwalls and pilasters are all constructed of masonry

2 Existing Information Search

2.1 Information Used to Form Assessment

The following documents were provided by BRB (Residuary) Ltd·

Historical bridge assessment report

2.2 Ground Investigation/ SI Results

An inspection pit was excavated on the bridge deck in the west verge

The excavation determined the depth and composition of the fill and the condition and level of the deck plate

A metallic sample was taken from the structure to establish the composition, see Appendix E for results

See Appendix E for the site investigation results

2.3 Existing Drawings

Sketch provided in historical bridge assessment report

3 Structure Condition

3.1 Main Girders

The east main girder is generally in good condition, the bottom flange shows no loss of section but some loss of paint system The web stiffeners are all in good condition, along with the top flange and angles The web generally shows no loss of section but some loss of paint to the external face Two holes in the web are located above each abutment (The hole above the north abutment is oval shaped and is 100mm wide and 30mm tall, the hole above the south abutment is oval shaped and is 90mm wide and 45mm tall).

The west main girder is also generally in good condition The bottom flange shows no loss of section apart from a small localised area adjacent to the face of the south abutment where a 50mm loss of width is apparent over 100mm The majority of the paint system is intact The web stiffeners are in good condition The top flange and angles are in good condition A large hole in the web is apparent over the south abutment, it is oval shaped and 170mm wide and 240mm tall and a second smaller hole is also apparent which measures 150mm wide and 30mm tall
Both inner faces of the main girders are in good condition with no section loss, but 30% of the paint system is missing

3.2 Cross Girders

The structure comprises of 10 transverse beams All except one girder are generally in good condition Some loss of paint system is recorded on the bottom flange of the beams Four of the beams span between the main girders forming a U frame, the remaining six span from the main girder to the abutment The eighth transverse beam counting from the north, shows a loss of section and a hole in the web above the abutment, the hole is oval shaped and measures 150mm wide and 30mm tall

3.3 Deck plates

The buckle plates show some loss of paint system recorded in the central area of the structure but no loss of section is apparent The plates are domed in shape and are connected by Tee sections

3.4 Bearings

The main girders bear onto steel plates and the transverse girders bear onto timber sections above the masonry abutments, the timber is in fair to poor condition

3.5 Abutments

The north abutment is constructed from masonry and is found to be in good condition with no spalling or open joints The centre third of the wall shows signs of dampness but no water is dripping The wall has been recently re-mortared beneath the north west main girder, these joints are showing signs of becoming loose and are open to approximately 30mm

The south abutment is also in good condition with no spalled masonry or open joints

3.6 Wingwalls

3.6.1 North East Wingwall

The north east wingwall is constructed of masonry and is in good condition with generally no spalled masonry or open joints The copingstones show some minor spalling and an open joint is present where they bed onto the wingwall Small trees are growing behind the wall but the wall face is free from vegetation

3.6.2 South East Wingwall

The south east wingwall is in good condition Approximately 1m^2 of open joints are apparent towards the top of the wall to a depth of 50mm No vegetation growth is apparent on the face of the wall and minor vegetation growth is found behind the wall.

3.6.3 North West Wingwall

The north west wingwall is also in good condition 2m^2 of open joints are apparent No vegetation growth is apparent on the face of the wall but some minor vegetation is growing behind the wall

3.6.4 South west wingwall

The south west wingwall is in good condition There is no vegetation growth on the face but a small tree is growing behind the wall

3.7 Carriageway

The structure carries a public road which is found to be in good condition with no surface fractures A verge is apparent on both sides of the road, the west verge is larger than the east

3.8 Pilasters

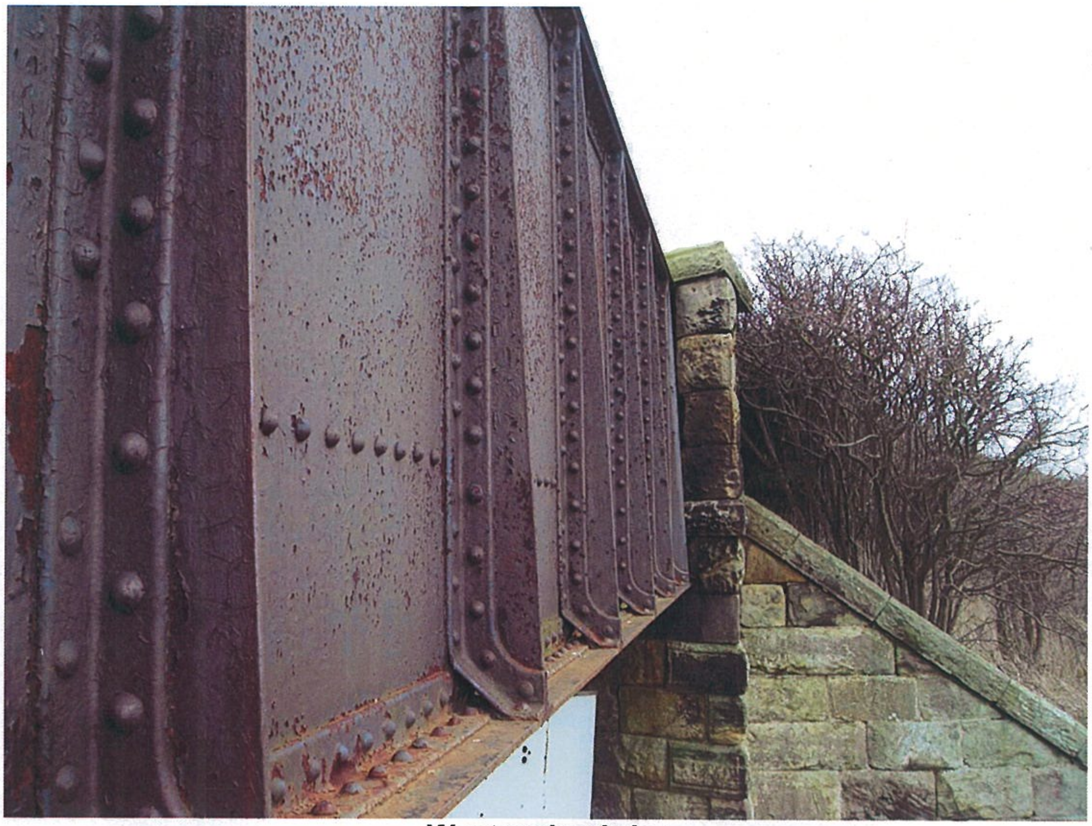
The north east, south east and south west pilasters are generally in good condition, the external face of the north west pilaster shows signs of movement and deep open joints are present

3.9 Formation

The old disused railway formation is on flat land which is now farmed either side of the structure, the formation is clear of vegetation Two trailers are parked below the structure and the bridge provides access between the two fields

Appendix A - Photographs

West elevation



West main girder

Hole in east main girder

Holes in main and cross girders in south west corner of structure


Corrosion to bottom flange of west main girder adjacent to south abutment


Deck plates and cross girders

Open joints in north west pilaster

North east wing wall

South east wing wall

South west wing wall

North west wing wall

South abutment

Trial pit exposing deck plate and tee section

Carriageway and trial pit reinstatement looking south

Appendix B - Form AA

FORM 'AA' (BRIDGES)

GC/TP0356
Appendix: 4
Issue· 1
Revision: B (Nov 2000)

ELR/ Bridge No ..**AGB 3 (VAR9 / 830)**

APPROVAL IN PRINCIPLE FOR ASSESSMENT

Bridge/Line Name . .Luffness Mains Bridge

ELR/Bridge No. . .AGB 3

Brief Description of Existing Bridge:

(a) Span Arrangement

Single skew span of 8 82m

(b) Superstructure Type

Steel girder overbridge with steel parapet walls The single span comprises of two longitudinal main girders supporting 10 no cross girders. In turn the cross girders support steel deck plates

(c) Substructure Type

Large coursed rockfaced masonry abutments.

(d) Details of any Special Features

None

Assessment Criteria

(a) Loadings and Speed

Dead loads and section sizes shall be determined from site measurements and existing drawings Vehicle loading obtained from and applied in accordance with BE4 Standard BE4 loading representative of 24 ton vehicles will be assessed

(b) Codes to be used

BE4 "The assessment of construction and use vehicles" Ministry of Transport, 1967 With Amendments to 1969

BS 153 . Parts 3B & 4 : 1958 "Steel Girder Bridges" British Standards Institution (with amendments to 12 Sept. 1968).

(c) Proposed Method of Structural Analysis

BE4/1967 will be used for the assessment

Dead and live loadings will be applied to the members by simple statics

The longitudinal girders will be treated as simply supported with compression flanges laterally supported by U-frames and assessed in accordance with BE4/1967 and BS153 Parts 3 & 4·1958.

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

Appendix: 4

ELR/ Bridge No ...**AGB 3 (VAR9 / 830)** Issue: 1

Revision: B (Nov 2000)

APPROVAL IN PRINCIPLE FOR ASSESSMENT

Measured dimensions taken from the site investigation and existing drawings are to be used in the BE4 calculations.

A flexibility coefficient of 0.5 x 10-10 Rad/Nmm is to be added as the third term for calculating δ in BS153 part 4

Capacities of the plate girders will be calculated using measurements of the reduced section sizes where corrosion is present Consequently, a general condition factor is not applied

The deck plate will be considered as an arch and assessed in compression

An inspection pit is to be excavated over the bridge deck The excavation is to determine the depth and composition of the fill above the girders and to establish their condition

A metallic sample is to be extracted from the structure to determine the composition.

(d) Details of any Special Requirements

None

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

Appendix 4
Issue 1
Revision B (Nov 2000)

ELR/ Bridge No **AGB 3 (VAR9 / 830)**

APPROVAL IN PRINCIPLE FOR ASSESSMENT

Senior Civil Engineer's Comments

None.

Proposed Category for Independent Check

Superstructure I

Substructure I

Name Of Checker Suggested If Cat 2 Or 3

Category 1

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

Signed [redacted]

Title Civil Engineer

Date 16/4/04

Category 2 and 3

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

Signed

Title

Date

Signed

Title

Date

Appendix C - Summary of BE4 Results and Recommendations

Summary of calculations (AGB 3)

Element· Transverse girders T4-T7 (full length)

Action	Location	Dead load effect	Full C&U load effect	Total load effect	Assessed resistance	Live load capacity
Bending	Mid-span	110 4 kN m	173 4 kN m	283 4 kN m	399 1 kN m	Full C&U 24 tons
Shear	Support	56 9 kN	75 6 kN	132 5 kN	355 kN	Full C&U 24 tons

Element Main girders

Action	Location	Dead load effect	Full C&U load effect	Total load effect	Assessed resistance	Live load capacity
Bending (Top flange compression buckling)	Mid-span	453 4 kN m	271 3 kN m	724 7 kN m	896 6 kN m	Full C&U 24 tons
Shear	Support	200 2 kN	126 1 kN	326 3 kN	1735 8 kN	Full C&U 24 tons
Rivet shear	Web/Angle	12 9 N/mm^2	8 1 N/mm^2	21 0 N/mm^2	85 0 N/mm^2	Full C&U 24 tons
Rivet shear	Flange /Angle	5 3 N/mm^2	3 4 N/mm^2	8 7 N/mm^2	85 0 N/mm^2	Full C&U 24 tons

Element Buckle plates

Action	Location	Allowable	Actual	Live load capacity
Plate*	Analogous strut	4090 lbs/in (714 N/mm)	907 lbs/in (207 N/mm)	Full C&U wheel (5t)
Connections	Support	106 2 N/mm^2	74 1 N/mm^2	Full C&U wheel (5t)

Element. Tee section connecting buckle plates

Action	Location	Dead load effect	Full C&U load effect	Total load effect	Assessed resistance	Live load capacity
Bending*	Mid-span	0 05 ton ft (0 16 kN m)	0 52 ton ft (1 58 kN m)	0 57 ton ft (1 73 kN m)	0 72 ton ft (2 18 kN m)	Full C&U wheel (5t)

* Thrust in the buckle plate and bending effects in the connecting tees have been derived in accordance with the findings from Jacobs report on the FE analysis of buckle plates (November 2005) See Appendix G

Recommendations
The bridge is adequate for Construction & Use loadings as defined by BE4 1967

There is extensive corrosion of the webs of both external girders and several internal girders Despite the corrosion, the girders still reach the required assessment capacity Nonetheless, the girders should be repaired regardless the outcome of the assessment This can be done by welding extra web plates over the holes The extra plates should be butt welded to the flange angles The corroded bottom flange of the west main girder, adjacent to the south abutment, is not critical because of the position, but should be treated to prevent further deterioration Maintenance painting is required throughout

The dampness of the abutments shows that the waterproofing is not effective There was no water leakage at the time of the inspection, but white calcium deposits were apparent on the abutment Deck re-waterproofing should be considered for the sustainability of the abutment/bridge

Minor masonry repairs and repointing are recommended as well as the removal of the tree behind the SW wingwall

Appendix D - Form BA

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

Appendix 4
Issue. 1
Revision A (Feb 1993)

ELR/ Bridge No **AGB 3 (VAR9 / 830)**

CERTIFICATION FOR ASSESSMENT CHECK

Assessment Group JacobsGIBB Ltd

Bridge/Line Name Luffness Mains Bridge

Category Of Check 1

ELR/Bridge No. AGB 3

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

It has been assessed in accordance with the Approval in Principle (where appropriate) as recorded

on Form AA approved on **16th April 2004** (date)

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

BE4 " The assessment of construction and use vehicles" Ministry of Transport, 1967 With Amendments to 1969

BS 153 Parts 3B & 4 1958 "Steel Girder Bridges" British Standards Institution (with amendments to 12 Sept 1968)

List any departures from the above, and additional methods or criteria adopted, with reference and justification for their acceptance (commenting on the results if appropriate)

None

Category 1

Name Signature Date

[redacted] 24/8/04

24/8/04
9.1.05

2005 Revisions

Assessor [redacted] 15/3/06

Assessment Checker [redacted] 15/03/06

Partner Of the Firm Of Consulting Engineers To Whom Assessor/ Checker Is Responsible [redacted]

FORM 'BA' (BRIDGES) **GC/TP0356**
Appendix 4
ELR/ Bridge No .. **AGB 3 (VAR9 / 830)** Issue: 1
Revision. A (Feb 1993)

CERTIFICATION FOR ASSESSMENT CHECK

Category 2 and 3 (Note Category 1 Check Must Also Be Signed)

(a) Assessment

Name	Signature	Date	
			Assessor
			Assessment Checker
			Partner Of the Firm Of Consulting Engineers To Whom Assessor/ Checker Is Responsible

(b) Check

Name	Signature	Date	
			Assessor
			Assessment Checker
			Partner Of the Firm Of Consulting Engineers To Whom Assessor/ Checker Is Responsible

This Certificate Is Accepted By .. . [redacted]

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

Appendix 4
Issue: 1
Revision· A (Feb 1993)

ELR/ Bridge No . .**AGB 3 (VAR9 / 830)**

CERTIFICATION FOR ASSESSMENT CHECK

Notification of Assessment Check

Assessment Group . JacobsGIBB Ltd

Bridge/Line Name Luffness Mains Bridge

ELR/Bridge No. AGB 3

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

Statement of Capacity

24 Tons full C&U capacity

Description of Structural Deficiencies and Recommended Strengthening

The bridge is adequate for Construction & Use loadings as defined by BE4 1967

There is extensive corrosion of the webs of both external girders and several internal girders Despite the corrosion, the girders still reach the required assessment capacity Nonetheless, the girders should be repaired regardless the outcome of the assessment Maintenance painting is required throughout

The dampness of the abutments shows that the waterproofing is not effective There was no water leakage at the time of the inspection, but white calcium deposits were apparent on the abutment Deck re-waterproofing should be considered for the sustainability of the abutment/bridge

Minor masonry repairs and repointing are recommended as well as the removal of the tree behind the SW wingwall

2005 Revisions

Name	Signature	Date
[redacted]		4/8/04
		24/8/04
		1.9.04

Assessor [redacted] 15/3/06

Assessment Checker [redacted] 15/03/06

Partner Of the Firm Of Consulting Engineers To Whom Assessor/ [redacted] 7/3/06

This Certificate Is Accepted By. .. [redacted]

Appendix E - Site Investigation Results

STRUCTURAL SOILS LTD

BRIDGE INVESTIGATION

BE4 ASSESSMENT PROGRAMME

CONTRACT NO. 36206

BRIDGE AGB/3

Site Description

The investigation on Bridge AGB/3 was carried out for and on the instructions of Jacobs The bridge is located on a minor road adjacent to Luffness Mains, off the A198, 1.5km south of Gullane, East Lothian, at National Grid Reference NT 483797

The bridge carries a minor road over a disused railway line and comprises half through main girders and cross girders with deck plates The investigation was carried out to provide information for the structural assessment of the bridge

Sitework was undertaken on 10th February 2004

Fieldwork

1 no metal sample, <25mm x 25mm in size, was taken from part of the main girder spanning between abutments at a position specified by Jacobs The sample was labelled and sent to a UKAS accredited laboratory for testing

Laboratory Testing

The chemical composition and grading of the sample was determined using combustion and ICP AES techniques The results are presented overleaf

SHEFFIELD
TESTING LABORATORIES
50 56 Nursery Street Sheffield S3 8GP U K
Telephone 0114 272 6581 Fax 0114 272 3248 e mail liq@sheffieldtesting com

TEST CERTIFICATE

UKAS TESTING
0136

Date 25 March 2004

Serial No 4030923

ORDER NO. [redacted]

OUR REF AB/AJH

CLIENT: STRUCTURAL SOILS LTD
CHEVET HOUSE, A1 GREAT NORTH ROAD
KNOTTINGLEY, WEST YORKS, WF11 0BS

Results of Chemical Analysis of Four Samples,

STL Test No.	B578	B579	B580	B581
Sample Description	Metal Sample	Metal Sample	Metal Sample	Metal Sample
Sample Identification	BE4 BRIDGES DAS 2/20	BE4 BRIDGES AGB/3	BE4 BRIDGES AGB/5	BE4 BRIDGES FHB/1043
	Mass %			
Carbon	0 15	0 16	0 15	0 14
Silicon	<0 02	<0 02	<0 02	<0 02
Manganese	0 55	0 53	0 53	0 44
Phosphorus	0 063	0 048	0 039	0 063
Sulphur	0 058	0 050	0 038	0 038
Chromium	<0 02	<0 02	<0 02	<0 02
Molybdenum	<0 02	<0 02	<0 02	<0 02
Nickel	<0 02	<0 02	<0 02	<0 02

Determined by Combustion & ICP OES Techniques

Sample B578 is mild steel similar to BS970 Part 1 Grade 040A12 but with phosphorus and sulphur above the specified maximum of 0 050%

Sample B579 is mild steel conforming to BS970 Part 1 Grade 040A12

Sample B580 is mild steel conforming to BS970 Part 1 Grade 040A12

Sample B581 is mild steel similar to BS970 Part 1 Grade 040A12 but with phosphorus above the specified maximum of 0 050%

This certificate is issued supplementary to and replaces certificate Serial No. 4020475.

Certified .... [redacted] . .
ry

[redacted] Materials Laboratory Manager

Form TC UP 01

STRUCTURAL SOILS

TRIAL PIT LOG

Contract	Client	Trialpit No
BE4 BRIDGES - AGB/3	JACOBS	TP 01

Job No	Date	Ground Level	Co-Ordinates	Sheet
36206	10.02.04			1 of 1

Samples and In-situ Tests: Depth	No	Type	Results	Water	Description of Strata	Depth (Thickness)
					Topsoil	0.28
					MADE GROUND Brown slightly clayey sandy fine to coarse GRAVEL of assorted lithology Occasional cobbles.	0.43
					MADE GROUND Brown slightly sandy slightly gravelly CLAY Gravel is sub angular fine to coarse shale and mudstone.	0.49
					MADE GROUND . Black tar.	0.52
					Inspection pit terminated at 0 52m on steel	

Plan (Not to Scale)

0 65 × 0.35

Bearing o

General Remarks

1 EASE OF EXCAVCATION EASY
2 SIMILARITY OF FACES ALL SIMILAR
3 STABILITY OF FACES : ALL STABLE
4 WATER : NONE ENCOUNTERED

All dimensions in metres Scale	Method	Logged By	Checked By
1:25	Inspection pit	MD	

Appendix F - Calculations

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CALCULATION COVER SHEET

Calc No.:	31
Project Title:	**VAR9/830 BE4 Assessments – Structure AGB 3**
Index Page:	
Job No:	**J20308B**
File:	

Subject: **BE4 Assessment of Structure AGB 3 Luffness Mains Bridge**

Elements assessed External girders
Internal girders
Buckle plates/Rivets

The referenced calculations have been reviewed against the requirements of the task specification It is confirmed comments and reservations have been resolved satisfactorily

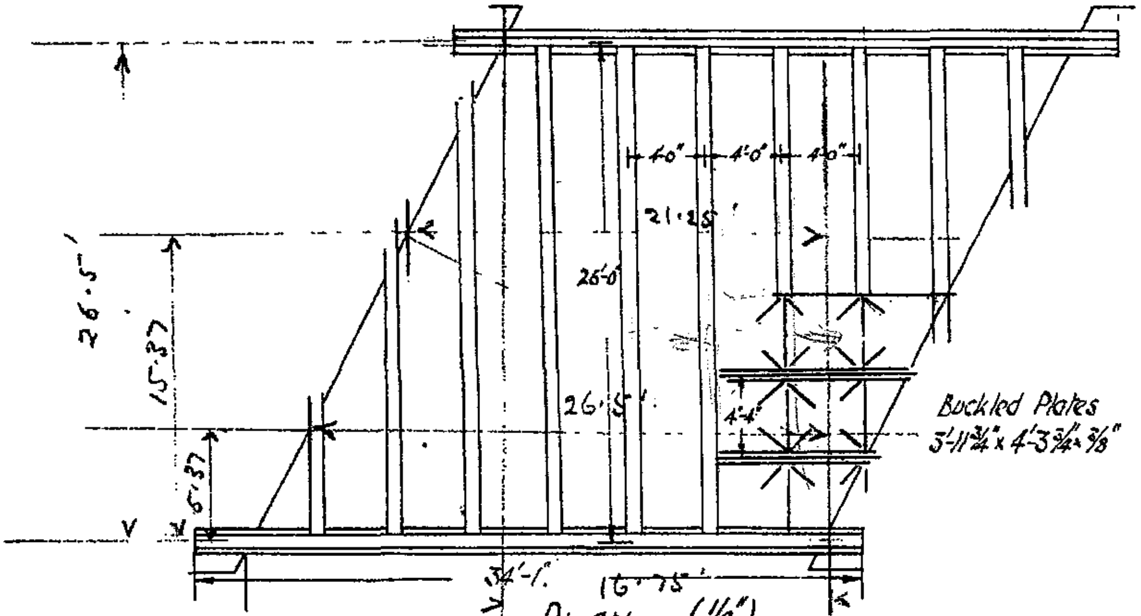
	Sheet Nos	Made by/ date	Checked by/ date	Reviewed by/ date
ORIGINAL	63	[redacted] July 2004	[redacted] August 2004	
REV ____				
REV ____				
REV ____				
REV ____				

Superseded by Calculation No Date:

Summary and conclusions:

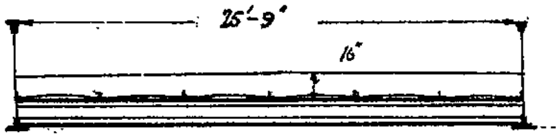
GULLANE BRANCH

BRIDGE No 3
LUFFNESS MAINS

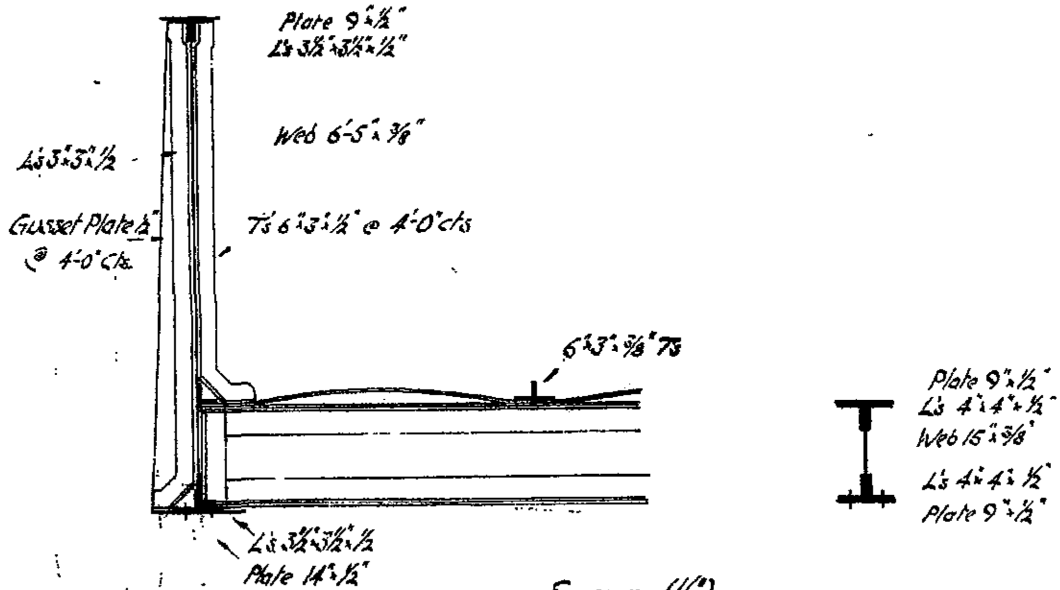


PLAN – (1/8")

2



SECTION – (1/8")



SCALE – (½")

Drawings 76/7 Store

¾" Rivets @ 4" Pitch

CALCULATION SHEET

Project Title. **VAR9-830 BE4 Assessments**			
Subject: **AGB 3 Luffness Mains Bridge BE4 Assessment**			Calc No 31
Job No. **J20308B-1142**			File.
Made by [redacted]	Date. **19/07/2004**	Checked by.	Date
Revised [redacted]	Date. Aug 04	Checked by.	Date.

AGB 3 LUFFNESS MAINS BRIDGE BE4 ASSESSMENT

Summary of Calculations

Assessment of External Girder

Dead Load Moment M_d =	149 24 tonft	
Moment of Resistance to Live Load M_l =	~~89 44~~ tonft	89 03
Unrestrained Bending Capacity M_{cr} =	~~1048.334~~ tonft	295.8
External girder is OK in unrestrained bending		
Shear Force due to Dead Load SF_d =	20 09 tons	✓
Shear Force due to Live Load SF_l =	12 657 tons	✓
Shear Capacity SF_c =	174 tons	✓
External girder is OK in shear		
Internal Girder Assessment Result	PASS	

Assessment of Internal Transverse Girder

Dead Load Moment M_d =	36 319 tonft	✓
Moment of Resistance to Live Load M_l =	~~57.724~~ tonft	57.078
Bending Capacity M_c =	399 118 tonft	✓
Internal girder is OK in bending		
Shear Force due to Dead Load SF_d =	~~5,711~~ tons	5.709
Shear Force due to Live Load SF_l =	~~7.636~~ tons	7.585
Shear Capacity SF_c =	36 tons	✓
Internal girder is OK in shear		
Internal Girder Assessment Result	PASS	

Assessment of Girder / Buckle Plate Connection

Number of Rivets on Plate n_P =	12	✓
Number of Rivets Required n_{reqd} =	5	
Girder / Buckle Plate Connection Assessment Result	PASS	

CALCULATION SHEET

Project Title. **VAR9-830 BE4 Assessments**			
Subject. **AGB 3 Luffness Mains Bridge BE4 Assessment**			Calc No.
Job No. **J20308B-1142**			File.
Made by. [redacted]	Date. **19/07/2004**	Checked by:	Date
Revised b[redacted]	Date. Aug 04	Checked by.	Date.

BE4 Assessment Result

BE4 Assessment Result PASS ✓

Calculation Contents

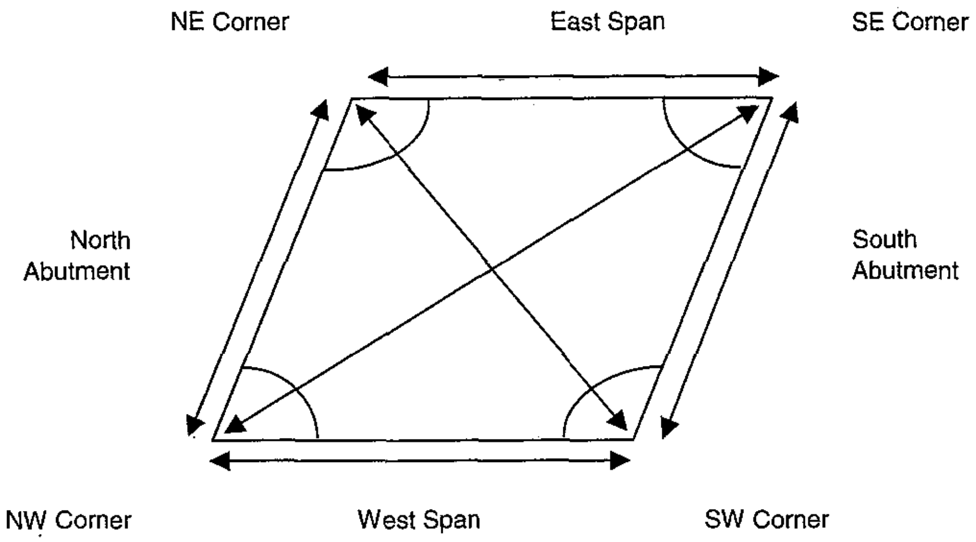
BD21/97 Assessment Summary

External Girder Assessment Result	N/A
Internal Transverse Girder Assessment Result	N/A
Girder / Buckle Plate Connection Assessment Res	N/A

CALCULATION SHEET

Project Title. **VAR9-830 BE4 Assessments**			
Subject. **AGB 3 Luffness Mains Bridge BE4 Assessment**			Calc No.
Job No. **J20308B-1142**			File
Made by [redacted]	Date. **19/07/2004**	Checked by.	Date
Revised [redacted]	Date. Aug 04	Checked by.	Date.

Skew Check



North Abutment Length L_N =	9280	mm	(Site notes)
South Abutment Length L_S =	9310	mm	(Site notes)
West Span L_W =	8750	mm	(Site notes)
East Span L_E =	8820	mm	(Site notes)
NW - SE Diagonal $L_{NW\text{-}SE}$ =	15400	mm	(Site notes)
NE - SW Diagonal $L_{NE\text{-}SW}$ =	9475	mm	(Site notes)

$L_E > L_W$

Skew Span $L_{skew} = L_E$ = 8820 mm

NW Corner Angle α_{NW} =	63 325	deg	(Cosine rule)
NE Corner Angle α_{NE} =	116 581	deg	(Cosine rule)
SW Corner Angle α_{SW} =	63 199	deg	(Cosine rule)
SE Corner Angle α_{SE} =	116 271	deg	(Cosine rule)

Total Angles $\alpha_t = \alpha_{NE} + \alpha_{NW} + \alpha_{SW} + \alpha_{SE}$ = 359 376 deg

Angle Misclosure $\alpha_{misc} = 360 - \alpha_t$ = 0 624 deg

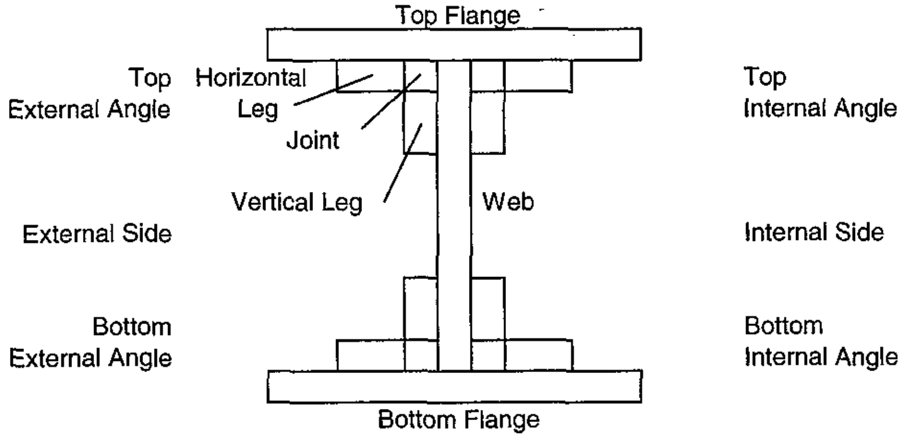
Skew Angle $\alpha_{skew} = (90 - \alpha_{NW}) + (\alpha_{NE} - 90) + (90 - \alpha_{SW}) + (\alpha_{SE} - 90) / 4$ = 26 582 deg ✓

CALCULATION SHEET

Project Title. **VAR9-830 BE4 Assessments**			
Subject. **AGB 3 Luffness Mains Bridge BE4 Assessment**			Calc No.
Job No. **J20308B-1142**			File.
Made by. [redacted]	Date. **19/07/2004**	Checked by.	Date
Revised [redacted]	Date. Aug 04	Checked by.	Date.

Assessment of External Girder

Section Properties of Girder



Top Flange

External Top Flange Width w_{top} =	9 in
External Top Flange Width w_{top} =	228 6 mm
External Top Flange Thickness t_{top} =	0 5 in
External Top Flange Thickness t_{top} =	12 7 mm

Bottom Flange

External Bottom Flange Width w_{bot} =	14 in
External Bottom Flange Width w_{bot} =	355 6 mm
External Bottom Flange Thickness t_{bot} =	0 5 in
External Bottom Flange Thickness t_{bot} =	12 7 mm

Web

External Web Thickness t_{web} =	0 375 in
External Web Thickness t_{web} =	9 525 mm
External Girder Depth Midspan d =	78 in
External Girder Depth Midspan d =	1981 2 mm
External Web Depth $d_{web} = d - t_{top} - t_{bot}$ =	1955 8 mm

CALCULATION SHEET

Project Title: **VAR9-830 BE4 Assessments**			
Subject. **AGB 3 Luffness Mains Bridge BE4 Assessment**			Calc No
Job No. **J20308B-1142**			File.
Made by [redacted]	Date. **19/07/2004**	Checked by.	Date.
Revised [redacted]	Date. Aug 04	Checked by.	Date

Top External Angle Horizontal Leg

Horizontal Leg Width w_{TEAHL} =	3 in
Horizontal Leg Width w_{TEAHL} =	76 2 mm
Horizontal Leg Thickness t_{TEAHL} =	0 5 in
Horizontal Leg Thickness t_{TEAHL} =	12 7 mm

Top External Angle Vertical Leg

Vertical Leg Thickness t_{TEAVL} =	0 5 in
Vertical Leg Thickness t_{TEAVL} =	12 7 mm
Vertical Leg Depth d_{TEAVL} =	3 in
Vertical Leg Depth d_{TEAVL} =	76 2 mm

Bottom External Angle Horizontal Leg

Horizontal Leg Width w_{BEAHL} =	3 in
Horizontal Leg Width w_{BEAHL} =	76 2 mm
Horizontal Leg Thickness t_{BEAHL} =	0 5 in
Horizontal Leg Thickness t_{BEAHL} =	12 7 mm

Bottom External Angle Vertical Leg

Vertical Leg Thickness t_{BEAVL} =	0 5 in
Vertical Leg Thickness t_{BEAVL} =	12 7 mm
Vertical Leg Depth d_{BEAVL} =	3 in
Vertical Leg Depth d_{BEAVL} =	76 2 mm

Top Internal Angle Horizontal Leg

Horizontal Leg Width w_{TIAHL} =	3 in
Horizontal Leg Width w_{TIAHL} =	76 2 mm
Horizontal Leg Thickness t_{TIAHL} =	0 5 in
Horizontal Leg Thickness t_{TIAHL} =	12 7 mm

Top Internal Angle Vertical Leg

Vertical Leg Thicknesst$_{TIAVL}$ =	0 5 in
Vertical Leg Thicknesst$_{TIAVL}$ =	12 7 mm
Vertical Leg Depth d_{TIAVL} =	3 in
Vertical Leg Depth d_{TIAVL} =	76 2 mm

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Bottom Internal Angle Horizontal Leg

Horizontal Leg Width w_{BIAHL} = 3 in

Horizontal Leg Width w_{BIAHL} = 76 2 mm

Horizontal Leg Thickness t_{BIAHL} = 0 5 in

Horizontal Leg Thickness t_{BIAHL} = 12 7 mm

Bottom Internal Angle Vertical Leg

Vertical Leg Thickness t_{BIAVL} = 0.5 in

Vertical Leg Thickness t_{BIAVL} = 12 7 mm

Vertical Leg Depth d_{BIAVL} = 3 in

Vertical Leg Depth d_{BIAVL} = 76 2 mm

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Rivet Diameter dia_R = 0 75 in

Rivet Diameter dia_R = 19 05 mm

Element	b (mm)	d (mm)	Area A (mm^2)
Top Flange	228 6	12 7	2903 22
Bottom Flange	355 6	12 7	4516 12
Web	9 525	1955 8	18628 995
Top External Angle Horizontal Leg	76 2	12 7	967 74
Top External Angle Joint	12 7	12 7	161 29
Top External Angle Vertical Leg	12 7	76 2	967 74
Bottom External Angle Horizontal Leg	76 2	12 7	967 74
Bottom External Angle Joint	12 7	12 7	161 29
Bottom External Angle Vertical Leg	12 7	76 2	967 74
Top Internal Angle Horizontal Leg	76 2	12 7	967 74
Top Internal Angle Joint	12 7	12 7	161 29
Top Internal Angle Vertical Leg	12 7	76 2	967 74
Bottom Internal Angle Horizontal Leg	76 2	12 7	967 74
Bottom Internal Angle Joint	12 7	12 7	161 29
Bottom Internal Angle Vertical Leg	12 7	76 2	967 74
Bottom External Angle Horizontal Leg Rivet	19 05	25 4	-483.87
Bottom Internal Angle Horizontal Leg Rivet	19.05	25 4	-483 87
Bottom Angles Vertical Leg Rivet	34 925	19 05	-665.321

External Girder Area with Rivets $A_{EGR} = \Sigma A$ = 34435 415 mm^2 ✓

External Girder Area $A_{EG} = \Sigma A$ = 32802.354 mm^2 ✓

Element	A (mm^2)	y (mm)	Ay (mm^3)
Top Flange	2903.22	1974 85	5733424 017
Bottom Flange	4516 12	6 35	28677 362
Web	18628 995	990 6	18453882 447
Top External Angle Horizontal Leg	967 74	1962 15	1898851 041
Top External Angle Joint	161 29	1962 15	316475 174
Top External Angle Vertical Leg	967 74	1917 7	1855834 998
Bottom External Angle Horizontal Leg	967.74	19 05	18435 447
Bottom External Angle Joint	161 29	19 05	3072 575
Bottom External Angle Vertical Leg	967 74	63 5	61451 49
Top Internal Angle Horizontal Leg	967 74	1962 15	1898851 041
Top Internal Angle Joint	161 29	1962 15	316475 174
Top Internal Angle Vertical Leg	967 74	1917.7	1855834 998
Bottom Internal Angle Horizontal Leg	967 74	19 05	18435 447
Bottom Internal Angle Joint	161 29	19 05	3072 575
Bottom Internal Angle Vertical Leg	967 74	63 5	61451 49
Bottom External Angle Horizontal Leg Rivet	-483 87	12 7	-6145 149
Bottom Internal Angle Horizontal Leg Rivet	-483 87	12 7	-6145 149
Bottom Angles Vertical Leg Rivet	-665.321	63.5	-42247.899

$y_{bar} = \Sigma Ay / A_{EG}$ = 989 858 mm ✓

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Element	bd³ / 12 (mm⁴)	A(y-y_bar)² (mm⁴)	I_{xx} (mm⁴)
Top Flange	39022	2816728290	2816767311 449
Bottom Flange	60700	4368392627	4368453327 617
Web	5938230669	10244	5938240913 469
Top External Angle Horizontal Leg	13007	914853837	914866844 608
Top External Angle Joint	2168	152475640	152477807 435
Top External Angle Vertical Leg	468260	833117630	833585890 692
Bottom External Angle Horizontal Leg	13007	912064997	912078004.517
Bottom External Angle Joint	2168	152010833	152013000 753
Bottom External Angle Vertical Leg	468260	830456384	830924644 592
Top Internal Angle Horizontal Leg	13007	914853837	914866844 608
Top Internal Angle Joint	2168	152475640	152477807 435
Top Internal Angle Vertical Leg	468260	833117630	833585890 692
Bottom Internal Angle Horizontal Leg	13007	912064997	912078004 517
Bottom Internal Angle Joint	2168	152010833	152013000 753
Bottom Internal Angle Vertical Leg	468260	830456384	830924644 592
Bottom External Angle Horizontal Leg Rivet	-26014	-462017772	-462043786 537
Bottom Internal Angle Horizontal Leg Rivet	-26014	-462017772	-462043786.537
Bottom Angles Vertical Leg Rivet	-20121	-570938764	-570958884.726

$I_{xx} = \Sigma I_{xx} =$ 19220307480 mm^4 ✓

Girder is symmetrical, therefore the neutral axis is located at half of web thickness

Element	A (mm²)	x (mm)
Top Flange	2903 22	0
Bottom Flange	4516 12	0
Web	18628 995	0
Top External Angle Horizontal Leg	967 74	55 563
Top External Angle Joint	161 29	11 113
Top External Angle Vertical Leg	967 74	11 113
Bottom External Angle Horizontal Leg	967 74	55 563
Bottom External Angle Joint	161 29	11 113
Bottom External Angle Vertical Leg	967 74	11 113
Top Internal Angle Horizontal Leg	967 74	55 563
Top Internal Angle Joint	161 29	11 113
Top Internal Angle Vertical Leg	967 74	11 113
Bottom Internal Angle Horizontal Leg	967 74	55 563
Bottom Internal Angle Joint	161 29	11 113
Bottom Internal Angle Vertical Leg	967.74	11.113

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Element	$db^3/12$ (mm^4)	Ax^2 (mm^4)	I_{yy} (mm^4)
Top Flange	12643030	0	12643029 553
Bottom Flange	47589126	0	47589126 327
Web	140844	0	140843 935
Top External Angle Horizontal Leg	468260	2987599	3455858.965
Top External Angle Joint	2168	19917	22085 196
Top External Angle Vertical Leg	13007	119504	132511 177
Bottom External Angle Horizontal Leg	468260	2987599	3455858 965
Bottom External Angle Joint	2168	19917	22085 196
Bottom External Angle Vertical Leg	13007	119504	132511 177
Top Internal Angle Horizontal Leg	468260	2987599	3455858 965
Top Internal Angle Joint	2168	19917	22085 196
Top Internal Angle Vertical Leg	13007	119504	132511 177
Bottom Internal Angle Horizontal Leg	468260	2987599	3455858 965
Bottom Internal Angle Joint	2168	19917	22085 196
Bottom Internal Angle Vertical Leg	13007	119504	132511.177

$I_{yy} = \Sigma I_{yy} =$ 74814821 mm^4 ✓

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Dead Loads

	External Girder Area A_{EGR} =	34435 415 mm^2	
	External Girder Area A_{EGR} =	0 034 m^2	
BE4	Steel Density Y_{steel} =	490 lb/ft^3	
	Steel Density Y_{steel} =	76 996 kN/m^3	
	External Girder Self Weight $W_{EG} = A_{EG} \times Y_{steel}$ =	2 651 kN/m	
	Add 10% for stiffners, splice plates and bolts		
	$W_{EG} = W_{EG} \times 1.1$ =	**2.917 kN/m** ✓	
	Internal Transverse Girder Spacing S_I =	4 ft	
	Internal Transverse Girder Spacing S_I =	1 219 m	
	Verge Depth d_V =	174 mm	(Trial pit logs and site notes)
	Verge Depth d_V =	0.174 m	
	Verge Area $A_V = S_I \times d_V \times$ =	0 212 m^2	
BE4	Fill Density Y_{fill} =	135 lb/ft^3	
	Fill Density Y_{fill} =	21 213 kN/m^3	
	Verge Self Weight $W_V = A_V \times Y_{fill}$ =	**4.5 kN/m** ✓	
	Carriageway Depth d_C =	50 mm	(Site notes)
	Carriageway Depth d_C =	0 05 m	
	Carriageway Area $A_C = S_I \times d_C$ =	0 061 m^2	
BE4	Macadam Density $Y_{macadam}$ =	144 lb/ft^3	
	Macadam Density $Y_{macadam}$ =	22 627 kN/m^3	
	Carriageway Self Weight $W_C = A_C \times Y_{macadam}$ =	**1.379 kN/m** ✓	
	Fill Depth d_F =	346 mm	(Site notes)
	Fill Depth d_F =	0 346 m	
	Fill Area $A_F = S_I \times d_F$ =	0 422 m^2	
BE4	Fill Density Y_{fill} =	135 lb/ft^3	
	Fill Density Y_{fill} =	21 213 kN/m^3	
	Fill Self Weight $W_F = A_F \times Y_{fill}$ =	**8.949 kN/m** ✓	

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T-Section Flange Width $w_{Tflange}$ = 6. in

T-Section Flange Width $w_{Tflange}$ = 0 152 m

T-Secton Flange Thickness $t_{Tflange}$ = 0 375 in

T-Secton Flange Thickness $t_{Tflange}$ = 0 01 m

T-Section Web Thickness t_{Tweb} = 0 375 in

T-Section Web Thickness t_{Tweb} = 0 01 m

T-Section Depth d_T = 3 in

T-Section Depth d_T = 0 076 m

T-Section Web Depth $d_{Tweb} = d_T - t_{Tflange}$ = 0 067 m

T-Section Area $A_T = (w_{Tflange} \times t_{Tflange}) + (t_{Tweb} \times t_{Tweb})$ = 0 002 m^2

T-Section Volume $V_T = S_l \times A_T$ = 0 003 m^3

BE4 Steel Density Y_{steel} = 490 lb/ft^3

Steel Density Y_{steel} = 76 996 kN/m^3

T-Section Self Weight $W_T = V_T \times Y_{steel}$ = 0 196 kN

Add 5% for bolts.

T-Section Self Weight $W_T = W_T \times 1.05$ = 0.206 kN ✓

Buckle Plate Depth d_{BP} = 0 375 in

Buckle Plate Depth d_{BP} = 0 01 m

Buckle Plate Area $A_{BP} = S_l \times d_{BP}$ = 0 012 m^2

BE4 Steel Density Y_{steel} = 490 lb/ft^3

Steel Density Y_{steel} = 76 996 kN/m^3

Buckle Plate Self Weight $W_{BP} = A_{BP} \times Y_{steel}$ = 0.894 kN/m ✓

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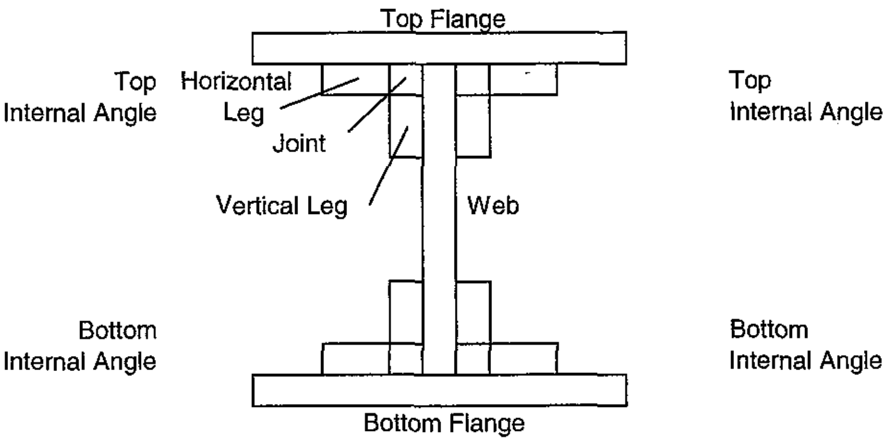
Section Properties of Internal Girder



Top Flange

Internal Top Flange Width w_{top} = 9 in
Internal Top Flange Width w_{top} = 228 6 mm

Internal Top Flange Thickness t_{top} = 0 5 in
Internal Top Flange Thickness t_{top} = 12 7 mm

Bottom Flange

Internal Bottom Flange Width w_{bot} = 9 in
Internal Bottom Flange Width w_{bot} = 228 6 mm

Internal Bottom Flange Thickness t_{bot} = 0 5 in
Internal Bottom Flange Thickness t_{bot} = 12 7 mm

Web

Internal Web Thickness t_{web} = 0 375 in
Internal Web Thickness t_{web} = 9 525 mm

Internal Girder Depth d_I = 16 in
Internal Girder Depth d_I = 406 4 mm

Internal Web Depth $d_{web} = d - t_{top} - t_{bot}$ = 381 mm

Top Angle Horizontal Leg

Horizontal Leg Width w_{TAHL} = 3 5 in
Horizontal Leg Width w_{TAHL} = 88 9 mm

Horizontal Leg Thickness t_{TAHL} = 0 5 in
Horizontal Leg Thickness t_{TAHL} = 12 7 mm

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Top Angle Vertical Leg

Vertical Leg Thickness t_{TAVL} = 0 5 in
Vertical Leg Thickness t_{TAVL} = 12 7 mm

Vertical Leg Depth d_{TAVL} = 3 5 in
Vertical Leg Depth d_{TAVL} = 88 9 mm

Bottom Angle Horizontal Leg

Horizontal Leg Width w_{BAHL} = 3 5 in
Horizontal Leg Width w_{BAHL} = 88 9 mm

Horizontal Leg Thickness t_{BAHL} = 0 5 in
Horizontal Leg Thickness t_{BAHL} = 12 7 mm

Bottom Angle Vertical Leg

Vertical Leg Thickness t_{BAVL} = 0 5 in
Vertical Leg Thickness t_{BAVL} = 12 7 mm

Vertical Leg Depth d_{BAVL} = 3 5 in
Vertical Leg Depth d_{BAVL} = 88 9 mm

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Rivet Diameter dia_R = 0 75 in

Rivet Diameter dia_R = 19 05 mm

Breadth b of angles is multiplied by 2 due to them existing on both sides of the web

Element	b (mm)	d (mm)	Area A (mm²)
Top Flange	228 6	12 7	2903 22
Bottom Flange	228 6	12 7	2903 22
Web	9.525	381	3629 025
Top Angle Horizontal Leg	177 8	12 7	2258 06
Top Angle Joint	25 4	12 7	322 58
Top Angle Vertical Leg	25.4	88 9	2258 06
Bottom Angle Horizontal Leg	177 8	12 7	2258 06
Bottom Angle Joint	25 4	12 7	322 58
Bottom Angle Vertical Leg	25 4	88 9	2258 06
Bottom Internal Angle Horizontal Leg Rivet	38 1	25 4	-967 74
Bottom Angles Vertical Leg Rivet	34.925	19.05	-665.32125

Internal Girder Area with Rivets $A_{IGR} = \Sigma A$ = 19112 865 mm² ✓

Internal Girder Area without Rivets $A_{IG} = \Sigma A$ = 17479 804 mm² ✓

Element	A (mm²)	y (mm)	Ay (mm³)
Top Flange	2903 22	400 05	1161433 161
Bottom Flange	2903 22	6 35	18435 447
Web	3629 025	203.2	737417 88
Top Angle Horizontal Leg	2258 06	387 35	874659.541
Top Angle Joint	322 58	387 35	124951 363
Top Angle Vertical Leg	2258 06	336 55	759950 093
Bottom Angle Horizontal Leg	2258 06	19 05	43016 043
Bottom Angle Joint	322 58	19 05	6145 149
Bottom Angle Vertical Leg	2258 06	69 85	157725 491
Bottom Internal Angle Horizontal Leg Rivet	-967 74	12 7	-12290 298
Bottom Angles Vertical Leg Rivet	-665.321	69 85	-46472.68931

$y_{bar} = \Sigma Ay / A_{IG}$ = 218 822 mm ✓

Element	bd³ / 12 (mm⁴)	A(y-y_bar)² (mm⁴)	Ixx (mm⁴)
Top Flange	39022	95351827	95390848 22
Bottom Flange	39022	131064375	131103396 52
Web	43899408	885688	44785096 51
Top Angle Horizontal Leg	30350	64132471	64162820 78
Top Angle Joint	4336	9161782	9166117 25
Top Angle Vertical Leg	1487160	31296276	32783436 06
Bottom Angle Horizontal Leg	30350	90116870	90147219 72
Bottom Angle Joint	4336	12873839	12878174 25
Bottom Angle Vertical Leg	1487160	50112565	51599724 95
Bottom Internal Angle Horizontal Leg Rivet	-52029	-41115799	-41167827 46
Bottom Angles Vertical Leg Rivet	-20121	-14765309 ✓	-14785429.82

$I_{xx} = \Sigma I_{xx}$ = 476063577 mm⁴ ✓

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Girder is symmetrical, therefore the neutral axis is located at half of web thickness

Element	A (mm^2)	x (mm)
Top Flange	2903 22	0
Bottom Flange	2903 22	0
Web	3629 025	0
Top Angle Horizontal Leg	2258 06	61.913
Top Angle Joint	322 58	11 113
Top Angle Vertical Leg	2258 06	11 113
Bottom Angle Horizontal Leg	2258 06	61 913
Bottom Angle Joint	322 58	11 113
Bottom Angle Vertical Leg	2258.06	11.113

Element	$db^3 / 12$ (mm^4)	Ax^2 (mm^4)	I_{yy} (mm^4)
Top Flange	12643030	0	12643029 55
Bottom Flange	12643030	0	12643029 55
Web	27437	0	27437 13
Top Angle Horizontal Leg	1487160	8655500	10142660 17
Top Angle Joint	4336	39835	44170 39
Top Angle Vertical Leg	30350	278843	309192 75
Bottom Angle Horizontal Leg	1487160	8655500	10142660 17
Bottom Angle Joint	4336	39835	44170 39
Bottom Angle Vertical Leg	30350	278843	309192.75

$I_{yy} = \Sigma I_{yy} =$ 46305543 mm^4

Internal Girder Area A_{IGR} = 19112 865 mm^2

Internal Girder Area A_{IGR} = 0 019 m^2

BE4

Steel Density Y_{steel} = 490 lb/ft^3

Steel Density Y_{steel} = 76 996 kN/m^3

Internal Transverse Girder Self Weight $W_{IG} = A_{IG} \times Y_{steel}$ = 1.472 kN/m

Add 5% for stiffners, splice plates and bolts

$W_{IG} = W_{IG} \times 1.05$ = 1.545 kN/m ✓

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Internal Transverse Girders 4 - 7

External Girder Spacing S_E =	26 5 ft	
External Girder Spacing S_E =	8 077 m	
Verge 1 Width w_{V1} =	2100 mm	(Site notes)
Verge 1 Width w_{V1} =	2 1 m	
Verge 2 Width w_{V2} =	1400 mm	(Site notes)
Verge 2 Width w_{V2} =	1 4 m	
Take smallest verge		
Verge Width w_V =	1 4 m	
Verge Dead Load $DL_V = W_V \times w_V$ =	6 3 kN	
Carriageway Width $w_C = (S_E / 2) - w_V$ =	2 639 m	
Carriageway Dead Load $DL_C = W_C \times w_C$ =	3 64 kN	
Fill Width $w_F = S_E / 2$ =	4 039 m	
Fill Dead Load $DL_F = W_F \times w_F$ =	36 14 kN	
Number of T-Section Supported n_T =	5	
T-Section Dead Load $DL_F = (n_T / 2) \times W_T$ =	0 514 kN	
Buckle plate is assumed to be carried entirely on Internal Transerve Girder		
Buckle Plate Width $w_{BP} = (S_E / 2)$ =	4 039 m	
Buckle Plate Dead Load $DL_{BP} = W_{BP} \times w_{BP}$ =	3 611 kN	
Internal Girder Dead Load $DL_{IG} = W_{IG} \times (S_E / 2)$ =	6 24 kN	
Total Dead Load $DL_{4-7} = DL_V + DL_C + DL_F + DL_T + DL_{BP} + DL_{IG}$ =	**56.446 kN**	✓

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Internal Transverse Girder 8

Skew Span L_{skew} =	8820 mm
Skew Span L_{skew} =	8 82 m
Distance Internal Transeverse Girder from Abutment $d_{abut} = (L_{skew} - (6 \times S_I)) / 2$	0 752 m
Skew Angle α_{skew} =	26 582 deg
Girder Length $L_8 = S_E - ((S_I - d_{abut}) / \tan \alpha_{skew})$ =	7 144 m
Internal Girder Depth d_I =	406 4 mm
Internal Girder Depth d_I =	0 406 m

Add bearing length on abutment

$L_8 = L_8 + ((1 / 3) \times (1 / 4) \times d_I)$	7 178 m
Verge Width w_V =	1 4 m
Verge Dead Load $DL_V = W_V \times w_V$ =	6 3 kN
Carriageway Width $w_C = (L_8 / 2) - w_V$ =	2 189 m
Carriageway Dead Load $DL_C = W_C \times w_C$ =	3 02 kN
Fill Width $w_F = (L_8 / 2)$ =	3 589 m
Fill Dead Load $DL_F = W_F \times w_F$ =	32 117 kN
Number of T-Section Supported n_T =	4
T-Section Dead Load $DL_F = (n_T / 2) \times W_T$ =	0 411 kN
Buckle Plate Width $w_{BP} = (L_8 / 2)$ =	3 589 m
Buckle Plate Dead Load $DL_{BP} = W_{BP} \times w_{BP}$ =	3 209 kN
Internal Girder Dead Load $DL_{IG} = W_{IG} \times (L_8 / 2)$ =	5 546 kN
Total Dead Load $DL_8 = DL_V + DL_C + DL_F + DL_T + DL_{BP} + DL_{IG}$ =	**50.604 kN** ✓

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Internal Transverse Girder 9

Girder Length $L_9 = S_E - (((2 \times S_I) - d_{abut}) / \tan \alpha_{skew}) =$	4 708 m
Add bearing length on abutment	
$L_9 = L_9 + ((1/3) \times (1/4) \times d_I)$	4 742 m
Verge Width $w_V =$	1 4 m
Verge Dead Load $DL_V = W_V \times w_V =$	6 3 kN
Carriageway Width $w_C = (L_9 / 2) - w_V =$	0 971 m
Carriageway Dead Load $DL_C = W_C \times w_C =$	1 339 kN
Fill Width $w_F = (L_9 / 2) =$	2 371 m
Fill Dead Load $DL_F = W_F \times w_F =$	21 215 kN
Number of T-Section Supported $n_T =$	2
T-Section Dead Load $DL_F = (n_T / 2) \times W_T =$	0 206 kN
Buckle Plate Width $w_{BP} = (L_9 / 2) =$	2 371 m
Buckle Plate Dead Load $DL_{BP} = W_{BP} \times w_{BP} =$	2 12 kN
Internal Girder Dead Load $DL_{IG} = W_{IG} \times (L_9 / 2) =$	3 663 kN
Total Dead Load $DL_9 = DL_V + DL_C + DL_F + DL_T + DL_{BP} + DL_G =$	**34.843 kN** ✓

CALCULATION SHEET

Project Title. **VAR9-830 BE4 Assessments**			
Subject. **AGB 3 Luffness Mains Bridge BE4 Assessment**			Calc No
Job No. **J20308B-1142**			File.
Made by. [redacted]	Date. **19/07/2004**	Checked by.	Date
Revised b[redacted]	Date. Aug 04	Checked by.	Date.

Internal Transverse Girder 10

Girder Length $L_{10} = S_E - (((3 \times S_I) - d_{abut}) / \tan \alpha_{skew}) =$	2 271 m
Add bearing length on abutment	
$L_{10} = L_{10} + ((1/3) \times (1/4) \times d_I)$	2 305 m
Verge Width $w_V = L_{10} / 2 =$	1 152 m
Verge Dead Load $DL_V = W_V \times w_V =$	5 186 kN
Fill Width $w_F = (L_{10} / 2) =$	1 152 m
Fill Dead Load $DL_F = W_F \times w_F =$	10 313 kN
Number of T-Section Supported $n_T =$	1
T-Section Dead Load $DL_F = (n_T / 2) \times W_T =$	0 103 kN
Buckle Plate Width $w_{BP} = (L_{10} / 2) =$	1 152 m
Buckle Plate Dead Load $DL_{BP} = W_{BP} \times w_{BP} =$	1 03 kN
Internal Girder Dead Load $DL_{IG} = W_{IG} \times (L_{10} / 2) =$	1 781 kN
Total Dead Load $DL_{10} = DL_V + DL_F + DL_T + DL_{BP} + DL_{IG} =$	**18.414 kN** ✓

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Dead Load Moment

Internal Transverse Girder Positions

Internal Transverse Girder Spacing S_I = 1 219 m

Internal Transverse Girder 4 Position $d_4 = d_{abut}$ = 0 752 m

Internal Transverse Girder 5 Position $d_5 = d_4 + S_I$ = 1 972 m

Internal Transverse Girder 6 Position $d_6 = d_5 + S_I$ = 3 191 m

Internal Transverse Girder 7 Position $d_7 = d_6 + S_I$ = 4 41 m

Internal Transverse Girder 8 Position $d_8 = d_7 + S_I$ = 5 629 m

Internal Transverse Girder 9 Position $d_9 = d_8 + S_I$ = 6 848 m

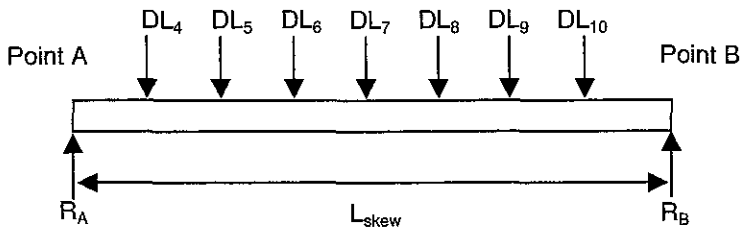
Internal Transverse Girder 10 Position $d_{10} = d_9 + S_I$ = 8 068 m

Distance over Bearings d_B = 34 083 ft

Distance over Bearings d_B = 10 389 m

Skew Span L_{skew} = 8 82 m

External Bearing Length $L_{EB} = (1 / 3) \times (1 / 4) \times ((d_B - L_{skew}) / 2)$ = 0 065 m



CALCULATION SHEET

Project Title. **VAR9-830 BE4 Assessments**			
Subject. **AGB 3 Luffness Mains Bridge BE4 Assessment**			Calc No
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BE4 Part 1-303(a)(iv)	Effective Length EL = $L_{skew} + (2 \times L_{EB})$ =	8 951 m
	Self Weight W_{EG} =	2.917 kN/m
	Point Load 1 DL_4 =	56 446 kN
	Point Load 1 Position $d_4 = d_4 + L_{EB}$ =	0 818 m
	Point Load 2 DL_5 =	56 446 kN
	Point Load 2 Position $d_5 = d_5 + L_{EB}$ =	2 037 m
	Point Load 3 DL_6 =	56 446 kN
	Point Load 3 Position $d_6 = d_6 + L_{EB}$ =	3 256 m
	Point Load 4 DL_7 =	56 446 kN
	Point Load 4 Position $d_7 = d_7 + L_{EB}$ =	4 475 m
	Point Load 5 DL_8 =	50 604 kN
	Point Load 5 Position $d_8 = d_8 + L_{EB}$ =	5 695 m
	Point Load 6 DL_9 =	34 843 kN
	Point Load 6 Position $d_9 = d_9 + L_{EB}$ =	6 914 m
	Point Load 7 DL_{10} =	18 414 kN
	Point Load 7 Position $d_{10} = d_{10} + L_{EB}$ =	8 133 m

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Project Title: **VAR9-830 BE4 Assessments**			
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Revised b[redacted]	Date Aug 04	Checked by.	Date

Taking moments about Point A

BM at A due to SW $BM_{ASW} = W_{EG} \times (EL^2 / 2) =$	116 829 kNm
BM at A due to DL_4 $BM_{APL1} = DL_4 \times d_4 =$	46 158 kNm
BM at A due to DL_5 $BM_{APL2} = DL_5 \times d_5 =$	114 977 kNm
BM at A due to DL_6 $BM_{APL3} = DL_6 \times d_6 =$	183 796 kNm
BM at A due to DL_7 $BM_{APL4} = DL_7 \times d_7 =$	252 615 kNm
BM at A due to DL_8 $BM_{APL5} = DL_8 \times d_8 =$	288 165 kNm
BM at A due to DL_9 $BM_{APL6} = DL_9 \times d_9 =$	240 899 kNm
BM at A due to DL_{10} $BM_{APL7} = DL_{10} \times d_{10} =$	149 759 kNm
BM at A due to loads $BM_{ALoad} = BM_{ASW} + BM_{APL1} + BM_{APL2} + BM_{APL3} + BM_{APL4} + BM_{APL5} + BM_{APL6} + BM_{APL7} =$	1393 198 kNm
BM at A $BM_A =$	0 kNm
$R_B = BM_{ALoad} / EL =$	155 652 kN
Load at Point A due to SW $Load_{SW} = W_{EG} \times EL =$	26 105 kN
Loads at Point A $Load_A = Load_{SW} + DL_4 + DL_5 + DL_6 + DL_7 + DL_8 + DL_9 + DL_{10} =$	355.748 kN
$R_A = Load_A - R_B =$	200 096 kN

Bending Moment at Point 1

BM at 1 due to SW $BM_{1SW} = W_{EG} \times (d_4^2 / 2) =$	0 975 kNm
BM at 1 due to R_A $BM_{1RA} = R_A \times d_4 =$	163 628 kNm
BM at 1 $BM_1 = BM_{1SW} - BM_{1RA} =$	-162 653 kNm

Not checked we know that BM max is at Point 4

CALCULATION SHEET

Project Title. **VAR9-830 BE4 Assessments**			
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Bending Moment at Point 2

BM at 2 due to SW $BM_{2SW} = W_{EG} \times (d_5^2 / 2) =$ 6 051 kNm

BM at 2 due to DL_4 $BM_{2PL1} = DL_4 \times (d_5 - d_4) =$ 68 819 kNm

BM at 2 due to R_A $BM_{2RA} = R_A \times d_5 =$ 407 587 kNm

BM at 2 $BM_2 = BM_{2SW} + BM_{2PL1} - BM_{2RA} =$ -332 717 kNm

Bending Moment at Point 3

BM at 3 due to SW $BM_{3SW} = W_{EG} \times (d_6^2 / 2) =$ 15 461 kNm

BM at 3 due to DL_4 $BM_{3PL1} = DL_4 \times (d_6 - d_4) =$ 137 638 kNm

BM at 3 due to DL_5 $BM_{3PL2} = DL_5 \times (d_6 - d_5) =$ 68 819 kNm

BM at 3 due to R_A $BM_{3RA} = R_A \times d_6 =$ 651 545 kNm

BM at 3 $BM_3 = BM_{3SW} + BM_{3PL1} + BM_{3PL2} - BM_{3RA}$
$=$ -429 627 kNm

Not checked

Bending Moment at Point 4

BM at 4 due to SW $BM_{4SW} = W_{EG} \times (d_7^2 / 2) =$ 29 207 kNm ✓

BM at 4 due to DL_4 $BM_{4PL1} = DL_4 \times (d_7 - d_4) =$ 206 456 kNm ✓

BM at 4 due to DL_5 $BM_{4PL2} = DL_5 \times (d_7 - d_5) =$ 137 638 kNm ✓

BM at 4 due to DL_6 $BM_{4PL3} = DL_6 \times (d_7 - d_6) =$ 68 819 kNm

BM at 4 due to R_A $BM_{4RA} = R_A \times d_7 =$ 895 503 kNm ✓

BM at 4 $BM_4 = BM_{4SW} + BM_{4PL1} + BM_{4PL2} +$
$BM_{4PL3} - BM_{4RA} =$ -453 383 kNm ✓

CALCULATION SHEET

Project Title. **VAR9-830 BE4 Assessments**			
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Made by. [redacted]	Date. **19/07/2004**	Checked by.	Date
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Bending Moment at Point 5

BM at 5 due to SW $BM_{5SW} = W_{EG} \times (d_8^2 / 2) =$ 47 289 kNm

BM at 5 due to DL_4 $BM_{5PL1} = DL_4 \times (d_8 - d_4) =$ 275 275 kNm

BM at 5 due to DL_5 $BM_{5PL2} = DL_5 \times (d_8 - d_5) =$ 206 456 kNm

BM at 5 due to DL_6 $BM_{5PL3} = DL_6 \times (d_8 - d_6) =$ 137 638 kNm

BM at 5 due to DL_7 $BM_{5PL4} = DL_7 \times (d_8 - d_7) =$ 68 819 kNm

BM at 5 due to R_A $BM_{5RA} = R_A \times d_8 =$ 1139 461 kNm

BM at 5 $BM_5 = BM_{5SW} + BM_{5PL1} + BM_{5PL2} + BM_{5PL3} + BM_{5PL4} - BM_{5RA} =$ -403 985 kNm

Bending Moment at Point 6

BM at 6 due to SW $BM_{6SW} = W_{EG} \times (d_9^2 / 2) =$ 69 705 kNm

BM at 6 due to DL_4 $BM_{6PL1} = DL_4 \times (d_9 - d_4) =$ 344 094 kNm

BM at 6 due to DL_5 $BM_{6PL2} = DL_5 \times (d_9 - d_5) =$ 275 275 kNm

BM at 6 due to DL_6 $BM_{6PL3} = DL_6 \times (d_9 - d_6) =$ 206 456 kNm

BM at 6 due to DL_7 $BM_{6PL4} = DL_7 \times (d_9 - d_7) =$ 137 638 kNm

BM at 6 due to DL_8 $BM_{6PL5} = DL_8 \times (d_9 - d_8) =$ 61 696 kNm

BM at 6 due to R_A $BM_{6RA} = R_A \times d_9 =$ 1383 419 kNm

BM at 6 $BM_6 = BM_{6SW} + BM_{6PL1} + BM_{6PL2} + BM_{6PL3} + BM_{6PL4} + BM_{6PL5} - BM_{6RA} =$ -288 555 kNm

Not checked as Point 4 is the critical for max BM

CALUCLATION SHEET

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Bending Moment at Point 7

BM at 7 due to SW $BM_{7SW} = W_{EG} \times (d_{10}^2 / 2) =$ 96 457 kNm

BM at 7 due to DL_4 $BM_{7PL1} = DL_4 \times (d_{10} - d_4) =$ 412 913 kNm

BM at 7 due to DL_5 $BM_{7PL2} = DL_5 \times (d_{10} - d_5) =$ 344 094 kNm

BM at 7 due to DL_6 $BM_{7PL3} = DL_6 \times (d_{10} - d_6) =$ 275 275 kNm

BM at 7 due to DL_7 $BM_{7PL4} = DL_7 \times (d_{10} - d_7) =$ 206 456 kNm

BM at 7 due to DL_8 $BM_{7PL5} = DL_8 \times (d_{10} - d_8) =$ 123 392 kNm

BM at 7 due to DL_9 $BM_{7PL6} = DL_9 \times (d_{10} - d_9) =$ 42 481 kNm

BM at 7 due to R_A $BM_{7RA} = R_A \times d_{10} =$ 1627 377 kNm

BM at 7 $BM_7 = BM_{7SW} + BM_{7PL1} + BM_{7PL2} + BM_{7PL3} + BM_{7PL4} + BM_{7PL5} + BM_{7PL6} - BM_{7RA} =$ -126 309 kNm

Taking moments about Point B (Check)

BM at B due to SW $BM_{BSW} = W_{EG} \times (EL^2 / 2) =$ 116 829 kNm

BM at B due to DL_4 $BM_{BPL1} = DL_4 \times (EL - d_4) =$ 459 071 kNm

BM at B due to DL_5 $BM_{BPL2} = DL_5 \times (EL - d_5) =$ 390 252 kNm

BM at B due to DL_6 $BM_{BPL3} = DL_6 \times (EL - d_6) =$ 321 434 kNm

BM at B due to DL_7 $BM_{BPL4} = DL_7 \times (EL - d_7) =$ 252 615 kNm

BM at B due to DL_8 $BM_{BPL5} = DL_8 \times (EL - d_8) =$ 164 773 kNm

BM at B due to DL_9 $BM_{BPL6} = DL_9 \times (EL - d_9) =$ 70 974 kNm

BM at B due to DL_{10} $BM_{BPL7} = DL_{10} \times (EL - d_{10}) =$ 15 058 kNm

BM at B due to R_A $BM_{BRA} = R_A \times EL =$ 1791 006 kNm

BM at B $BM_B = BM_{BSW} + BM_{BPL1} + BM_{BPL2} + BM_{BPL3} + BM_{BPL4} + BM_{BPL5} + BM_{BPL6} + BM_{BPL7} - BM_{BRA} =$ 0 kNm

Not checked

CALCULATION SHEET

Project Title. **VAR9-830 BE4 Assessments**			
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Made by. [redacted]	Date: **19/07/2004**	Checked by.	Date.
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Bending Moment at Midspan

BM at Midspan due to SW $BM_{MSW} = W_{EG} \times ((EL / 2)^2 / 2) =$ 29.207 kNm

BM at Midspan due to DL_4 $BM_{MPL1} = DL_4 \times ((EL / 2) - d_4) =$ 206 456 kNm

BM at Midspan due to DL_5 $BM_{MPL2} = DL_5 \times ((EL / 2) - d_5) =$ 137 638 kNm

BM at Midspan due to DL_6 $BM_{MPL3} = DL_6 \times ((EL / 2) - d_6) =$ 68 819 kNm

BM at Midspan due to DL_7 $BM_{MPL4} = DL_7 \times ((EL / 2) - d_7) =$ 0 kNm

BM at Midspan due to R_A $BM_{MRA} = R_A \times (EL / 2) =$ 895 503 kNm

BM at Midspan $BM_M = BM_{MSW} + BM_{MPL1} + BM_{MPL2} + BM_{MPL3} + BM_{MPL4} - BM_{MRA} =$ -453 383 kNm

Dead load moment is equal to maximum bending moment

Dead Load Moment $M_d = -BM_4 =$ 453.383 kNm ✓

Dead Load Moment $M_d =$ 149.24 tonft ✓

CALCULATION SHEET

JE JACOBS

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Subject AGB 3 Live Load assessment			Calc No
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Made By [redacted]	Date Aug 04	Revised By	Date
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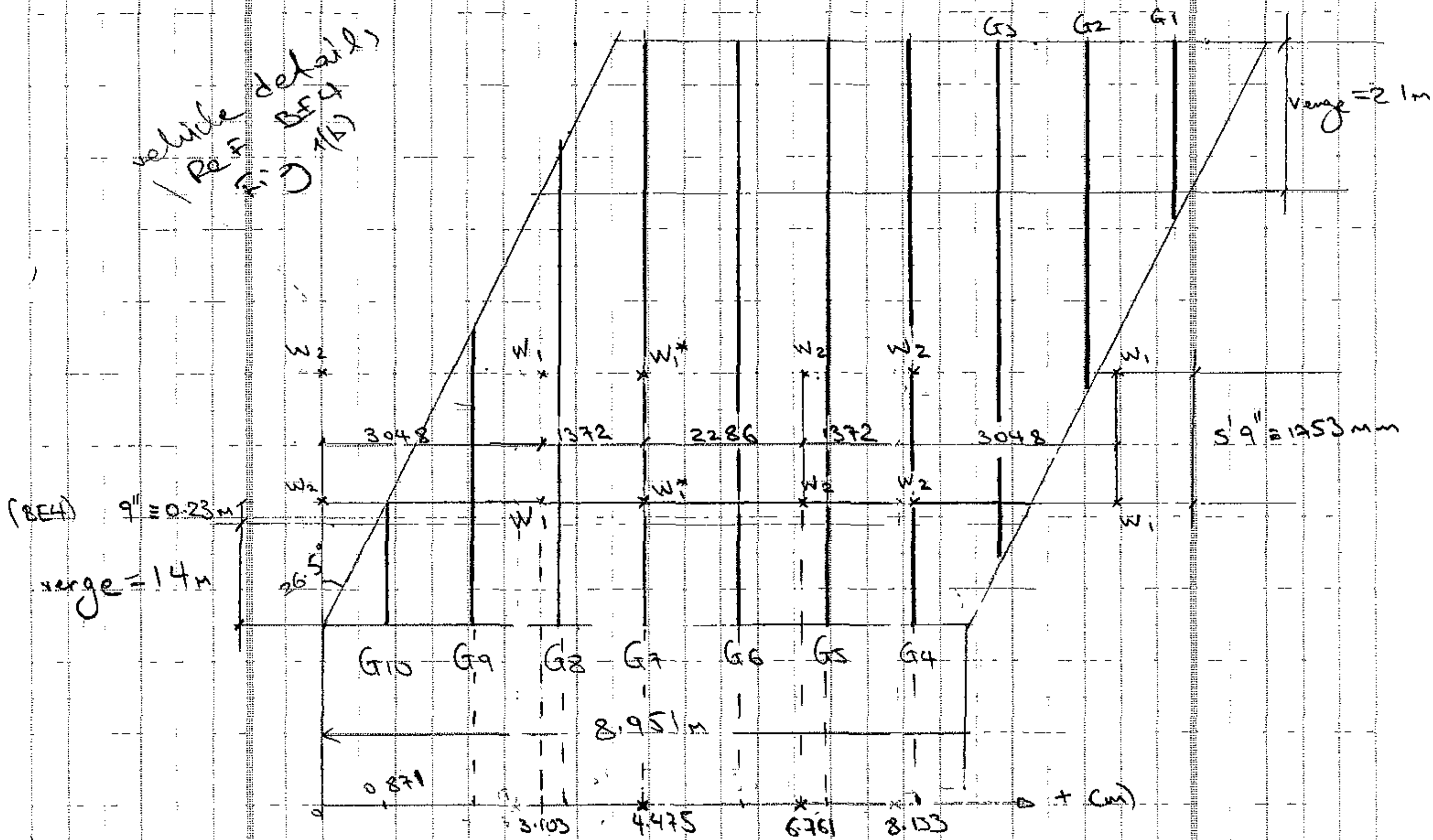
Live load assessment



Width of carriageway = 8.077 − verges
= 8.077 − 1.4 − 2.1 = 4.577m

From BE4-1967, Part I, cl 302 only one lane of vehicle should be applied, cause 4.577 < 18'-0"

For max BM, we should apply the impact load wheel at G7

Ref p.16 for position of G10 – G4 girders

G10 is at 0.817m, G9 at 2.036m, G8 = 3.255m

G7 = 4.475m, G6 = 5.694m, G5 = 6.913m and G4 = 8.132m

CALCULATION SHEET

JACOBS

Project Title BE4 assessment		Sheet No	28
Subject AGB 3 Live Load assessment		Calc No	
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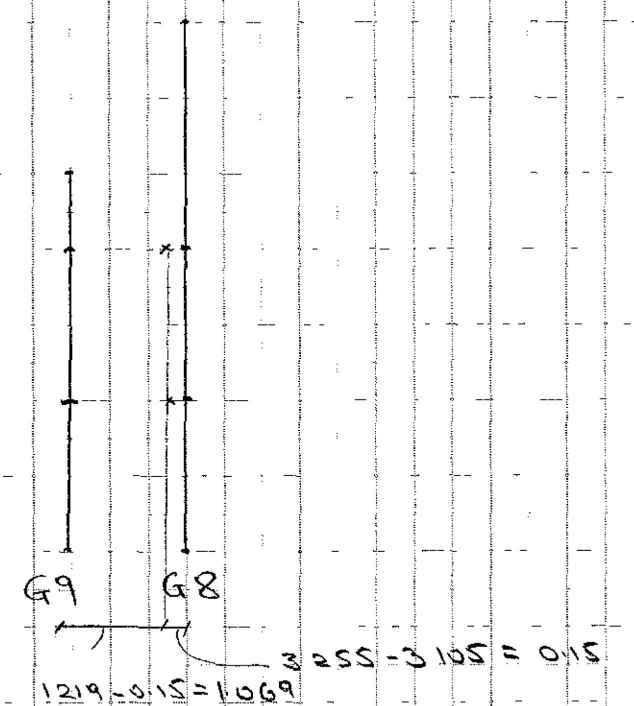
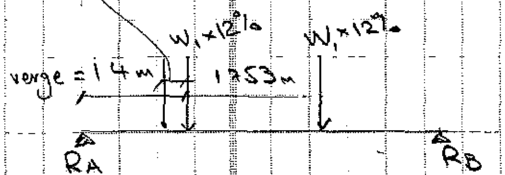
Distribution of LL
on G9 and G8



Distribution on G9 = $\frac{0.15}{1.219} \times 100 = 12.3\%$

Distribution on G8 = 100 − 12.3 = 87.7 %

$W_1 = 0.33 \times 12 \times 9.967 = 39.47$ KN (12 → tons (wheel load); 9.967 → conversion to KN)



L9 = 4.742m (Ref p14)

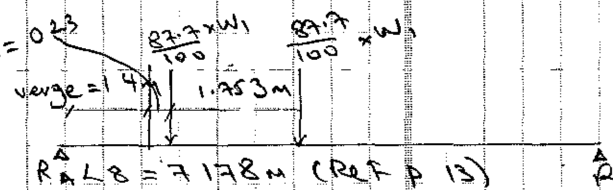
(G9)

$M_A = 0 \Rightarrow (1.63 \times W_1 + 3.383 W_1) \times \frac{12.3}{100} - 4.742 R_B = 0$

$R_B = \frac{5.013 W_1 \times \frac{12.3}{100}}{4.742} = 5.1$ KN

Total load = $2 \times W_1 \times \frac{12.3}{100} = 9.7$ KN

$R_A = 4.6$ KN



L8 = 7.178m (Ref p 13)

$M_A = 0 \Rightarrow (1.4 \times W_1 + 3.153 W_1) \frac{87.7}{100} - 7.178 =$

$R_B = \frac{5.013 W_1 \times 87.7/100}{7.178} = 24.2$ KN

Total load = $2 \times W_1 \times \frac{87.7}{100} = 69.2$ KN

$R_A = 69.2 - 24.2 = 45.0$ KN

CALCULATION SHEET

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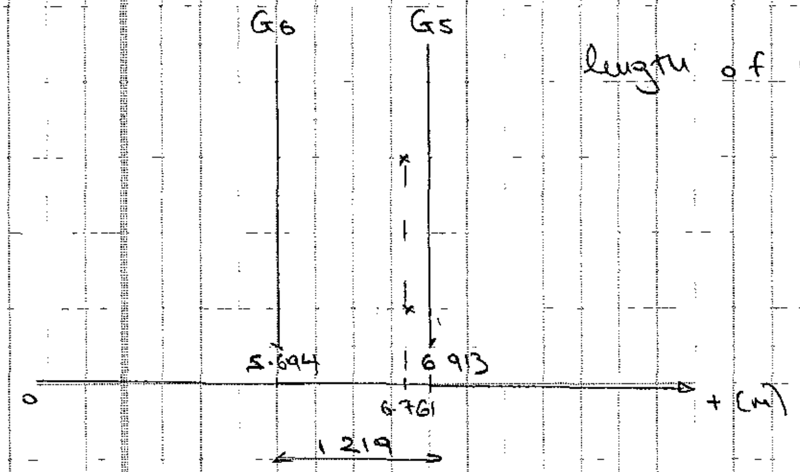
Distribution to G6 and G5

Position of Girders (Ref p 17)

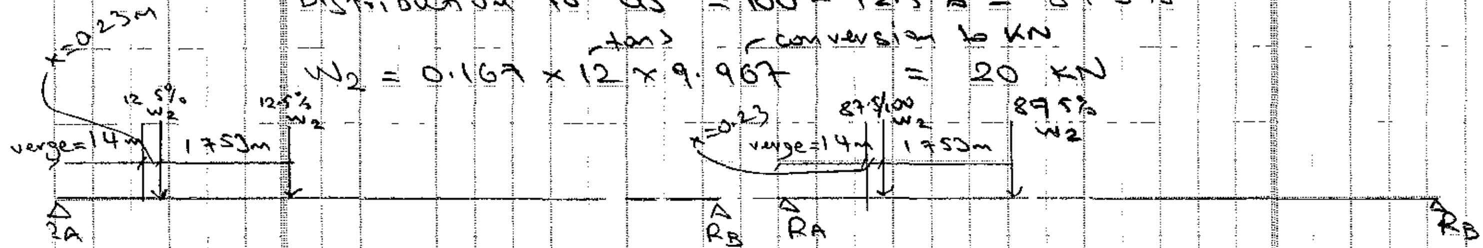
length of G6, G5 (Ref p12) = 8.077m



Distribution to G6 = $\frac{6.913 - 6.761}{1.219} = 0.125 \equiv 12.5\%$

Distribution to G5 = 100 − 12.5% = 87.5%

$W_2 = 0.167 \times 12 \times 9.967 = 20$ kN (tons; conversion to kN)



$M_A = 0 \Rightarrow (1.63 \times W_2 + 3.383 W_2) \times \frac{12.5}{100} - 8.077 R_B = 0$

$R_B = \frac{5.013 W_2 \times \frac{12.5}{100}}{8.077} = 1.6$ kN

Total load = $\frac{12.5}{100} \times 2 \times W_2 = 5$ kN

$R_A = 5 - 1.6 = 3.4$ kN

$M_A = 0 \Rightarrow 4.553 W_2 \times \frac{87.5}{100} - 8.077 R_B = 0$

$R_B = \frac{5.013 \times 20 \times 87.5/100}{8.077} = 10.9$ kN

Total load = $\frac{87.5}{100} \times 2 \times W_2 = 35$ kN

$R_A = 35 - 10.9 = 24.1$ kN

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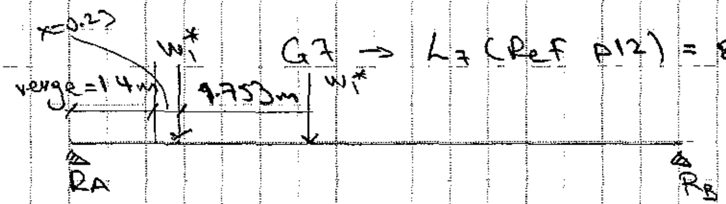
Girder G7 is directly loaded with

W_1^*. W_1^* (include 25% for impact) = 0.33 × 1.25 × 12 × 9.967 ⇒

$W_1^* = 49.3$ KN

G7 → L_7 (Ref p12) = 8.077 m



$\curvearrowright + \; M_A = 0 \Rightarrow 1.63 W_1^* + 3.383 W_1^* - 8.077 R_B = 0 \Rightarrow$

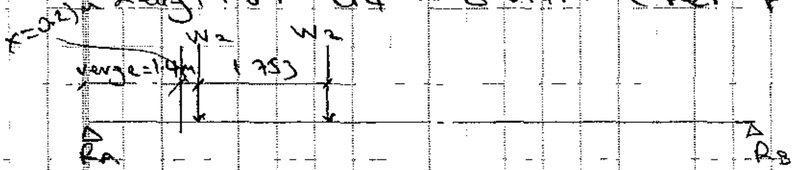
$R_B = \frac{5.013 W_1^*}{8.077} = \frac{5.013 \times 49.3}{8.077} = 30.6$ KN

Total load = 2 × 49.3 = 98.6 KN

$R_A = 98.6 - 30.6 = 68.0$ KN

G4 is directly loaded with $W_2 = 20$ KN

Length of G4 = 8.077m (Ref p12)



$\curvearrowright + \; M_A = 0 \Rightarrow 1.63 W_2 + 3.383 W_2 - 8.077 R_B = 0 \Rightarrow$

$R_B = \frac{5.013}{8.077} \times W_2 = 12.4$ KN

Total load = 2 × W_2 = 40 KN

$R_A = 40 - 12.4 = 27.6$ KN

CALCULATION SHEET

JACOBS

Project Title. BE4 assessment		Sheet No 31
Subject AGB 3 Live Load assessment		Calc No
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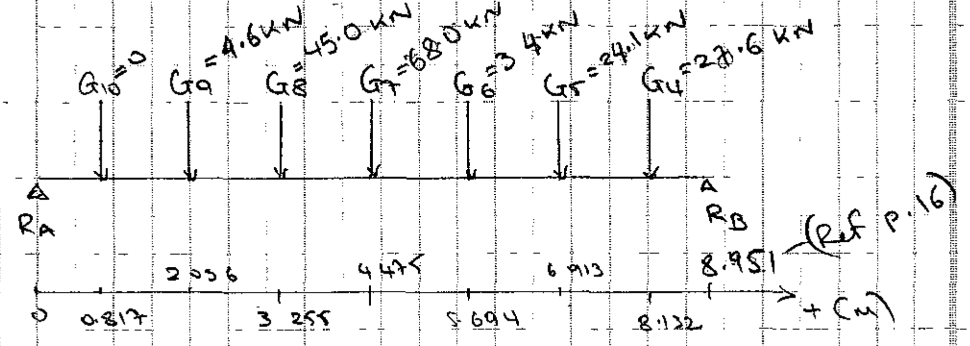
Made By [redacted]	Date Aug 04	Revised By / Date
Checked By [redacted]	Date Aug 04	Checked By / Date

LL assessment (Ref p 1,2,3,4)



$$+\circlearrowright\ M_A = 0 \Rightarrow 4.6 \times 2.036 + 45.0 \times 3.255 + 68.0 \times 4.475$$
$$+ 3.4 \times 5.694 + 24.1 \times 6.913 + 27.6 \times 8.132$$
$$- 8.951 \cdot R_B = 0 \Rightarrow$$

$$R_B = \frac{9.4 + 146.5 + 304.3 + 19.4 + 166.6 + 224.4}{8.951} \Rightarrow$$

$$R_B = 97.3 \text{ kN}$$

$$\text{Total load} = 4.6 + 45 + 68 + 3.4 + 24.1 + 27.6$$
$$= 172.7 \text{ kN}$$

$$R_A = 172.7 - 97.3 = 75.4 \text{ kN}$$

$$+\circlearrowright\ BM_{G7} = 75.4 \times 4.475 - 4.6 \times (4.475 - 2.036)$$
$$- 45.0\,(4.475 - 3.255) \Rightarrow$$

$$BM_{G7} = 337.4 - 11.2 - 54.9 = 271.3 \text{ kNm} \Rightarrow$$

$$\underline{BM_{G7} = 271.3 \text{ kNm}}$$

CALCULATION SHEET

Project Title. **VAR9-830 BE4 Assessments**		
Subject. **AGB 3 Luffness Mains Bridge BE4 Assessment**		Calc No
Job No. **J20308B-1142**		File.
Made by. [redacted]ate. **19/07/2004**	Checked by.	Date
Revised b[redacted]ate. Aug 04	Checked by.	Date.

Bending Capacity

Total Moment due to Dead and Live loads $M_t = M_d + M_l$ = ~~725.097~~ kNm

453 4 DL (P.26)
271.3 LL (P.31)
724 7 (DL+LL)

Unrerstrained condition

Total moments (DD+LL) = 7247 KNm

BS 153 Part 4 21(a)

For member unrestrained in lateral bending, effective length of compression flange is equal to span (load is not applied on compression flange)

	Span l = EL =	8 951 m ✓
	Span l =	352 389 in
	I_{yy} =	74814821 mm^4 ✓
	A = A_{EG} =	32802 354 mm^2 ✓
	Radius of Gyration about y-y Axis $r_y = \sqrt{(I_{yy} / A)}$ =	47 757 mm ✓
	Radius of Gyration about y-y Axis r_y =	1 88 in
	Compression Flange Thickness $T = t_{top}$ =	12 7 mm
	Compression Flange Thickness T =	0 5 in
	Overall Depth D = d =	1981 2 mm
	Overall Depth D =	78 in
BS153 Pt 3B-28(b)(i)	$C_s = 170000 / (l / r_y)^2 \times \sqrt{[1 + (1 / 20) \times ((l \times T) / (r_y \times D))^2]}$ =	5 011 ton/in^2
	Web Thickness $t = t_{web}$ =	9 525 mm ✓
	Web Thickness t =	0 375 in
BS153 Pt 3B-28(b)(i)	T / t =	1 333 ✓
	Clear Distance Between Flange Angles $d_1 = d_{web} - t_{TEAHL} - d_{TEAVL} - t_{BEAHL} - d_{BEAVL}$ =	1778 mm ✓
	Clear Distance Between Flange Angles d_1 =	70 in ✓
	d_1 / t =	186 667 ✓

T / t < 2 and d_1 / t < 85 therefore C_s ~~can~~ NOT be increased by 20%

$C_s = C_s$ ~~x 1.2~~ = ~~6.014~~ 5 011 ton/in^2

CALUCLATION SHEET

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BS 153 Pt 3B Table 8	Allowable Stress p_{bc} =	~~2.8~~ 2.4 ton/in^2
	Allowable Stress p_{bc} =	~~43256 606~~ 37077 kN/m^2
BE4 Part 1-304(a)	Apply 25% enhancement for steel	
BS153 Pt 3B Table 1	$p_{bc} = p_{bc} \times 1\ 25$ =	~~54070 757~~ 46346·6 kN/m^2
	Girder Depth Midspan d =	1981 2 mm ✓
	$y_{bot} = y_{bar}$ =	989 858 mm ✓
	$y_{top} = d - y_{bot}$ =	991 342 mm ✓
	I_{xx} =	19220307480 mm^4 ✓
	$Z_{top} = I_{xx} / y_{top}$ =	19388179 mm^3 ✓
	Z_{top} =	0 019 m^3
	Unrestrained Bending Capacity $M_{cu} = p_{bc} \times Z_{top}$ =	~~**1046.334 KNm**~~ 898·6 KNm > 724 7 KNm
	Unrestrained Bending Capacity M_{cu} =	~~**345.079**~~ 295·8 **tonft**
	$M_{cu} > M_t$	OK

External girder is OK in unrestrained bending.

Restrained condition

It is not necessary to check restrained bending capacity as the member has sufficient capacity in the unrestrained condition

CALCULATION SHEET

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Dead Load Shear Force

Span L = L_{skew} =	8820 mm	✓
Span L =	8 82 m	
Self Weight W_{EG} =	2 917 kN/m	✓
Point Load 1 DL_4 =	56 446 kN	✓
Point Load 1 Position d_4 =	0 752 m	✓
Point Load 2 DL_5 =	56 446 kN	✓
Point Load 2 Position d_5 =	1 972 m	✓
Point Load 3 DL_6 =	56 446 kN	✓
Point Load 3 Position d_6 =	3 191 m	✓
Point Load 4 DL_7 =	56 446 kN	✓
Point Load 4 Position d_7 =	4 41 m	✓
Point Load 5 DL_8 =	50 604 kN	✓
Point Load 5 Position d_8 =	5 629 m	✓
Point Load 6 DL_9 =	34 843 kN	✓
Point Load 6 Position d_9 =	6 848 m	✓
Point Load 7 DL_{10} =	18 414 kN	✓
Point Load 7 Position d_{10} =	8 068 m	✓

Shear Force at Point A

SF at A due to SW $SF_{ASW} = W_{EG} \times (L / 2)$ =	12 862 kN	✓
SF at A due to DL_4 $SF_{APL1} = DL_4 \times ((L - d_4) / L)$ =	51 631 kN	✓
SF at A due to DL_5 $SF_{APL2} = DL_5 \times ((L - d_5) / L)$ =	43.828 kN	✓

CALCULATION SHEET

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SF at A due to DL_6 $SF_{APL3} = DL_6 \times ((L - d_6) / L) =$ 36 025 kN

SF at A due to DL_7 $SF_{APL4} = DL_7 \times ((L - d_7) / L) =$ 28 223 kN

SF at A due to DL_8 $SF_{APL5} = DL_8 \times ((L - d_8) / L) =$ 18 307 kN

SF at A due to DL_9 $SF_{APL6} = DL_9 \times ((L - d_9) / L) =$ 7 789 kN

SF at A due to DL_{10} $SF_{APL7} = DL_{10} \times ((L - d_{10}) / L) =$ 1 571 kN

SF at A $SF_A = SF_{ASW} + SF_{APL1} + SF_{APL2} + SF_{APL3} + SF_{APL4} + SF_{APL5} + SF_{APL6} + SF_{APL7} =$ 200 235 kN

Shear Force at Point B

SF at B due to SW $SF_{BSW} = W_{EG} \times (L / 2) =$ 12 862 kN

SF at B due to DL_4 $SF_{BPL1} = DL_4 \times (d_4 / L) =$ 4 815 kN

SF at B due to DL_5 $SF_{BPL2} = DL_5 \times (d_5 / L) =$ 12 618 kN

SF at B due to DL_6 $SF_{BPL3} = DL_6 \times (d_6 / L) =$ 20 42 kN

SF at B due to DL_7 $SF_{BPL4} = DL_7 \times (d_7 / L) =$ 28 223 kN

SF at B due to DL_8 $SF_{BPL5} = DL_8 \times (d_8 / L) =$ 32.297 kN

SF at B due to DL_9 $SF_{BPL6} = DL_9 \times (d_9 / L) =$ 27 055 kN

SF at B due to DL_{10} $SF_{BPL7} = DL_{10} \times (d_{10} / L) =$ 16 843 kN

SF at B $SF_B = SF_{BSW} + SF_{BPL1} + SF_{BPL2} + SF_{BPL3} + SF_{BPL4} + SF_{BPL5} + SF_{BPL6} + SF_{BPL7} =$ 155 132 kN

$SF_B > SF_A$

Shear force due to dead load is equal to maximum shear force

Shear Force due to Dead Load $SF_d = SF_B =$ 200.235 kN
Shear Force due to Dead Load $SF_d =$ 20.09 tons

CALCULATION SHEET

JE JACOBS

Project Title BE4 assessment				Sheet No 36
Subject AGB 3 Live Load assessment				Calc No
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Made By [redacted]	Date Aug 04	Revised By		Date
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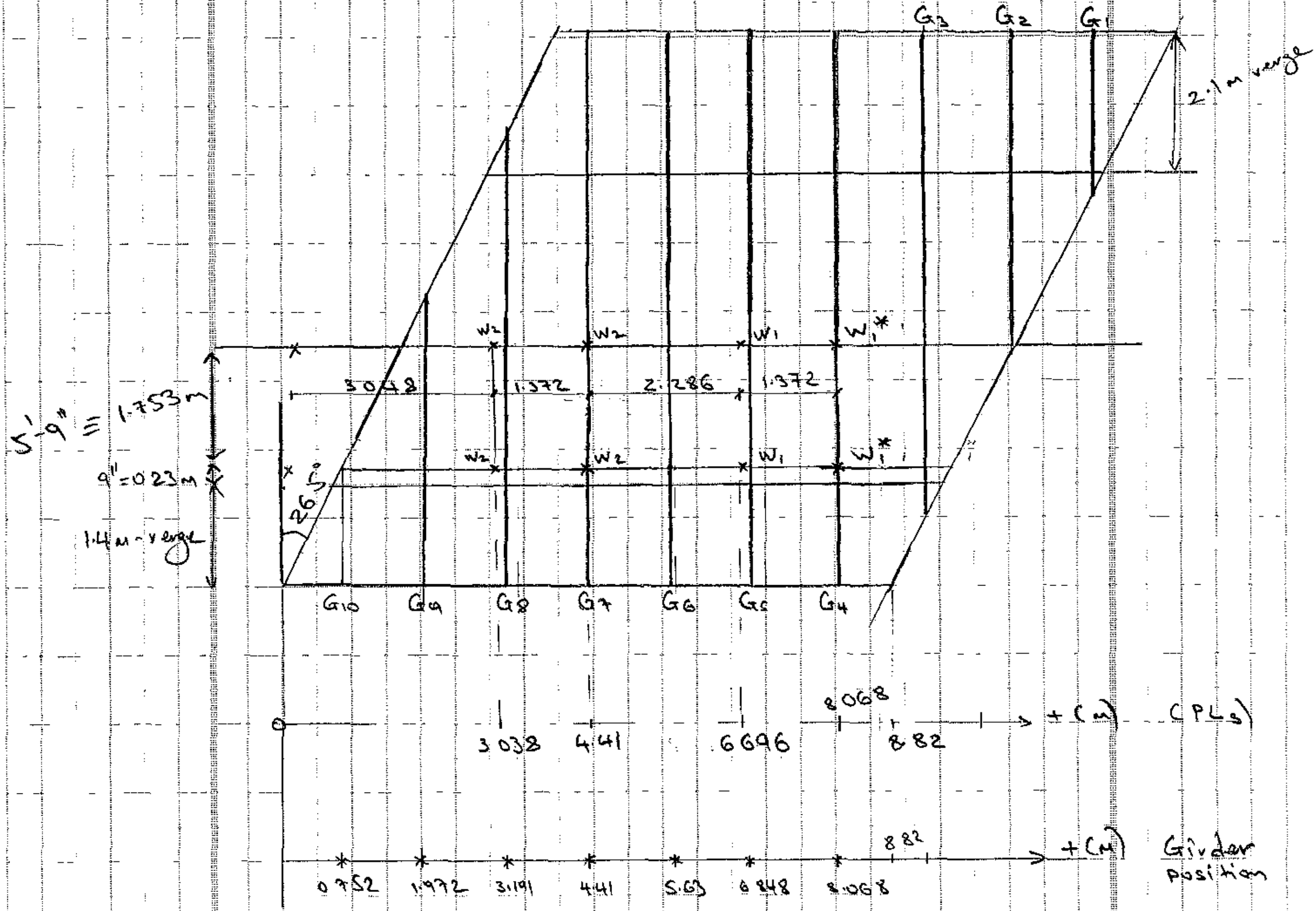
Shear force - LL assessment



Girders G4 and G7 are directly loaded with LL PLs, for the rest distribution will be assumed.

Only one vehicle train will be applied as carriageway < 18 ft (Ref BM assessment)

CALCULATION SHEET

JACOBS

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Subject AGB 3 Live Load assessment		Calc No	
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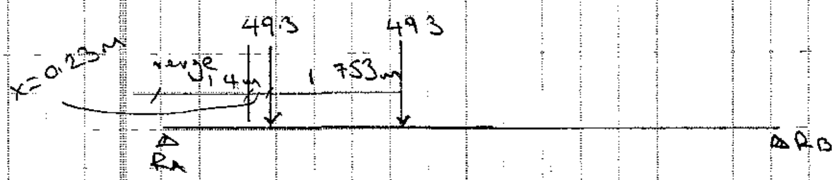
G4

Ref BM assessment

$W_i^* =$ includes 25% for impact $= 0.33 \times 1.25 \times 12 \times 9.967$
$= 49.3$ kN

length of G9 = 8.077m (Ref p14)



$+\circlearrowright\ M_A = 0 \Rightarrow (1.63 \times 49.3 + 3.383 \times 49.3) - 8.077\, R_B = 0 \Rightarrow$

$R_B = \dfrac{5.013 \times 49.3}{8.077} = 30.6$ kN

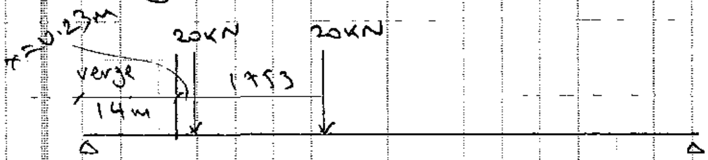
Total load $= 2 \times 49.3 = 98.6$ kN

$R_A = 98.6 - 30.6 = 68.0$ kN

G7

$W_2 = 0.167 \times 12 \times 9.967 = 20$ kN

length of G6 = 8.077 m



$+\circlearrowright\ M_A = 0 \Rightarrow 1.63 \times 20 + 3.383 \times 20 - 8.077\, R_B = 0 \Rightarrow$

$R_B = \dfrac{5.013 \times 20}{8.077} = 12.4$ kN

Total load $= 2 \times 20 = 40$ kN

$R_A = 40 - 12.4 = 27.6$ kN

CALCULATION SHEET

JACOBS

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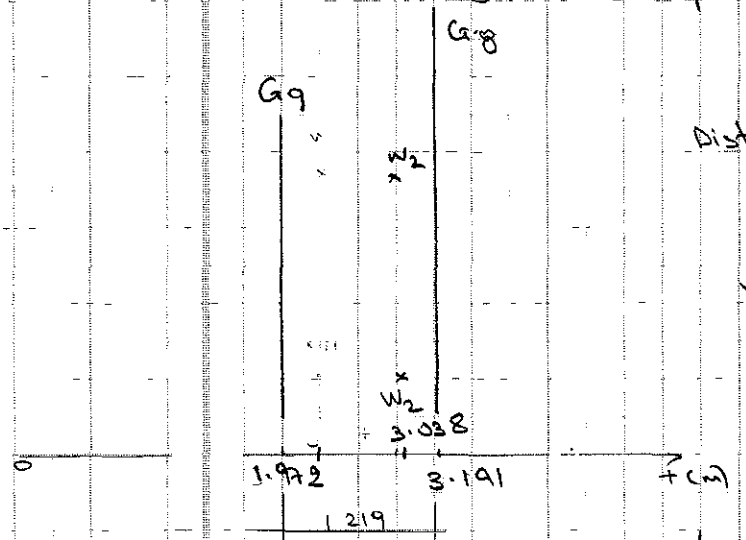
Checked By [redacted]	Date Aug 04	Checked By	Date

Distribution G9 – G8

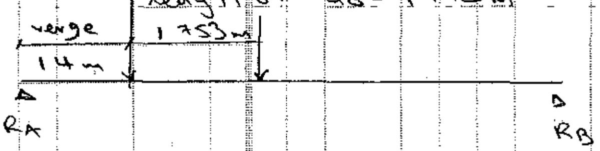


Distribution to G9 = $\frac{3.191 - 3.038}{1.219} = 12.6\%$

" " G8 = 100 – 12.6 = 87.4%

$W_1 = 0.167 \times 12 \times 9.967 = 20.0$ kN

length of G8 = 7.178 m



+) $M_A = 0 \Rightarrow (1.63 + 3.383) W_1 \times 87.4\% - 7.178 R_B = 0$ ⇒

$R_B = \frac{5.013 \times 20.0 \times 0.874}{7.178} = 12.2$ kN

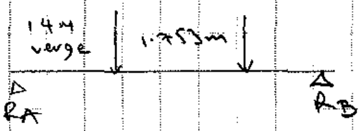
Total load = 20.0 × 2 × 87.4%

= 35.0 kN

$R_A = 35.0 - 12.2 = 22.8$ kN

G9

length of G9 = 4.742 m



+) $M_A = 0 \Rightarrow$

$R_B = \frac{5.013 \times 20.0 \times 0.126}{4.742} = 2.7$ kN

Total load = 20.0 × 2 × 12.6%

= 5.0 kN

$R_A = 5.0 - 2.7 = 2.3$ kN

CALCULATION SHEET

JACOBS

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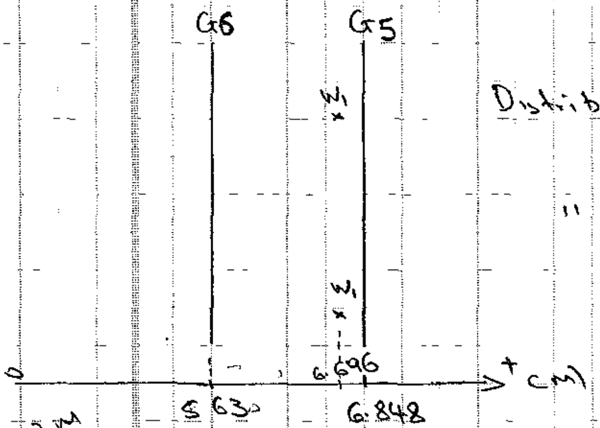
Distribution G6 - G5

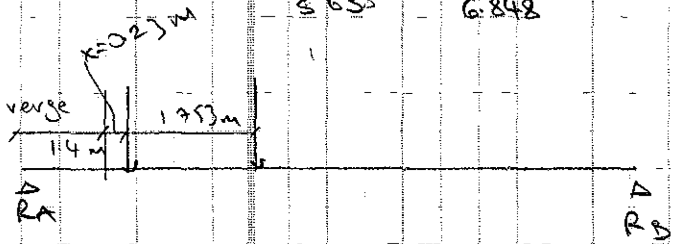
lengths of G5 = G4 = 8.077 m

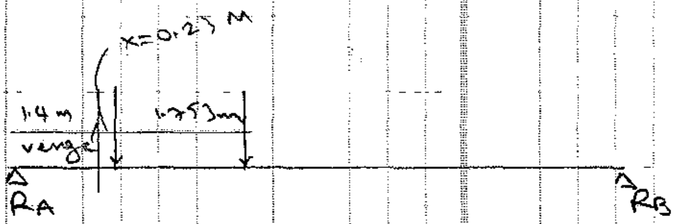


Distribution to G5 = $\frac{6.696 - 5.63}{1.219}$ = 87.4 %

" " G6 = 100 - 87.6 = 12.6 %

W_1 = 0.33 × 12 × 9.967 = 39.5 kN





+) $M_A = 0 \Rightarrow R_B = \frac{5.013 \times 39.5}{8.077} \times 87.4\%$

= 21.4 kN

Total load = 87.4% × 2 × 39.5

= 69.0 kN

R_A = 69 - 21.4 = 47.6 kN

(G5)

$\Rightarrow R_B = \frac{5.013 \times 39.5}{8.077} \times 12.6\%$

= 3.1 kN

Total load = 12.6% × 39.5 × 2

= 10 kN

R_A = 10 - 3.1 = 6.9 kN

(G6)

CALCULATION SHEET

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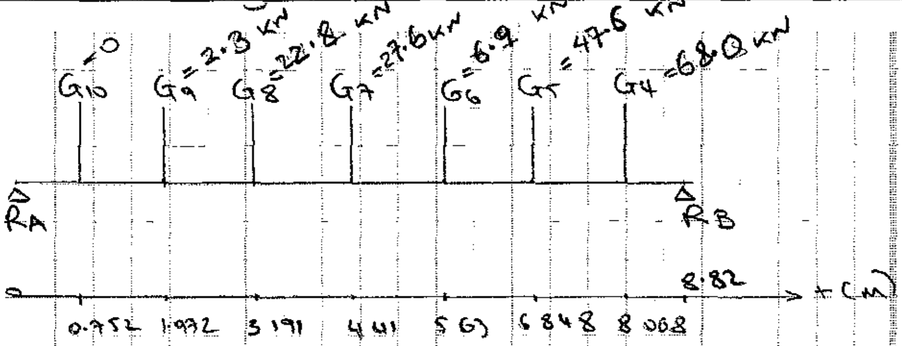

$\overset{+}{\curvearrowright} M_A = 0 \Rightarrow 2.3 \times 1.992 + 22.8 \times 3.191 + 27.6 \times 4.41$

$+ 6.9 \times 5.63 + 47.6 \times 6.848 + 68.0 \times 8.068$

$- R_B \times 8.82 = 0 \Rightarrow$

$$R_B = \frac{4.54 + 72.8 + 121.7 + 38.8 + 326.0 + 548.6}{8.82}$$

$$= \frac{1112.4}{8.82} = 126.1 \text{ kN}$$

CALCULATION SHEET

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Shear Capacity

Total Shear Force due to Dead and Live loads SF_t $= SF_d + SF_l =$ 326 383 kN

200.2 kN (DL)
126.1 kN (LL)
326.3 kN ✓

Web Area A_{web} = 18628.995 mm² ✓
Web Area A_{web} = 0 019 m²

Reduce web area to take account of holes

External Web Thickness t_{web} = 9 525 mm ✓
External Web Thickness t_{web} = 0 01 m

Maximum Hole Depth d_{hole} = 240 mm ✓ (Hole found at SW corner)
Maximum Hole Depth d_{hole} = 0 24 m

$A_{web} = A_{web} - (t_{web} \times d_{hole}) =$ 0 016 m² — 16343 mm² ✓

~~Average shear stess p_{ave} = SF_t / A_{web} = 19970.849 kN/m²~~

BS153 Pt 3B Table 3 — Permissable Shear p_s = 5 5 ton/in² ✓
Permissable Shear p_s = 84968 332 kN/m²

BE4 Part 1-304(a) — Apply 25% enhancement for steel

BS153 Pt 3B Table 1 — $p_s = p_s \times 1\ 25 =$ ✓ 106210 416 kN/m² ✓

Shear Capacity $SF_c = A_{web} \times p_s$ = **1735.796 KN** ✓ > 326.3 kN ✓
Shear Capacity SF_c = **174.154 tons**

$SF_c > SF_t$

External girder is OK in shear.

External Girder Assessment Result: **PASS** ✓

CALCULATION SHEET

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Assessment of Internal Girder

Dead Load Moment

Girder span is calculated from spacing between external girders

External Girder Spacing S_E	8 077 m ✓
Internal Girder Self Weight W_{IG} =	1 545 kN/m ✓
Fill Self Weight W_F =	8 949 kN/m ✓
Buckle Plate Self Weight W_{BP} =	0 894 kN/m ✓
Total Dead Loads $W_{UDL} = W_{IG} + W_F + W_{BP}$ =	11 388 kN/m ✓
Verge Self Weight W_V =	4 5 kN/m ✓
Verge 1 Width w_{V1} =	2100 mm ✓
Verge 1 Width w_{V1} =	2 1 m
Verge 2 Width w_{V2} =	1400 mm ✓
Verge 2 Width w_{V2} =	1 4 m
Carriageway Self Weight W_C =	1 379 kN/m ✓
Carriageway Width $w_C = S_E - w_{V1} - w_{V2}$ =	4 577 m ✓

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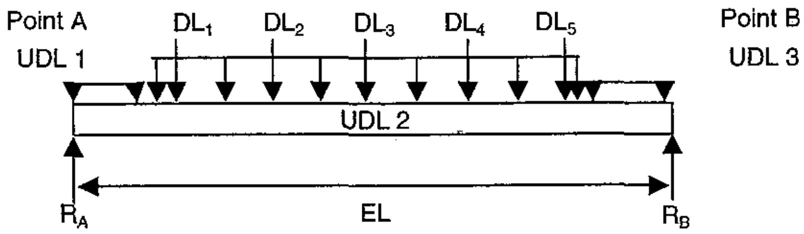

BE4 Part 1-303(a)(i)	Effective Span EL = S_E =	8 077 m	✓
	W_{UDL} =	11 388 kN/m	✓
	UDL 1 $W_{UDL1} = W_V$ =	4.5 kN/m	✓
	UDL 1 Width $w_{UDL1} = w_{V1}$ =	2 1 m	✓
	UDL 1 Position $L_{UDL1} = w_{V1} / 2$ =	1 05 m	✓
	UDL 2 $W_{UDL2} = W_C$ =	1 379 kN/m	✓
	UDL 2 Width $w_{UDL2} = w_C$ =	4 577 m	✓
	UDL 2 Position $L_{UDL2} = w_{V1} + (w_C / 2)$ =	4 389 m	✓
	UDL 3 $W_{UDL3} = W_V$ =	4 5 kN/m	✓
	UDL 3 Width $w_{UDL3} = w_{V2}$ =	1 4 m	✓
	UDL 3 Position $L_{UDL3} = w_{V1} + w_C + (w_{V2} / 2)$ =	7 377 m	✓
	Point Load 1 $DL_1 = W_T$ =	0 206 kN	✓
	Point Load 1 Position $L_1 = (1 / 6) \times S_E$ =	1 346 m	✓
	Point Load 2 $DL_2 = W_T$ =	0 206 kN	✓
	Point Load 2 Position $L_2 = (2 / 6) \times S_E$ =	2 692 m	✓
	Point Load 3 $DL_3 = W_T$ =	0 206 kN	✓
	Point Load 3 Position $L_3 = (3 / 6) \times S_E$ =	4 039 m	✓
	Point Load 4 $DL_4 = W_T$ =	0 206 kN	✓
	Point Load 4 Position $L_4 = (4 / 6) \times S_E$ =	5 385 m	✓
	Point Load 5 $DL_5 = W_T$ =	0 206 kN	✓
	Point Load 5 Position $L_5 = (5 / 6) \times S_E$ =	6 731 m	✓

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Taking Moments about Point A

BM at A due to UDL $BM_{AUDL} = W_{UDL} \times (EL^2 / 2) =$ 371 485 kNm ✓

BM at A due to UDL 1 $BM_{AUDL1} = W_{UDL1} \times w_{UDL1} \times L_{UDL1} =$ 9 923 kNm ✓

BM at A due to UDL 2 $BM_{AUDL2} = W_{UDL2} \times w_{UDL2} \times L_{UDL2} =$ 27 708 kNm ✓

BM at A due to UDL 3 $BM_{AUDL3} = W_{UDL3} \times w_{UDL3} \times L_{UDL3} =$ 46 478 kNm ✓

BM at A due to DL_1 $BM_{APL1} = DL_1 \times L_1 =$ 0 277 kNm ✓

BM at A due to DL_2 $BM_{APL2} = DL_2 \times L_2 =$ 0 554 kNm ✓

BM at A due to DL_3 $BM_{APL3} = DL_3 \times L_3 =$ 0 831 kNm ✓

BM at A due to DL_4 $BM_{APL4} = DL_4 \times L_4 =$ 1 108 kNm ✓

BM at A due to DL_5 $BM_{APL5} = DL_5 \times L_5 =$ 1 384 kNm ✓

BM at A due to loads $BM_{ALoad} = BM_{AUDL} + BM_{AUDL1} + BM_{AUDL2} + BM_{AUDL3} + BM_{APL1} + BM_{APL2} + BM_{APL3} + BM_{APL4} + BM_{APL5} =$ 459 747 kNm ✓

BM at A $BM_A =$ 0 kNm

$R_B = BM_{ALoad} / EL =$ 56 919 kN ✓

Total Load due to UDL $Load_{UDL} = W_{UDL} \times EL =$ 91 983 kN ✓

Total Load at due to UDL 1 $Load_{UDL1} = W_{UDL1} \times w_{UDL1} =$ 9 45 kN ✓

Total Load at due to UDL 2 $Load_{UDL2} = W_{UDL2} \times w_{UDL2} =$ 6 314 kN ✓

Total Load at due to UDL 3 $Load_{UDL3} = W_{UDL3} \times w_{UDL3} =$ 6 3 kN ✓

Loads at Point A $Load_A = Load_{UDL} + Load_{UDL1} + Load_{UDL2} + Load_{UDL3} + DL_1 + DL_2 + DL_3 + DL_4 + DL_5 =$ 115 076 kN ✓

$R_A = Load_A - R_B =$ 58 157 kN ✓

CALCULATION SHEET

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Revised b[redacted]	Date. Aug 04	Checked by.	Date.

Bending Moment at Point 1

BM at 1 due to UDL $BM_{1UDL} = W_{UDL} \times (L_1^2 / 2) =$	10 319 kNm
BM at 1 due to UDL 1 $BM_{1UDL1} = W_{UDL1} \times w_{UDL1} \times (L_1 - L_{UDL1}) =$	2 799 kNm
BM at 1 due to UDL 2 $BM_{3UDL2} = W_{UDL2} \times (L_1 - w_{UDL1}) \times ((L_1 - w_{UDL1}) / 2) =$	0 392 kNm
BM at 1 due to R_A $BM_{1RA} = R_A \times L_1 =$	78 291 kNm
BM at 1 $BM_1 = BM_{1UDL} + BM_{1UDL1} + BM_{1UDL2} + BM_{1UDL2} - BM_{1RA} =$	-64 781 kNm

Bending Moment at Point 2

BM at 2 due to UDL $BM_{2UDL} = W_{UDL} \times (L_2^2 / 2) =$	41 276 kNm
BM at 2 due to UDL 1 $BM_{2UDL1} = W_{UDL1} \times w_{UDL1} \times (L_2 - L_{UDL1}) =$	15 521 kNm
BM at 2 due to UDL 2 $BM_{3UDL2} = W_{UDL2} \times (L_2 - w_{UDL1}) \times ((L_2 - w_{UDL1}) / 2) =$	0 242 kNm
BM at 2 due to DL_1 $BM_{2PL1} = DL_1 \times (L_2 - L_1) =$	0 277 kNm
BM at 2 due to R_A $BM_{2RA} = R_A \times L_2 =$	156 582 kNm
BM at 2 $BM_2 = BM_{2UDL} + BM_{1UDL1} + BM_{1UDL2} + BM_{2PL1} - BM_{2RA} =$	-99 266 kNm

Not critical to check points 1, 2, 4 and 5. The max BM would be found at PL3 or midspan since the points coincide

CALCULATION SHEET

Project Title. **VAR9-830 BE4 Assessments**			
Subject. **AGB 3 Luffness Mains Bridge BE4 Assessment**			Calc No.
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Made by. [redacted]	Date. **19/07/2004**	Checked by.	Date.
Revised b[redacted]	Date Aug 04	Checked by.	Date

Bending Moment at Point 3

BM at 3 due to UDL $BM_{3UDL} = W_{UDL} \times (L_3^2 / 2) =$ 92 871 kNm ✓

BM at 3 due to UDL 1 $BM_{3UDL1} = W_{UDL1} \times w_{UDL1} \times (L_3 - L_{UDL1}) =$ 28 244 kNm ✓

BM at 3 due to UDL 2 $BM_{3UDL2} = W_{UDL2} \times (L_3 - w_{UDL1}) \times ((L_3 - w_{UDL1}) / 2) =$ 2 592 kNm ✓

BM at 3 due to DL_1 $BM_{3PL1} = DL_1 \times (L_3 - L_1) =$ 0 554 kNm ✓

BM at 3 due to DL_2 $BM_{3PL2} = DL_2 \times (L_3 - L_2) =$ 0.277 kNm ✓

BM at 3 due to R_A $BM_{3RA} = R_A \times L_3 =$ 234 873 kNm ✓

BM at 3 $BM_3 = BM_{3UDL} + BM_{3UDL1} + BM_{3UDL2} + BM_{3PL1} + BM_{3PL2} - BM_{3RA} =$ -110 336 kNm ✓

Bending Moment at Point 4

BM at 4 due to UDL $BM_{4UDL} = W_{UDL} \times (L_4^2 / 2) =$ 165 104 kNm

BM at 4 due to UDL 1 $BM_{4UDL1} = W_{UDL1} \times w_{UDL1} \times (L_4 - L_{UDL1}) =$ 40 966 kNm

BM at 4 due to UDL 2 $BM_{3UDL2} = W_{UDL2} \times (L_4 - w_{UDL1}) \times ((L_4 - w_{UDL1}) / 2) =$ 7 442 kNm

BM at 4 due to DL_1 $BM_{4PL1} = DL_1 \times (L_4 - L_1) =$ 0 831 kNm

BM at 4 due to DL_2 $BM_{4PL2} = DL_2 \times (L_4 - L_2) =$ 0 554 kNm

BM at 4 due to DL_3 $BM_{4PL3} = DL_3 \times (L_4 - L_3) =$ 0 277 kNm

BM at 4 due to R_A $BM_{4RA} = R_A \times L_4 =$ 313 165 kNm

BM at 4 $BM_4 = BM_{4UDL} + BM_{4UDL1} + BM_{4UDL2} + BM_{4PL1} + BM_{4PL2} + BM_{4PL3} - BM_{4RA} =$ -97 992 kNm

Not checked.

CALCULATION SHEET

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Made by. [redacted]	Date. **19/07/2004**	Checked by.	Date.
Revised by.	Date.	Checked by.	Date.

Bending Moment at Point 5

BM at 5 due to UDL $BM_{5UDL} = W_{UDL} \times (L_5^2 / 2) =$	257 975 kNm
BM at 5 due to UDL 1 $BM_{5UDL1} = W_{UDL1} \times w_{UDL1} \times (L_5 - L_{UDL1}) =$	53 688 kNm
BM at 5 due to UDL 2 $BM_{3UDL2} = W_{UDL2} \times (L_5 - w_{UDL1}) \times ((L_5 - w_{UDL1}) / 2) =$	14 791 kNm
BM at 5 due to DL_1 $BM_{5PL1} = DL_1 \times (L_5 - L_1) =$	1 108 kNm
BM at 5 due to DL_2 $BM_{5PL2} = DL_2 \times (L_5 - L_2) =$	0 831 kNm
BM at 5 due to DL_3 $BM_{5PL3} = DL_3 \times (L_5 - L_3) =$	0 554 kNm
BM at 5 due to DL_4 $BM_{5PL4} = DL_4 \times (L_5 - L_4) =$	0 277 kNm
BM at 5 due to R_A $BM_{5RA} = R_A \times L_5 =$	391 456 kNm
BM at 5 $BM_5 = BM_{5UDL} + BM_{5UDL1} + BM_{5UDL2} + BM_{5PL1} + BM_{5PL2} + BM_{5PL3} + BM_{5PL4} - BM_{5RA} =$	-62 232 kNm

Not checked

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Taking Moments about Point B (Check)

BM at B due to UDL $BM_{BUDL} = W_{UDL} \times (EL^2 / 2) =$	371 485 kNm
BM at B due to UDL 1 $BM_{BUDL1} = W_{UDL1} \times w_{UDL1} \times (EL - L_{UDL1}) =$	66 41 kNm
BM at B due to UDL 2 $BM_{BUDL2} = W_{UDL2} \times w_{UDL2} \times (EL - L_{UDL2}) =$	23 289 kNm
BM at B due to UDL 3 $BM_{BUDL3} = W_{UDL3} \times w_{UDL3} \times (EL - L_{UDL3}) =$	4 41 kNm
BM at B due to DL_1 $BM_{BPL1} = DL_1 \times (EL - L_1) =$	1 384 kNm
BM at B due to DL_2 $BM_{BPL2} = DL_2 \times (EL - L_2) =$	1 108 kNm
BM at B due to DL_3 $BM_{BPL3} = DL_3 \times (EL - L_3) =$	0 831 kNm
BM at B due to DL_4 $BM_{BPL4} = DL_4 \times (EL - L_4) =$	0 554 kNm
BM at B due to DL_5 $BM_{BPL5} = DL_5 \times (EL - L_5) =$	0 277 kNm
BM at B due to R_A $BM_{BRA} = R_A \times EL =$	469 747 kNm
BM at B $BM_B = BM_{BUDL} + BM_{BUDL1} + BM_{BUDL2} + BM_{BUDL3} + BM_{BPL1} + BM_{BPL2} + BM_{BPL3} + BM_{BPL4} + BM_{BPL5} - BM_{BRA} =$	0 kNm

No need to check.

CALCULATION SHEET

Project Title. **VAR9-830 BE4 Assessments**			
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Job No. **J20308B-1142**			File
Made by. [redacted]	Date. **19/07/2004**	Checked by.	Date.
Revised b[redacted]	Date. Aug 04	Checked by.	Date.

Bending Moment at Midspan

BM at Midspan due to UDL $BM_{MUDL} = W_{UDL} \times ((EL / 2)^2 / 2) =$ 92 871 kNm

BM at Midspan due to UDL 1 $BM_{MUDL1} = W_{UDL1} \times w_{UDL1} \times ((EL / 2) - L_{UDL1}) =$ 28 244 kNm

BM at Midspan due to UDL 2 $BM_{MUDL2} = W_{UDL2} \times ((EL / 2) - w_{UDL1}) \times (((EL / 2) - w_{UDL1}) / 2) =$ 8 207 kNm

BM at Midspan due to DL_1 $BM_{MPL1} = DL_1 \times ((EL / 2) - L_1) =$ 0 554 kNm

BM at Midspan due to DL_2 $BM_{MPL2} = DL_2 \times ((EL / 2) - L_2) =$ 0 277 kNm

BM at Midspan due to DL_3 $BM_{MPL3} = DL_3 \times ((EL / 2) - L_3) =$ 0 kNm

BM at Midspan due to DL_4 $BM_{MPL4} = DL_4 \times ((EL / 2) - L_4) =$ -0 277 kNm

BM at Midspan due to DL_5 $BM_{MPL5} = DL_5 \times ((EL / 2) - L_5) =$ -0 554 kNm

BM at Midspan due to R_A $BM_{MRA} = R_A \times (EL / 2) =$ 234 873 kNm

BM at Midspan $BM_M = BM_{MUDL} + BM_{MUDL1} + BM_{MUDL2} + BM_{MPL1} + BM_{MPL2} + BM_{MPL3} + BM_{MPL4} + BM_{MPL5} - BM_{MRA} =$ -105 551 kNm

No need to be checked

Dead load moment is equal to maximum bending moment

Dead Load Moment $M_d = -BM_3 =$ 110.336 kNm ✓

Dead Load Moment $M_d =$ 36.319 tonft ✓

CALCULATION SHEET

JACOBS

Project Title BE4 assessment			Sheet No 50
Subject AGB 3			Calc No
Job No J20308b			File
Made By [redacted]	Date Aug 04	Revised By.	Date
Checked By [redacted]	Date Aug 04	Checked By	Date

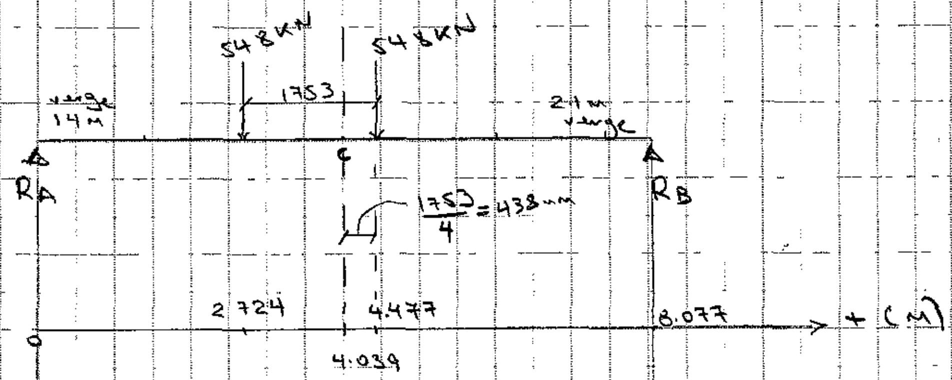
Transverse girders
LL assessment — BM at midspan

According to BE4-1967 section 202, Part I a single 11 ton axle load would be applied on the transverse girder (impact load is included)

Hence, wheel load $= \frac{11}{2} = 5.5$ ton

$= 5.5 \times 9.967 = 54.8$ kN/axle

From LL assessment of the external girder sketch we can see that only one axle can be loaded on the transverse girder.



CALCULATION SHEET

JACOBS

Project Title BE4 assessment			Sheet No 51
Subject AGB 3			Calc No
Job No J20308b			File
Made By [redacted]	Date Aug 04	Revised By	Date
Checked By [redacted]	Date Aug 04	Checked By	Date

$M_A = 0$ (+) ⇒ $54.8 \times 2.724 + 54.8 \times 4.477 - R_B \times 8.077 = 0$ ⇒

$R_B = \frac{54.8(2.724 + 4.477)}{8.077} = 48.9$ kN

Total load $= 2 \times 54.8 = 109.6$ kN

$R_A = 109.6 - 48.9 = 60.7$ kN

(+) $M_{MID\text{-}span} = -54.8 \times 0.438 + 48.9 \times 4.038$

$= -24.0 + 197.4 = 173.4$ kNm

CALCULATION SHEET

Project Title. **VAR9-830 BE4 Assessments**			
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Bending Capacity

Total Moment due to Dead and Live loads $M_t = M_d + M_l$ = → 173.4 KN M — ~~286.295 kNm~~ 283.7 KN m (110.3 KN m)

External girder spacing S_E = 8 077 m

External girder spacing S_E = 8077.224 mm

External Bottom Flange Width w_{bot} = 355 6 mm

Span $l = S_E - w_{bot}$ = 7721 624 mm

Span l = 304 001 in

I_{yy} = 46305543 mm^4

$A = A_{IG}$ = 17479 804 mm^2

Radius of Gyration about y-y Axis $r_y = \sqrt{(I_{yy} / A)}$ = 51 469 mm

Radius of Gyration about y-y Axis r_y = 2 026 in

Compression Flange Thickness $T = t_{top}$ = 12 7 mm

Compression Flange Thickness T = 0 5 in

Overall Depth D = d = 406 4 mm

Overall Depth D = 16 in

Wrong procedure !

BS153 Pt 3B-28(b)(i) $C_s = 170000 / (l / r_y)^2 \times \sqrt{[1 + (1 / 20) \times ((l \times T) / (r_y \times D))^2]}$ = 10 943 ton/in^2

Web Thickness $t = t_{web}$ = 9 525 mm ✓

Web Thickness t = 0 375 in ✓

BS153 Pt 3B-28(b)(i) T / t = 1 333 ✓

Clear Distance Between Flange Angles $d_1 = d_{web} - t_{TEAHL} - d_{TEAVL} - t_{BEAHL} - d_{BEAVL}$ = 177 8 mm ✓

Clear Distance Between Flange Angles d_1 = 7 in ✓

d_1 / t = 18 667 ✓

~~T / t < 2 and d_1 / t < 85 therefore C_s can be increased by 20%~~

~~$C_s = C_s \times 1\ 2$ = 13 131 ton/in^2~~

CALCULATION SHEET

Project Title. **VAR9-830 BE4 Assessments**			
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BS 153 Pt 3B Table 3	Allowable Stress p_{bc} =	9 5 ton/in² ✓	Table 3 Parts in bending (ii), $\frac{d_1}{t}$ < 85 For BS 15 = p_{bc} = 9.5 ton/in² ✓
	Allowable Stress p_{bc} =	146763 483 kN/m² ✓	
BE4 Part 1-304(a)	Apply 25% enhancement for steel		
BS153 Pt 3B Table 1	$p_{bc} = p_{bc} \times 1\ 25$ =	183454 354 kN/m² ✓	
	Girder Depth Midspan d =	406 4 mm ✓	
	$y_{bot} = y_{bar}$ =	218 822 mm ✓	
	I_{xx} =	476063577 mm⁴ ✓	
	$Z_{bot} = I_{xx} / y_{top}$ =	2175571 mm³ ✓	
	Z_{bot} =	0 002 m³	
	Bending Capacity $M_{cu} = p_{bc} \times Z_{top}$ =	**399.118 KNm** ✓	> 283.7 KNm ok
	Bending Capacity M_c =	**131.377 tonft**	
	$M_c > M_t$		
	Internal girder is OK in bending. ✓		

CALCULATION SHEET

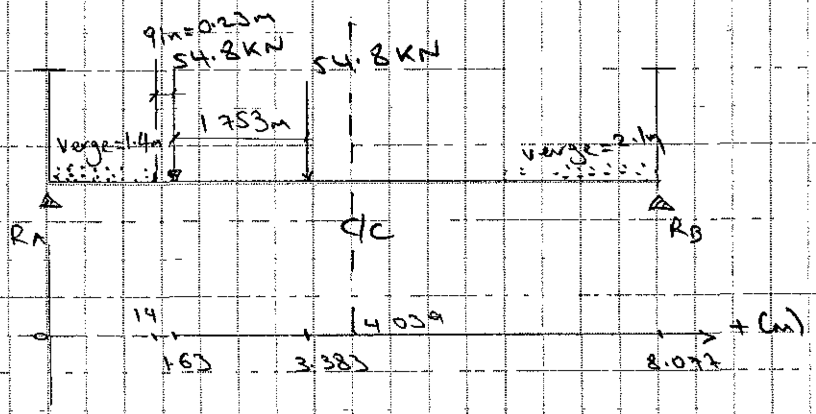
JE JACOBS

Project Title BE4 assessment			Sheet No 54
Subject. AGB 3			Calc No
Job No J20308b			File
Made By [redacted]	Date Aug 04	Revised By	Date
Checked By [redacted]	Date Aug 04	Checked By	Date

Cross girder

LL assessment - Shear force

Similarly to the assessment for BM at mid span, we will use 11 ton axle load. Hence, $W = 5.5\ \text{ton} = 54.8\ \text{kN/axle}$



$M_A = 0$ ⇒ $R_B = \frac{54.8 \times 1.63 + 54.8 \times 3.383}{8.077}$ ⇒

$R_B = 34.0$ kN

Total load = $2 \times 54.8 = 109.6$ kN

$R_A = 109.6 - 34.0 = 75.6$ kN

$SF_{LL} = 75.6$ kN

CALCULATION SHEET

Project Title. **VAR9-830 BE4 Assessments**		
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Made by. [redacted]ate. **19/07/2004**	Checked by.	Date.
Revised b[redacted]ate Aug 04	Checked by.	Date

Shear Capacity

	Total Shear Force due to Dead and Live loads SF_t = SF_d + SF_l = 56.9 ⟶ 75.6 kN	~~133.024~~ 132.5 kN	
	Web Area A_{web} =	3629 025 mm^2	3/8" x 15" ✓
	Web Area A_{web} =	0 004 m^2	
	Reduce web area to take account of holes Maximum hole depth is found on Internal Transverse Girder 8 which does not have a full span However the reduction in the web area should not adversly affect the capacity of the full span girders		
	External Web Thickness t_{web} =	9 525 mm	✓
	External Web Thickness t_{web} =	0 01 m	
	Maximum Hole Depth d_{hole} =	30 mm	– (AGB3 Report)
	Maximum Hole Depth d_{hole} =	0 03 m	
	$A_{web} = A_{web} - (t_{web} \times d_{hole})$ =	0 003 m^2	— 3343.3 mm^2
BS153 Pt 3B Table 3	Permissable Shear p_s =	5 5 ton/in^2	✓
	Permissable Shear p_s =	84968 332 kN/m^2	
BE4 Part 1-304(a)	Apply 25% enhancement for steel		
BS153 Pt 3B Table 1	$p_s = p_s \times 1\ 25$ =	106210 416 kN/m^2	✓
	Shear Capacity $SF_c = A_{web} \times p_s$ =	**355 KN**	✓ > 132.5 kN ok
	Shear Capacity SF_c =	**36 tons**	✓

$SF_c > SF_t$

Internal girder is OK in shear.

Internal Girder Assessment Result: **PASS**

CALCULATION SHEET

Project Title. **VAR9-830 BE4 Assessments**		
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Assessment of Girder / Buckle Plate Connection

Dead Loads

	Carriageway Self Weight W_C =	1 379 kN/m
	Fill Self Weight W_F =	8 949 kN/m
	Buckle Plate Self Weight BP_{sw} =	0 894 kN/m
	Total Dead Load $W_d = W_C + W_F + W_{BP}$ =	**11.222 kN/m** ✓

Buckle plate is 4'-4" wide and can not fit a ton axles 2No according to BE4 sect 202 ie use single axle so 11 ton

Live Loads

	Internal Transverse Girder Spacing S_I =	1 219 m
	Distributed Dead Load $load_d = W_d / S_I$ =	9 204 kN/m^2
	The worst case in when one of the wheels of the vehicle is in the middle of the plate.	
	Axle Weight W_{axle} =	11 tons ✓
	Axle Weight W_{axle} =	109.637 kN
	Wheel Weight $W_w = W_{axle} / 2$ =	54 819 kN ✓
	Wheel Load $load_w$ =	33 in^2/ton ✓
	Wheel Load $load_w$ =	0 002 m^2/kN
	Contact Area $A_{con} = W_w$ x $load_w$ =	0 117 m^2 ✓
BE4 Part 1-302(e)	$A_{con} = 1\ 4 \times b_{con}^2$	✓
	Therefore $b_{con}^2 = A_{con} / 1\ 4$	0 084 m^2
	b_{con} =	0 289 m ✓
	$l_{con} = A_{con} / b_{con}$ =	0 405 m ✓
	Carriageway Depth d_C =	0 05 m
	Carriageway Depth d_F =	0 346 m
	Depth of Load Distribution $d_{dist} = d_C + d_F$ =	0 396 m ✓
	Distribution of loads at 45° in transverse plane	
	$b_{dist} = b_{con} + (2 \times d_{dist})$ =	1 081 m ✓

CALCULATION SHEET

Project Title. **VAR9-830 BE4 Assessments**			
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Made by. [redacted]	Date. **19/07/2004**	Checked by.	Date.
Revised b[redacted]	Date. Aug 04	Checked by.	Date

Distribution of loads at 45° in longitudinal plane

$l_{dist} = l_{con} + (2 \times d_{dist}) =$ 1 197 m ✓

Distributed Live Load $load_l = W_w / (b_{dist} \times l_{dist}) =$ 42 361 kN/m^2 ✓

Total Distributed Load $load_t = load_d + load_l =$ 51 565 kN/m^2 ✓

T-Section Flange Width $w_{Tflange} =$ 0 152 m

Buckle Plate Span $L_{BP} = Sl - w_{Tflange} =$ 1 067 m ✓

Buckle Plate Rise $r_{BP} =$ 72 mm ✓ (Site notes)

Buckle Plate Rise $r_{BP} =$ 0 072 m

To calculate the thrust, the total applied load will be taken as it occupies the full span of the plate

Thrust $T = (load_t \times L_{BP}^2) / (8 \times r_{BP}) =$ 101.883 kN/m ✓

Thrust T = 0.26 ton/in ✓

Rivet Spacing $R_s =$ 4 in ✓

Rivet Spacing $R_s =$ 0 102 m

Number of Rivets on Plate $n_P = S_l / R_s =$ 12. ✓

Number of Rivets on Plate $n_P =$ 12.

→ Ref RT/CE/C/025
Appendix F
cl 5.7.5.3.1
For span of 1.2m or less

CALCULATION SHEET

Project Title. **VAR9-830 BE4 Assessments**			
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BS153 Pt 3B Table 3	Permissable Shear p_s =	5 5 ton/in^2 ✓	(Hand-driven rivets assumed)
	Permissable Shear p_s =	84968.332 kN/m^2 ✓	
BE4 Part 1-304(a)	Apply 25% enhancement for steel		
BS153 Pt 3B Table 1	$p_s = p_s \times 1\ 25$ =	106210 416 kN/m^2 ✓	
	Rivet Diameter dia_R =	0.75 in ✓	
	Rivet Diameter dia_R = ✓	0 019 m ✓	
	Rivet Area $A_R = (\pi \times dia_R^2) / 4$ =	0 000285 m^2 ✓	
	Allowable Load per Rivet $p_R = p_s \times A_R$ =	30 273 kN ✓	
	Required Number of Rivets per m $n_{reqdm} = T / p_R$ =	3 366 ✓	
	Number of Rivets Required $n_{reqd} = S_l \times n_{reqdm}$ =	**4.103** ✓	
	Number of Rivets Required n_{reqd} =	**5.**	
	$n_{reqd} < n_p$		
	Girder / Buckle Plate Connection Assessment Result:	**PASS** ✓	
	BE4 Assessment Result:	**PASS** ✓	

Allowable shear stress = 5.5 × 1.25 = 6.875 $\frac{ton}{in^2}$ ≡ 106.2 N/mm²

Actual " " (by interpolation) = $\frac{4.103}{12}$ × 106.2 = 36.3 N/mm²

CALCULATION SHEET

JACOBS

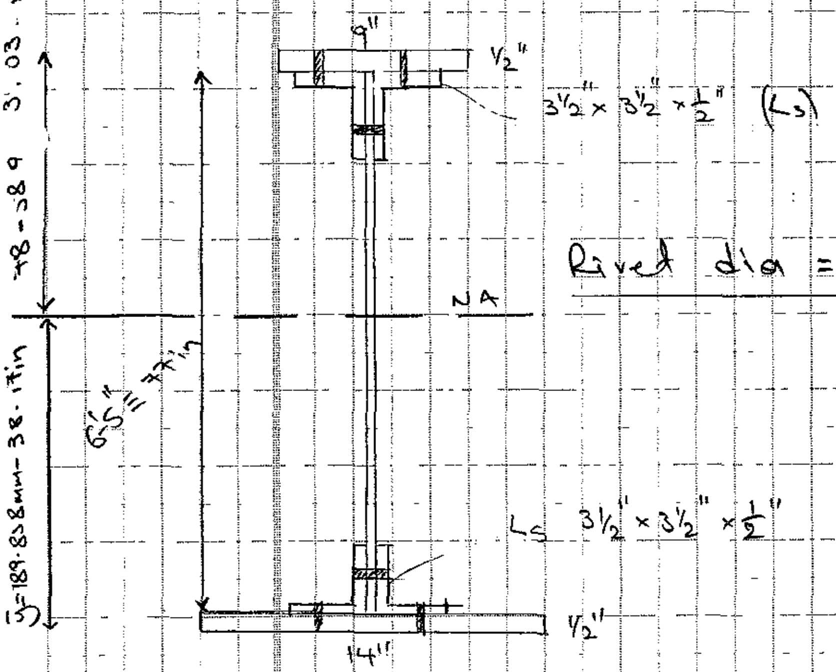
Project Title. BE4 assessment		Sheet No	59
Subject AGB 3	Rivets shear capacity MAIN GIRDER	Calc No	
Job No J20308b		File	
Made By [redacted]	Date Aug 04	Revised By	Date
Checked By	Date. Aug 04	Checked By	Date

Rivet shear capacity

MAIN GIRDER

→ see section properties at main girder assessment

web/angle rivets



Rivet dia = 3/4"

Attaching area = (Flange + 2 angles) = 9" × ½" = 4.5 in²

+ 3½" × ½" × 2 = 3.5 in²

+ 3" × ½" × 2 = 3.0 in²

Area$_{TOT}$ = 11.0 in²

$$y' = \frac{\Sigma A \times y\text{-}y'}{\Sigma A_{TOT}} =$$

$$= \frac{\left(9\times\frac{1}{2}\right)\times\left(39.03-\frac{0.5}{2}\right)+\left(3\tfrac{1}{2}\times\frac{1}{2}\times2\right)\times\left(39.03-0.5-\frac{0.5}{2}\right)+\left(3\times\frac{1}{2}\times2\right)\times\left(39.0-\tfrac{1}{2}-\frac{1}{2}-\frac{3}{2}\right)}{11}$$

$$= \frac{174.51+133.98+109.59}{11} = 38\text{ in}$$

Calcsheet1

CALCULATION SHEET

JACOBS

Project Title BE4 assessment		Sheet No 60	
Subject AGB 3		Calc No	
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Made By [redacted]	Date Aug 04	Revised By	Date
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$I_{xx} = \frac{1922030480 \text{ mm}^4}{25.4^4} = 46177 \text{ in}^4$ ✓

Total Shear = Shear due to DL + LL (main girder)

= 200.2 + 126.1 = 326.3 KN

(≡ 32.74 tons)

Long. shear = $\frac{\text{Shear} \times \text{Area} \times y}{I}$ ⇒

Long. shear = $\frac{32.74 \times 11 \times 38}{46177}$ = 0.3 ton/in

Rivet pitch = 4" No. of rivets = 1

No of shear planes/rivet = 2

$0.3 \times 4''/2 = 0.6$ tons/rivet plane

Rivet capacity = permissible shear × Area

BS 153 table 3 (Ref buckle plate calcs)

$= 5.5 (\text{ton}/\text{in}^2) \times \frac{\pi}{4}\left(\frac{3}{4}\right)^2 (\text{in}^2)$

= 5.5 × 0.44 = 2.42 tons

Hence, capacity of rivet = 2.42 tons > 0.6 tons/rivet

OK

CALCULATION SHEET

JACOBS

Project Title BE4 assessment		Sheet No 61	
Subject AGB 3		Calc No	
Job No J20308b		File	
Made By [redacted]	Date Aug 04	Revised By	Date
Checked By	Date Aug 04	Checked By	Date

Flange/angle rivets

p.1 Attaching area ≡ (top flange) = $9'' \times \frac{1}{2}'' = 4.5 \text{ in}^2$

$\bar{y} = \frac{(9 \times \frac{1}{2}) \times (39.03 - \frac{0.5}{2})}{4.5} = 38.78 \text{ in}$

$I_{xx} = 46177 \text{ in}^4$

Total shear = 32.74 tons

Long shear = $\frac{\text{shear} \times A \times \bar{y}}{I} = \frac{32.74 \times 4.5 \times 38.78}{46177} \Rightarrow$

Long. shear = 0.124 ton/rivet

Rivet pitch = 4" No of rivets = 2

No of shear plane/rivet = 1

$0.124 \times \frac{4''}{2} = 0.247$ tons/rivet

Rivet capacity = 2.42 tons/rivet > assessed shear/rivet (= 0.247 tons/rivet)

<u>OK</u> For 24 ton vehicle train

CALCULATION SHEET

JACOBS

Project Title BE4 assessment			Sheet No 62
Subject AGB 3			Calc No
Job No J20308b			File
Made By [redacted]	Date Aug 04	Revised By	Date
Checked By	Date 8/04	Checked By	Date

Summary

Web/angles

Permissible stress ~~Assessed resistance~~ = 5.5 ton/in² = 85 N/mm² (85.153)

p 60 Total load effect = 0.6 ton/rivet/plane

$= \frac{0.6 \times 9.967}{\text{Area of rivet}} \times 10^3 = 21.0$ N/mm²

Area of rivet = 0.44 in² = $0.44 \times \frac{1}{25.4^2}$ (mm²)

Dead load effect $= \frac{21.1 \times 200.2}{326.3}$

= 12.9 N/mm²

Live load effect = 21.1 − 12.9 = 8.1 N/mm²

CALCULATION SHEET

JE JACOBS

Project Title BE4 assessment		Sheet No 63	
Subject AGB 3		Calc No	
Job No J20308b		File	
Made By [redacted]	Date Aug 04	Revised By	Date
Checked By	Date 8/04	Checked By	Date

Summary

flange angle

(BS 153)

Permissible stress ~~Assessed resistance~~ $= 5.5 \frac{ton}{in^2} = 85\ N/mm^2$

p. 61 Total load effect = 0.247 ton/rivet plane

$= \frac{0.247 \times 9.967 \times 10^3}{\text{Area of rivet}}$

Total load effect $= \frac{0.247 \times 9.967 \times 10^3}{0.44} \times \frac{1}{25.4^2} = 8.7\ \frac{N}{mm^2}$

Dead load effect $= \frac{8.7 \times 2002}{3263} = 5.3\ N/mm^2$

Live load effect $= 8.7 - 5.3 = 3.4\ N/mm^2$

CALCULATION COVER SHEET

JacobsGIBB Ltd.
Reading

Project Title:	BRB (Residuary) Ltd - BE4 assessments - 2004	Calc. No.:	31.2
Job No:	J20308A	File:	R6
Project Manager	[redacted]	Subject:	AGB3
Designer			Addendum to calc. 31.1
Project Group	31400		Buckle plate checks

	Total Sheets	Made by	Date	Checked by	Date	Reviewed by	Date		
Original	4	JLR	Apr-05	[redacted]	May 05				
Rev									
Rev									
Rev									
Rev									
Rev									

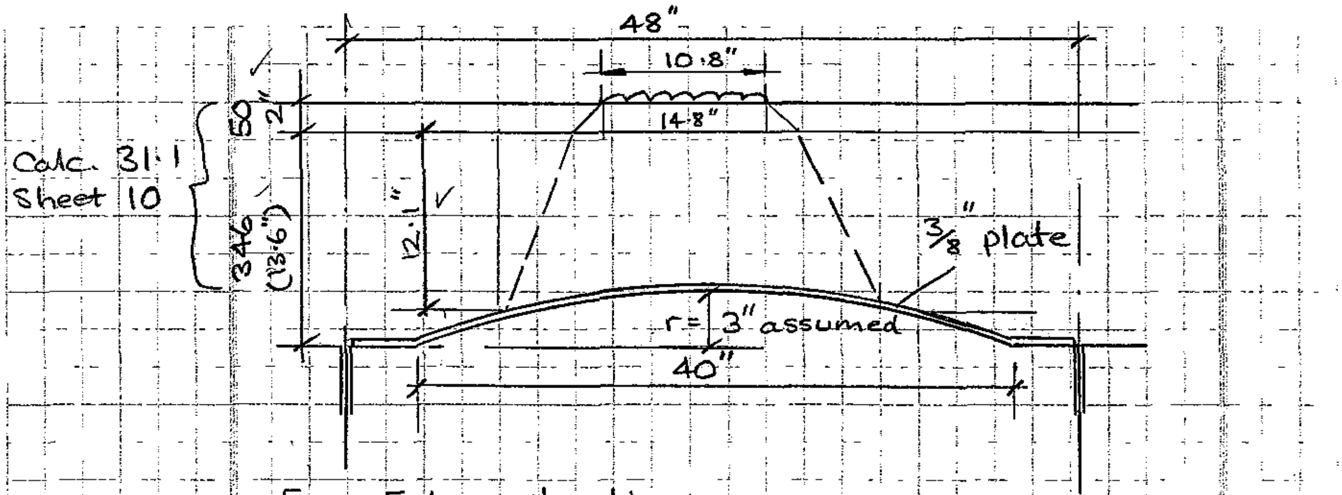
Superseded by Calculation No. Date

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

JACOBS

CALCULATION SHEET

Project Title	BE4 ASSESSMENTS 2004 - VAR9/830	Sheet No	A 1
Subject	AGB3 - BUCKLE PLATE CHECK	Calc No	31.2
Job No	J20308B	File	
Made by	[redacted]	Date 27/4/05	Revised	Date
Checked by	[redacted]	Date May 05	Checked by	Date



For 5 ton wheel:

footprint area = 5 × 33 = 165 in^2

$1.4b^2 = 165$, $b = \sqrt{165/1.4} = 10.8"$ ($1.4b = 15.12$ in)

Take 1:1 dispersal through surface, 1:2 through fill to mid-height of plate:

Dispersed length = 10.8 + 4 + 12.1 = 26.9 in

= 15.12 + 4 + 12.1 = 31.2 in

Intensity of wheel pressure (+ impact)

$$= \frac{5 \times 1.25}{26.9 \times 31.2} = 0.00745 \text{ ton/in}^2$$

$$= 16.68 \text{ lbs/in}^2$$

Dead load

$$= \frac{14.1 \times 135}{12} = 158.6 \text{ lbs/ft}^2 = 1.10 \text{ lbs/in}^2$$

Total = 17.78 lbs/in^2

Take as uniform load intensity over plate:

$$\text{Thrust} = \frac{wL^2}{8r} = \frac{17.78 \times 40^2}{8 \times 3} = 1185 \text{ lbs/in}$$

JACOBS

CALCULATION SHEET

Project Title	BE4 - VAR9/830		Sheet No	A 2
Subject	AGB3 -		Calc No	31·2
Job No	J20308B		File	
Made by	[redacted]	Date 27/4/05	Revised	Date
Checked by	[redacted]	Date May 05	Checked by	Date

BA56. method.

Take plate to be acting as a strut

Take l_e as distance from end of span to the intersection point of the wheel distribution:

$$l_e = 40'' - 26.9'' = 13.1''$$

Radius of gyration for plate:

$$I = \frac{1 \times 0.375^3}{12} = 0.0044 \text{ in}^4 \checkmark$$

$$A = 0.375 \text{ in}^4$$

$$r = \sqrt{\frac{0.0044}{0.375}} = 0.108 \text{ in} \checkmark$$

Slenderness ratio

$$l/r = \frac{13.1}{0.108} = 121. \checkmark$$

BS153: Pt 3B Table 4

$$P_{ac} = 3.9 \text{ ton/in}^2$$

Rest of bridge is steel, so steel plates assumed

Apply Case II enhancement:

$$P_{ac} = 3.9 \times 1.25 = 4.87 \text{ ton/in}^2. \checkmark$$

Strut capacity $= P_{ac} \times A$

$$= (4.87 \times 2240) \times 0.375 = 4090 \text{ lbs/in} \checkmark$$

$4090 > 1185$ ✓

∴ Plate is satisfactory for max. C&U wheel loading. ✓

JACOBS

CALCULATION SHEET

Project Title			Sheet No A 3
Subject AGB3			Calc No 31·2
Job No J20308B			File
Made by. [redacted]	Date. 27/4/05	Revised.	Date.
Checked by [redacted]	Date May 05	Checked by.	Date

Check connecting rivets:

Thrust from plate = 1185 lbs/in

Rivet spacing = 4"

Shear on rivet = 1185 × 4 = 4740 lbs = 2.12 tons ✓

Rivet diameter = $\frac{3}{4}$"

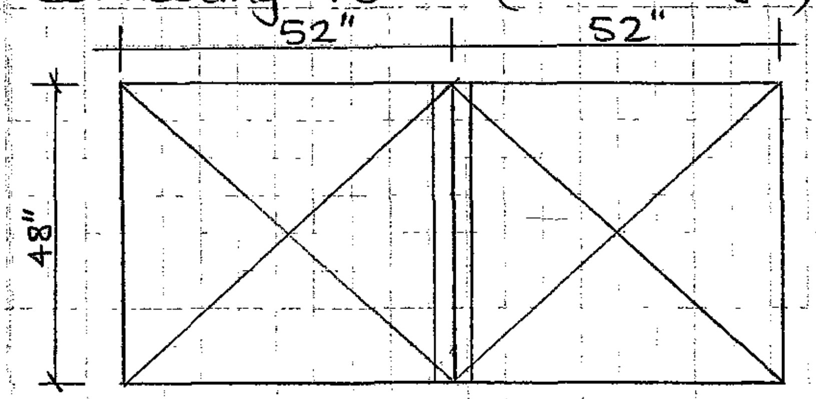
Assume hand driven field rivets

BS 153: 3B Table 3 — Permissible stress = 5·5 ton/in² × 1·25 (case II stress) ✓

Rivet capacity (single shear) = $\frac{\pi \times 0.75^2}{4} \times 5.5 \times 1.25$ = 3·04 tons ✓

Connecting rivets satisfactory.

Connecting Ts. (6 × 3 × $\frac{3}{8}$")



Applied dead load = 1·10 lbs/in² × ½ × 48 × 52 (ignore self weight, too small) ✓
= 1372·8 lbs = 0·613 tons. ✓

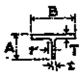
Wheel load = 5 × 1·25 = 6·25 tons ✓

As udl: $M = \frac{(0.613 + 6.25) \times 4}{8}$ = 3·43 ton ft ✓

JACOBS

CALCULATION SHEET

Project Title				Sheet No	A 4
Subject	AGB3			Calc No	31·2
Job No	J20308A			File	
Made by	[redacted]	Date 27/4/05	Revised	[redacted]	Date 15/11/05
Checked by	[redacted]	Date May 05	Checked by	[redacted]	Date Nov 05



STRUCTURAL TEES

Cut from Universal Columns

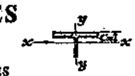
DIMENSIONS AND PROPERTIES

Serial Size	Weight per foot	Width of Section B	Depth of Section A	Thickness		Root Radius r	Slope inside Flange	Area of Section
				Web t	Flange T			
ins	lbs	ins	ins	ins	ins	ins	per cent	ins²
16 ×7	79	15·550	7·500	·730	1·188	·60	0	23·24
14½×7	68	14·740	7·375	·660	1·063	·60	0	19·99
	59·5	14·650	7·250	·570	·938	·60	0	17·49
	51·5	14·575	7·125	·495	·813	·60	0	15·13
	43·5	14·500	7·000	·420	·688	·60	0	12·78
12 ×6	53	12·230	6·440	·620	·986	·60	0	15·59
	46	12·155	6·310	·545	·856	·60	0	13·53
	39·5	12·080	6·190	·470	·736	·60	0	11·61
	32·5	12·000	6·060	·390	·606	·60	0	9·55
10 ×5	44·5	10·275	5·440	·615	·998	·50	0	13·09
	36	10·170	5·250	·510	·808	·50	0	10·59
	30	10·075	5·125	·415	·683	·50	0	8·83
	24·5	10·000	5·000	·340	·558	·50	0	7·20
8 ×4	29	8·222	4·375	·510	·808	·40	0	8·53
	24	8·117	4·250	·405	·683	·40	0	7·06
	20	8·077	4·125	·365	·558	·40	0	5·88
	17·5	8·027	4·060	·315	·493	·40	0	5·15
	15·5	8·000	4·000	·288	·433	·40	0	4·56
6 ×3	12·5	6·080	3·185	·320	·454	·30	0	3·67
	10	6·018	3·100	·258	·369	·30	0	2·96
	7·85	6·000	3·000	·240	·269	·30	0	2·31

3/16" table 1

STRUCTURAL TEES

Cut from Universal Columns



DIMENSIONS AND PROPERTIES

Gravity Centre Distance Cx	Moment of Inertia		Radius of Gyration		Elastic Modulus		Nominal Size	Cut from Universal Column
	Axis x-x	Axis y-y	Axis x-x	Axis y-y	Axis x-x	Axis y-y		
ins	ins⁴	ins⁴	ins	ins	ins³	ins³	ins.	ins×ins×lbs
1·34	69·3	372·5	1·73	4·00	11·3	47·9	15 9/16×7 1/2	14×16 ×158
1·31	60·1	283·9	1·73	3·77	9·89	38·5	14 3/4 ×7 3/8	14×14½×136
1·22	50·4	245·9	1·70	3·75	8·36	33·6	14 5/8 ×7 1/4	119
1·15	42·4	209·9	1·47	3·72	7·10	28·8	14 9/16×7 1/8	103
1·08	34·9	174·8	1·45	3·70	5·88	24·1	14½ ×7	87
1·20	36·7	150·4	1·53	3·11	7·01	24·6	12 1/4 ×6 7/16	12×12 ×106
1·13	31·0	128·2	1·51	3·08	5·99	21·1	12 1/8 ×6 5/16	92
1·06	25·8	108·2	1·49	3·05	5·03	17·9	12 1/16×6 3/16	79
·98	20·6	87·3	1·47	3·02	4·06	14·6	12 ×6 1/16	65
1·07	21·3	90·3	1·28	2·63	4·88	17·6	10 1/4 ×5 7/16	10×10 × 89
·97	16·4	70·9	1·24	2·59	3·83	13·9	10 3/16×5 1/4	72
·88	12·8	58·2	1·21	2·57	3·03	11·6	10 1/16×5 1/8	60
·81	9·99	46·5	1·18	2·54	2·38	9·30	10 ×5	49
·87	9·12	37·5	1·03	2·10	2·61	9·12	8 1/4 ×4 3/8	8× 8 × 58
·78	6·92	30·5	·99	2·08	2·00	7·51	8 1/8 ×4 1/4	48
·74	5·80	24·5	·99	2·04	1·71	6·07	8 1/16×4 1/8	40
·69	4·88	21·3	·97	2·03	1·45	5·30	8 ×4 1/16	35
·67	4·32	18·5	·97	2·01	1·30	4·62	8 ×4	31
·61	2·27	8·51	·79	1·52	·88	2·80	6 1/16×3 3/16	6× 6 × 25
·56	1·75	6·71	·77	1·50	·69	2·23	6 ×3 1/8	20
·56	1·47	4·85	·80	1·45	·60	1·62	6 ×3	15·7

For steel Tee (table 3: 10 + 1·25 = 12·5; table 1)

$M = 12{\cdot}5 \times 0{\cdot}69 / 12 = 0{\cdot}718$ ton.ft ✓

~~Buckle plates have ample capacity for load when considered as spanning one way. Therefore tees can be regarded as connectors and stiffeners, not as load bearing structural members~~

Modifying moment applied in accordance with the recommendations of Jacobs FE analysis of buckle plates (Nov. 05)

Revised moment $= M/6 = 3{\cdot}43/6 = 0{\cdot}572$ ton ft

Applied moment $0{\cdot}572 < 0{\cdot}718$ ton ft capacity

∴ Adequate for single 5T wheel load (C & U loading)

Appendix G - Buckle plate FE analysis

Finite element analysis of buckle plates and tee connections AGB/3 AGB/5 ELW/9 WTD/1

1. Problem Definition

There are a number of bridges with two-way spanning buckle plates for which a satisfactory assessment approach has proved difficult to define The plates in question have very similar characteristics They are typically about four feet (1220mm) square In one direction they are riveted to principal structural members, e g. transverse RSJs or plate girders and in the other direction they are connected to each other with an inverted tee section, usually 6" x 3" x $^3/_8$" Plate rise is usually 3" There is variation with the amount of overburden, but a typical value is 15" to 20" Typical rivet connection is ¾" rivets at 4" pitch

There are simple hand methods of analysis available for one-way spanning plates The method outlined in BA56/96 equates the buckle plate to an arch, the thrust in the arch is evaluated and plate capacity is calculated assuming that it is strut in compression The method defines how the effective length is obtained The method is conservative and makes no allowance for any additional capacity from the plate having a double curvature

Work undertaken by Gifford and incorporated into Bridgeguard 3 CIS No 35 appeared to offer a potential approach, but closer examination of the work revealed that again it only applied to one-way spanning plates and as such only represented a refinement of the BA 56 approach

While accepting that the BA56 approach offers a conservative method for evaluating the plate capacity, because the plates are actually two-way spanning, concerns were raised on the shedding of load to the tee sections A simple distribution of load (quarter triangle from each adjacent plate) can apply up to 50% of the total load being considered onto the tee and under this condition the tee is invariably overstressed This approach is far too conservative In reality, once load starts to be applied to the tee, it deflects and the load distribution between the tee and the main girder support alters This problem was not amenable to hand calculation, therefore a FE approach was suggested It was agreed that panel of three plates should be modelled

A further consideration was the condition where the wheel load is applied directly over the tee section The mechanism of interest here is how the plates attract load away from the tee

It was hoped to obtain a typical distribution of load which could be applied to buckle plate systems with similar parameters

2. Distribution of load from plate to supporting girders and tees

a. Modelling

A three dimensional finite element model representing three in-line buckle plates supported along the long edge by plate girders and transversely by tee sections was created The model was restrained at the four outside corners both vertically and horizontally The plates were allowed rotational freedom at their interface with the supporting girders in consideration of the single line of rivets used in the connections See figure 1

The following load cases were examined:

- A patch load of 0.1155 N/mm^2 over an area of 793 x 683mm was applied in the centre of the centre plate (Figure 2). This represented the distribution through 2" of surfacing and 13.6" of fill from a BE4 5T wheel load plus impact, the most intense loading from the four particular bridges under consideration.

- In order to test the sensitivity of the distribution to the size of the patch load, a single point load was applied at the centre of the plate.

- A uniformly distributed load over all the plates representing the dead load condition. A udl value of 0.0075 N/mm^2 was used, being commensurate with the construction thickness used to derive the live load intensity.

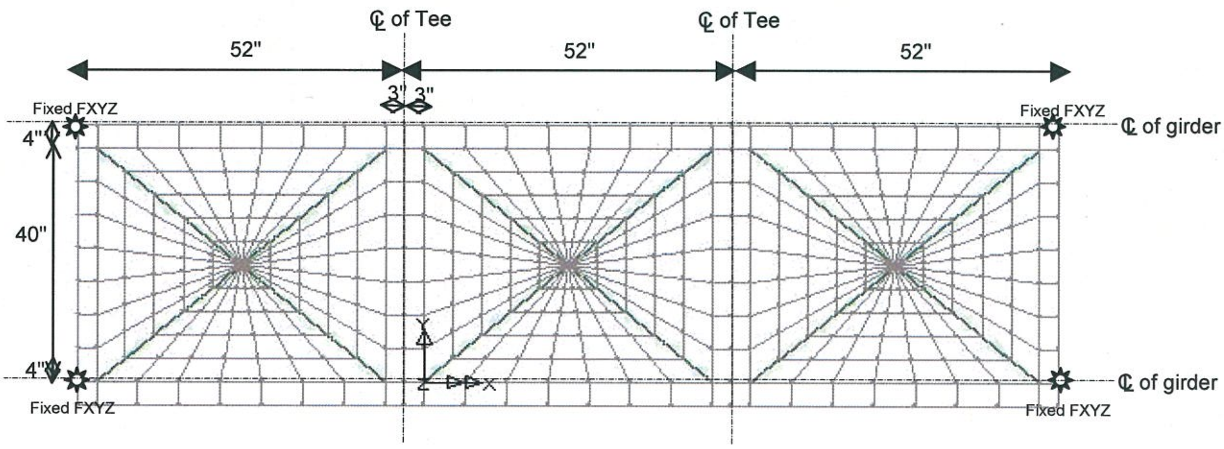


Figure 1 – FE mesh plan

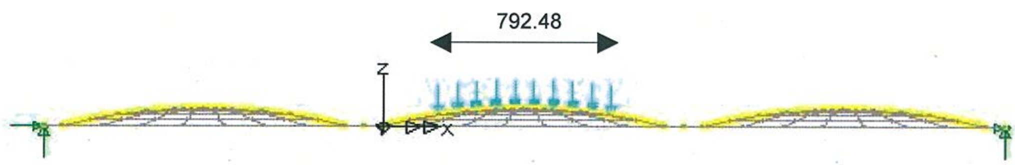


Figure 2 – FE mesh section

The model was analysed using LUSAS finite element software.

b. Results

Patch load (live load)

The thrust generated on the neutral axis of the plate in the Y direction (i.e. towards the supporting girders – Figure 3) is typically (-) 6 N/mm^2, (dark green/cyan colours) The minus sign indicates compression. In comparison, the thrust in the X direction towards the tees (Figure 4) is typically –2 N/mm^2 (yellow / light green), indicating that three times more load is migrating to the main girders than to the tees.

The stress plots show that plates and tees are acting together as a strut and tie system, with tensile stresses being generated in the tee sections (the yellow and orange colours in Figure 3).

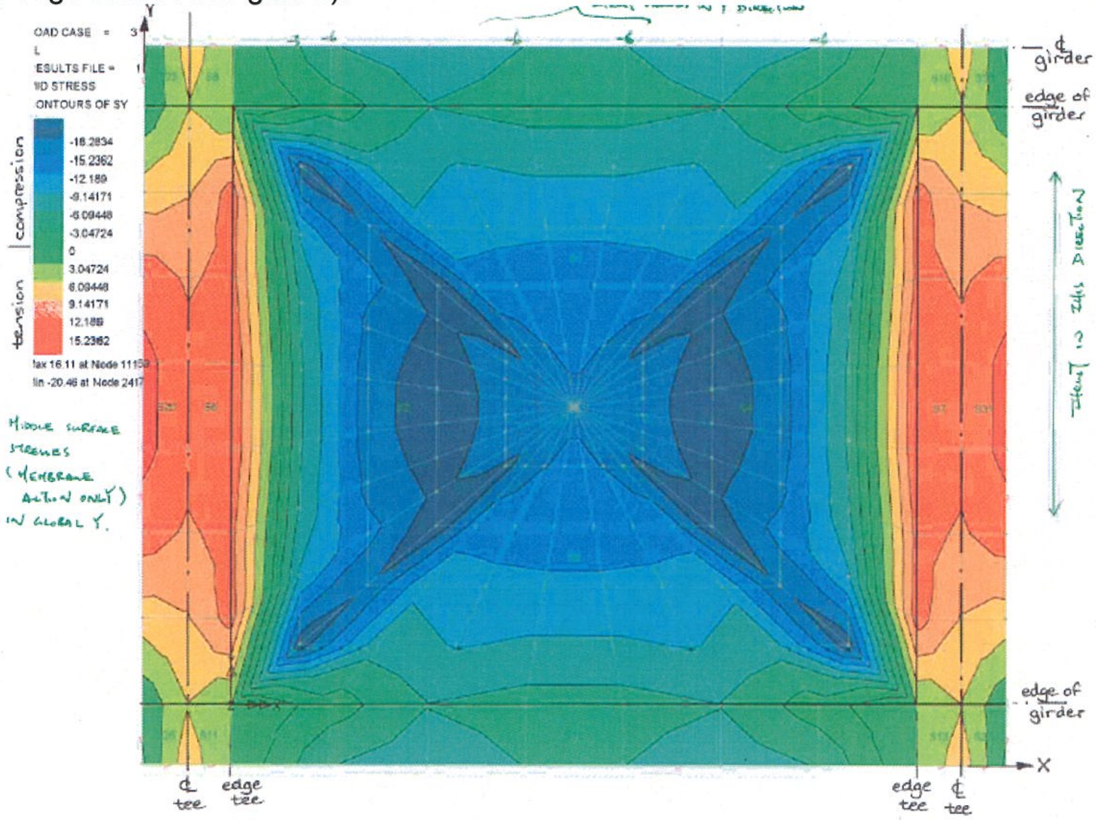


Figure 3 – Thrust in plate in Y-direction (towards girders)

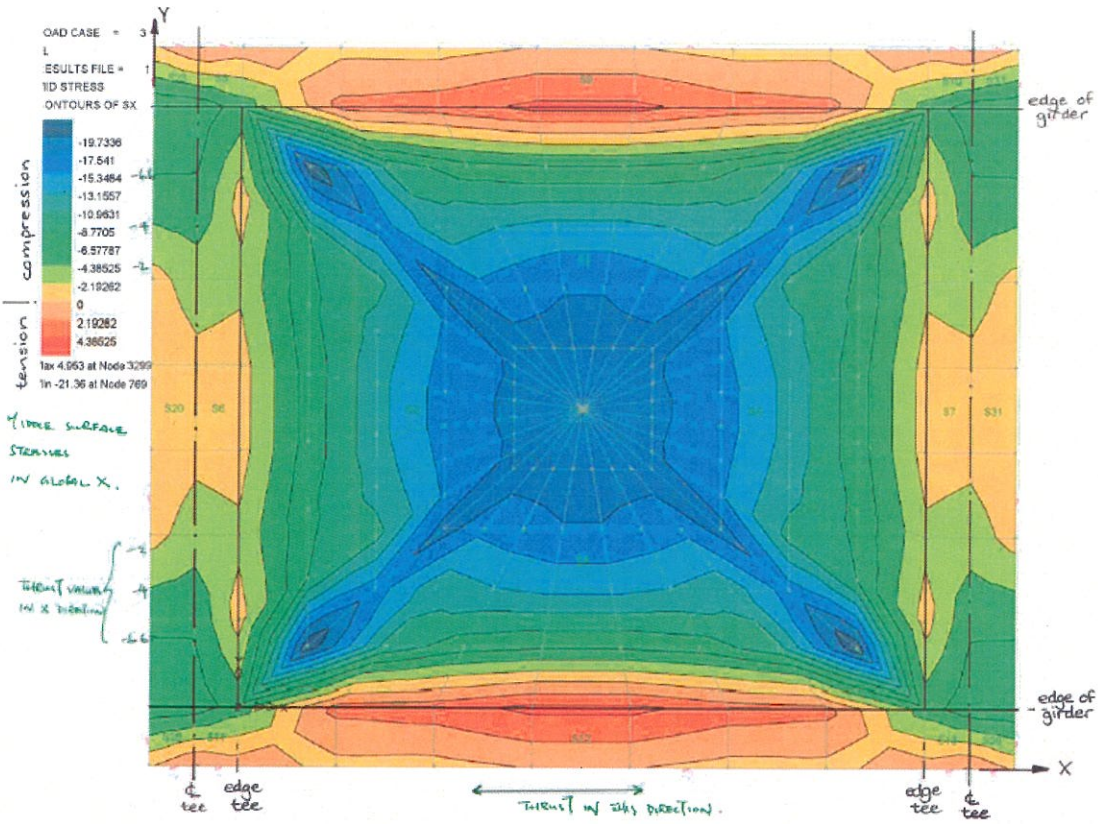


Figure 4 – Thrust in plate in X-direction (towards tees)

The load distribution can be considered in terms of vertical load being applied to the supporting members (shear in the plate). Figures 5 and 6 indicate the Y direction and X direction shear forces respectively. Note, although symmetrical about mid-span, the sign of the shear force changes at mid-span giving the colour inversion.

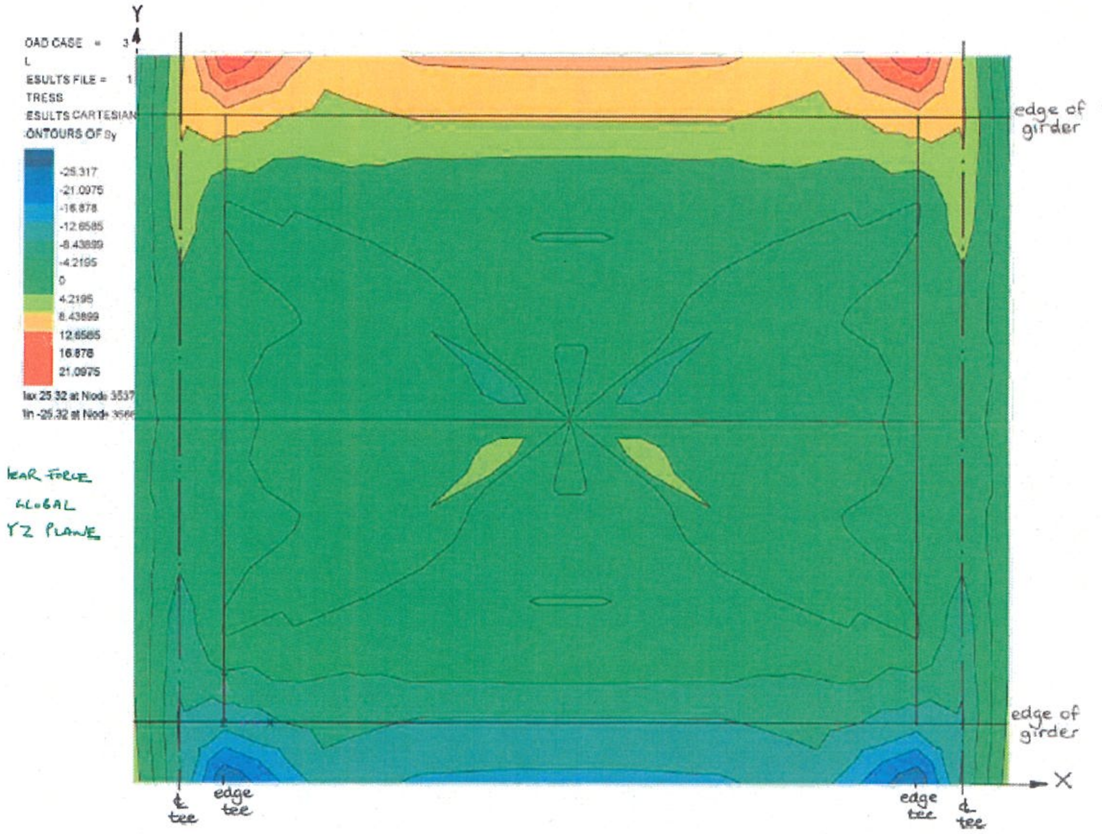


Figure 5 – Patch load - shear force in plate in Y-direction (towards girders)

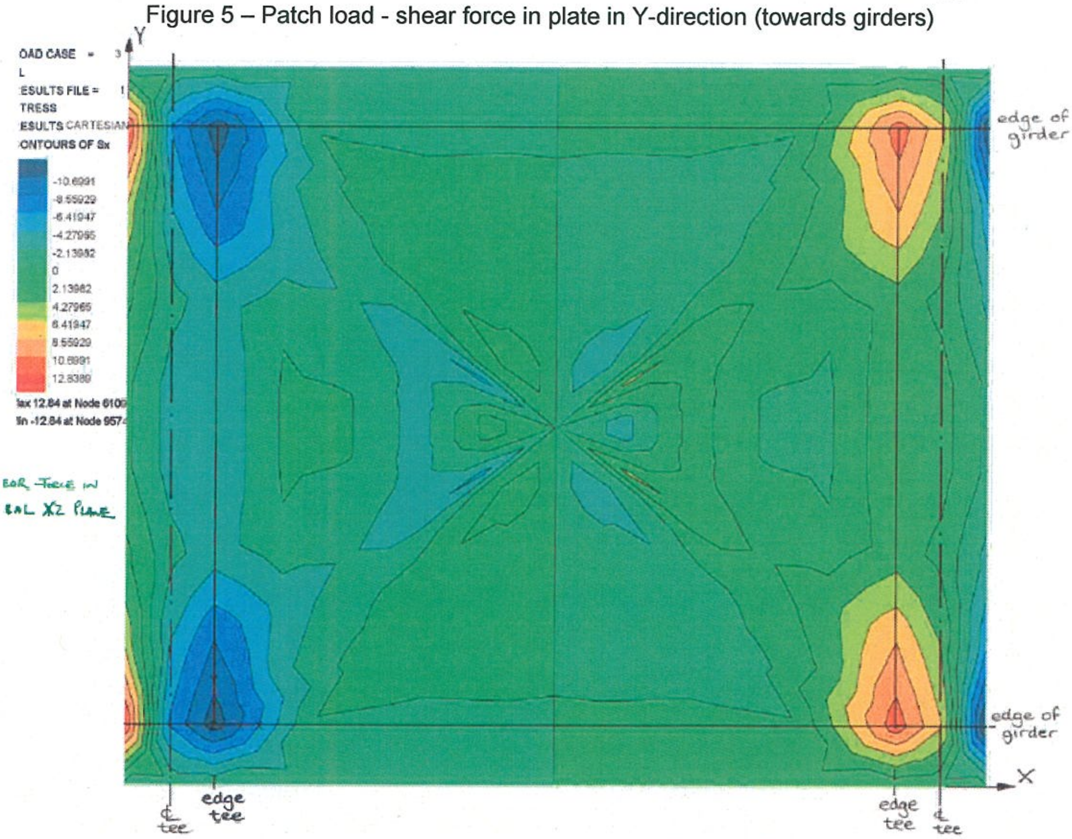


Figure 6 – Patch load - shear force in plate in X-direction (towards tees)

Taking a section through the shear plot adjacent to the supporting members enables the total load applied to each member to be evaluated.

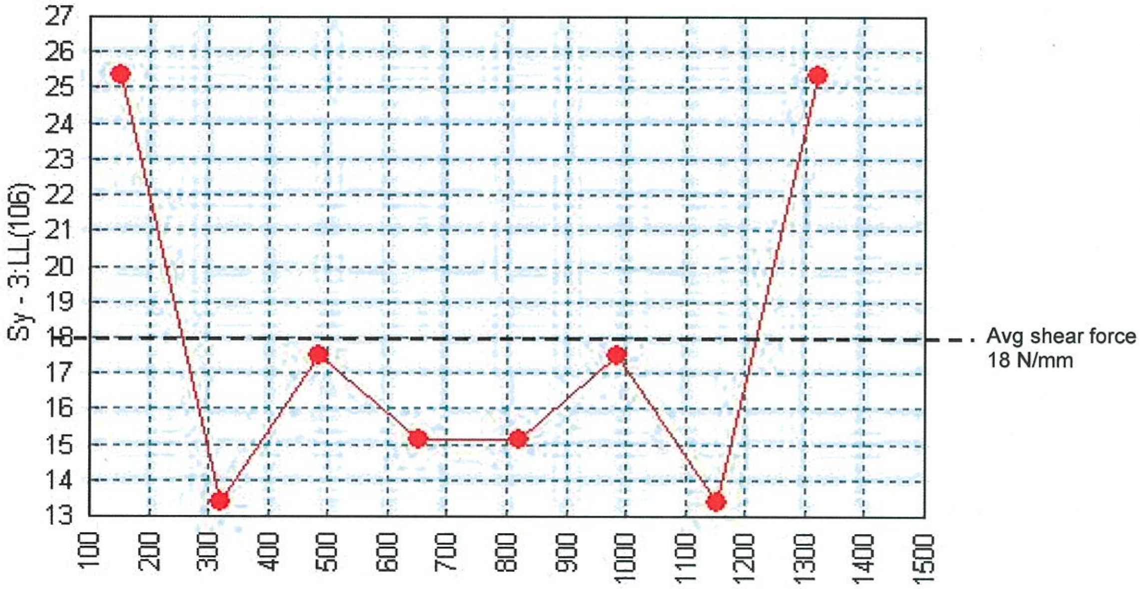


Figure 7: Cross section through shear contour plot at edge of girder (from Fig. 5)

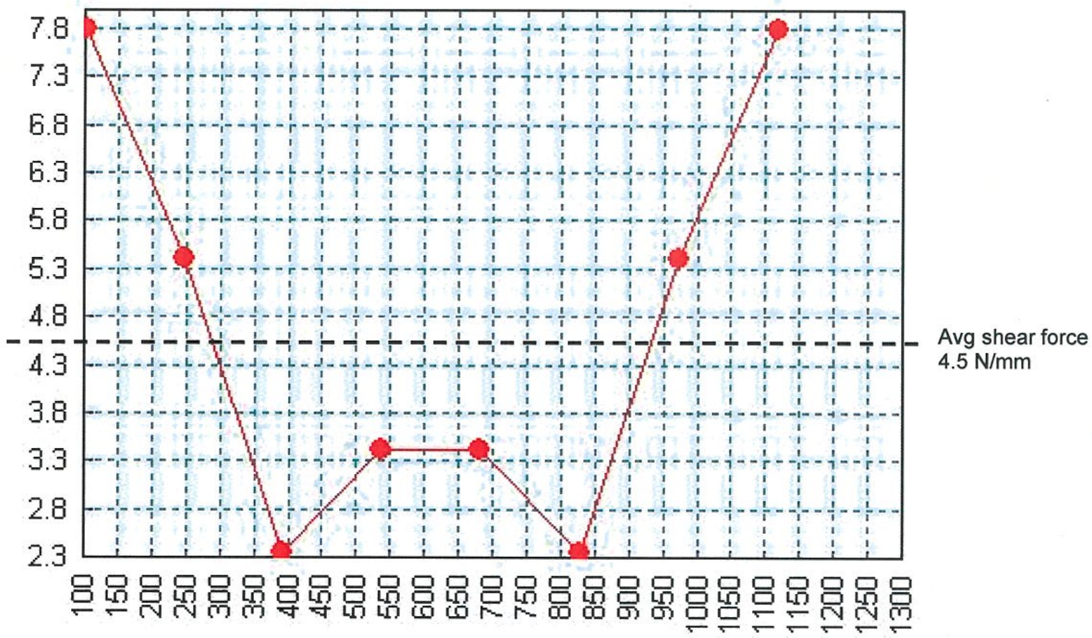


Figure 8: Cross section through shear contour plot at edge of tee (from Fig. 6)

The average load applied from the plate to the girder is 18.0 N/mm. From the plate to the tee, it is 4.5 N/mm, a ratio of 4 : 1.

The moment effect induced in the tee are small because most of the load is applied near the ends of the tee.

Point Load

In order to check the sensitivity of this result to varying patch size, the load was applied as a single point load at the centre of the plate:

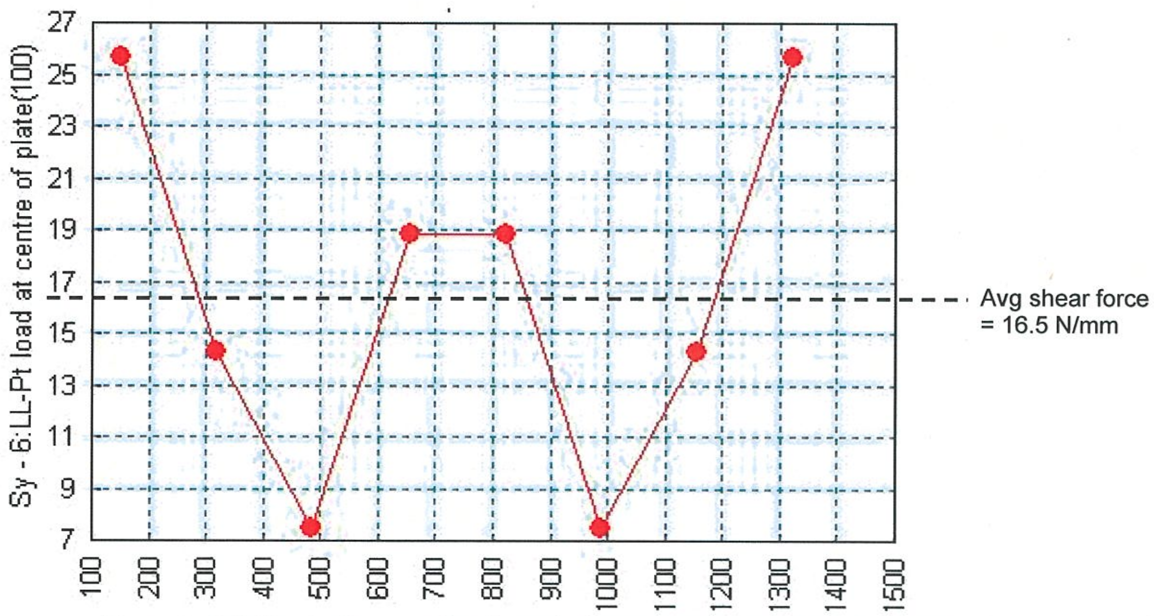


Figure 9: Point load - Cross section through shear contour plot at edge of girder

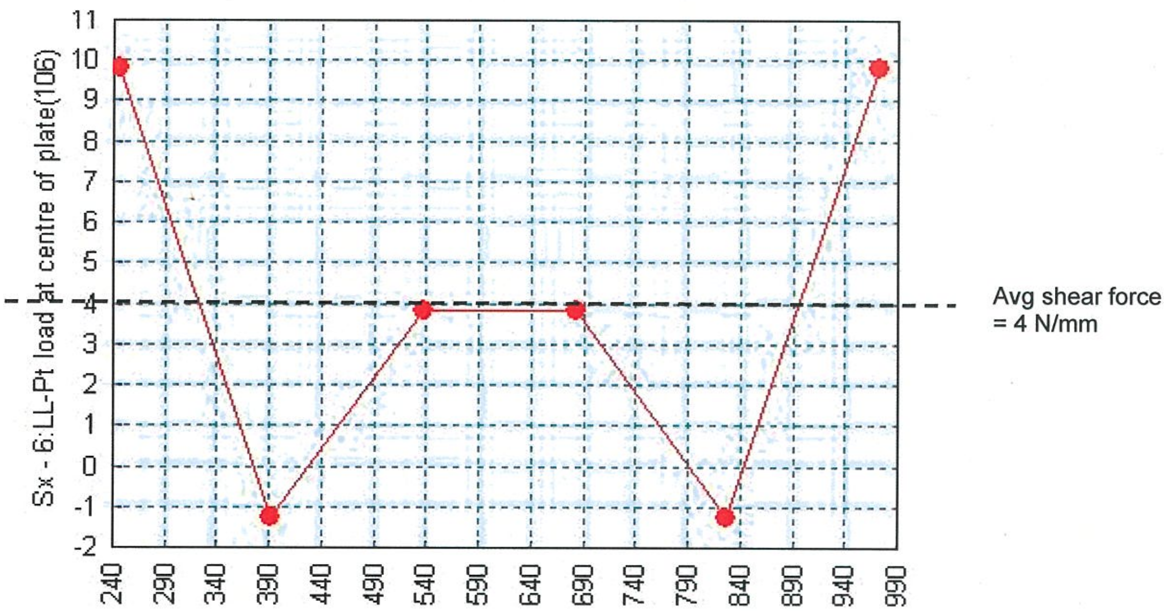


Figure 10: Point load - Cross section through shear contour plot at edge of tee

The proportion of load applied to each supporting member is almost exactly the same as for the patch load, i.e. 4 : 1 girder/tee.

Uniformly distributed load on all plates
A similar exercise was carried out for a uniformly distributed load applied on all three plates, representing the dead load condition:

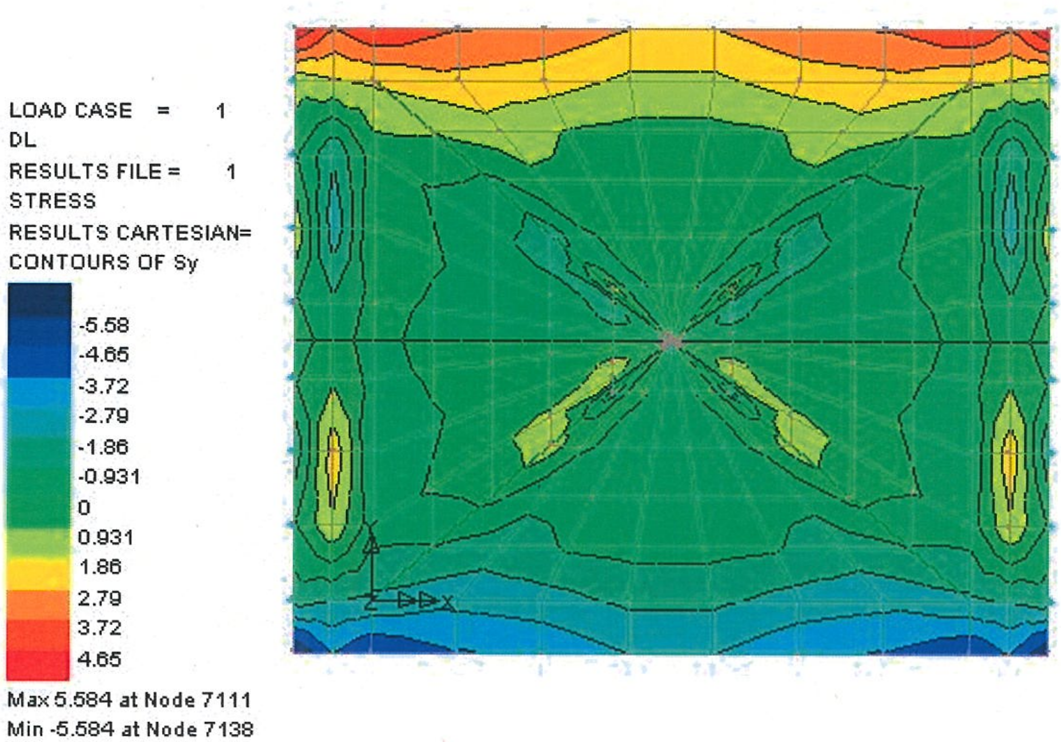


Figure 11 – Uniform load - shear force in plate in Y-direction (towards girders)

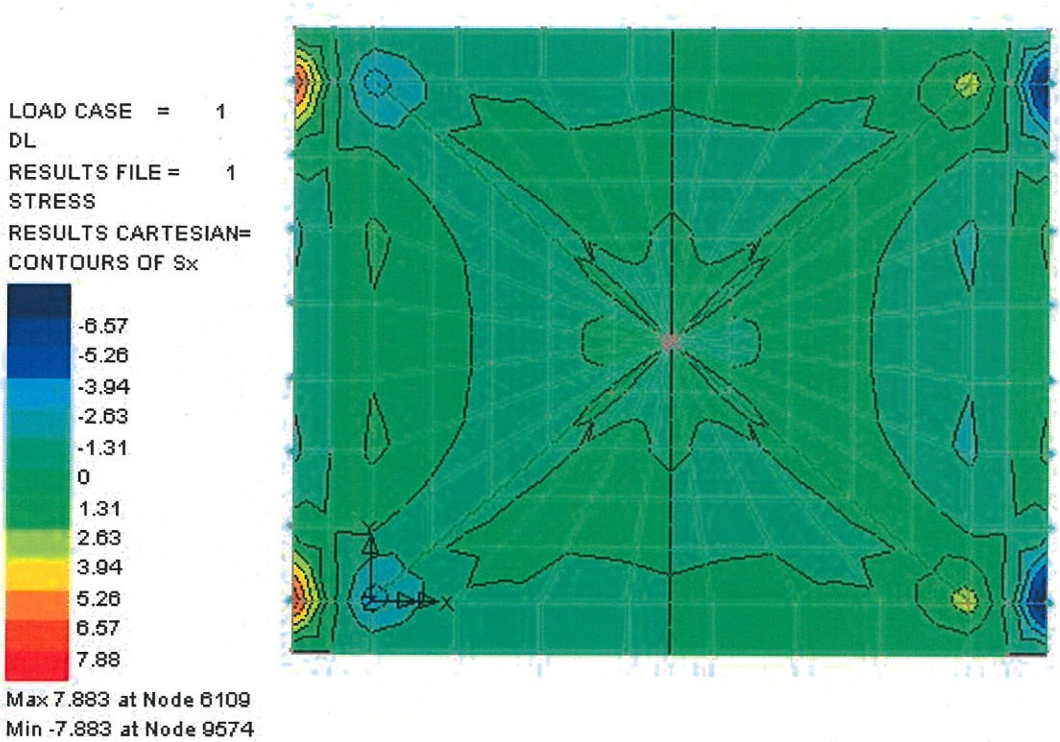


Figure 12 – Uniform load - shear force in plate in X-direction (towards tees)

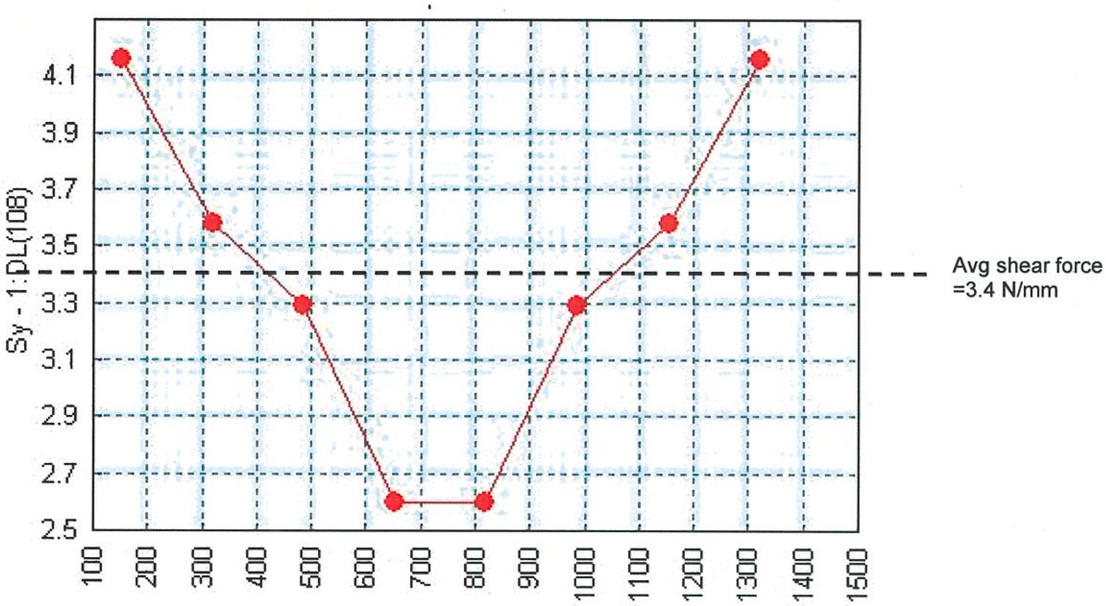


Figure 13: Cross section through shear contour plot at edge of girder (from Fig. 9)

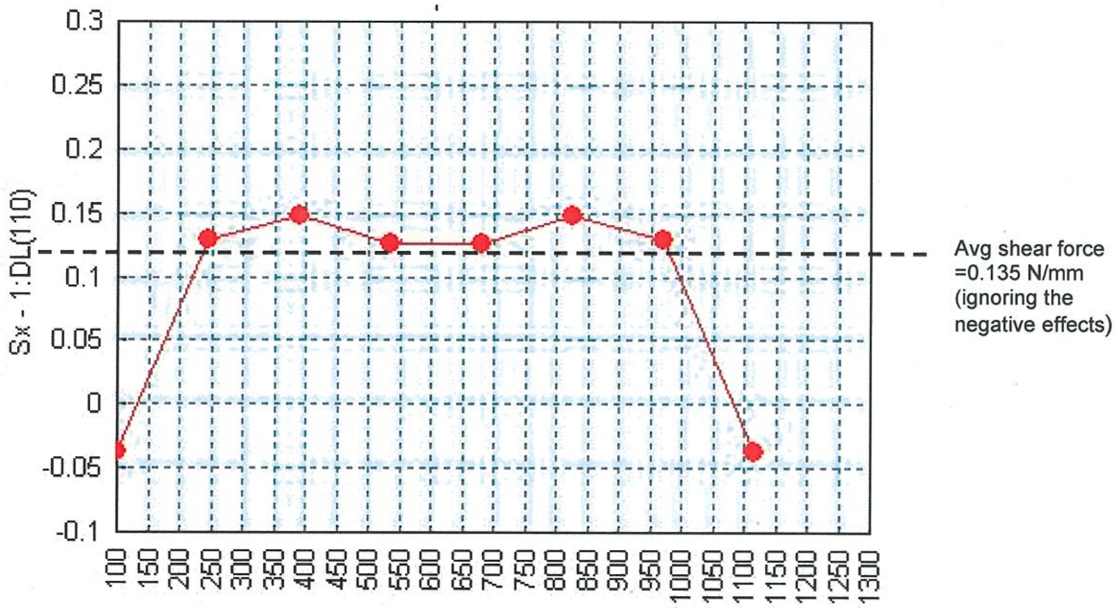


Figure 14: Cross section through shear contour plot at edge of tee (from Fig. 10)

In this case the load is mainly carried to the girders, with only a small proportion (less than 5%) carried through the tees.

3. Distribution of load when wheel load is applied directly over the tee.

a. Modelling

To assess the contribution of the plates when the wheel load is located directly above the tee, two models were set up. The first looked at the framework of girders and tees without the plates being present, the second model reintroduced the plates, so that the load effects could be compared and the contribution of the plates determined.

Two load cases were examined on the first model, a point load, representing the BE4 5T wheel load positioned at mid-span of the tee and the same load applied as a uniformly distributed load along the tee   On the second model the same load cases were examined for comparison purposes and a third load case representing a patch load applied partly on the tee and partly on the plate, as would occur in a real distribution, was also considered (Figure 15)

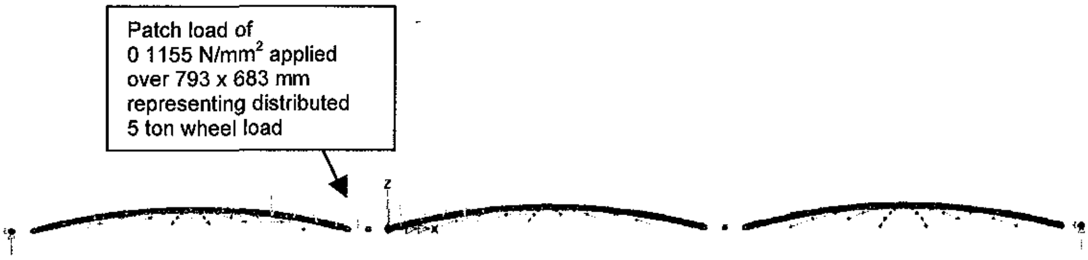


Figure 15 – Application of patch load

b. Results

The following results were obtained from the model

	Tee alone		Tee with buckle plates	
	Mid-span bending	Deflection	Mid-span bending	Deflection
Point load	19 05 kN m	16 7 mm	5 02 kN m	2 8 mm
UDL	9 6 kN m*	10 6 mm	1 41 kN m	1 8 mm
Patch Load	-	-	1 56 kN m	2 1 mm

* 9 5 kN m by hand calculation

It can be seen that the presence of the buckle plates dramatically reduces the load effects in the tee   Transfer of load between the tee and its adjacent plates is dependent on the capacity of the connecting rivets in tension   Permissible load per ¾" rivet in BE4 assessment would be 6 ton/in^2 x 1 25 (case II uplift) x 0 442 in^2 = 3 3 tons/rivet With a typical arrangement of ¾" rivets at 4" centres, this is amply provided for where the wheel load including impact is 6 25 tons

4. Conclusions

In a typical buckle plate system with double curvature plates and connecting tees spanning between larger support girders, where the load is applied to a single plate as a patch (i e. a typical situation under single wheel loading), it is concluded that about four times as much load transfers directly to the larger girders than through a load path via the tees There is little variation to this distribution with the size of the patch

With a uniformly distributed load applied to all plates, more than 95% of the load is applied directly to the main girders

The bending moment in the tees with the wheel load in the most onerous load position is less than one sixth of that calculated by simple statical principles

In practical application, the following rules may be applied to simple calculations

- If a buckle plate spans in two directions, the thrust for a patch load on one plate calculated by the simple method outlined in BA56, using the formula

  $$Thrust = \frac{wL^2}{8r}$$

  may be reduced by 25%

- For dead load effects no change to the simple BA56 formula is required

- The bending moment at mid-span of the connecting tee sections may be taken as the moment calculated on the assumption of a uniformly distributed load applied to the tee, divided by 6

5. Further investigation

Further work could be carried out to refine the approach by investigating

- Establishment of a better algorithm for basic thrust in the plate
- Varying plate thickness
- Effects of varying depths of overburden
- Varying type of tee connectors
- The effects of corrosion and holing in the plates

It may be possible to develop a series of adjustment factors to be applied on a basic formula, similar to the Bridgeguard CIS 35 approach

Application of results to particular bridges

	AGB/3	AGB/5	ELW/9	WTD/1
Plate clear span	40"	40"	42"	42"
Plate rise	3"	3"	3"	3"
Plate thickness	$^3/_8$"	$^3/_8$"	¼"	¼"
Overburden	15 6"	16.5"	19 0"	19 5"
Plate material	Steel	Steel	WI	WI
Connecting tees	6 x 3 x $^3/_8$"	6 x 3 x $^3/_8$"	4 x 4 x $^3/_8$"	6 x 3 x $^3/_8$"
BA56 dead load thrust	73 lbs/in	86 lbs/in	101 lbs/in	103 lbs/in
BA56 live load thrust	1112 lbs/in	979 lbs/in	922 lbs/in	846 lbs/in
Total BA56 thrust	1185 lbs/in	1065 lbs/in	1023 lbs/in	949 lbs/in
Strut (plate) capacity	4090 lbs/in	5040 lbs/in	1265 lbs/in	1646 lbs/in
Result	Pass	Pass	Pass	Pass
Revised thrust (Dead + 0.75 live)	**907 lbs/in**	**820 lbs/in**	**792 lbs/in**	**737 lbs/in**
Result	**Pass**	**Pass**	**Pass**	**Pass**
UDL moment on tee	3 43 ton ft	3 48 ton ft	3 48 ton.ft	3 46 ton ft
Tee capacity	0 72 ton ft	0 72 ton ft	1 02 ton ft	0 72 ton ft
Result	Fail	Fail	Fail	Fail
Revised moment	**0.57 ton.ft**	**0.58 ton.ft**	**0.58 ton.ft**	**0.58 ton.ft**
Result	**Pass**	**Pass**	**Pass**	**Pass**