ASSESSMENT REPORT FOR BOXTED BRIDGE

(ECC No. 59)

Report No.1302C/92/2375/B59/5-A

Author: [redacted]

Checked by:

Approved by:

Taywood Engineering Limited,
Taywood House,
345 Ruislip Road,
Southall,
Middlesex UB1 2QX.

May 1992

CONTENTS

CERTIFICATE NO. F OF BD 2/89 PART 1

STRUCTURAL ASSESSMENT REPORT (PART 1) - AHS/2i

STRUCTURAL ASSESSMENT REPORT (PART 2) - AHS/2ii

IN ACCORDANCE WITH BD 34/90

ASSESSMENT CERTIFICATE

CERTIFICATE F

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

   i. it has been assessed in accordance with the Approval in Principle dated 16.4.92.

   ii. the assessed capacity of the structure is as follows: 3 Te.

   iii. the unique numbers of the drawings used for the assessment are:

   TEL/2375/59/301/A

Signed ..[redacted].....
TEAM LEADER

Name .....[redacted].....

Signed ...[redacted].....
PARTNER/DIRECTOR (Consulting Engineer)

Name ....[redacted]..............

Date ....15/7/92..........

2. The certificate is accepted by the TAA

Signed .[redacted]

Name ...[redacted]

Date ....28/11/95.........

Department of Transport

.tructure Assessment Report (. art One)

AHS/2

Date Assessment Completed 29 - MAY - 1992

Structure Key

Structure Number / / / • / /

Structure Name BOXTED BRIDGE

Region Code | Region

Agent Code | Agent Name ESSEX COUNTY COUNCIL

Consultant Code | Consultant Name TAYWOOD ENGINEERING LTD

Previous Assessments

1 BE4

2 BE3/73

3 Other

4 None

Existing Restrictions on Structure

5 Restricted

6 Basis of Restriction

7 Vehicle Weight (tons) •

8 Axle Weight (tons) •

9 Lane Width

10 Single Lane Working

11 Structure Being Monitored

Assessed Capacity of Structure to Assessment Code

12 Assessment Live Loading (tonnes) 3•0

13 FE Group —

Assessed Capacity to Carry Exceptional Loads

14 Structure assessed for HB N

15 Level of HB Loading which can be carried (units) •

16 Structure assessed for AIL

Major Factors Governing Load Capacity

17

E	IF	E	IF	E	IF	E	AF	E	AF	E	AF
10	02					10	02				

Signature of Team Leader [redacted]

Date 29th May 1992

Signature of Chief Officer or Partner [redacted]

Date 15/7/92

Signature of Director (DTp)

Date

Structure Assessment Report (Part Two)

Structure Key

Structure Number ___ / ___ / ___ / ___ • ___ / ___ / ___

Structure Name BOXTED BRIDGE

Region Code | Region

Agent Code | Agent Name ESSEX COUNTY COUNCIL

Consultant Code | Consultant Name TAYWOOD ENGINEERING LTD.

Proposed Remedial Measures

1	*Strengthen Superstructure*	Y
2	Reconstruct Superstructure	N
3	*Strengthen Substructure*	N
4	Reconstruct Substructure	N
5	Strengthen Independent Retaining Wall	N
6	Reconstruct Independent Retaining Wall	N

Affected Spans

7	Number of Spans to be Strengthened	1
8	Number of Spans to be Reconstructed	0

Interim Measures

9	Required	Y
10	Impose Weight Restriction (tonnes)	3•0
11	*Impose Lane Width Restriction*	N
12	Impose One Way Working	N
13	Prop Structure	N
14	Close Structure	N
15	Provide Temporary Alternative	N
16	Monitor Structure	Y

Estimated Resources for Proposed Remedial Measures

17 Estimated Cost of Remedial Measures £

Signature of Team Leader [redacted]

Date 29th May 1992

Signature of Chief Officer or Partner [redacted]

Date 15/7/92

Signature of Director (DTp)

Date

1.0 INTRODUCTION

This is a report on the assessment of Boxted Bridge, by Taywood Engineering Limited, for Essex County Council. The object is to prove the capacity of the bridge in terms of traffic loading.

To this end the bridge was inspected and surveyed in order to determine the condition and dimensions of the structure. The results of this work are presented here and using this data, calculations were performed to determine the capacity of the structural members of the bridge, and the forces in these members resulting from traffic loading. The calculation, which is presented in Appendix A of the report, is in accordance with the Department of Transport and other relevant codes of practice.

2.0 BRIEF DESCRIPTION OF THE STRUCTURE

The date of construction of Boxted Bridge is unknown (probably 1939-1945). It carries on unclassified road (Boxted - Thorington Street) over the River Stour. It is a single span structure supported by brick abutments. The deck is formed of the two main plate girder edge beams with the floor formed partly from transverse RSJ's and partly from RSJ's spanning longitudinally between the brick abutments and the outer transverse RSJ's. Steel hogging plates span between the bottom flanges of the RSJ's above which is mass concrete and a tarmac topping. Connections are generally by rivets with steel angles and T sections forming the connecting plates and stiffeners for the plate girders. The bridge is marked for two lanes.

ESSEX COUNTY COUNCIL – BRIDGE ASSESSMENT **BOXTED BRIDGE**

3 0 SUMMARY OF RESULTS & MEMBER CAPACITIES FOR ULTIMATE LIMIT STATE

ELEMENT	LOAD TYPE	WORST LOAD EFFECT	WORST CO–ACTING EFFECT	MEMBER CAPACITY	MEMBER CO–ACTING CAPACITY	REDUCTION FACTOR 'K' TO BD 21/84	UNITY RATIOS		ASSESSED HA CAPACITY	NOTES
							WORST LOAD EFFECT	CO–ACT EFFECT		
Longitudinal RSJ	Vertical Bending Moment	98 5 kNm	23 6 kN	105 8 kNm	268 2 kN	0 91	0 93	0 09	FULL	a
	Vertical Shear Force	84 2 kN	0 0 kNm	268 2 kN	105 8 kNm	0 91	0 31	0 0	FULL	a
	Vertical Bending Moment	71 4 kNm	26 1 kN	105 8 kNm	268 2 kN	N/A	0 67	0 10	3Te Axle	a
x	Vertical Shear Force	89 2 kN	0 0 kNm	268 2 kN	105 8 kNm	N/A	0 33	0 0	3Te Axle SF	a
	Vertical Bending Moment	98 5 kNm	23.6 kN	81 1 kNm	206 1 kN	0 91	1 21	0 11	FULL	b
	Vertical Shear Force	84 2 kN	0 0 kNm	206 1 kN	81 1 kNm	0 91	0 41	0 0	FULL	b
	Vertical Bending Moment	71 4 kNm	26 1 kN	81 1 kNm	206 1 kN	N/A	0 88	0 13	3Te Axle	b
x	Vertical Shear Force	89 2 kN	0 0 kNm	206 1 kN	81 1 kNm	N/A	0 43	0 0	3Te Axle SF	b
Longitudinal Channel	Vertical Bending Moment	39 0 kNm	15 0 kN	36 2 kNm	202 2 kN	N/A	1 08	0 07	3Te Axle	a
	Vertical Shear Force	48 7 kN	0 0 kNm	202 2 kN	36 2 kNm	N/A	0 24	0 0	3Te Axle SF	a
Transverse Beams (Double Flange)	Vertical Bending Moment	318 8 kNm	54 6 kN	592 7 kNm	499 0 kN	N/A	0 53	0 11	40Te Axle	a
	Vertical Shear Force	340 7 kN	0 0 kNm	499 0 kN	–	N/A	0 68	0 0	40Te Axle	a
	Vertical Bending Moment	318 8 kNm	54 6 kN	454 4 kNm	382 6 kN	N/A	0 69	0 14	40Te Axle	b
	Vertical Shear Force	340 7 kN	0 0 kNm	382 6 kN	–	N/A	0 89	0 0	40Te Axle	b

NOTES ·

a) Assessed using characteristic yield stress of 300 N/mm^2 As test results
b) Assessed using characteristic yield stress of 230 N/mm^2

GENERAL NOTES : All loadings include load factor f_3 =1 1

ESSEX COUNTY COUNCIL – BRIDGE ASSESSMENT BOXTED BRIDGE

3 0 SUMMARY OF RESULTS & MEMBER CAPACITIES FOR ULTIMATE LIMIT STATE

ELEMENT	LOAD TYPE	WORST LOAD EFFECT	WORST CO–ACTING EFFECT	MEMBER CAPACITY	MEMBER CO–ACTING CAPACITY	REDUCTION FACTOR 'K' TO BD 21/84	UNITY RATIOS		ASSESSED HA CAPACITY	NOTES
							WORST LOAD EFFECT	CO–ACT EFFECT		
Transverse Beams (Tnple Flange)	Vertical Bending Moment	613 5 kNm	94 1 kN	805 4 kNm	499 0 kN	N/A	0 76	0 19	40Te Axle	a
	Vertical Shear Force	453.7 kN	0 0 kNm	499 0 kN	–	N/A	0 91	0 0	40Te Axle	a
Plate Girders	Vertical Bending Moment	2928 0 kNm	833 5 kN	1828 0 kNm	2030 0 kN	0 91	1 6	0 06	FULL	a
	Vertical Shear Force	833 5 kN	0 0 kNm	2030 0 kN	–	0 91	0 41	–	FULL	a
	Vertical Bending Moment	1773 0 kNm	83 0 kN	1828 0 kNm	2030 0 kN	0 20	0 97	0.04	3Te	a
	Vertical Shear Force	464 0 kN	0 0 kNm	2030 0 kN	–	0 20	0 23	–	3Te	SF a
	Vertical Bending Moment	1701 0 kNm	104 0 kN	1828 0 kNm	2030 0 kN	N/A	0 93	0 05	3Te Axle	a
	Vertical Shear Force	422 0 kN	0 0 kNm	2030 0 kN	–	N/A	0 21	–	3Te Axle	SF a

NOTES :

a) Assessed using characteristic yield stress of 300 N/mm^2 As test results
b) *Assessed using characteristic yield stress of 230 N/mm^2*

GENERAL NOTES : All loadings include load factor f_3=1 1

4.0 CONCLUSIONS & RECOMMENDATIONS

The calculations show that the bridge is not capable of carrying the 40 Te assessment live loading, chiefly because of the weakness of the main plate girders. The bridge has been assessed to be able to carry only 3 te loading and even that is with reservations. To achieve this it was necessary to adopt the steel yield stress of 300 N/mm^2, the test value. With the lower bound stress of 230 N/mm^2, 3 te loading cannot be carried except possibly by limiting the bridge to a single lane.

Additionally, assumptions regarding the determination of the effective length of the structure have been necessary. A small enough effective length and subsequently adequate limiting stress in the top flange can only be achieved by assuming that the transverse beams and double-T web stiffeners to the plate girder act as U-frames restraining the top flange. The connection detail between transverse beams and stiffeners is unknown but without the assumption of U-frames the bridge girders are shown to be quite inadequate for dead loading alone - which does not seem to be the case. For the 3 te loading the stiffeners are marginally overstressed, but the assistance of alternate transverse beams and single T stiffeners has been ignored.

There is a fair degree of corrosion to the structure which has contributed to its reduced loading capacity. The longitudinal channels in each corner of the structure are badly corroded which has resulted in a considerable reduction of load capacity. Consequently they are marginally overloaded under 3 te accident loading but the spread of loads is conservatively estimated and the overstress not considered significant.

It is recommended that the bridge be restricted to 3 te loading and monitored with a view to introducing a single lane if necessary.

5.0 VISUAL INSPECTION REPORT

Date of Inspection: 12/03/92

Inspected by: [redacted]
(Taywood Engineering Limited)

Supervising Agent: [redacted]Essex County Council)

Weather: Cool, overcast, intermittent rain and windy.

Colchester direction is considered as South for the purposes of this inspection.

5.1 Foundations

The type of foundation is unknown. An underwater inspection was not carried out, however the river bed was probed and this showed that there was no serious undermining or scour. The depth of water varied from 100 mm next to the abutments to about 1000 mm at the centre of the channel. The river bed was found to be quite solid and remained fairly constant in level upstream and downstream to the bridge.

5.2 Abutments

The brick abutments of the bridge appear to be in a reasonable condition. The mortar joints show no significant deterioration. There are slight vertical cracks at the ends of the abutments sloping up toward the edge beam bearings. The crack at the East end of the South abutment varies in width from 2 mm at the base to 5 mm beneath the bridge soffit. There is no sign of settlement across the crack and it does not appear to have led to further deterioration of the adjacent brickwork. There is significant ivy growth at this end of the abutment with roots embedded in the mortar joints. The other cracks are hairline.

5.3 Bridge Deck

The hogging plates generally appear to be in a reasonable condition except for quite severe corrosion at their junction with the flanges of the RSJ's.

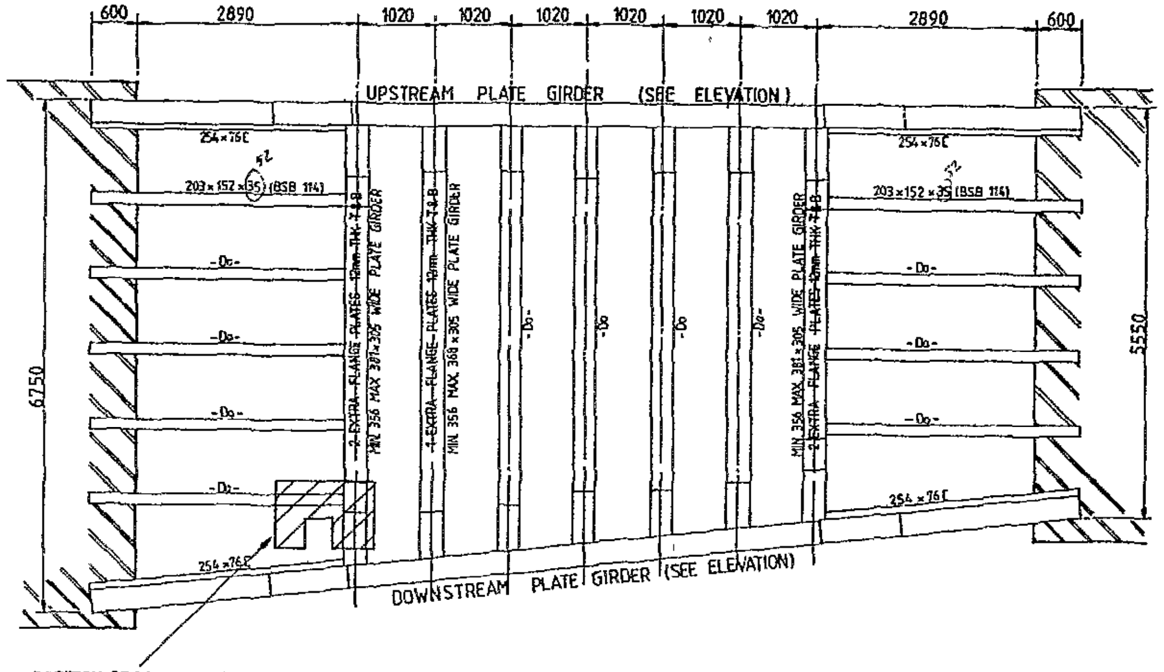
The RSJ's at 1/4 and 3/4 spans, have suffered severe localised corrosion of the bottom flange. This coincides with the position where the plates to reinforce the bottom flange begin. In one instance it can be seen that there has been approximately 35% loss of section of the bottom flange. Where the bottom flanges are reinforced by two additional plates over the centre half of their span the plates appear to have buckled and spaces have opened between them back to the line of the securing rivets. The plates themselves appear to be quite pitted and corrosion products are visible at their edges.

The longitudinally spanning channels at the edge of the deck are quite severely corroded, particularly at the South East corner of the bridge where approximately 50% of the section of the bottom flange has been lost. The two main plate girders at the edge of the deck appear to be in a reasonable condition. Where the hogging plates rest on the bottom flange of the girders leakage can be seen which has caused corrosion locally. The loss of section due to this corrosion is not great. The connecting angles between the bottom flanges and webs of the girders appear to have corroded but remain intact around their rivet connections. Subsequent to this deterioration the girders have been painted. The condition of the paintwork is good and would suggest that no further deterioration of the angles has occurred. The girder webs, T-section stiffeners and channel top flange all appear to be in a good condition. The end plate stiffeners at the North West and South East of the bridge appear to have suffered impact damage causing buckling of the plates, however this is the extent of the damage and it also appears to have occurred prior to painting, the paintwork remaining unmarked. These impacts may have caused or worsened the cracks in the abutments beneath.

The road surface is generally in a reasonable condition except for some slight settlement and surface deterioration at the South end of the bridge.

DRAWING No

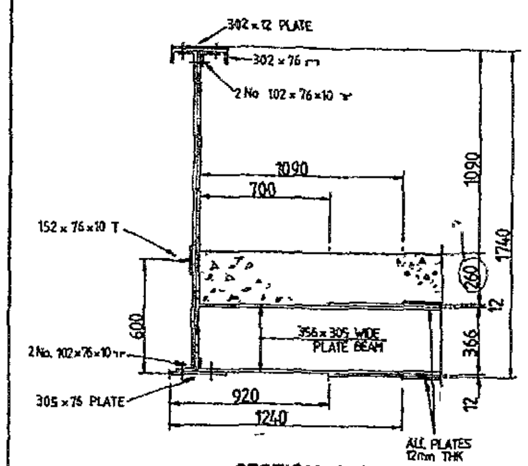
TEL / 2375 / B59 / 301 / A



PLAN OF DECK

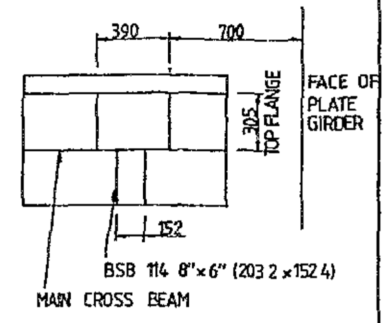
1:50

DETAIL 1



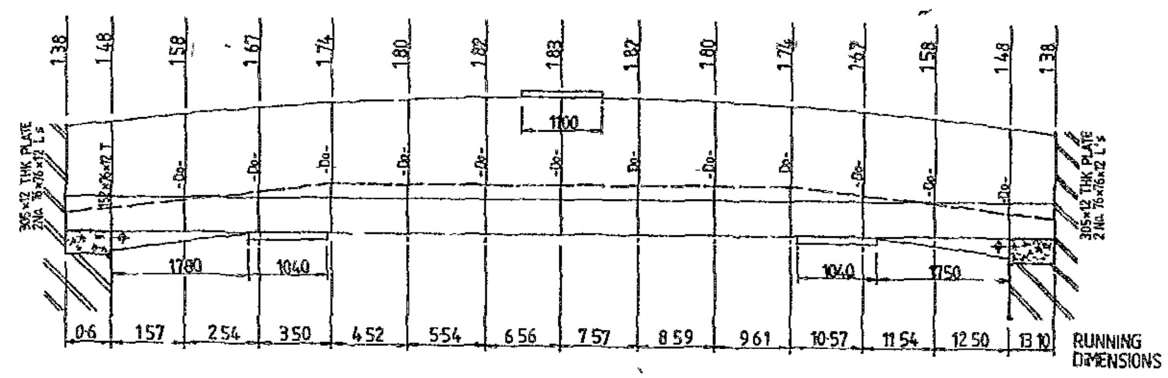
SECTION A-A

1:20



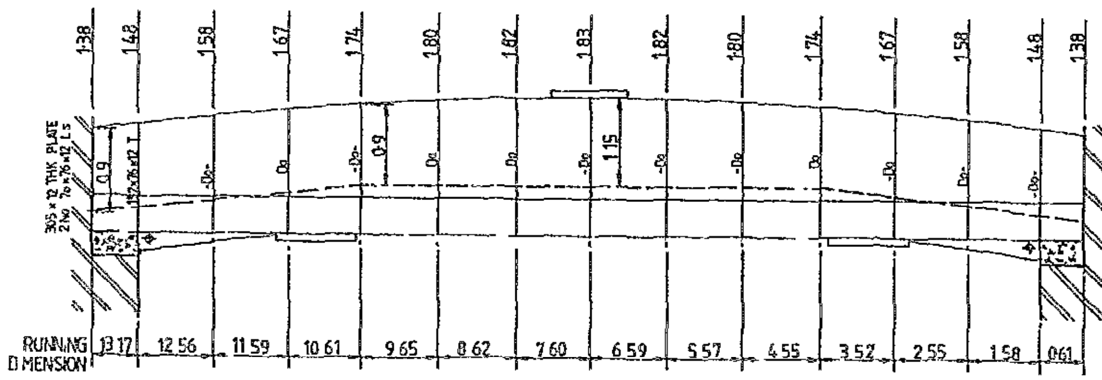
TRIAL HOLE

1:20



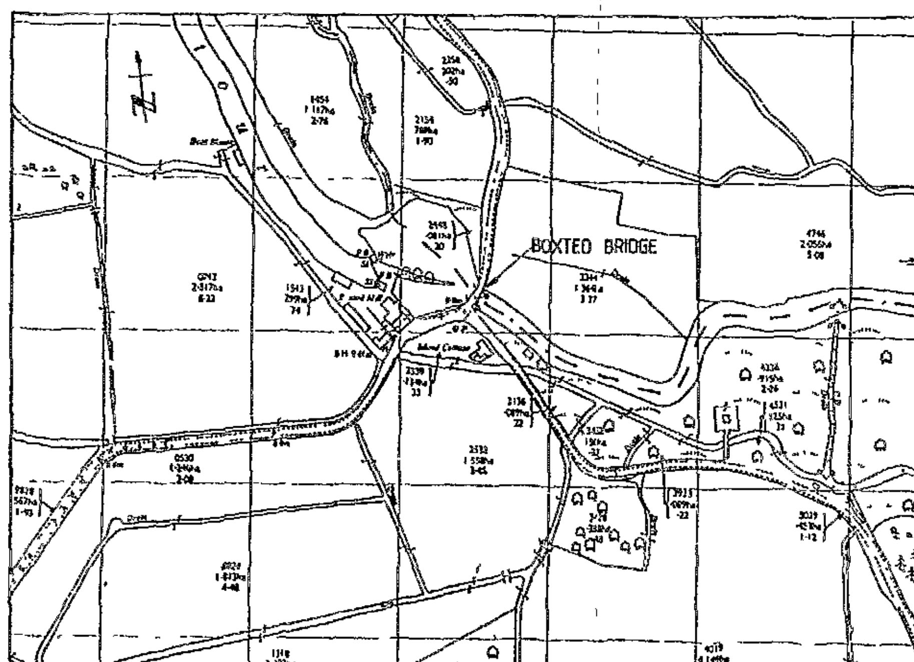
UPSTREAM PLATE GIRDER

1:50



DOWNSTREAM PLATE GIRDER

1:50



NOTES

1. Figured dimensions only are to be taken from this drawing.
2. BRIDGE LOCATION IS AT GRID REF 012 3
3. TRIAL HOLE DETAILS TAKEN FROM ESSEX COUNTY COUNCIL SURVEY DRG B32/59/1 (31 10 85)
4. DIMENSIONS SHOWN ON THIS DRG HAVE BEEN TAKEN FROM EXISTING RECORD DRGS B32/59/1,2,3 AND HAVE BEEN VERIFIED ON SITE
5. ALL RIVETS ARE AT 102mm CENTRES IN 2s (127 PITCH)

REVISION	DATE	INL	APP
FOR APPROVAL	MAR 92	C.G.M.	

SCALE 1:50, 1:20

Drawn C.G.M. Date MAR

Checked Date

APPROVED

PROJECT

ESSEX BRIDGE ASSESSMENT

DRAWING TITLE

BOXTED BRIDGE (B59)
GENERAL ARRANGEMENT
Sheet 1 of 1

Taywood Engineering Limited
345 Ruislip Road,
Southall, Middlesex UB1 2QX
Tel 081-578 2366

DRAWING No

TEL / 2375 / B59 / 301 / B

PLATE 1 EAST ELEVATION

PLATE 2 WEST ELEVATION

PLATE **3** VIEW FROM NORTH

PLATE **4** SOUTH ABUTMENT AND DECK SOFFIT

PLATE **5** NORTH ABUTMENT AND DECK SOFFIT

PLATE **6**
PLATE GIRDER EDGE BEAM

PLATE 7 DECK SOFFIT SHOWING DEFORMED FLANGE PLATES

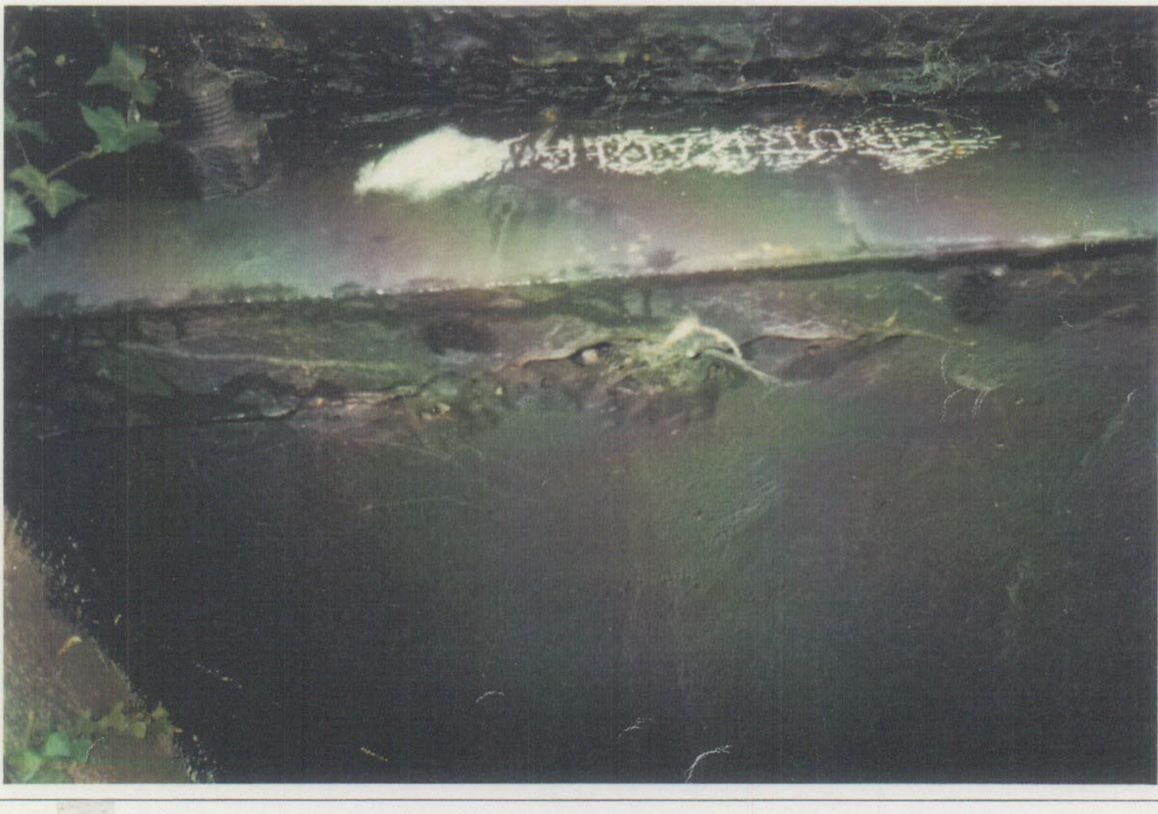


PLATE 8 CORROSION OF LONGITUDINAL CHANNEL AT THE EDGE OF THE DECK

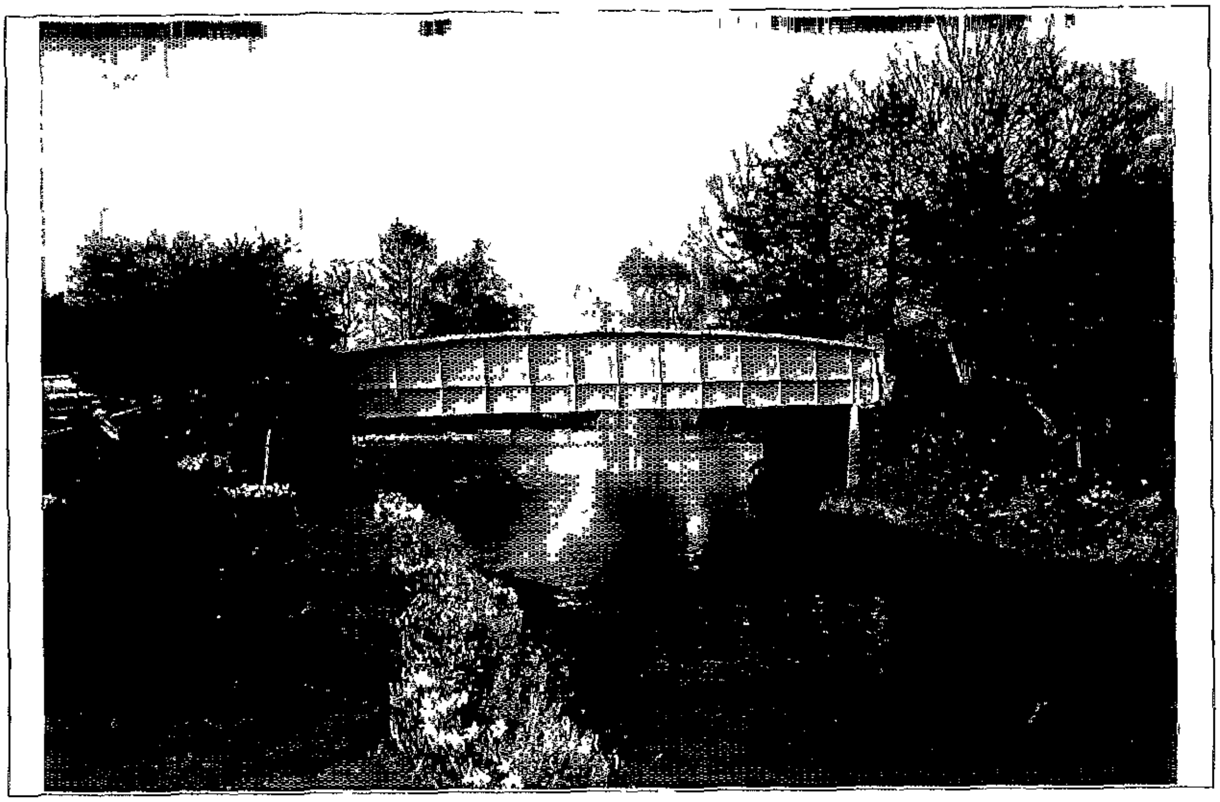


PLATE 1 EAST ELEVATION

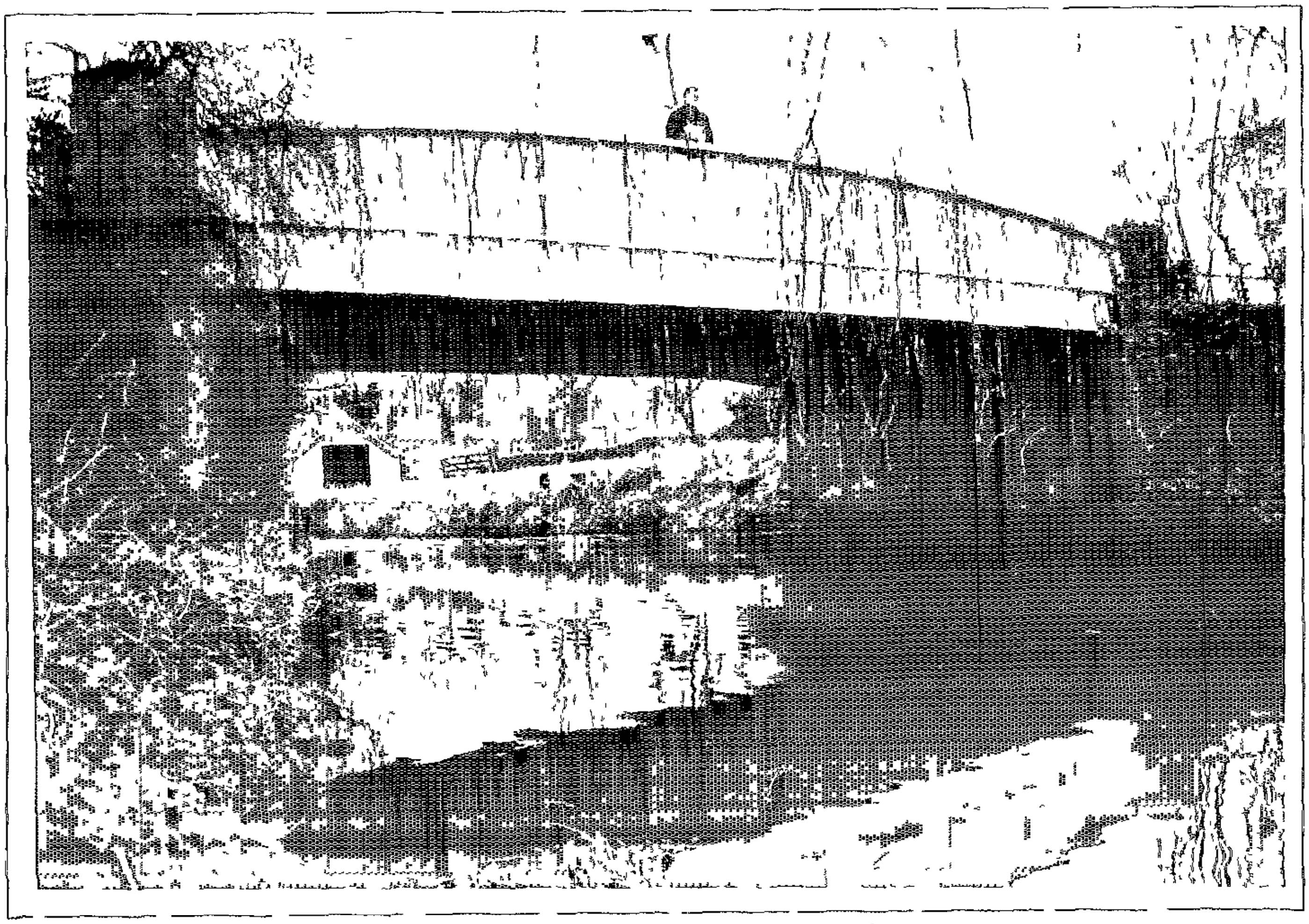


PLATE 2 WEST ELEVATION

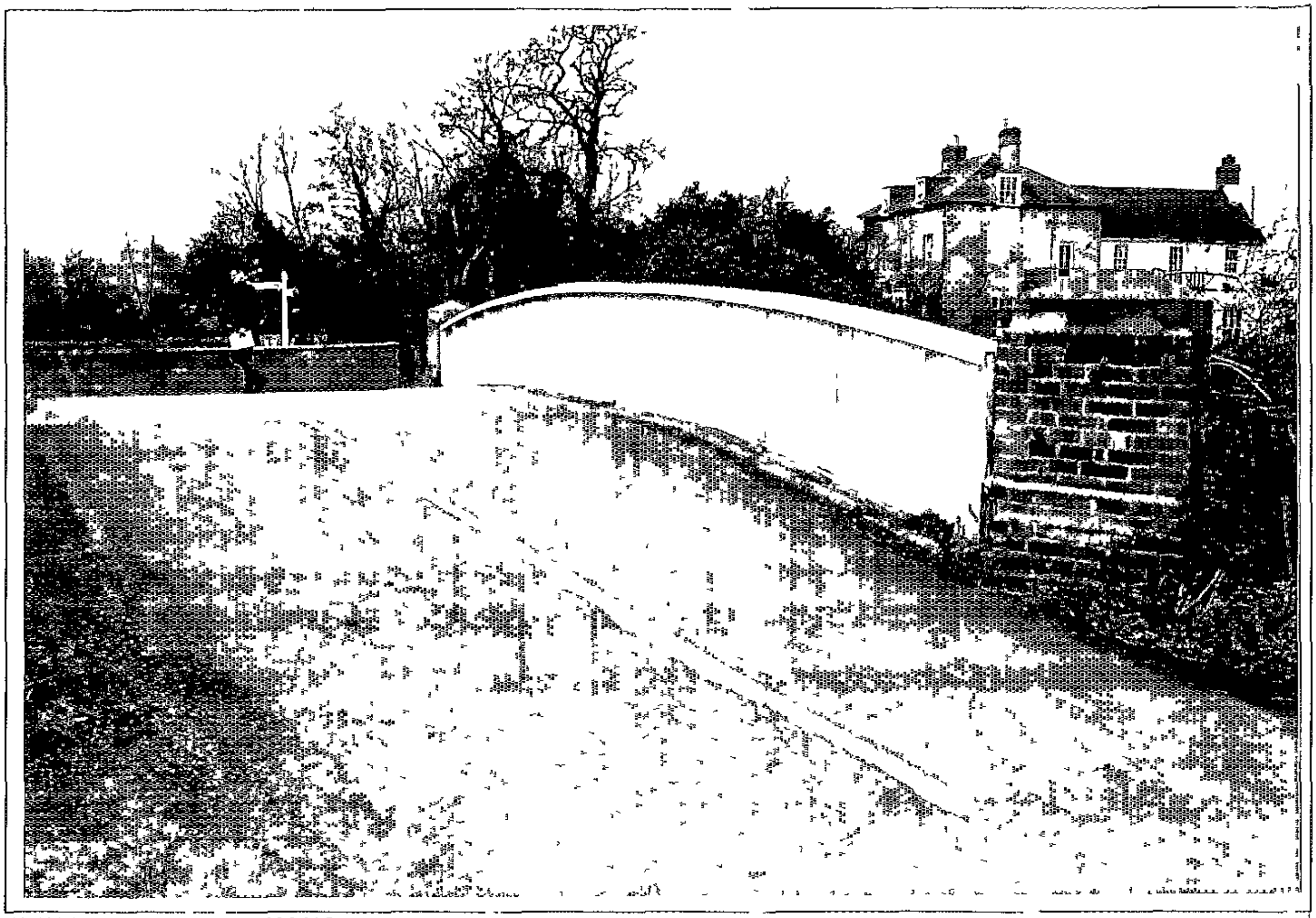


PLATE 3 VIEW FROM NORTH

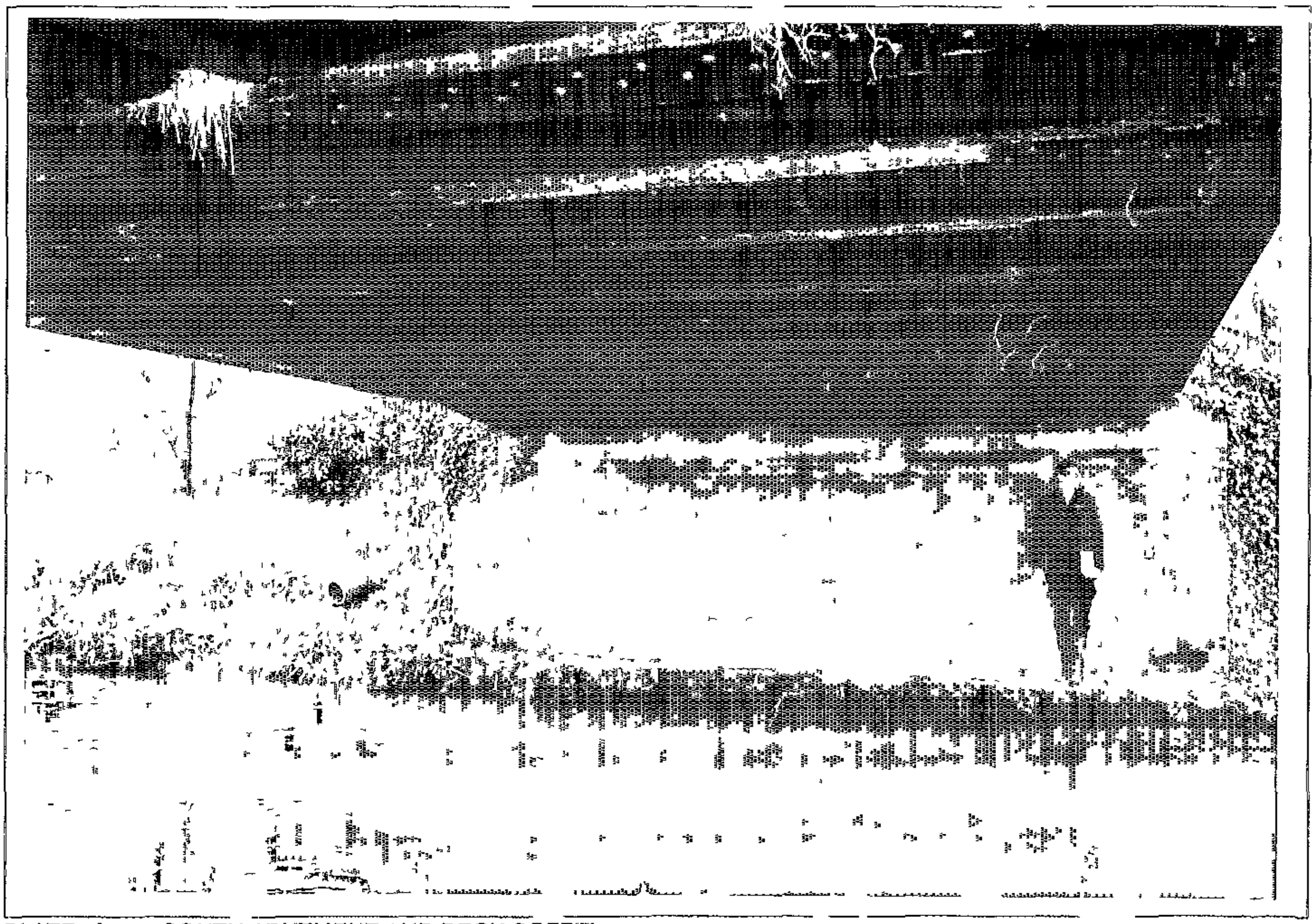


PLATE 4 SOUTH ABUTMENT AND DECK SOFFIT

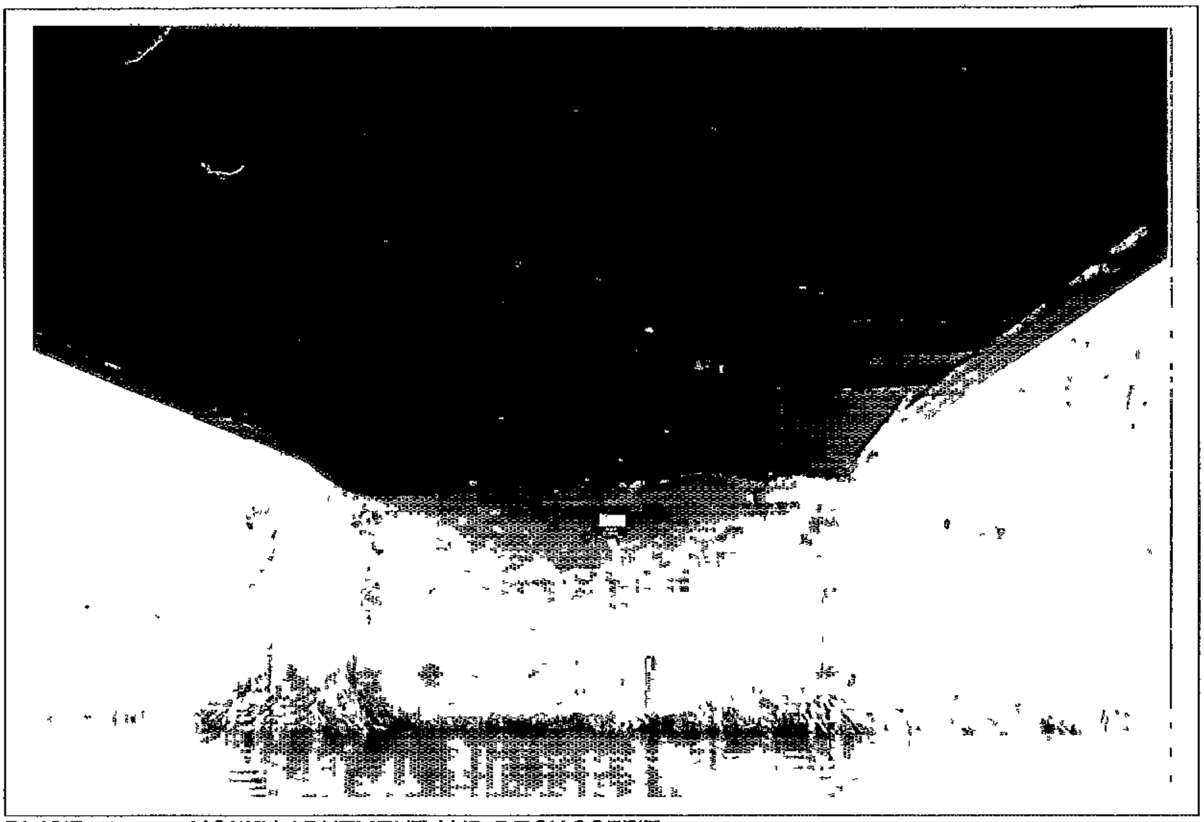


PLATE 5 NORTH ABUTMENT AND DECK SOFFIT

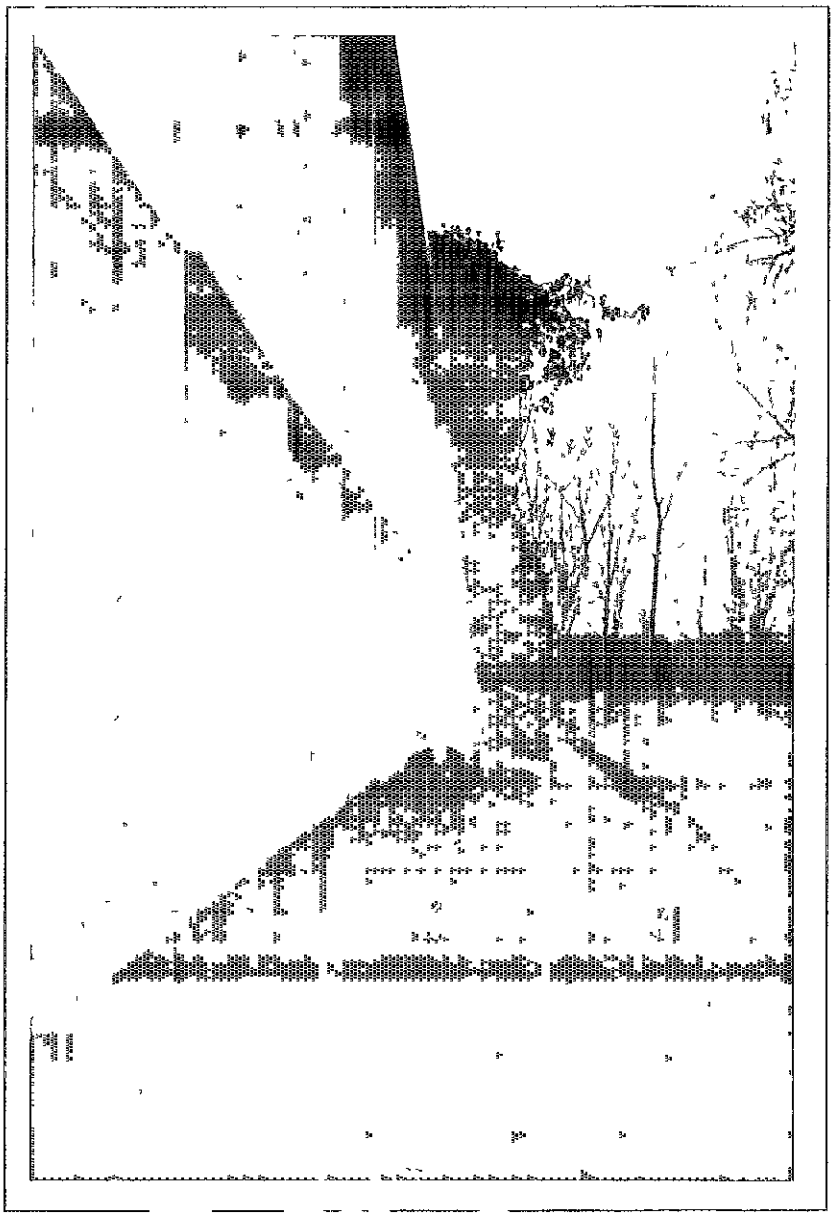


PLATE 6
PLATE GIRDER EDGE BEAM

PLATE 7 DECK SOFFIT SHOWING DEFORMED FLANGE PLATES

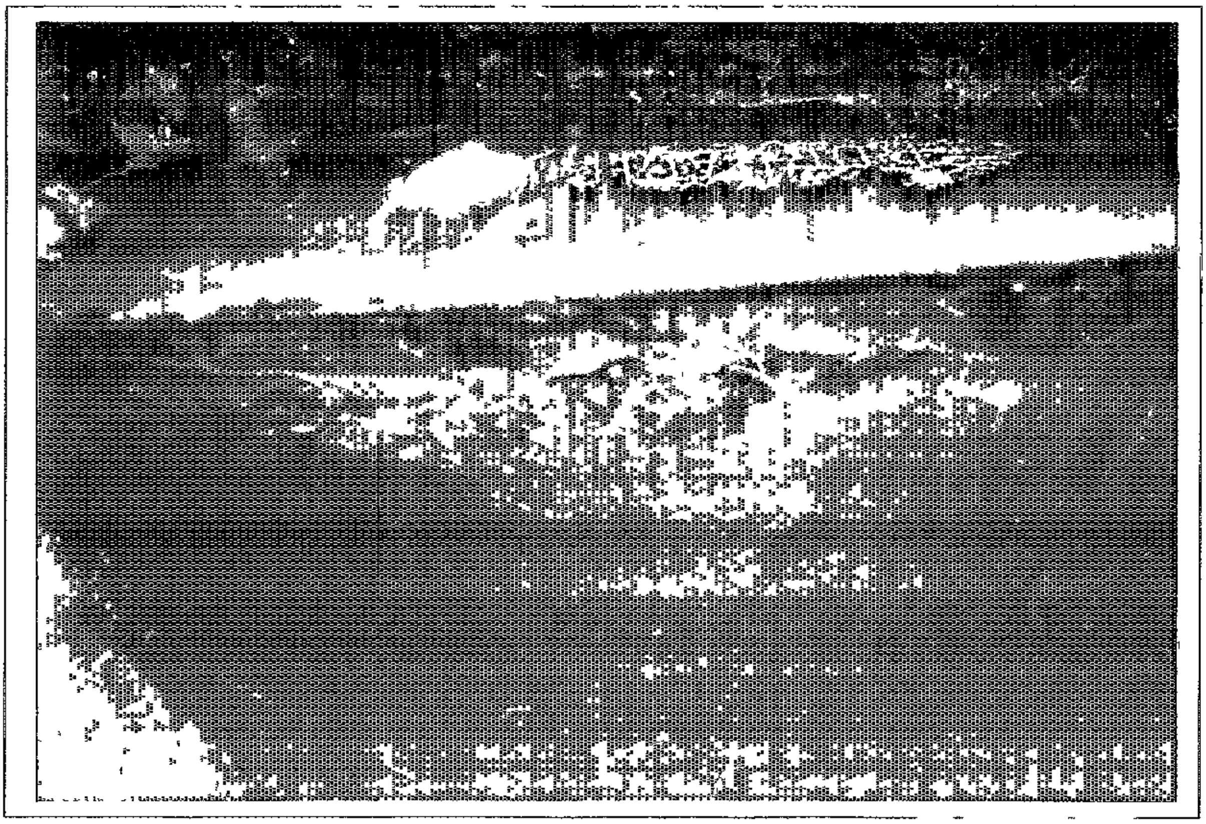


PLATE 8 CORROSION OF LONGITUDINAL CHANNEL AT THE EDGE OF THE DECK

APPENDIX A

ASSESSMENT CALCULATIONS

Prepared by	Date	TAYLOR WOODROW GROUP	Sheet No
[redacted]	May 92	CLIENT ESSEX COUNTY COUNCIL	1 of 67
[redacted]	May 92	CONTRACT/TENDER ASSESSMENT OF 21 BRIDGES	Calculation No 1302C/92/2375/B59/5-A
[redacted]	July '92	Calculation Title BOXTED BRIDGE ASSESSMENT	Ref Drawing No

BOXTED BRIDGE ASSESSMENT

CALCULATION TO DETERMINE THE LOAD CARRYING CAPACITY OF THE STRUCTURE

Prepared by	Date	TAYLOR WOODROW GROUP	Sheet No.
[redacted]	May 92	CONTINUATION SHEET	2 of 67
Checked by [redacted]	May 92	Calculation Title BOXTED BRIDGE ASSESSMENT	Calculation No 359/5-A

CONTENTS.

Prepared by	Date	TAYLOR WOODROW GROUP	Sheet No
[redacted]	May 92	CONTINUATION SHEET	3 of 67
Checked by [redacted]	May 92	Calculation Title BOXTED BRIDGE ASSESSMENT	Calculation No B59/5-A

1.0. INTRODUCTION

Boxted Bridge carries an unclassified road over the River Stour. It is a single span structure supported by brick abutments. The deck is formed of tarmac surfacing on mass concrete infill on logging plates. The logging plates span between the bottom flanges of RSJ's and plated steel beams. The RSJ's which are in the two end quarters of the span are in the longitudinal direction, whilst the plated beams in the middle half of the span are in the transverse direction. These span onto a pair of deep plate girder edge beams which span between the abutments. Connections are by rivets with angles and T-sections forming the connecting plates and stiffeners for the girders.

The bridge was surveyed on 12-03-92 the results of which are used in the analyses - carried out in accordance with D.O.T Standard BD 21/84 and BS 5400: Pt3 Ref 1

The bridge will be assessed for 40 Te assessment live loading which will include:

i) a UDL together with KEL

ii) single axle/wheel load

If the structure is shown to be capable of withstanding the 40 Te assessment loading it will be assessed to determine the number of units of HB loading it can carry.

Prepared by	Date	TAYLOR WOODROW GROUP	Sheet No
[redacted]	May 92	CONTINUATION SHEET	4 of 47
Checked by [redacted]	May 92	Calculation Title BOXTED BRIDGE ASSESSMENT	Calculation No B59/5-A

2.0 REFERENCES.

1 Survey Details ([redacted] 12-03-92)

2. City Consultancy Services - Test Results

3 Department of Transport Standard BD 21/84 - The Assessment of Highway Bridges and Structures - Amendment No 1

4. BS 5400 - Steel Concrete and Composite Bridges - Pt 3. Code of Practice for the Design of Steel Bridges

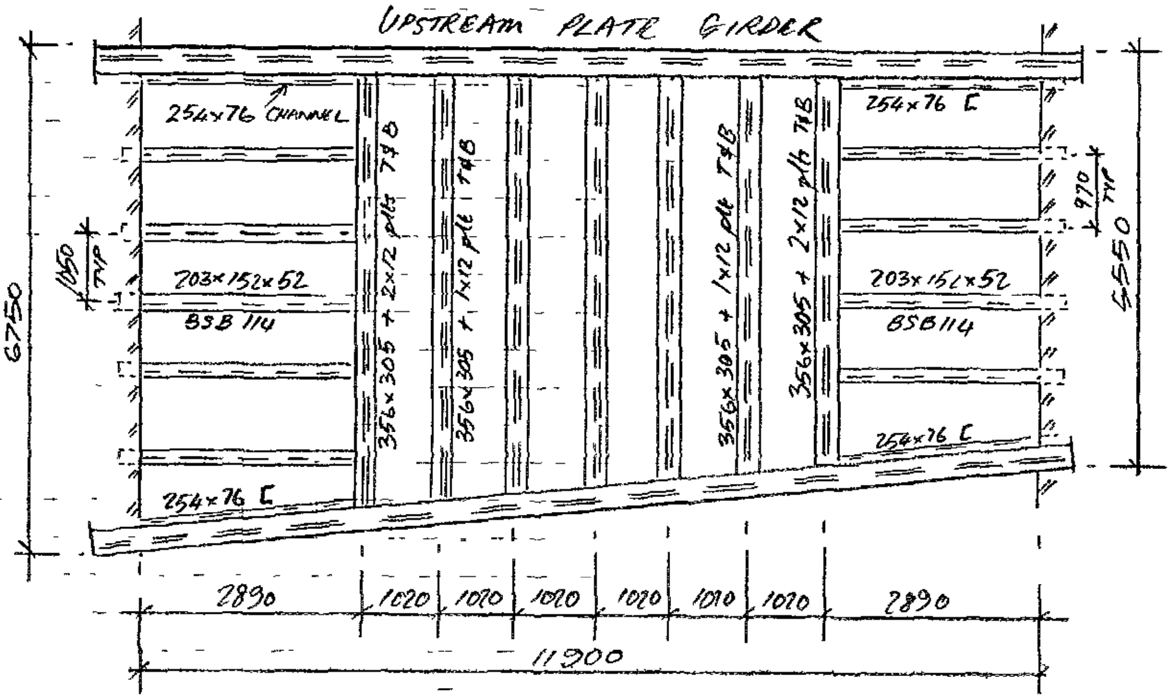
5. Drawing No TEC/2375/B59/301/B.

6 Historical Structural Steelwork Handbook - BCSA Publication No. 11/84

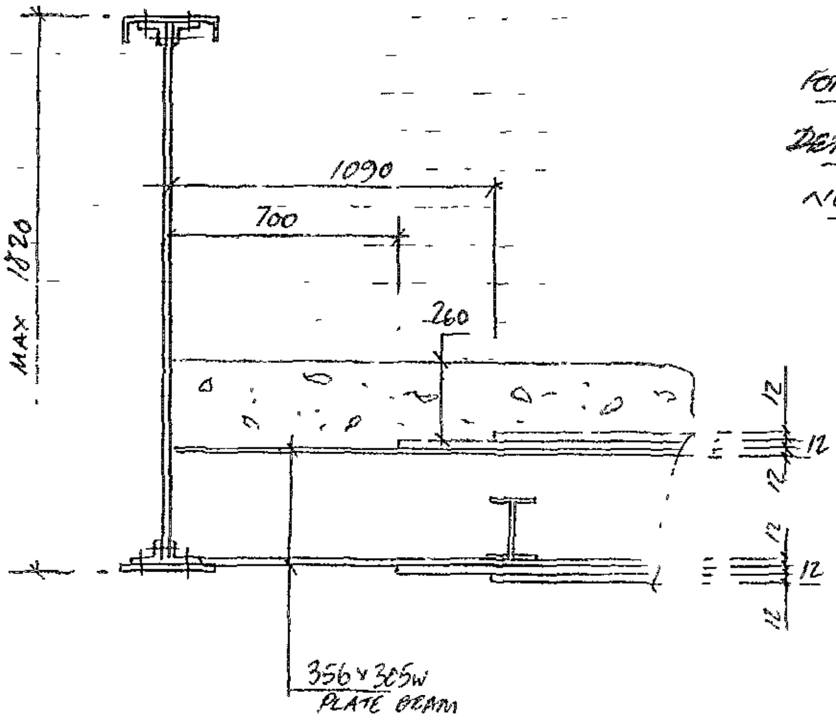
7. Dept of Transport Advice Note BA 16/84

Prepared by	Date	TAYLOR WOODROW GROUP	Sheet No.
[redacted]	May 92	CONTINUATION SHEET	5 of 67
[redacted]	May 92	Calculation Title BOXTED BRIDGE ASSESSMENT	Calculation No 859/5-A

3.0 GENERAL DIMENSIONS



PLAN ON STEELWORK



FOR ALL OTHER DETAILS SEE DRG NO TEL/2375/859/301

SECTION

4.0 ASSESSMENT CRITERIA

The Structure is deemed to be capable of carrying the assessment loading when the following relationship is satisfied

$$R_A^* \geqslant S_A^*$$ BD 21/84 5.5 Eqn 1

where R_A^* = Assessment Resistance

S_A^* = Assessment Load Effects

This can be written as

$$F_c \text{ function} \left(\frac{f_K}{\gamma_m}\right) \geqslant \gamma_{f3} \left(\text{effects of } \gamma_{fL} \cdot Q_K\right)$$

Where γ_{f3} = partial factor for inaccurate assessment of load effects

γ_m = partial factor for material strength

f_K = characteristic strength

γ_{fL} = partial factor for loads

Q_K = nominal loads

F_c = condition factor

[redacted]	Date	TAYLOR WOODROW GROUP	Sheet No
	May 92	CONTINUATION SHEET	7 of 67
	May 92	Calculation Title BOLTED BRIDGE ASSESSMENT	Calculation No B59/5-A

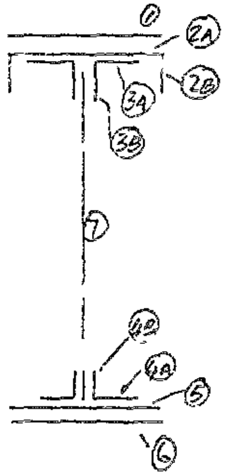
5.0 CONDITION FACTORS

The following three pages record material thicknesses and estimated extent of corrosion.

The actual measured thicknesses as shown in the tables will be used in the calculations with $F_c = 1.0$ where possible, since the measured values already account for material loss from corrosion. Generally UPV meter readings are relied on, with caliper checks as a back-up. As the UPV values are only in localised parts of the members F_c will be reduced in accordance with any visual observations of corrosion

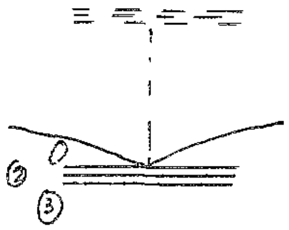
Prepared by	Date	TAYLOR WOODROW GROUP	Sheet No
[redacted]	May 92	CONTINUATION SHEET	8 of 67
	May 92	Calculation Title BOXTED BRIDGE ASSESSMENT	Calculation No B59/5-A

STRUCTURAL ELEMENT	THICKNESS CALIPER	THICKNESS UPV	REMARKS	LIKELY ORIGINAL SECTION	ESTIMATED ADDITIONAL LOSS	THICKNESS CALCULATION	CONDITION FACTOR
MAIN PLATE GIRDER			All steel painted				
1 LAP PLATE	15 + 10	11.7, 11.7	Badly corroded at edge to 10 mm or less Assume linear decrease to centre	15 mm Plt	15%	11.7 mm	0.85
2A TOP FLANGE CHANNEL	10	9.3, 9.5 9.3, 9.4 9.3, 9.3 9.5, 9.6 9.2, 9.2	Top face smooth, underside rough (badly pitted and cleaned ?) Rusted through in 2 locations by abutments	12 mm	20%	9.6	0.80
2B TOP FLANGE CHANNEL LEG	10.5	–	Smooth, good condition	12 mm	0%	10.0	1.0
3A L	14	–	Appears in good condition	12 mm	0%	10.0	1.0
3B L	11	–	" " "	12 mm	0%	10.0	1.0
4A L	–	12.2		12 mm	0%	10.0	1.0
4B L	–			12 m	0%	10.0	1.0
5 BOTTOM FLANGE PLATE	17	15.2, 16.0, 15.0 IN CENTRE	At abutment bearing corroded back to L Pieces (70x15) corroded out of edges Some pitting on top	17.5 mm	10%	15.0	0.9



M NO TW45

STRUCTURAL ELEMENT	THICKNESS CALIPER	THICKNESS UPV	REMARKS	LIKELY ORIGINAL SECTION	ESTIMATED ADDITIONAL LOSS	THICKNESS CALCULATION	CONDITION FACTOR
MAIN GIRDER (CONT)							
⑥ BOTTOM FLANGE LAP PLATES 7.5m FROM ABUTMENTS	–	15.2 AT CENTRE	No visible corrosion, probably some at edges similar to flange bottom plate	17	10%	15.0	0.90
⑦ WEB PLATE	–	12.7, 12.9, 12.8	Appears good. However part is buried on inside (in concrete) there is rainwater leakage here - assume some hidden damage	15	10%	12.7	0.9
VERTICAL STIFFENERS	12	11.4, 11.7, 11.8	Appear in good condition	12.0	0	11.5	1.0
MAIN GIRDER O/A							0.9.
TRANSVERSE FLOOR BEAMS							
① BEAM BOTTOM FLANGE	–	10.7, 11.8	Some corrosion	12.7		11.0	0.7
②/③ STRENGTHENING PLATES		11.7, 10.8, 11.8, 12.7	Considerable distortion, rust between plates. Some ends corroded away	12.7	30%	11.0	0.7



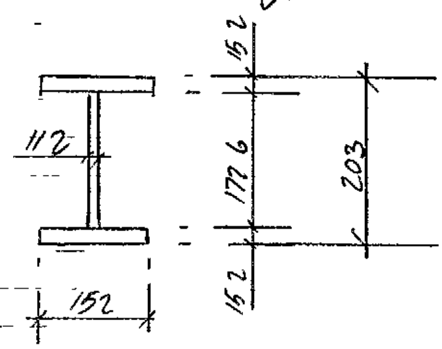
STRUCTURA ELEMENT	THICKNESS CALIPER	THICKNESS UPV	REMARKS	LIKELY ORIGINAL SECTION	ADDITIONAL LOSS	THICKNESS CALCULATION	CONDITION FACTOR.
TRANSVERSE FLOOR BEAMS (cont.)							
TOP FLANGE PLATES	HIDDEN	HIDDEN	Cast in concrete. Presume better condition than bottom	12.7	0	11.0	1.0
WEB	HIDDEN	HIDDEN	Cast-in	12.7		12.0	1.0
TRANSVERSE FLOOR BEAMS O/A							0.80 (0.95 shear)
LONGITUDINAL FLOOR BEAMS							
BOTTOM FLANG	Toe 12 Mid pt 15	Toe 12	Cast in concrete, only bottom flange visible. Appear good condition except for some leakage at abutment - so some corrosion could be hidden	203x152 x52 BSB114 15mm		15.2 flange 11.2 web	0.9
LONGITUDINAL CHANNEL							
BOTTOM FLANGE	–	–	Considerable corrosion	BSC 113 154x76 x 29 11.4mm		11.4mm flange 8.1mm web	0.5 bending 0.75 say shear

6.0. MEMBER RESISTANCES

6.1 LONGITUDINAL R.S.J's

BSB 114 = 203 × 152 × 52 kg/m — Ref 6

Idealised Section



Plastic modulus

$$Z_p = 2 \times 152 \times 15.2 \times \left(\frac{203 - 15.2}{2}\right) + 11.2 \times 177.6^2/4$$

$$= (4.34 + 0.83) \times 10^5 \text{ mm}^3 = \underline{5.17 \times 10^5 \text{ mm}^3}$$

Section Classification

Check for compact section

i) Web Cl. 9.3.7.2 — BS 5400 Pt 3

$$28\, t_w \sqrt{\frac{355}{\sigma_{yw}}} = 28 \times 11.2 \sqrt{\frac{355}{300}}$$ $\sigma_{yw} = 300$ best value — Ref 2

$= 341$ mm > 151.5 mm OK

ii) compression flange Cl 9.3.7.3

$$7 t_{fo} \sqrt{\frac{355}{300}} = 7 \times 15.2 \times \sqrt{\frac{355}{300}} = 115.7 > 76 \quad OK$$

∴ SECTION IS COMPACT.

Bending capacity.

$$M_D = \frac{Z_{pe}\, \sigma_{lc}}{\gamma_m \gamma_{f3}}$$

$\sigma_{lc} = \sigma_y$ since compression flange is restrained making $l_e = 0$

$$\therefore \frac{\sigma_{li}}{\sigma_{lc}} = 1.0 \quad \text{(fig 10 BS 5400)}$$

Member Resistances (cont)

$\therefore \sigma_{lc} = 300 \text{ N/mm}^2$ (ref 2)

or $= 230 \text{ N/mm}^2$ (BD 21/84 Cl 6.2.2)

$\therefore M_D = \dfrac{517 \times 10^5 \times 300 \times 10^{-6}}{1.2 \times 1.1} = \underline{117.5} \text{ kNm}$ $(\sigma_{lc} = 300)$

$M_D = 117.5 \times \dfrac{230}{300} = \underline{90.1 \text{ kNm}}$ $(\sigma_{lc} = 230)$

Flange resistances

$M_R = \dfrac{F_f \cdot d_f}{\gamma_m \gamma_{f3}} = \dfrac{\sigma_{lc} A_{fe} \cdot d_f}{\gamma_m \gamma_{f3}}$ BS 5400 Pt 3 cl. 9.9.3

$M_R = \dfrac{300 \times 152 \times 15.2 \times 187.8 \times 10^{-6}}{1.2 \times 1.1} = \underline{98.6} \text{ kNm}$ $(\sigma_{lc} = 300)$

$M_R = 98.6 \times 230/300 = \underline{75.6} \text{ kNm}$ $(\sigma_{lc} = 230)$

Shear Capacity

$V_D = \dfrac{t_w (d - h_h) \tau_l}{\gamma_m \gamma_{f3}}$ $\lambda = \dfrac{d_{we}}{t_w}\sqrt{\dfrac{\sigma_{yw}}{355}} = \dfrac{172.6}{11.2}\sqrt{\dfrac{300}{355}}$

$= 14.2 \therefore \dfrac{\tau_l}{\tau_y} = 1.0$

$\therefore \tau_l = 300/\sqrt{3} = 173 \text{ N/mm}^2$ $(\sigma_{yw} = 300)$

$\tau_l = 230/\sqrt{3} = 133 \text{ N/mm}^2$ $(\sigma_{yw} = 230)$

$\therefore V_D = \dfrac{11.2 \times 203 \times 173 \times 10^{-3}}{1.2 \times 1.1} = \underline{298} \text{ kN}$ $(\sigma_{yw} = 300)$

$V_D = 298 \times 133/173 = \underline{229} \text{ kN}$ $(\sigma_{yw} = 230)$

Member Resistances (cont.)

BSB 114 – Assessment Resistance

from pg 10 F_c is taken as 0.9

$\therefore R_A^*$,

$M_D = 0.9 \times 117.5 = \underline{105.8 \text{ kNm}}$ $(\sigma_{lc} = 300)$

$M_D = 0.9 \times 90.1 = \underline{81.1 \text{ kNm}}$ $(\sigma_{lc} = 230)$

$M_R = 0.9 \times 98.6 = \underline{88.7 \text{ kNm}}$ $(\sigma_{lc} = 300)$

$M_R = 0.9 \times 75.6 = \underline{68.0 \text{ kNm}}$ $(\sigma_{lc} = 230)$

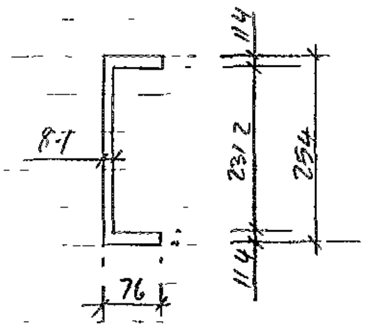
$V_D = 0.9 \times 298 = \underline{268.2 \text{ kN}}$ $(\sigma_y = 300)$

$V_D = 0.9 \times 229 = \underline{206.1 \text{ kN}}$ $(\sigma_y = 230)$

G2 LONGITUDINAL CHANNEL

BSC 113 254×76×29 kg/m — Ref 6

Idealised section



Section cast in concrete $\therefore$ compression flange restrained

$\sigma_{lc} = \sigma_y$ and by inspection section is compact

Plastic Modulus

$$Z_p = 2 \times 76 \times 11.4 \times \left(\frac{254 - 11.4}{2}\right) + 8.1 \times 231.2^2/4 = (2.10 + 1.08) \times 10^5$$

$$= \underline{3.18 \times 10^5 \text{ mm}^3}$$

Member Resistances (cont.)

Bending Capacity

$M_D = \dfrac{3.18 \times 10^5 \times 300 \times 10^{-6}}{1.2 \times 1.1} = \underline{72.3}$ kN m ($\sigma_{lc} = 300$)

$M_D = 72.3 \times 230/300 = \underline{55.4}$ kN m ($\sigma_{lc} = 230$)

Flange resistances

$M_R = \dfrac{300 \times 76 \times 11.4 \times 242.6 \times 10^{-6}}{1.2 \times 1.1} = \underline{47.8}$ kN m ($\sigma_{lc} = 300$)

$= 47.8 \times 230/300 = \underline{36.6}$ kN m ($\sigma_{lc} = 230$)

Shear capacity

$V_D = \dfrac{8.1 \times 254 \times 173 \times 10^{-3}}{1.2 \times 1.1} = \underline{269.6}$ kN ($\sigma_{yw} = 300$)

$V_D = 269.6 \times 230/300 = \underline{206.7}$ kN ($\sigma_{yw} = 230$)

Assessment Resistances

From pg 10 F_C = 0.5 bending, 0.75 shear.

$\therefore R_A^*$,

$M_D = 0.5 \times 72.3 = \underline{\underline{36.2}}$ kN m ($\sigma_{lc} = 300$)

$M_D = 0.5 \times 55.4 = \underline{\underline{27.7}}$ kN m ($\sigma_{lc} = 230$)

$M_R = 0.5 \times 47.8 = \underline{\underline{23.9}}$ kN m ($\sigma_{lc} = 300$)

$M_R = 0.5 \times 36.6 = \underline{\underline{18.3}}$ kN m ($\sigma_{lc} = 230$)

$V_D = 0.75 \times 269.6 = \underline{\underline{202.2}}$ kN ($\sigma_{yw} = 300$)

$V_D = 0.75 \times 206.7 = \underline{\underline{155.0}}$ kN ($\sigma_{yw} = 230$)

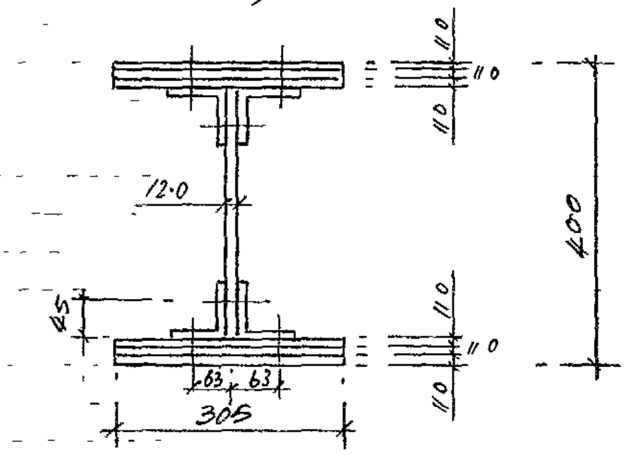
Prepared by	Date	TAYLOR WOODROW GROUP	Sheet No
[redacted]	May 92	CONTINUATION SHEET	15 of 67
	May 92	Calculation Title BOXTED BRIDGE ASSESSMENT	Calculation No 859/5-A

Member Resistances (cont)

6.3 TRANSVERSE BEAMS - TWO ADDITIONAL FLANGE PLATES (TRIPLE PLATE BEAMS)

Idealised section

Angles assumed to be 102×76×9.4



Section classification

Web $28 t_w \sqrt{\frac{355}{\sigma_{yw}}} = 28 \times 12 \sqrt{\frac{355}{300}} = 365.5 > 200$ OK

Flange $7 t_f \sqrt{\frac{355}{300}} = 7 \times 33 \sqrt{\frac{355}{300}} = 251.3 > 162.5$ OK

∴ Section is Compact

Plastic Modulus

Axis of equal area - assume at depth x from top of web

Area of top plates $= 33 \times 305 = 10065\ mm^2$

top angles $= 2 \times (102 \times 9.4 + 66.6 \times 9.4) = 3170\ mm^2$

upper web $= 12x$

lower web $= (334 - x)12 = 4008 - 12x$

lower angles $= 3170\ mm^2$

bottom plates $= 10065$

lower rivet holes (18mm rivets) $= 18(30.8 + 2 \times 42.4) = 2081\ mm^2$

$\therefore\ 10065 + 3170 + 12x = 4008 - 12x + 3170 + 10065 - 2081$

Member Resistances (cont)

$24x = 1927 \quad \therefore x = 80.3$ mm

$$\therefore Z_{pe} = 10065(80.3+16.5+253.7+16.5) + 2\times102\times9.4(80.3-4.7+253.7-4.7)$$

$$+ 2\times66.6\times9.4(80.3-42.7+253.7-42.7) - 18\times30.8\times(253.7-45)$$

$$- 2\times18\times42.4\times(253.7+16.5) + 120\times334^2/4$$

$$= (3.69 + 0.62 + 0.31 - 0.11 - 0.41 + 0.33)\times10^6 \text{ mm}^3$$

$$= \underline{4.43\times10^6 \text{ mm}^3}$$

Bending Capacity $\quad \sigma_{lc} = \sigma_y \quad$ top flange fully restrained

$$M_D = \frac{4.43\times10^6\times300\times10^{-6}}{1.2\times1.1} = \underline{1006.8} \text{ kNm} \qquad (\sigma_{lc} = 300)$$

$$M_D = 1006.8\times230/300 = \underline{771.9} \text{ kNm} \qquad (\sigma_{lc} = 230)$$

Flange resistances

$$M_R = \frac{\sigma_f A_{fe} d_f}{\gamma_m \gamma_{f3}} \qquad \text{where } \sigma_f A_{fe} \text{ is lower flange force}$$

$$= \frac{300(10065-18\times2\times33)\times367\times10^{-6}}{1.2\times1.1} = \underline{740.0} \text{ kNm} \qquad (\sigma_{lc} = 300)$$

$$M_R = 740\times230/300 = \underline{568.0} \text{ kNm} \qquad (\sigma_{lc} = 230)$$

Shear capacity at support $\quad (D = 334$, clear between flanges$)$

$$V_D = \frac{12.0\times334\times173\times10^{-3}}{1.2\times1.1} = \underline{525.3} \text{ kN} \qquad (\sigma_{yw} = 300)$$

$$V_D = 525.3\times230/300 = \underline{402.7} \text{ kN} \qquad (\sigma_{yw} = 230)$$

Member Resistances (cont)

Assessment resistances

from fig 10 F_c = 0.8 bending , 0.95 shear

R_d^*,

M_D	=	0.8 × 1006.8	=	805.4 KNm	(σ_{lc} = 300)
M_D	=	0.8 × 771.9	=	617.5 KNm	(σ_{lc} = 230)
M_R	=	0.8 × 740	=	592 KNm	(σ_{lc} = 300)
M_R	=	0.8 × 568	=	454 KNm	(σ_{lc} = 230)
V_D	=	0.95 × 525.3	=	499.0 KN	(σ_{yw} = 300)
V_D	=	0.95 × 402.7	=	382.6 KN	(σ_{yw} = 230)

6.4 TRANSVERSE BEAMS - ONE ADDITIONAL FLANGE PLATE.
(DOUBLE PLATE BEAMS)

Section as fig. 15 but o/a depth = 378 mm

Check for compact flange

$$7 t_0 \sqrt{\frac{355}{\sigma_{yw}}} = 7 \times 22 \times \sqrt{\frac{355}{300}} = 167.5 \quad \text{OK}$$

Plastic modulus

Area of flange plates = 22×305 = 6710 mm²

lower rivet holes = 18(30.8 + 2×31.4) = 1685 mm²

$$24x = 4008 - 1685 \qquad \therefore x = 96.8 \text{ mm}$$

$$\therefore Z_{pe} = 6710(334+22) + 0.62^{E6} + 0.31^{E6} + 0.33^{E6} - 18 \times 30.8 \times (334 - 96.8 - 4.5) \qquad \text{fig 16}$$

$$- 2 \times 18 \times 31.4 \times (334 - 96.8 + 11.0)$$

$$= (2.39 + 0.62 + 0.31 + 0.33 - 0.11 - 0.28) \times 10^6 \text{ mm}^3$$

$$= \underline{3.26 \times 10^6 \text{ mm}^3}$$

Member Resistances (cont.)

Bending Capacity

$M_D = \frac{326 \times 10^6 \times 300 \times 10^{-6}}{1.2 \times 1.1} = 740.9$ kNm $(\sigma_{lc} = 300)$

$M_D = 740.9 \times 230/300 = 568.0$ kNm $(\sigma_{lc} = 230)$

Flange resistances

$M_R = \frac{300(6710 - 18 \times 2 \times 22) \times 356 \times 10^{-6}}{1.2 \times 1.1} = 479$ kNm $(\sigma_{lc} = 300)$

$M_R = 479 \times 230/300 = 367.2$ kNm $(\sigma_{lc} = 230)$

Shear capacity at support $= 525.3$ kN $(\sigma_{yw} = 300)$ Pg 16

$= 402.7$ kN $(\sigma_{yw} = 230)$ Pg 16

Assessment Resistances

$F_c = 0.8$ bending , 0.95 Shear

γ_3^*,

$M_D = 0.8 \times 740.9 = \underline{\underline{592.7}}$ kNm $(\sigma_{lc} = 300)$

$M_D = 0.8 \times 568.0 = \underline{\underline{454.4}}$ kNm $(\sigma_{lc} = 230)$

$M_R = 0.8 \times 479 = \underline{\underline{383.2}}$ kNm $(\sigma_{lc} = 300)$

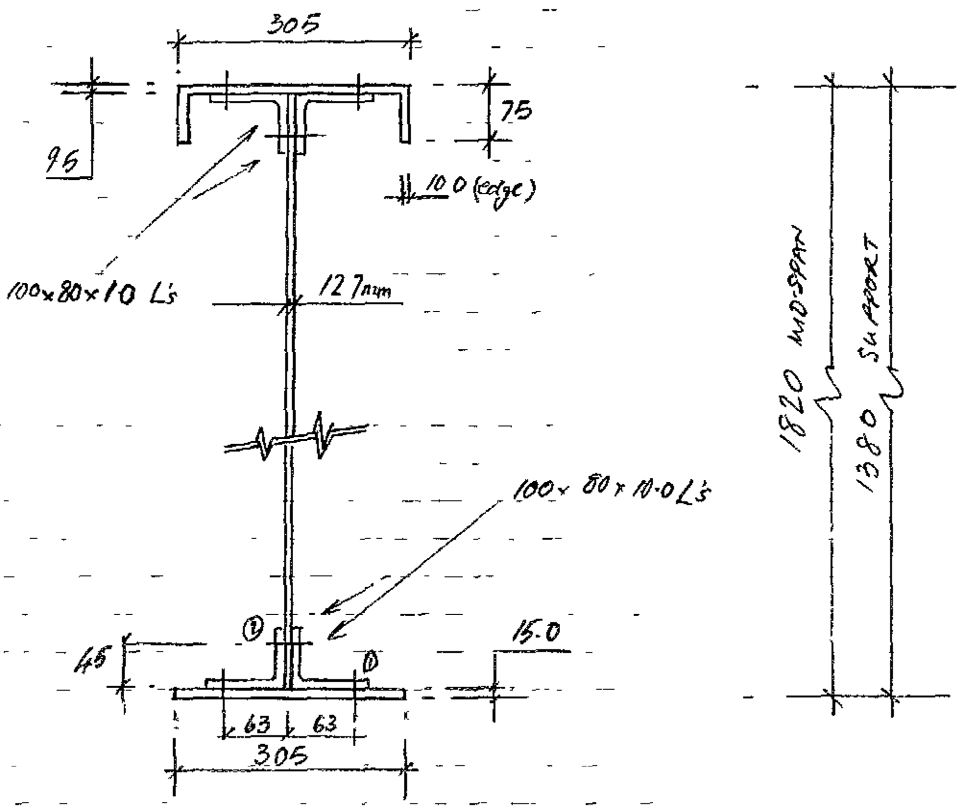
$M_R = 0.8 \times 367 = \underline{\underline{293.6}}$ kNm $(\sigma_{lc} = 230)$

$V_D = 0.95 \times 525.3 = \underline{\underline{499.0}}$ kN $(\sigma_{yw} = 300)$

$V_D = 0.95 \times 402.7 = \underline{\underline{382.6}}$ kN $(\sigma_{yw} = 230)$

member resistances (cont.)

6.5 PLATE GIRDERS



Top Channel - non standard - seems to be bent from uniform plate (legs have parallel sides)

Top flange = 9.5mm thick

legs - assume = 10.0mm thick for assessment

Angles measured 100 × 82

100mm leg is 14mm thick
82mm leg is 11mm thick } Non standard.

Take as 100 × 80 × 10mm thick for assessment (conservative)

Bottom plate

Measured thickness >15mm U.P.V average = 15.4mm

Use 15mm thick for assessment

PLATE GIRDERS. (CONT.)

Elastic Neutral Axis

From top of girder

Section	Area (mm^2)	Centroid mm	$A\bar{x}$ mm^3
Channel Web	305×9.5 = 2897.5	4.75	13763.1
Channel flanges	2x10.0x65.5 = 1310.0	42.25	55347.5
Upper Angle top legs	2×100×10.0 = 2000.0	14.50	29000.0
Angle bott. legs	2×70×10.0 = 1400.0	54.50	76300.0
Web	1795.5×12.7 = 22802.9	907.25	20687931.0
Lower Angle top legs	2×70×10.0 = 1400.0	1760.00	2464000.0
Angle bott. legs	2×100×10.0 = 2000.0	1800.00	3600000.0
Bottom plate	305×15.0 = 4575.0	1812.50	8292187.5
Bottom rivets ①	-2×18×25.0 = -900.0	1807.5	-1626750.0
Bottom rivets ②	-18×32.7 = -588.6	1760.0	-1035936.0

ΣA 36896.8 $\quad\quad$ $\Sigma A\bar{x}$ = 32555843

$\therefore \bar{X} = 32555843 / 36896.8$

$= \underline{882.35 \text{ mm}}$

Section Classification

Cl 9.3.7 BS 5400

Web Depth between elastic neutral axis and compression edge of web — Cl. 9.3.7.2

= 882.35 - 9.5 - 80.0 = 792.9 mm

PLATE GIRDERS (CONT)

Depth to compressive edge 793.3 $\not> 28\, t_w \sqrt{\frac{355}{\sigma_{yw}}}$

σ_{yw} = nominal yield stress = 300 N/mm² Test result

or = 230 N/mm² BD 21/84 Cl.6.2.2

$$28 t_w \sqrt{\frac{355}{\sigma_{yw}}} = 28 \times 12.7 \times \sqrt{\frac{355}{230}} = 441.8$$

∴ Web is not compliant

∴ Section is not compact

Bending Resistance

Cl 9.9.1.3 Non-compact sections

M_D is the lesser of

$$M_D = \frac{Z_{xc}\, \sigma_{lc}}{\gamma_m \gamma_{f3}}$$

or $$M_D = \frac{Z_{xt}\, \sigma_{yt}}{\gamma_m \gamma_{f3}}$$

Z_{xc}, Z_{xt} are elastic moduli based on effective section derived in accordance with 9.4.2

σ_{lc} = limiting compressive stress derived from 9.8.3.

σ_{yt} = nominal yield stress of tension flange material.

Effective section Cl 9.4.2

9.4.2.4 Flange = $K_c\, b\, t$

PLATE GIRDERS (CONT.)

K_c determined from fig 5

$$\lambda = \frac{b}{t}\sqrt{\frac{\sigma_y}{355}} \qquad b = 152.5\,mm \quad t = 9.5\,mm$$

$$\sigma_y = 300$$

$$\therefore \lambda = \frac{152.5}{9.5}\sqrt{\frac{300}{355}} = 14.75 \qquad \therefore K_c = 1.0$$

∴ Area of compression flange is full area of channel and angles

9.4.7.5 Web

Ignoring longit stiffener

$$t_{we} = t_w \text{ if } \frac{y_c}{t_w}\sqrt{\frac{\sigma_{yw}}{355}} \le 68$$

$y_c = 792.9\,mm \qquad t_w = 12.7 \qquad \sigma_{yw} = 300.$ fig 20

$$\therefore \frac{y_c}{t_w}\sqrt{\frac{\sigma_{yw}}{355}} = \frac{792.9}{12.7}\sqrt{\frac{300}{355}} = 57.4 \le 68$$

$$\therefore t_{we} = t_w = \underline{12.7\,mm}$$

Limiting compressive stress

cl 9.8.3

$$\sigma_{lc} \text{ taken as lesser of } \frac{D\sigma_{li}}{2y_t} \text{ or } \sigma_{yc}$$

σ_{li} as derived in cl 9.8.1

σ_{yc} as defined in cl. 9.8.1

D is overall depth

y_t is as defined in 9.7.3.1

Prepared by	Date	TAYLOR WOODROW GROUP	Sheet No
[redacted]	May 92	CONTINUATION SHEET	23 of 67
	May 92	Calculation Title BOXTED BRIDGE ASSESSMENT	Calculation No B59/5-A

PLATE GIRDERS (CONT)

Cl. 9.8.1 $\sigma_{yc} = 300\ N/mm^2$

$\frac{\sigma_{li}}{\sigma_{yc}}$ dependent on $\lambda_{LT}\sqrt{\frac{\sigma_{yc}}{355}}$

λ_{LT} determined from 9.7

Cl 9.7.4 Varying sections

$\lambda_{LT} = (1.5 - 0.5\rho_f) \times$ value obtained from 9.7.2

$\rho_f = \frac{\text{min flange area}}{\text{max flange area}} = 1.0 \qquad 1.5 - 0.5\rho_f = 1.0$

from 9.7.2

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

l_e = effective length determined in accordance with 9.6

r_y = radius of gyration of gross cross section about Y-Y axis

$k_4 = 1.0 \qquad \eta = 1.0$

v depends on I_c & I_t

$i = I_c / I_c + I_T \qquad$ & $\lambda_F = \frac{l_e}{r_y}\left(\frac{t_f}{D}\right)$

$I_c = 2 \times 75 \times 10 \times \left(\frac{295}{2}\right)^2 = 32.63 \times 10^6\ mm^4$

$+ \frac{1}{12} \times 9.5 \times 285^3 = 18.33 \times 10^6$

$+ 2 \times \frac{100^3 \times 10}{12} = 1.67 \times 10^6$

$+ 2 \times 100 \times 10 \times 56.35^2 = 6.35 \times 10^6$

$+ 2 \times 70 \times 10 \times 11.35^2 = 0.18 \times 10^6$

$59.16 \times 10^6\ mm^4$

$I_T = 15 \times 305^3/12 + 1.67E^6 + 6.35E^6 + 0.18E^6 - 2 \times 18 \times 25 \times 63^2$ (3.57E6)

$= 40.09 \times 10^6\ mm^4$

FORM NO TW45

Prepared by	Date	TAYLOR WOODROW GROUP	Sheet No
[redacted]	May 92	CONTINUATION SHEET	24 of 67
	May 92	Calculation Title BOXTED BRIDGE ASSESSMENT	Calculation No B59/5-A

Plate Girders (Cont).

Radius of gyration (of gross cross-section)

Gross $I_{y-y} = I_c + I_T + I_{web} + I_{rivet\ holes}$

$I_{web} = 1795.5 \times 12.7^3/12 = 0.31 \times 10^6 \ mm^4$

$I_{rivet\ holes} = 3.57 \times 10^6 \ mm^4$ pg 23

$\therefore I_{y-y} = (59.16 + 40.09 + 0.31 + 3.57) \times 10^6 = 103.13 \times 10^6 \ mm^4$

Gross Area = 36896.8 + 900.0 + 588.6

= 38385.4 mm²

$r^2 = \dfrac{I_{y-y}}{A} = \dfrac{103.13 \times 10^6}{38385.4} = 2686.7$

$\therefore \underline{r = 51.8 \ mm}$

Effective length, l_e

Ref. Clause 9.6 BS. 5400 Pt 3.

First consider top flange restrained by deck cl. 9.6.6.2

$l_e = 2.5 k_3 (E I_c \delta)^{0.25}$ for unit length

$k_3 = 1.0 \quad I_c = 59.16 \times 10^6 \ mm^4$ pg 23

$\& \ \delta = \dfrac{d_1^3}{3EI_1} + \dfrac{uBd_2^2}{EI_2} \qquad E = 205\,000 \ N/mm^2$

$d_1 = 1820 - 9.5/2 - 650 = 1165.25 \ mm$ (650 = ave deck thickness)

$d_2 = 1820 - 9.5/2 - 650/2 = 1490.25 \ mm$

Plate Girders (cont)

$u = 0.5$ (outer beam) — BS 5400 Pt 3 Cl. 9.6.5

$B = 6050$ mm (c/c of girders at longest two flange plate beam)

I_2 = inertia of deck $= \frac{650^3}{12} = 2.29 \times 10^7$ mm⁴

E for concrete $= 14000$ N/mm² — BD 21/84 Pg 21

$I_1 = \frac{t_w^3}{12} = \frac{12.7^3}{12} = 170.7$ mm³

$\therefore \delta = \frac{1165.25^3}{3 \times 2.05 \times 10^5 \times 170.7} + \frac{0.5 \times 6050 \times 1490.25^2}{14000 \times 2.29 \times 10^7}$

$= 15.07 + 0.02 = 15.09$

$\therefore l_e = 2.5 (2.05 \times 10^5 \times 59.16 \times 10^6 \times 15.09)^{0.25}$

$= 9.2$ m

$\lambda_f = \frac{l_e}{r_y}\left(\frac{t_f}{D}\right) = \frac{9200}{518}\left(\frac{9.5+15}{2 \times 1820}\right) = 1.19$ — BS 5400 Pt3 Cl. 9.7.2 Pg 24

$i = \frac{I_c}{I_c + I_T} = \frac{59.16}{59.16 + 40.09} = 0.6$

$\therefore v = 0.916$ — table 9 BS 5400

$\lambda_{LT} = \frac{l_e}{r_y} k_4 \eta v = \frac{9200}{518} \times 1.0 \times 1.0 \times 0.916 = 162.7$ — 9.7.2

$\lambda_{LT}\sqrt{\frac{\sigma_{yc}}{355}} = 162.7\sqrt{\frac{300}{355}} = 149.6$ for $\sigma_{yc} = 300$ N/mm²

$= 162.7\sqrt{\frac{230}{355}} = 131.0$ for $\sigma_{yc} = 230$ N/mm²

$\therefore \frac{\sigma_{li}}{\sigma_{yc}} = 0.215$ or 0.275 — fig 10 BS 5400

Plate Girders (cont)

$\sigma_{li} = 0.215 \times 300 = 64.5 \ N/mm^2 \quad (\sigma_{yc} = 300)$

$\sigma_{li} = 0.275 \times 230 = 63.2 \ N/mm^2 \quad (\sigma_{yc} = 230)$

$y_T = 1820 - 882.35 = 937.65$ mm — Pg 20

$\dfrac{D\sigma_{li}}{2y_T} = \dfrac{1820 \times 64.5}{2 \times 937.65} = 62.6 \ N/mm^2 \quad (\sigma_{yc} = 300)$ — BS5400 Pt3 cl 9.8.3

$= \dfrac{62.6 \times 63.2}{64.5} = 61.3 \ N/mm^2 \quad (\sigma_{yc} = 230)$

Elastic Modulus of Section

NA at 882.35 mm from top of girder — Pg. 20

Section	I about own axis	Area mm²	Centroid from NA mm	Ax^2 mm⁴
Channel web	$305 \times 9.5^3/12 = 21792$	2897.5	877.6	2.23 E 9
Channel flanges	$2 \times 10 \times 65.5^3/12 = 468352$	1310.0	840.1	0.93 E 9
Upper Angle top legs	$2 \times 100 \times 10^3/12 = 16666$	2000.0	867.85	1.51 E 9
bottom legs	$2 \times 10 \times 70.0^3/12 = 571666$	1400.0	827.85	0.96 E 9
Web	$12.7 \times 1795.5^3/12 = 6.126 E^9$	22807.9	24.93	0.01 E 9
lower angle top legs	$2 \times 10 \times 70.0^3/12 = 571666$	1400.0	877.65	1.08 E 9
bottom legs	$2 \times 100 \times 10^3/12 = 16666$	2000.0	923.15	1.70 E 9
Bottom Plate	$305 \times 15^3/12 = 85781$	4575.0	930.15	3.95 E 9
lower rivets (1)	neg	−900.0	925.15	−0.77 E 9
lower rivets (2)	neg	−588.6	877.65	−0.45 E 9

Ref table on Pg 20

$\Sigma = 6.128 E9$ $\qquad \Sigma = 11.16 E9$

$\text{Total } I_{x-x} = (6.128 + 11.16) E^9 = 17.29 \times 10^9 \ mm^4$

$Z_{xc} = \dfrac{I_{x-x}}{\bar{y}} = \dfrac{17.29 \times 10^9}{882.35} = 1.96 \times 10^7 \ mm^3$

Plate Girders (cont)

Bending Resistance

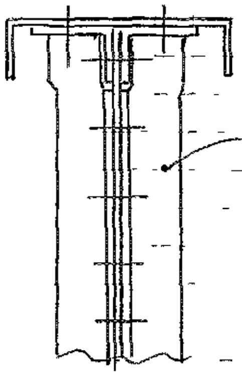
BS 5400 Pt3 9.9.1.3

$$M_D = \frac{Z_{xc} \times \sigma_{lc}}{\gamma_m \gamma_{f3}} = \frac{1.96\times10^7 \times 62.6\times10^{-6}}{1.1\times1.2} = 929.5 \text{ KNm } (\sigma_{yc}=300)$$

$$= 929.5 \times \frac{61.3}{62.6} = 910.2 \text{ KNm } (\sigma_{yc}=230)$$

These values are much lower than the bending moment due to dead load alone which suggests the value of l_e has been conservatively evaluated — Pg 42

∴ Derive a new l_e based on the top flange being restrained by U-frames made up of transverse beams and vertical T-section stiffeners



T-section stiffeners
79 leg, 145 table, 11.5 thick (survey ref.1)

take as 140×75×11.5 and include
16×web thickness = 203.2 mm on either side
= 406.4 mm total

Cl 9.6.5

$$l_e = 2.5 k_3 (E I_c l_u \delta)^{0.25}$$

Consider only double T-stiffeners at 2.04 m crs.

$k_3 = 1.0$ $\quad I_c = 59.16\times10^6 \text{ mm}^4$ (Pg 23)

$l_u = 2.04$ m

$$\delta = \frac{d_1^3}{3EI_1} + \frac{uBd_2^2}{EI_2} + f d_1^2$$

Plate Girders (cont.)

$E = 205,000$ N/mm²

$I_1 = 2\times 11.5\times 75^3/12 + 4\times 64.25\times 11.5^3/12 + 406.4\times 12.7^3/12 + 4\times 64.25\times 11.5\times 121^2$
$+ 2\times 11.5\times 43.85^2 = (0.81 + 0.03 + 0.07 + 0.43 + 3.32)\times 10^6$ mm⁴

$= 4.66\times 10^6$ mm⁴

I_2 = Inertia of transverse beam (double flange plates)

$= 2\times 305\times 22^3/12 + 2\times 305\times 22\times 178^2 + 4\times 102\times 9.4\times 162.3^2 + 4\times 9.4\times 66.6^3/12$
$+ 4\times 9.4\times 66.6\times 124.3^2 + 12\times 334^3/12$

$= (0.54 + 425.2 + 101.1 + 0.92 + 38.67 + 37.26)\times 10^6$

$= 603.61\times 10^6$ mm⁴

$d_1 = 1820 - 15 - 356 - 9.5/2 = 1444.25$ mm — BS 5400 Pt 3 fig 8

$d_2 = 1820 - 9.5/2 - 15 - 356/2 = 1622.25$ mm

$u = 0.5$ (outer beam)

$B = 6050$ mm (c/c of girders at longest two flange plt beam)

$f = 0.5\times 10^{-10}$ rad/N mm

$$\delta = \frac{1444.25^3}{3\times 2.05\times 10^5\times 4.66\times 10^6} + \frac{0.5\times 6050\times 1622.25^2}{2.05\times 10^5\times 603.61\times 10^6} + 0.5\times 10^{-10}\times 1622.25^2$$

$= 1.05\times 10^{-3} + 6.43\times 10^{-5} + 1.32\times 10^{-4}$

$= 1.24\times 10^{-3}$ mm/N

$\therefore\ l_e = 2.5\times 10\times (2.05\times 10^5\times 59.16\times 10^6\times 2040\times 1.25\times 10^{-3})^{0.25}$

$= 2.5\times 235.82 = \underline{5.895\text{ m}}$

Plate Girders (Cont.)

$$\lambda_F = \frac{l_e}{r_y}\left(\frac{t_f}{D}\right) = \frac{5895}{51.8}\left(\frac{9.5+150}{2\times1820}\right) = 0.766$$

$c = 0.6$ (fig 25) $\therefore v = 0.924$ — table 9 BS 5400

$$\lambda_{LT} = \frac{l_e}{r_y} k_4 \eta v = \frac{5895}{51.8} \times 1.0 \times 1.0 \times 0.924 = 105.2$$

$$\lambda_{LT}\sqrt{\frac{\sigma_{yc}}{355}} = 105.2\sqrt{\frac{300}{355}} = 96.7 \quad \text{for } \sigma_{yc} = 300 \text{ N/mm}^2$$

$$= 105.2\sqrt{\frac{230}{355}} = 84.7 \quad \text{for } \sigma_{yc} = 230 \text{ N/mm}^2$$

$$\therefore \frac{\sigma_{li}}{\sigma_{yc}} = 0.47 \text{ or } 0.57$$ — fig 10 BS 5400

$$\therefore \sigma_{li} = 0.47 \times 300 = 141 \text{ N/mm}^2 \quad (\sigma_{yc} = 300)$$

$$= 0.57 \times 230 = 131 \text{ N/mm}^2 \quad (\sigma_{yc} = 230)$$

$$Y_T = 1820 - 882.35 = 937.65$$

$$\frac{D\sigma_{li}}{2Y_T} = \frac{1820 \times 141}{2 \times 937.65} = 136.8 \text{ N/mm}^2 \quad (\sigma_{yc} = 300)$$

$$= 136.8 \times 131/141 = 127.1 \text{ N/mm}^2 \quad (\sigma_{yc} = 230)$$

$$Z_{xc} = 1.96 \times 10^7 \text{ mm}^3$$

Bending resistance $M_D = \frac{1.96\times10^7 \times 136.8\times10^{-6}}{1.1\times1.2} = 2031$ kN m $(\sigma_{yc} = 300)$

$$M_D = \frac{2031 \times 127.1}{136.8} = 1887 \text{ kN m} \quad (\sigma_{yc} = 230)$$

Plate Girders (cont.)

Flange Resistances

$$M_R = \frac{\sigma_f A_{fe} d_f}{\gamma_m \gamma_{f3}}$$ 9.9.3.1 BS 5400

$\sigma_f = \sigma_{lc} = 136.8 \ N/mm^2$ ($\sigma_{yc} = 300$)

$= 127.1 \ N/mm^2$ ($\sigma_{yc} = 230$)

$A_{fe} = 2897.5 + 1310.0 + 2000.0 + 1400.0$ Pg 20

$= 7607.5 \ mm^2$

centroid top flange $= (13763 + 55347 + 29000 + 76300) / 7607.5 = 22.9$ mm Pg. 20

Centroid bottom flange $= \frac{(2471700 + 3611000 + 8292187 - 1626750 - 1035936)}{1400 + 2000 + 4575 - 900 - 588.6} = \frac{11712201}{6486}$

$= 1805$ mm

$\therefore d_f = 1805 - 22.9 = 1782$ mm

$\therefore M_R = \frac{136.8 \times 7607.5 \times 1782}{1.2 \times 1.1 \times 10^6} = 1405$ kN m ($\sigma_{yc} = 300$)

or $\frac{1405 \times 127.1}{136.8} = 1305$ kN m ($\sigma_{yc} = 230$)

Shear capacity

Ignore longitudinal stiffener and check resistance under pure shear in accordance with cl 9.9.2.2

$V_D = \left[\frac{t_w (d_w - h_h)}{\gamma_m \gamma_{f3}}\right] \tau_l$ D = 1380 at support

τ_l dependent on $\lambda = \frac{d_{we}}{t_w} \sqrt{\frac{\sigma_{yw}}{355}}$

Plate girders (cont)

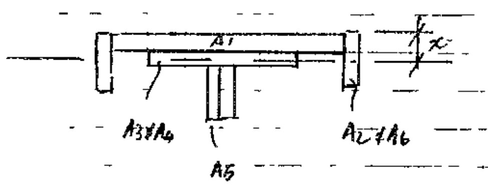
$d_{we} = 1380 - 9.5 - 15 - 2\times80 = 1195.5$

$a = 1020 - 140 = 880 \quad \phi = \dfrac{880}{1195.5} = 0.74$

$m_{fw} = \dfrac{\sigma_{yf} \cdot 2Z_p}{2\sigma_{yw}\, d_{we}^2\, t_w}$ as modified by cl 9.2 of BD 21/84

Z_p = plastic modulus of flange section + angles

Axis of equal area of flange



$A_1 = 285\times9.5 = 2707.5$

$A_2 = 2\times10\times x = 20x$

$A_3 = 200\times(x-9.5) = 200x - 1900$

$A_4 = 200\times(19.5-x) = 3900 - 200x$

$A_5 = 2\times70\times10 = 1400$

$A_6 = 2\times(75-x)\times10 = 1500 - 20x$

$A_1 + A_2 + A_3 = A_4 + A_5 + A_6$

$\therefore\ 2707.5 + 20x + 200x - 1900 = 3900 - 200x + 1400 + 1500 - 20x$

$440x = 5992.5 \quad \therefore\ x = 13.6$ mm

$Z_p = 285\times9.5\times(13.6 - 9.5/2) + 200\times10\times(14.5-13.6) + 2\times70\times10\times(54.5-13.6)$
$+\ 2\times75\times10\times(37.5-13.6)$

$= 23961 + 1800 + 57260 + 35850 = 118871\ mm^3$

$\therefore\ m_{fw} = \dfrac{2\times118871}{2\times1195.5^2\times12.7} = 0.0065$

for $\sigma_{yw} = 300 \quad \lambda = \dfrac{1195.5}{12.7}\sqrt{\dfrac{300}{355}} = 86.5$

Prepared by	Date	TAYLOR WOODROW GROUP	Sheet No
[redacted]	May 92	CONTINUATION SHEET	32 of 67
	May 92	Calculation Title BOXTED BRIDGE ASSESSMENT	Calculation No B59/5-A

Plate girders (cont)

– for $m/_w = 0.005$ τ_{y}/τ_y c = 1.0

$\therefore$ $\tau_c = 300/\sqrt{3} = \underline{173\ N/mm^2}$ (for $\sigma_{yw} = 300$)

$= 230/\sqrt{3} = \underline{133\ N/mm^2}$ (for $\sigma_{yw} = 230$)

$d_w = 1380 - 9.5 - 15 = 1355.5$

$\therefore$ $V_D = \dfrac{12.7 \times 1355.5 \times 173 \times 10^{-3}}{1.2 \times 1.1} = \underline{2256\ kN}$ ($\sigma_{yw} = 300$)

$V_D = 2256 \times 133/173 = \underline{1735\ kN}$ ($\sigma_{yw} = 230$)

ASSESSMENT RESISTANCES

$F_c = 0.9$ — Pg 9

R_A^*, $M_D = 0.9 \times 2031 = \underline{\underline{1828\ kNm}}$ ($\sigma_{yc} = 300$)

$M_D = 0.9 \times 1887 = \underline{\underline{1698\ kNm}}$ ($\sigma_{yc} = 230$)

$M_R = 0.9 \times 1405 = \underline{\underline{1265\ kNm}}$ ($\sigma_{yc} = 300$)

$M_R = 0.9 \times 1306 = \underline{\underline{1175\ kNm}}$ ($\sigma_{yc} = 230$)

$V_D = 0.9 \times 2256 = \underline{\underline{2030\ kN}}$ ($\sigma_{yw} = 300$)

$V_D = 0.9 \times 1735 = \underline{\underline{1562\ kN}}$ ($\sigma_{yw} = 230$)

NOTE: Further checks to be carried out for increased shear due to sloping flanges and transverse stresses in flanges due to curvature (top flange) and loading from transverse beams on the bottom flange

7.0 ANALYSIS

7.1 LOADING

DEAD LOAD AND SUPERIMPOSED DEAD LOAD

Depth of fill and surfacing

Centre = 1.83 - 1.15 = 0.68 m

1/4 Span = 1.74 - 0.9 = 0.84 m

Abutment = 1.48 + 0.25 - 0.9 = 0.83 m

2.35

Ave = 2.35/3 = 0.78 Say 0.8 m.

Jack Arch rise = 100 mm

Surfacing = 100 mm

∴ Average fill thickness = 800 - 100 - 50 (half JA rise)

= 650 mm

∴ Concrete fill (plain)

$Q_K = 0.65 \times 2300 \times 9.81/10^3 = \underline{14.67}$ kN/m²

Surfacing

$Q_K = 0.1 \times 2400 \times 9.81/10^3 = \underline{2.35}$ kN/m²

Extra 150×750 strip along inside main girder

$Q_K = 0.15 \times 0.75 \times 2300 \times 9.81/10^3 = \underline{2.54}$ kN/m

Assessment loads

$Q_A^* = \gamma_{fL} \cdot Q_K$ γ_{fL} table 5.1 BD21/84

$= 1.2 \times 14.67 + 1.75 \times 2.35 = \underline{21.72}$ kN/m²

Extra over = 2.54 × 1.2 = $\underline{3.05}$ kN/m

Loadings (cont)

Steel Beam Self weight

Longitudinal Beams

203 × 152 × 52 kg/m $\qquad Q_K = 52 \times 9.81/10^3 = 0.51$ kN/m

$Q_A^* = 1.05 \times 0.34 = \underline{0.54}$ kN/m

Longitudinal Channels

254 × 76 × 29 kg/m $\qquad Q_K = 29 \times 9.81/10^3 = 0.285$

$Q_A^* = 1.05 \times \qquad = \underline{0.3}$ kN/m

Transverse Beams (Double flange plts)

Area $= 2 \times 6710 + 2 \times 3170 + 334 \times 12 = 23768\ mm^2$ $\qquad$ pg 15 + 17

$\therefore Q_K = 23768 \times 10^{-6} \times 7850 \times 9.81/10^3 = 1.83$ kN/m

$Q_A^* = 1.05 \times 1.83 = \underline{1.92}$ kN/m

Transverse Beams (Triple flange plts)

$A = 23768 + 2 \times 305 \times 11 = 30478\ mm^2$

$Q_K = 30478 \times 10^{-6} \times 7850 \times 9.81/10^3 = 2.35$ kN/m

$Q_A^* = 1.05 \times 2.35 = \underline{2.46}$ kN/m

Plate Girders

$A = 38385\ mm^2$ max $\qquad$ pg 24

$Q_K = 38385 \times 10^{-6} \times 7850 \times 9.81/10^3 = 2.96$ kN/m

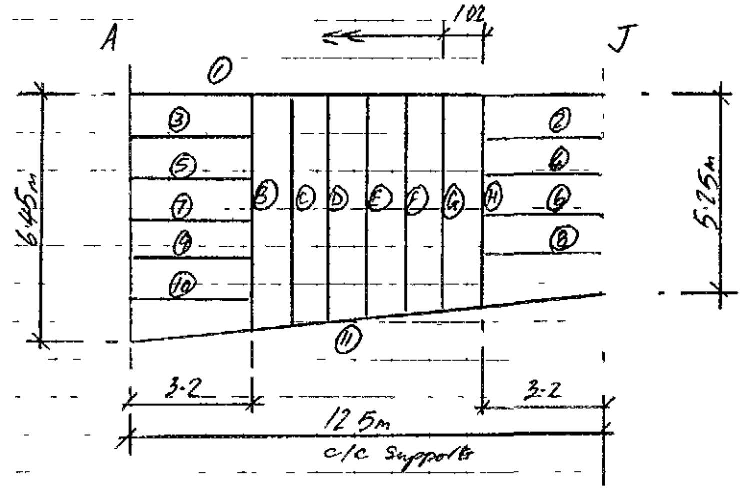
$Q_A^* = 1.05 \times 2.96 = \underline{3.1}$ kN/m

((Conservative as depth reduces to support
but ignores stiffeners)

Loadings (cont.)

Dead & Superimposed loads

On individual beams



Beam spans 2-10 = 2.89 + 0.3 = 3.19 say 3.2 m
(Channels similar)

Beams 1, 11 = 13.1 - 0.3×2 = 12.5 m

Beam H = $5.25 + \frac{3.2}{12.5} \times 1.2$ = 5.55 m

Beam G = $5.55 + \frac{1.02}{12.5} \times 1.2$ = 5.65 m

Beam F = 5.75 m

Beam E = 5.85 m

Beam D = 5.95 m

Beam C = 6.05 m

Beam B = 6.15 m

Channels load width = 1.05/2 - Dead load = 21.72×0.525 + 0.3 = 11.7 kN/m Pg 33

Total load = 3.2×11.7 = 37.4 kN Reactions = 18.7 kN

Beams 2-10 load width = 1.05m

- Dead load - 21.72×1.05 + 0.54 = 23.3 kN/m Pg 33, 34

Total load - 3.2×23.2 = 74.2 kN Reactions - 37.1 kN

Loadings (cont)

Dead load on beams.

Beams C – G

load width = 1.02 m

∴ load/m = 21.72 × 1.02 + 1.92 = 24.1 kN/m

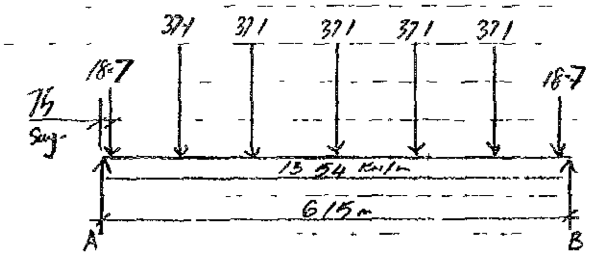
∴ Total loads

Beam C = 6.05 × 24.1 = 145.8 kN		R = 72.9 kN
Beam D = 5.95 × 24.1 = 143.4 kN		R = 71.7 kN
Beam E = 5.85 × 24.1 = 141.0 kN		R = 70.5 kN
Beam F = 5.75 × 24.1 = 138.6 kN		R = 69.3 kN
Beam G = 5.65 × 24.1 = 136.2 kN		R = 68.1 kN

Beam B.

UDL = 21.72 × $\frac{1.07}{2}$ + 2.46 = 13.54 kN/m   Pg 33, 34

Reactions from Beams 3–11 + 2 channels



Total load = 6.15 × 13.54 + 6 × 37.1 + 3 = 83.3 + 222.9 = 306 kN

$R_A = R_B = 153$ kN

Beam H

Total load = 5.55 × 13.54 + 5 × 37.1 = 75.1 + 185.5 = 260.6 kN

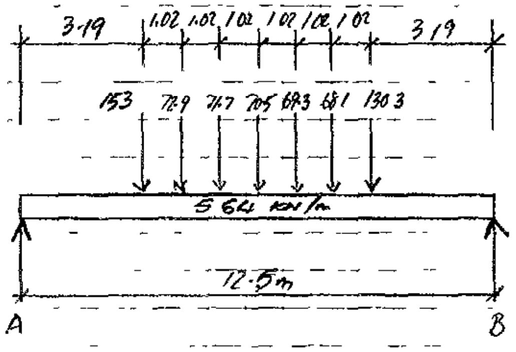
R = 130.3 kN

Loadings (cont)

Dead load on beams.

Plate Girders - beams 1 & 11

$U.D.L = \underset{(e/o)}{2.54} + \underset{(s\ wt)}{3.1} = 5.64$ kN/m Pg 33, 34



$$R_A = \frac{5.64\times12.5}{2} + \frac{1}{12.5}\Big(153\times9.31 + 72.9\times8.29 + 71.7\times7.27 + 70.5\times6.25 + 69.3\times5.23 + 68.1\times4.21 + 130.3\times3.19\Big)$$

$$= 35.25 + 324.44 = \underline{359.7\ kN}$$

$$R_B = 70.5 + 153 + 72.9 + 71.7 + 70.5 + 69.3 + 68.1 + 130.3 - 359.7$$

$$= \underline{346.6\ kN}$$

Loadings (cont)

LIVE LOADING.

i) HA type loading

Span = 12.5m ∴ UDL = $336\left(\frac{1}{12.5}\right)^{0.67}$ = 61.8 kN/m — Fig 7.2 BD 21/84

Carriageway width ~ 5.0 m at narrow end

∴ Two notional lanes 2.5m wide

Lane factor $\beta_1 \beta_2$ = 0.274×2.5 = 0.685

∴ UDL = 61.8×0.685 = 42.3 kN/m /lane

KEL = 120×0.685 = 82.2 kN /lane

Reduction factor for 40 Te assessment = 0.91

∴ UDL Q_K = 42.3×0.91 = 38.5 kN/m

KEL Q_K = 82.2×0.91 = 74.8 kN

Assessment loads

$Q_A^* = \gamma_{fL} \cdot Q_K$ $\gamma_{fL} = 1.5$

UDL Q_A^* = 38.5×1.5 = 57.8 kN/m /lane

KEL Q_A^* = 74.8×1.5 = 112.2 kN /lane

ii) Single Axle loads

Nominal load = 200 kN $\gamma_{fL} = 1.5$ — table 7.3 BD 21/84

Q_A^* = 200×1.5 = 300 kN

iii) Single Wheel loads

Nominal load = 100 kN $\gamma_{fL} = 1.5$

Q_A^* = 100×1.5 = 150 kN

iv) Accidental wheel loads (edge channels)

Nominal $W_1 = 100$ kN $W_2 = 60$ kN $\gamma_{fL} = 1.5$ Table 7.3A BD 21/84

$\therefore Q_A^*$ $W_1 = 150$ kN $W_2 = 90$ kN

IV) Equivalent Axle loads for transverse girders Cl 7 1 BD21/84 & Appendix DS/D & Section 4 BA 16/84

for Bending - Fig 4.6 BA 16/84 Amended Aug 1989

Internal girder Axle load = 76 kN (5.75-6.0 m span)

for shear table 41 BA 16/84

Axle load = 160 kN (1m spacing)

Converted to wheel loads 1.8m apart

for bending = 38 kN each

for shear = 80 kN each

$\therefore Q_A^* = 38 \times 1.5 = \underline{57 \text{ kN}}$ for bending

$= 80 \times 1.5 = \underline{120 \text{ kN}}$ for shear

v) Equivalent axle loads for transverse girders B & H

For internal girder axle load = 76 kN for bending

Residual from 200 kN Axle = 124 kN carried by adjacent beams - 62 kN per beam

for girders B & H only one adjacent beam (transverse)

so half residual carried by girders themselves

$\therefore$ Equiv Axle load = 76 + 62 = 138 kN = 69 kN/wheel

$\therefore Q_A^* = 69 \times 1.5 = \underline{103.5}$ kN for bending

for shear Axle load = $160 + \frac{40}{2} = 180$ kN

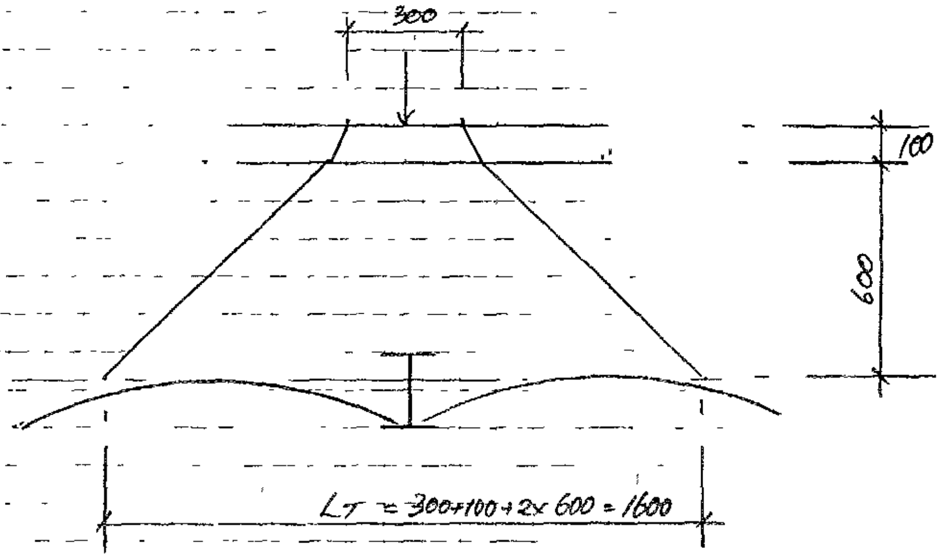
$Q_A^* = 90 \times 1.5 = \underline{135 \text{ kN}}$ wheel load for shear

Prepared by	Date	TAYLOR WOODROW GROUP	Sheet No
[redacted]	May 92	CONTINUATION SHEET	40 of 67
	May 92	Calculation Title BOXTED BRIDGE ASSESSMENT	Calculation No B59/5-A

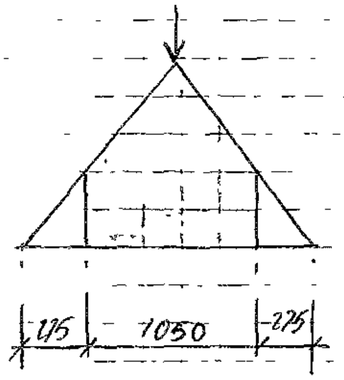
7.2 DISPERSAL OF LIVE LOADING

i) No dispersal of HA type loading.

ii) Single Wheel loads (longitudinal beams)



Idealised as a triangle of loading



Proportion carried by beam

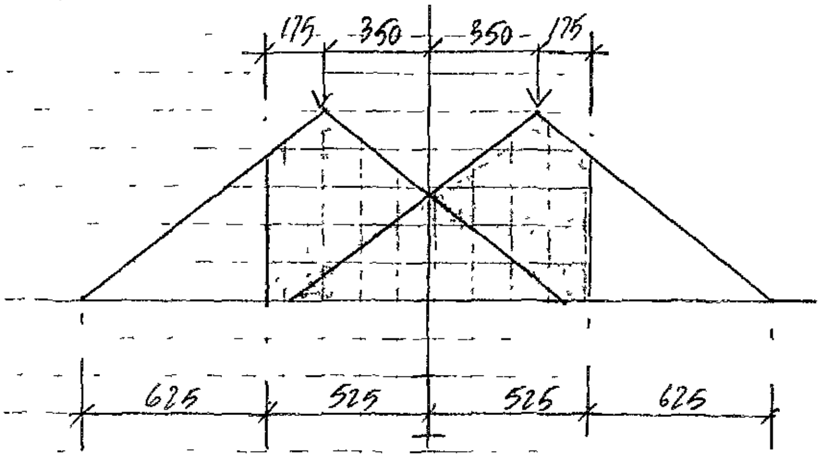
$$\text{Shaded area} = \frac{1600}{2} - \frac{275^2}{800} = 705.5$$

$\therefore$ Proportion of total load $= 705.5/800 = 0.88$

$\therefore$ Wheel loading carried $= 0.88 \times 150 = \underline{132 \text{ kN}}$

Dispersal of loading (cont.)

iii) Adjacent axle loads

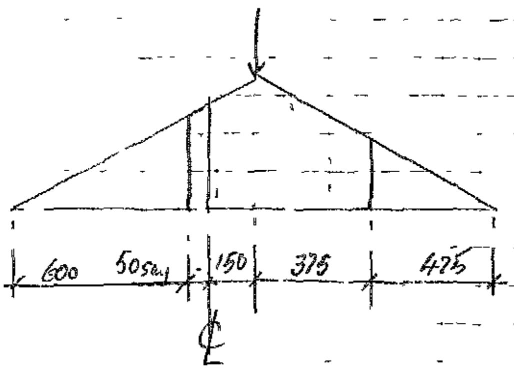


$$\text{Shaded area} = \frac{2 \times 1600}{2} - \frac{625^2}{800} = 1111.7$$

$\therefore$ proportion of load carried $= 1111.7/800 = 1.39$

$\therefore$ load carried $= 150 \times 1.39 = \underline{208.5\ kN}$

iv) Edge Channel. (wheel ℄ within 150 of ℄ of channel)
Accidental loading



$$\text{Shaded area} = 800 - \frac{425^2}{1600} - \frac{600^2}{1600}$$

$$= 462.1$$

$$\text{Truncated area} = 800 - \frac{600^2}{1600} = 575$$

$\therefore$ proportion carried by channel

$$= 462.1/575 = 0.8$$

$\therefore$ load on channel $= 0.8 \times 150 = \underline{120\ kN}$ W_1

$= 0.8 \times 90 = \underline{72\ kN}$ W_2

7.3. MOMENTS & SHEARS.

7.3.1 MOMENTS & SHEARS DUE TO DEAD LOADS

i) Longitudinal beams (Nos 2-10, pg 33)

$M = 74.2 \times 3.2/8 = \underline{29.7 \text{ kN m}}$ pg 35

$V = \underline{37.1 \text{ kN}}$ pg 35

ii) Channels

$M = 37.4 \times 3.2/8 = \underline{15.0 \text{ kN m}}$

$V = \underline{18.7 \text{ kN}}$ pg 35

iii) Transverse Double flange plts (Beams C-G)

Consider Beam C

$M = 145.8 \times 6.05/8 = \underline{110.3 \text{ kN m}}$ pg 36

$V = \underline{72.9 \text{ kN}}$

iv) Transverse Triple flange plts (Beams B & H)

Beam B

$M = 153 \times \frac{6.15}{2} - 18.7 \times 3.0 - 37.1 \times 2.1 - 37.1 \times 1.05 - 13.54 \times \frac{3.075^2}{2}$ pg 36

$= \underline{233.5 \text{ kN m}}$

$V = \underline{153 \text{ kN}}$ pg 36

v) Plate girders

$M = 359.7 \times 6.25 - 3.64 \times \frac{6.25^2}{2} - 153 \times 3.06 - 72.9 \times 2.04 - 71.7 \times 1.02$

$= \underline{1448 \text{ kN.m}}$

$V = \underline{360 \text{ kN}}$

73.2 MOMENTS & SHEARS DUE TO LIVE LOADS

1) LONGITUDINAL BEAMS (Nos 2-10)

a) HA type loading

UDL = 57.8/2.5 = 23.1 kN/m² (2.5m lane width) Pg 38

KEL = 112.2/2.5 = 44.9 kN/m

load width = 1.05m

UDL = 23.1×1.05 = 24.3 kN/m

KEL = 44.9×1.05 = 47.1 kN

∴ Moment $= \frac{24.3 \times 3.2^2}{8} + \frac{47.1 \times 3.2}{4} = 31.1 + 37.7$

$= \underline{68.8 \text{ kN.m}}$

Co-ext Shear 47.1/2 = 23.6 kN

Max Shear = 47.1 kN

b) Adjacent axles

$M = \frac{208.5 \times 3.2}{4} = 166.8$ kN.m Pg 41

Co ext shear = 104.25 kN

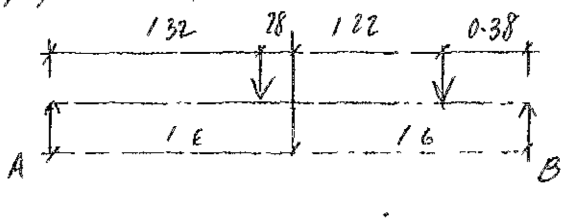
Max shear = 208.5 kN

ii) LONGITUDINAL CHANNELS.

Accident loading

Centroid of loads = $\frac{120}{(120+72)} \times 1.5 = 0.9375$ m from W_2 Pg 41

loading positions for max M



Prepared by	Date	TAYLOR WOODROW GROUP	Sheet No
[redacted]	May 92	CONTINUATION SHEET	44 of 67
	May 92	Calculation Title BOXTED BRIDGE ASSESSMENT	Calculation No 659/5-A

Live load moments + shears (cont.)

$R_A = 120 \times \frac{1.88}{3.2} + 72 \times \frac{0.38}{3.2} = 70.5 + 8.6 = 79.1$ kN

$\therefore M = 79.1 \times 1.32 = 104.4$ kNm

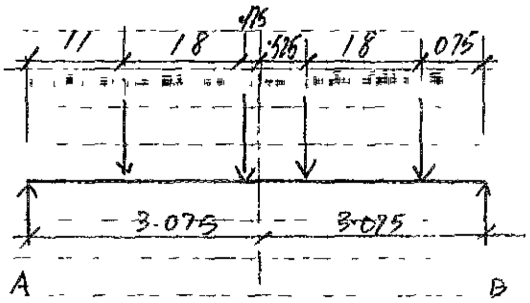
Co-act shear = 79.1 kN

Max shear = $120 + 72 \times \frac{1.7}{3.2} = 158.3$ kN

iii) TRANSVERSE BEAMS (TRIPLE FLANGE PLATES, BEAMS B & H)

for Beam B

Wheel loads 103.5 kN position for worst M — Pg 39



$R_A = \frac{103.5}{6.15}(5.05 + 3.25 + 2.55 + 0.75) = 195.2$ kN

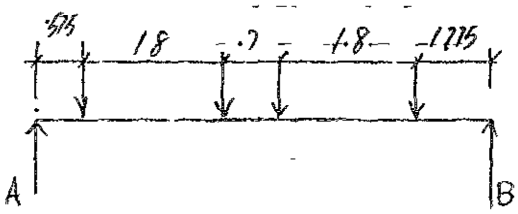
$\therefore M = 195.2 \times 2.9 - 103.5 \times 1.8 = 380.0$ kN m

Co-act shear = $195.2 - 103.5 = 91.7$ kN

Wheel loads 135 kN for worst shear (5m carriageway) — Pg 39

. closest approach to support = $(6.15 - 5.0)/2$

$= 0.575$m



$R_A \; \frac{135}{6.15}(5.575 + 3.775 + 3.075 + 1.275)$

$\therefore V = 300.7$ kN

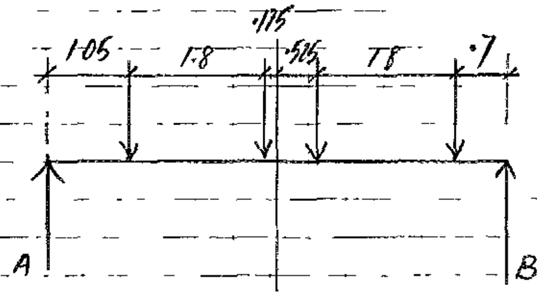
Prepared by	Date	TAYLOR WOODROW GROUP	Sheet No.
[redacted]	May 92	CONTINUATION SHEET	45 of 67
	May 92	Calculation Title BOXTED BRIDGE ASSESSMENT	Calculation No B59/5-A

Live load moments & shears (cont)

iv) TRANSVERSE BEAMS (DOUBLE FLANGE PLATES, BEAMS C TO G)

for Beam C (6.05 m span)

Wheel loads 57 kN positioned for worst M — Pg 39

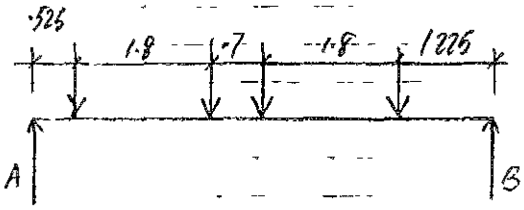


$R_A = \frac{57}{6.05}(5.0 + 3.2 + 2.5 + 0.7) = 107.4$ kN

$\therefore M = 107.4 \times 2.85 - 57 \times 1.8 = \underline{203.5 \text{ kN.m}}$

Co-act shear $= 107.4 - 57 = \underline{50.4 \text{ kN}}$

Wheel loads 120 kN for worst shear (Closest approach 0.525m) — Pg 39



$R_A = \frac{120}{6.05}(5.525 + 3.725 + 3.025 + 1.225)$

$= \underline{267.8 \text{ kN}}$

V) PLATE GIRDERS.

a) HA TYPE LOADING. – TWO LANES CENTRALLY PLACED

UDL = 57.8 kN/m — Pg 38

KEL = 112.2 kN

$\therefore M = 57.8 \times 12.5^2/8 + 112.2 \times 12.5/4 = \underline{1480 \text{ kNm}}$

Co-act shear = $\underline{56.1 \text{ kN}}$

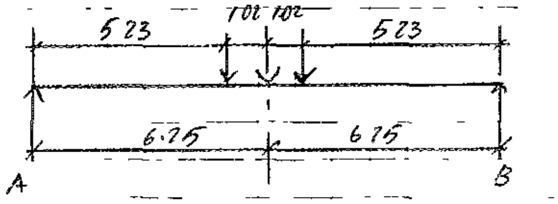
Max shear = $57.8 \times 12.5/2 + 112.2 = \underline{473.5 \text{ kN}}$

b) AXLE LOADING

120 kN wheel loads on transverse beam giving R = 267.8 kN — Pg 45

Allow 15 kN wheel loads on adjacent beams (Total = 150 kN)

∴ Reactions = $267.8 \times 15/120 = 33.5$ kN

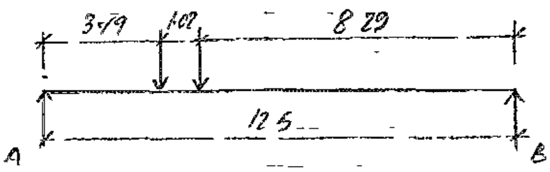


$R_A = 267.8/2 + 33.5 = \underline{167.4 \text{ kN}}$

$\therefore M = 167.4 \times 6.25 - 33.5 \times 1.02 = \underline{1012.1 \text{ kNm}}$

Co-act shear = $167.4 - 33.5 = \underline{133.9 \text{ kN}}$

Max shear Beams B, C, load from B = 300.7 kN C = 33.5 kN adjacent beam — Pg 44+46



$R_A = 300.7 \times \frac{9.31}{12.5} + 33.5 \times \frac{8.29}{12.5}$

$= \underline{246 \text{ kN}}$

7.3.3 COMBINED DEAD LOAD & LIVE LOAD

i) LONGITUDINAL BEAMS

HA type.

M = 29.7 + 68.8 = 98.5 kNm — Pg 42 & 43

Co-act Shear = 23.6 kN

Max Shear V = 37.1 + 47.1 = 84.2 kN

Axle loads

M = 29.7 + 166.8 = 196.5 kNm — Pg 42 & 43

Co-act Shear = 104.3 kN

Max Shear = 37.1 + 208.5 = 245.6 kN

ii) CHANNELS (Accidental Wheel loads)

M = 15.0 + 104.4 = 119.4 kNm — Pg 42, 44

Co-act Shear $79.1 + \frac{37.4}{3.2} \times 0.28 = 82.3$ kN

Max Shear = 18.7 + 158.3 = 177.0 kN

iii) TRANSVERSE DOUBLE PLATE BEAMS (Equivalent Axle loads)

M = 110.3 + 203.5 = 313.8 kNm. — Pg 42 & 45

Co-act Shear $50.4 + \frac{145.8}{6.05} \times 0.175 = 54.6$ kN

Max Shear = 72.9 + 267.8 = 340.7 kN — Pg 42 & 46

iv) TRANSVERSE TRIPLE PLATE BEAMS (Equivalent Axle loads)

M = 233.5 + 380.0 = 613.5 kNm — Pg 42 & 44

Co-act Shear 91.7 + 13.54 × .175 = 94.1 kN — Pg 36 & 44

Max Shear = 153 + 300.7 = 453.7 kN

Combined Dead & Live loads (cont)

v) PLATE GIRDERS

HA type

M = 1448 + 1480 = 2928 kN m — Pg 42 & 46

Co-act shear 70.5 + 56.1 (DL) = 126.6 kN — Pg 46

Max shear = 360 + 473.5 = 833.5 kN

Axle loading

M = 1448 + 1012 = 2460 kN m — Pg 46 & 42

Co-act shear 70.5 + 133.9 = 204.4 kN — Pg 46

Max Shear = 360 + 246 = 606 kN — Pg 42 & 46

7.3.4 SUMMARY OF BEAM CAPABILITIES TO CARRY 40Te ASSESSMENT

		σ_y = 300	230	
i) LONGITUDINAL BEAMS	HA Type	Y	N	Pg 13 & 47
	Axle	N	N	
ii) CHANNELS	Accident	N	N	Pg 14 & 47
iii) TRANSVERSE DOUBLE PLATE BEAMS	Axle	Y	Y	Pg 18 & 47
iv) TRANSVERSE TRIPLE PLATE BEAMS	Axle	Y	N (Shear)	Pg 17 & 47
v) PLATE GIRDERS	HA type	N	N	Pg 30 & 48
	Axle	N	N	

Most critical failures :-

Plate Girders HA & Channels Accident (very conservative)

Prepared by	Date	TAYLOR WOODROW GROUP	Sheet No
[redacted]	May 92	CONTINUATION SHEET	49 of 67
	May 92	Calculation Title BOXTED BRIDGE ASSESSMENT	Calculation No B59/5-A

8.0 REDUCED LOADING ASSESSMENT

Check external plate girders for a reduced loading group

M_D = 1828 kNm (σ_{yc} = 300) Pg 32

M_D = 1698 kNm (σ_{yc} = 230)

Moment due to dead load = 1448 kN·m Pg 42

∴ Moment available for live load

= 1828 - 1448 = 380 kN·m. (σ_{yc} = 300)

= 1698 - 1448 = 250 kNm (σ_{yc} = 230)

Live load moment - 40 te assessment

HA type = 1480 kNm

∴ Reduced loading

Reduction coeff k = $\frac{380}{1480}$ × 0.91 = 0.23 (σ_{yc} = 300)

k = $\frac{250}{1480}$ × 0.91 = 0.15 (σ_{yc} = 230)

3 te loading group k = 0.2 BD 21/84 Fig 73

Not acceptable for σ_{yc} = 230 N/mm²

Live load moment - 40 te assessment

Axle loading = 1012 kNm

∴ Reduced axle load = $\frac{380}{1012}$ × 100 = 37.5 kN (σ_{yc} = 300)

(→ FE Group 2 30 kN)

= $\frac{250}{1012}$ × 100 = 24.7 " (σ_{yc} = 230)

→ 3te group (25 kN)

Reduced loading assessment (cont)

MOMENTS & SHEARS FOR 3 Te LOADING

i) LONGITUDINAL BEAMS

AXLE LOADING $M = 29.7 + \frac{166.8 \times 25}{100} = \underline{71.4 \text{ kNm}}$

Co-act shear $= 104.3 \times 0.25 \quad \underline{26.1 \text{ kN}}$

Max Shear $= 37.1 + 208.5 \times .25 = \underline{89.2 \text{ kN}}$

ii) CHANNELS

Nominal loads $W_1 = 25 \quad W_2 = 0$

loads to beams $= 0.8 \times 25 \times 1.5 = 30 \text{ kN}$

Moment $= \frac{30 \times 3.2}{4} = \underline{24 \text{ kN m}}$

Co-act Shear $= \underline{15 \text{ kN}}$

Max shear $= \underline{30 \text{ kN}}$

With D L

$M = 15.0 + 24.0 = \underline{39.0 \text{ kN m}}$

Co-act shear $= \underline{15.0 \text{ kN}}$

Max shear $= 18.7 + 30.0 = \underline{48.7 \text{ kN}}$

iii) TRANSVERSE BEAMS

OK for 40 Te Assessment Don't check for 3 Te

Reduced loading assessment (cont)

(iv) PLATE GIRDERS.

HA type

$M = 1448 + \frac{1480 \times 0.20}{0.91} = \underline{1773}$ kN m

Co-act shear $= 70.5 + \frac{56.1 \times 0.20}{0.91} = \underline{83}$ kN

Max Shear $= 360 + \frac{473.5 \times .20}{.91} = \underline{464}$ kN

Axle loading

$M = 1448 + 1012 \times 0.25 = \underline{1701}$ kN m

Co-act shear $70.5 + 133.9 \times .25 = \underline{104}$ kN

Max shear $= 360 + 246 \times 0.25 = \underline{422}$ kN

9.0 ADDITIONAL CHECKS ON PLATE GIRDERS FOR 3TE LOADING

9.1 YIELDING OF FLANGE PLATES

Cl. 9.10.2 BS 5400 Pt 3

$$\sigma_f^2 + \sigma_2^2 - \sigma_f \sigma_2 + 3\tau^2 \leq \left(\frac{\sigma_{yf}}{\gamma_m \gamma_{f3}}\right)^2$$

σ_f = longitudinal stress

σ_2 = transverse stress - from curvature of top flange

No torsion $\therefore$ $\tau = 0.5\tau_2$ $\quad$ τ_2 = shear stress @ web junction due to shear force on beam

$$= \frac{VA\bar{y}}{Ib}$$

for compression flange

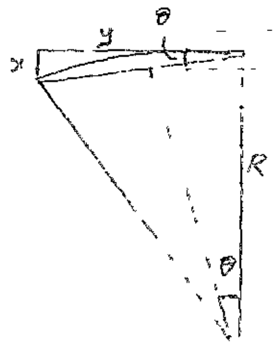
$$\sigma_f = \frac{M}{Z_{xc}} = \frac{1773 \times 10^6}{196 \times 10^7} = 90.5 \text{ N/mm}^2$$ Pg 26

$$\sigma_2 = \frac{3\sigma_f b_{fo}^2}{R_f t_{fo}}$$ b_{fo} = outstand $\quad$ t_{fo} = thickness $\quad$ Cl. 9.5.7 BS 5400

R_f = radius of curvature

b_{fo} from rivet line = $\frac{305}{2} - 63 = 89.5$ mm $\quad$ $t_{fo} = 9.5$ mm

R_f =



$\theta = \tan^{-1}\frac{x}{y}$ $\quad$ $R = \frac{x}{2\sin^2\theta}$

$x = 1820 - 1380 = 440$ mm

$y = 13100/2 = 6550$

$\theta = 3.84°$ $\quad$ $R = \underline{48973}$ mm

Yielding of flanges (cont)

$$\therefore \sigma_2 = \frac{3 \times 90.5 \times 89.5^2}{48973 \times 9.5} = 4.7 \text{ N/mm}^2$$

Shear at Mid span = 83 kN — Pg 51

+ Additional shear due to slope of flange.

Assume slope = 440 mm in 6550 mm

Force in compression flange

Area = 7607.5 mm² σ_f = 90.5 — Pg 30

$\therefore$ Force = $90.5 \times 7607.5 \times 10^{-3}$ = 688 kN

Vertical component = $\frac{688 \times 440}{6550}$ = 46.2 kN

Total shear force = 83 + 46.2 = 129.2 kN

Centroid of compression flange = 27.9 mm from top — Pg 30

$\therefore$ from N.A = 887.4 − 27.9 = 859.5 mm — Pg 20+26

$I = 17.29 \times 10^9$ mm⁴ — Pg 26

$$\tau_t = \frac{VA\bar{y}}{Ib} = \frac{129.2 \times 10^3 \times 7607.5 \times 859.5}{17.29 \times 10^9 \times 30.5} = 0.16 \text{ N/mm}^2$$

$\tau = 0.5\,\tau_t = 0.08$

$$\sigma_f^2 + \sigma_2^2 - \sigma_f\sigma_2 + 3\tau^2 = 90.5^2 + 4.7^2 + 90.5 \times 4.7 + 3 \times 0.08^2$$

$= 8637$

$$\left(\frac{\sigma_{yf}}{\gamma_m \gamma_{f3}}\right)^2 = \left(\frac{230}{1.1 \times 1.1}\right)^2 = 30360 > 8637 \quad \text{OK}$$

Prepared by	Date	TAYLOR WOODROW GROUP	Sheet No
[redacted]	May 92	CONTINUATION SHEET	54 of 67
	May 92	Calculation Title BOXTED BRIDGE ASSESSMENT	Calculation No B59/5-A

9.2 RIVET CAPACITY

$$\text{Shear flow} = \frac{VA\bar{y}}{\bar{I}}$$

Max V = 464 kN   B51

A = 7607.5 mm²   B30.

I at support   D = 1380 mm

Section	Area	Centroid	A x	x̄ - x	A(x̄ - x)²	Self I
Channel Web	2897.5	4.75	13763.1	661.05	1.27 E9	21792
Channel flanges	1310.0	42.25	55347.5	623.55	0.51 E9	468352
Upper Angle top legs	2000.0	14.50	29000.0	651.30	0.85 E9	16666
Upper Angle bott legs	1400.0	54.50	76300.0	611.30	0.52 E9	571666
Web	1355.5×12.7 = 17215.0	687.25	11831009.0	21.45	0.01 E9	2.64 E9
Lower Angle top legs	1400.0	1320.0	1848000.0	654.20	0.60 E9	571666
Lower Angle bott legs	2000.0	1360.0	2720000.0	694.20	0.96 E9	16666
Bottom plt	4575.0	1372.5	6279187.5	706.70	2.28 E9	85781
Bottom rivets (1)	-900.0	1367.5	-1230750.0	701.7	-0.44 E9	neg
" " (2)	-588.6	1320.0	-776952.0	654.2	-0.25 E9	neg
	Σ=31309		Σ=20844905		Σ=6.31 E9	Σ=2.64 E9

$$\bar{x} = 20844905 / 31309 = 665.8$$

$$\text{Total } I_{xx} = (6.31 + 2.64) \times 10^9 = \underline{8.95 \times 10^9 \text{ mm}^4}$$

$$\bar{y} = 665.8 - 22.9 = \underline{642.9 \text{ mm}}$$

$$\therefore \frac{VA\bar{y}}{\bar{I}} = \frac{464 \times 10^3 \times 7607.5 \times 642.9}{8.95 \times 10^9} = 254 \text{ N/mm}$$

Rivet capacity (cont)

Rivets @ 100 crs load/rivet = 25 4 kN

Cl 14.5 3.4 BS5400

$$\tau = \frac{V}{n A_{eq}}$$ A_{eq} for 18mm rivet = $\frac{\pi \times 18^2}{4} = 254 mm^2$

$n = 2$ for double shear

$$\tau = \frac{25.4 \times 10^3}{2 \times 254} = 50.0 \; N/mm^2$$

$$\frac{\sigma_q}{\gamma_m \gamma_{f3} \sqrt{2}} = \frac{300 \times 0.85}{1.1 \times 1.2 \times 1.414} = 136.6 N/mm^2 \text{ for } \sigma_{yc} = 300$$

$$= \frac{136.6 \times 230}{300} = 104.7 \; N/mm^2 > 50.0 \; N/mm^2 \text{ OK}$$

Prepared by [redacted]

Date: May 92 / May 92

TAYLOR WOODROW GROUP

CONTINUATION SHEET

Calculation Title: BOXTED BRIDGE ASSESSMENT

Calculation No: 559/5A

9.3 YIELDING OF BOTTOM FLANGE PLATE. 38t LOADING

At mid-span

Load from transverse beam at 1m c/s approx

DL = 72.9 kN

LL = 267.8 × 0.17 (0.17 = reduction factor table 4.2 BA 16/84 Amd No.1) Pg 46

= 45.5 kN

∴ Total reaction to bottom flange

= 72.9 + 45.5 = 118.4 kN

D.L. Moment = 1448 kN m Pg 42

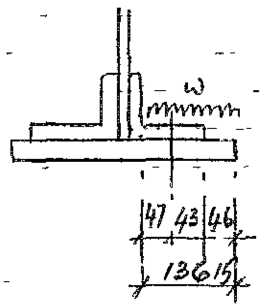
LL moment = 1012.1 × 0.17 = 172 kN m Pg 46

Total M = 1448 + 172 = 1620 kN m

longitudinal stress

$\sigma_f = \frac{M}{Z_{xT}} = \frac{1620 \times 10^6 \times 937.6}{17.29 \times 10^9} = 87.85$ N/mm² Pg 29

$\left(Z_{xT} = \frac{I_{x-x}}{y_T} = \frac{17.29 \times 10^9}{937.6} \right)$



Transverse stress

$w = \frac{118.4 \times 10^3}{136.5} = 869.6$ N/mm /m

Moment about rivet

$= \frac{w \times 96^2}{2} = \frac{869 \times 96^2}{2} = 4.00$ kN m /m

I for angle + plate

$= \frac{10^3 \times 10^3}{12} + \frac{10^3 \times 15^3}{12} = 0.36 \times 10^6$ mm⁴

$Z = I/y = \frac{0.36 \times 10^6}{12.5} = 2.88 \times 10^4$ mm³

$\therefore \sigma_2 = \frac{4.00 \times 10^6}{2.88 \times 10^4} = 139$ N/mm²

[redacted]

Date	TAYLOR WOODROW GROUP	Sheet No
May 92	CONTINUATION SHEET	57 of 67
May 92	Calculation Title BOXTED BRIDGE ASSESSMENT	Calculation No 859/5-A

Bottom flange (cont.)

τ negligible

$$\sigma_f^2 + \sigma_2^2 - \sigma_f \sigma_2 = 87.85^2 + 138^2 + 87.85 \times 138$$

$$= 38885$$

$$\left(\frac{\sigma_{yf}}{\gamma_m \gamma_{f3}}\right)^2 = \left(\frac{300}{1.1 \times 1.2}\right)^2 = 51653 > 38885 \quad \text{OK.}$$

$(\sigma_{yf} = 300\ N/mm^2)$

for $\sigma_{yf} = 230 \quad = \left(\frac{230}{1.1 \times 1.2}\right)^2 = 30360 < 38885$

However beam is over-stressed in bending for $\sigma_{yc} = 230\ N/mm^2$

9.4 RESTRAINT AT SUPPORT

Ref Cl. 9.12.4 BS5400

Bearing stiffener must be capable of carrying two equal & opposite forces F_R at beam flange levels

F_R is greater of F_1 or F_2

$$F_1 = \frac{\alpha_{LT}\, E\, D\, t_{f\,max}\, R_v\, \sigma_{fc}}{875\, W\, (\sigma_{ci} - \sigma_{fc})}$$

α_{LT} from fig 24 dependent on l_e/r_y

$l_e/r_y = 5895/51.8 = 113.8$ ∴ $\alpha_{LT} = 0.0023$ fig 24 & 28

$D = 1380$ $t_f = 9.5$ mm.

$R_v = 464$ kN $W = 2 \times 464 = 928$ kN

$\sigma_{fc} = \dfrac{M_1}{Z_{xc}} = \dfrac{1773 \times 10^6}{1.96 \times 10^7} = 90.5$ N/mm² fig 51 & 26

σ_{ci} from clause 9.12.2.2 $= \dfrac{\pi^2 E S}{\lambda_{LT}^2}$

$S = D/2y_c = \dfrac{1820}{2 \times 937.65} = 0.97$

$\lambda_{LT} = 105.2$ fig 29

∴ $\sigma_{ci} = \dfrac{\pi^2 \times 2.05 \times 10^5 \times 0.97}{105.2^2} = 177.3$ N/mm²

∴ $F_1 = \dfrac{0.0023 \times 2.05 \times 10^5 \times 1380 \times 9.5 \times 464 \times 90.5}{875 \times 928\ (177.3 - 90.5)}$

$= \underline{36.8 \text{ kN}}$

9.4 Support restraint (cont)

$F_2 = 2(F_u + F_c)$

F_u derived in Cl. 9.12.2.2

F_c derived in Cl. 9.12.2.3(a)

$F_u = \left(\frac{\sigma_{fc}}{\sigma_{ci} - \sigma_{fc}}\right)\frac{l_e}{667\delta}$ but not greater than $\left(\frac{\sigma_{fc}}{\sigma_{ci} - \sigma_{fc}}\right)\frac{E I_c}{16.5 l_e^2}$

$\frac{\sigma_{fc}}{\sigma_{ci} - \sigma_{fc}} \cdot \frac{l_e}{667\delta} = \frac{90.5 \times 5895}{(177.3 - 90.5) \times 667 \times 124 \times 10^{-3}} = 7.43$ kN

$\left(\text{compare} \left(\frac{90.5}{177.3 - 90.5}\right) \times \frac{2.05 \times 10^5 \times 59.6 \times 10^6}{16.5 \times 2.04^2 \times 10^6} = 184\text{ kN}\right)$

$F_c = \frac{3 E I_1 \theta}{d_2^2}$

θ = rotation of transverse beam neglecting interaction with vertical stiffener

$= \frac{\frac{1}{2}\Sigma A}{E I_2}$ where ΣA is area of Bm diag.

for UDL area of Bm diag $= \frac{2}{3} ML$

$F_c = \frac{3 E I_1 \times \frac{2}{3} M \cdot L}{d_2^2 \times 2 E I_2} = \frac{M \cdot L}{d_2^2} \frac{I_1}{I_2}$

Moment on transverse beam when stress in main girder $= \sigma_{fc}$

DL = 110.3 kNm — Pg 42

LL HA $= \frac{23.1 \times 0.2}{0.91} = 5.1$ kN/m^2 UDL

$= \frac{44.9 \times 0.2}{0.91} = 9.9$ kN/m — Pg 43

Spacing 1.02 m span 6.05 m UDL $= 5.1 \times 1.02 = 5.2$ kN/m

Support restraint (cont)

Transverse beam

$M = (99+52) \times \frac{6.05^2}{8} = 69.1$ kN.m

total $M = 69.1 + 110.3 = 179.4$ kN.m

for I_2 (for deflection) add steel I + I_{for} concrete deck.

$I_{steel} = 603.61 \times 10^6$ mm^4 (pg 28)

$I_{conc} = 2.29 \times 10^7 \times 1020$ mm^4 (pg 25)

in steel units $= 2.29 \times 102 \times 10^{10} \times \frac{14000 \ (E_c)}{205000 \ (E_s)}$

$= 1.595 \times 10^9$ mm^4

$\therefore I_2 = (1.595 + 0.604) \times 10^9 = 2.2 \times 10^9$ mm^4

$I_1 = \cancel{4.66 \times 10^6 \text{ mm}^4}$ $\quad d_2 = 1622.25$ pg. 28

$\therefore F_c = \frac{179.4 \times 10^6 \times 6050 \times 4.66 \times 10^6}{1622.25^2 \times 2.2 \times 10^9} = 0.87$ kN

$\therefore F_2 = 2(7.43 + 0.87) = 16.6$ kN

$F_R = F_1 = 36.8$ kN

Check U-frame strength

$F_u + F_c = 8.3$ kN

Moment on vertical stiffener $F d_2 = 8.3 \times 1.62 = 13.4$ kN m

$I_1 = 4.66 \times 10^6$ mm^4 $\quad y = \left(75 + \frac{12.7}{2}\right) = 81.35$ mm

$\sigma = \frac{13.4 \times 10^6 \times 81.35}{4.66 \times 10^6} = 234$ N/mm^2

Yielding of stiffener cl 9.13.5.2

$\sigma \not> \frac{\sigma_{ys}}{\gamma_m \gamma_{f3}} = \frac{300}{1.1 \times 1.2} = 227$ N/mm^2

Slightly overstressed.

Support Restraint (cont.)

End Plate = 305 × 9.5 mm at top flange

= 465 × 9.5 mm at bottom flange

Treat as cantilever from bottom flange level

Cl 9.12.4.2 (a)

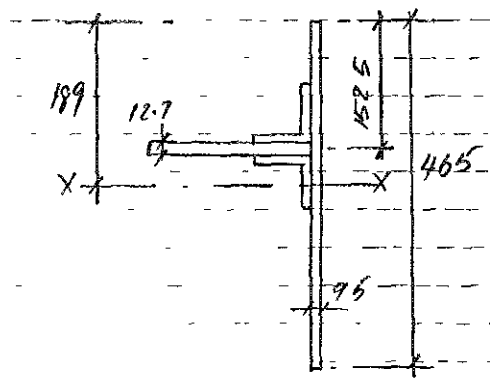
$$I_x \geq 0.67\ \alpha_{LT}\ D^3\ t_{f\,max}\ \frac{R_v}{W}$$

$\alpha_{LT} = 0.0023$ $\qquad$ $R_v/W = 0.5$ $\qquad$ B58

$\therefore I_x \geq 0.67 \times 0.0023 \times 1380^3 \times 9.5 \times 0.5$

$\geq 1.92 \times 10^7\ mm^4$

Effective section of Stiffener



Angles 75 × 75 × 9.5 mm

web section 16 tw = 16×12.7 = 203.2 mm

$$N.A = \frac{\left(\frac{465^2}{2} \times 9.5 + 203.2 \times 12.7 \times 152.5 + 75 \times 9.5 \times 108.65 + 75 \times 9.5 \times 116.35 + 65.5 \times 9.5 \times 141.4 + 65.5 \times 9.5 \times 163.6\right)}{(465 \times 9.5 + 203.2 \times 12.7 + 2 \times 75 \times 9.5 + 2 \times 65.5 \times 9.5)}$$

$= 1827857 / 9667.6 = 189$ mm

$$\therefore I_{xx} = \frac{9.5 \times 465^3}{12} + 9.5 \times 465 \times 43.5^2 + 75 \times 9.5 \times 80.15^2 + 203.2 \times 12.7 \times 36.5^2 + 65.5 \times 9.5 \times (47.6^2 + 25.4^2)$$

$= (7.96 + 0.84 + 0.46 + 0.34 + 0.18) \times 10^7$

$= \underline{9.78 \times 10^7\ mm^4} > \underline{1.92 \times 10^7\ mm^4}$ Ok.

Support Restraint (cont.)

Moment due to forces F_R

$= F_R \times \left(1380 - \frac{95}{2} - \frac{15}{2}\right)$

$= 36.8 \times 1367.75 \times 10^{-3}$ kN m

$= 50.3$ kN.m

$\therefore \sigma = \frac{My}{I}$ $y = 465 - 189 = 276$ mm

$\therefore \sigma = \frac{50.3 \times 10^6 \times 276}{9.78 \times 10^7} = \underline{142 \text{ N/mm}^2}$

$\sigma_S = \frac{300}{1.1 \times 1.2} = 227 \text{ N/mm}^2 > \underline{142 \text{ N/mm}^2}$ OK

or $\frac{230}{1.1 \times 1.2} = 174 \text{ N/mm}^2 > \underline{142 \text{ N/mm}^2}$ OK

10.0 UNITY RATIOS

Ref. BS 5400 Pt 3 Cl 9.9.3

Combined Bending & Shear

Cl. 9.9.3.2

$V \leq V_D$ for $M < 0.5 M_R$

$M \leq M_D$ for $V < 0.5 V_R$

Where V is shear force at any section of beam

M is co-acting bending moment at same point

V_D is the shear capacity

M_D is the bending resistance

V_R is V_D for $m_{fw} = 0$

M_R is bending resistance due to flanges alone

Also if $M \leq M_R$ and $2V \leq V_D$, $M/M_D \leq 1$

if $M > M_R$ and $2V > V_D$

$$M/M_D + \left(1 - M_R/M_D\right)\left(\frac{2V}{V_D} - 1\right) \leq 1$$

Prepared by	Date	TAYLOR WOODROW GROUP	Sheet No
[redacted]	May 92	CONTINUATION SHEET	64 of 67
	May 92	Calculation Title BOXTED BRIDGE ASSESSMENT	Calculation No B59/5-A

ELEMENT	SIZE	CLASS	MD KN m	MR KN m	VD KN	VR KN	MAX M KN·m	CO-ACT V KN	MAX V KN	CO-ACT M KN m	UNITY RATIOS MAX M	UNITY RATIOS CO-ACT V	UNITY RATIOS MAX V	UNITY RATIOS CO ACT M	COMMENTS
LONGITUDINAL RSJ	203 D × 152 W web 11.2 fl 15.2 × 52 kg/m	COMPACT	105.8	88.7	268.2	268.2	98.5	23.6	84.2	0	0.93	0.09	0.31	0	σ_{lc} = 300 40Te Assess
							196.5	104.3	245.6	0	1.86	0.39	0.91	0	40Te Axle
							71.4	26.1	89.2	0	0.67	0.10	0.33	0	3Te Axle
			81.1	68.0	206.1	206.1	98.5	23.6	84.2	0	1.21	0.11	0.41	0	σ_{lc} = 230 40Te Assess
							196.5	104.3	245.6	0	2.42	0.51	1.19	0	40Te Axle
							71.4	26.1	89.2	0	0.88	0.13	0.43	0	3Te Axle
LONGITUDINAL CHANNEL	254 d × 76 w × 29 kg/m Web 8.1 fl 11.4	COMPACT	36.2	23.9	202.2	202.2	119.4	82.3	177.0	0	3.30	0.40	0.88	0	σ_{lc} = 300 40Te Accident
							39.0	15.0	48.7	0	1.08	0.07	0.24	0	3Te Accident
			27.7	18.3	155.0	155.0	119.4	82.3	177.0	0	4.31	0.53	1.14	0	σ_{lc} = 230 40Te Accident
							39.0	15.0	48.7	0	1.41	0.10	0.31	0	3Te Accident

Prepared by [redacted]

Date: May 92 / May 92

TAYLOR WOODROW GROUP

CONTINUATION SHEET

Calculation Title: BOLTED BRIDGE ASSESSMENT

Sheet No. 65 of 67

Calculation No B59/5-A

ELEMENT	SIZE	CLASS	MD KNm	MR KNm	VD KN	VR KN	MAX M KNm	CO-ACT V KN	MAX V KN	COACT M KNm	UNITY RATIOS MAX M	UNITY RATIOS COACT V	UNITY RATIOS MAX V	UNITY RATIOS COACT M	COMMENTS
TRANSVERSE BEAMS DOUBLE FLANGE PLATES	378 D x305 w 22mm fl 12.0 web.	COMPACT	592.7	383.2	499.0	499.0	313.8	54.6	340.7	0	0.53	0.11	0.68	0	σ_{lc} = 300 40Te AXLE.
			464.4	293.6	382.6	382.6	313.8	54.6	340.7	0	0.67	0.14	0.89	0	σ_{lc} = 230 40Te AXLE
TRANSVERSE BEAMS TRIPLE FLANGE PLATES	400 D x305 w 33mm fl. 12.0 web	COMPACT	805.4	592.0	499.0	499.0	613.5	94.1	453.7	0	0.76	0.19	0.91	0	σ_{lc} = 300 40Te Axle
			617.5	454.0	382.6	382.6	613.5	94.1	453.7	0	0.99	0.25	1.18	0	σ_{lc} = 230 40Te Axle.
PLATE GIRDERS	1820d max x305w web 12.7 1380d min	NON COMPACT	1828	1265	2030	2030	2928	126.6	833.5	0	1.60	0.06	0.41	0	σ_{lc} = 300 40Te Assess
							2460	204.4	606	0	1.35	0.10	0.30	0	40Te Axle
							1773	83	464	0	0.97	0.04	0.23	0	3Te group
			1698	1175	1562	1562	2928	126.6	833.5	0	1.72	0.08	0.53	0	σ_{lc} = 230 40Te Assess
							2460	204.4	606	0	1.45	0.13	0.39	0	40Te Axle
							1773	83	464	0	1.04	0.05	0.30	0	3Te group

M NO TW45

11.0 CONCLUSIONS

The calculations show that the bridge is not capable of carrying the 40Te assessment live loading, chiefly because of the weakness of the main plate girders. The bridge has been assessed to be able to carry only 3te loading and even that is with reservations. To achieve this it was necessary to adopt the steel yield stress of 300 N/mm², the test value. With the lower bound stress of 230 N/mm², 3te loading cannot be carried except possibly by limiting the bridge to a single lane.

Additionally assumptions regarding the determination of the effective length of the structure have been necessary. A small enough effective length and subsequently adequate limiting stress in the top flange can only be achieved by assuming that the transverse beams and double-T web stiffeners to the plate girder act as U-frames restraining the top flange. The connection detail between transverse beams and stiffeners is unknown but without the assumption of U-frames the bridge girders are shown to be quite inadequate for dead loading alone - which does not seem to be the case. For the 3te loading the stiffeners are marginally overstressed, but the assistance of alternate transverse beams and single T stiffeners has been ignored.

There is a fair degree of corrosion to the structure

Prepared by	Date	TAYLOR WOODROW GROUP	Sheet No
[redacted]	May 92	CONTINUATION SHEET	67 of 67
	May 92	Calculation Title BOXTED BRIDGE ASSESSMENT	Calculation No B59/5-A

which as contributed to its reduced loading capacity. The longitudinal channels in each corner of the structure are badly corroded which has resulted in a considerable reduction of load capacity. Consequently they are marginally overloaded under 3te accident loading but the spread of loads is conservatively estimated and the overstress not considered significant.

It is recommended that the bridge be restricted to 3te loading and monitored with a view to introducing a single lane if necessary.

APPENDIX B

RESULTS OF TESTING

CITY CONSULTANCY SERVICES
CITY UNIVERSITY
LONDON EC1V OHB

<u>Results of Mechanical Tests on Bridge Samples</u>
<u>Samples Supplied by Taywood Engineering Ltd</u>

NAME OF SAMPLE: 7. BOXTED

(a) Tensile Test

A longitudinal sample was cut from the plate supplied. A shouldered specimen was used with dimensions as follows:

Reduced parallel section diameter (nominal) 5mm
Reduced parallel section length (nominal) 34mm
Gauge Length (for Elongation) 25mm
Extensometer Gauge Length 10mm

Test conditions - cross head speed for yield point 1mm/min
cross head speed for tensile strength 5mm/min

Methods generally were in compliance with BS EN 10 002-1 1990 and all measurements were fully traceable.

RESULTS

Lower Yield Strength MPa R_{eL}	Tensile Strength MPa R_m	Elongation %
300	450	30

(b) Hardness

A Vickers Hardness machine was used to measure hardness on a ground surface of a sample cut from the plates supplied. A 30kg load was employed and the machine and diamond are fully traceable.

RESULTS

Hv
128
128
133
130
134

Mean = 131

Technical and Quality Manager: J.D.R. Walker BSc, PhD, MIM

Signature .... Date 4/3/92