Rail Property LTD / Shropshire County Council

# BE3/73 Assessment Report

**Status of report :**

| | |
|---|---|
| Interim Draft | |
| Final Draft | |
| Final | ✓ |

**Bridge Name :** **Rowe**
**Bridge No. :** **GNQ4/14**
**Road :** **Unclassified**
**Location :** **5km East of Ellesmere**
**OS Ref :** **SJ 450 357**


**Report Prepared for and On Behalf of Shropshire County Council:**
Babtie Group Limited
Springfield
Maidstone
Kent ME14 2LQ

**Client:**

Rail Property Limited
Room C5
Hudson House
York YO1 6HP


Signed [redacted] ..........................
[redacted]
Technical Director, Structures

Signed [redacted]

Doc Ref: GNQ4/14.AssRep

# Contents

## BE3/73 PRINCIPLES

This report should be read in conjunction with the BD21/97 Assessment and Inspection Report which provides details relating to the current condition of the structure.

The following BE3/73 assessment has been undertaken using the principles given in Approval in Principle signed by [redacted] on 8 February 2000.

The main principles adopted are as follows :-

1. Only those elements of the structure which have failed to achieve a 40 tonne Assessment Live Loading capacity to BD21/97 have been re-assessed to BE3/73.

2. No computer analysis has been undertaken, only simple analysis methods have been used.

3. No pedestrian live loads or accidental wheel loads were required to be applied to the verges.

4. A Category 1 assessment check has been undertaken.

## BE3/73 RESULTS

The following elements were assessed using BE3/73 and the results are shown below :-

| | | |
|---|---|---|
| **Internal Beams** : | Bending under Dead Load & Live Load (24 ton vehicle) | Bending at midspan - **PASS**<br>Bending at ¼ span - **FAIL (10t)**<br>Max. shear - **PASS** |
| **Edge Beams :** | Bending under Dead Load Alone | Bending at midspan - **PASS**<br>Bending at ¼ span - **PASS**<br>Max. shear - **PASS** |

**Substructure :** The substructure is in very poor condition. A qualitative assessment would indicate that the substructure is capable of supporting the reduced assessment live loading transmitted from the superstructure.

**BE3/73 FAIL (10 tons)***

* Fails due to significant corrosion and loss of section to bottom flanges of beams. In addition, the substructure is in very poor condition.

BE3/73 Assessment Calculations are contained in Appendix B

# APPENDIX A

FORM AA - Approval in Principle

FORM AA (BRIDGES)

APPROVAL IN PRINCIPLE FOR ASSESSMENT

Babtie

| | |
|---|---|
| **STRUCTURE / LINE NAME** | Rowe<br>Dismantled Whitchurch to Welshpool Line |
| **ELR / STRUCTURE No.** | GNQ4/14 |

**BRIEF DESCRIPTION OF EXISTING BRIDGE :**

| | | |
|---|---|---|
| (a) | Span Arrangement | Single 8.740m clear span, 29° skew. |
| (b) | Superstructure Type | Steel beams with concrete jack arches |
| (c) | Substructure Type | Brick abutments and wingwalls. |
| (d) | Details of any Special Features | None |

**ASSESSMENT CRITERIA**

| | | |
|---|---|---|
| (a) | Loadings and Speed | Assessment loading to BE3/73. Speed 60mph. |
| (b) | Codes to be used | BE3/73<br>BS153 : Part 3B |
| (c) | Proposed Method of Structural Analysis | Hand calculations for individual beams using the distribution curves of BE3/73. Loss of section to the beams will be taken into account in the analysis. |
| (d) | Details of any Special Requirements | None. |

**STRUCTURAL ASSESSMENT ENGINEER'S COMMENTS**

**Superstructure**

There is severe corrosion and significant loss of section to the steel beams, particularly to the bottom flanges. The general condition of the beams can be described as poor and this section loss will be taken into account in the assessment.

There is loose and friable concrete to the outermost jack arches, with efflorescence and spalling evident. The inner jack arches are generally in fair condition.

**Substructure**

The wingwalls and abutments are in poor condition with mortar loss, missing brickwork and spalling noted. There is evidence of bulging to the wingwalls and abutments and cracking has been identified to the abutments.

FORM AA (BRIDGES)

APPROVAL IN PRINCIPLE FOR ASSESSMENT

Babtie

## CIVIL ENGINEER'S COMMENTS

—

## BRB WORKS GROUP COMMENTS - If applicable

## PROPOSED CATEGORY FOR INDEPENDENT CHECK :

| | |
|---|---|
| SUPERSTRUCTURE | **Category 1**<br>(Hand calculations for individual beams)<br>**Qualitative Assessment**<br>(Jack Arches) |
| SUBSTRUCTURE | **Qualitative Assessment** |

## CATEGORY 1

**THE ABOVE ASSESSMENT, WITH AMENDMENTS SHOWN, IS APPROVED IN PRINCIPLE :**

SIGNED [redacted]

TITLE Senior Civil Engineer

DATE 8/2/00

# APPENDIX B

Assessment and Check Calculations

**BABTIE** CALCULATION SHEET

| | | | | | |
|---|---|---|---|---|---|
| OFFICE | MAIDSTONE | PAGE No. | | CONT'N PAGE No. | |
| JOB No. & TITLE | 015528 : BE3/73 ASSESSMENTS | ORIGINATOR | [redacted] | DATE | Jan 00. |
| SECTION | ROWE GNQ4/14 | CHECKER | [redacted] | DATE | FEB 00 |

INDEX

**BABTIE** **CALCULATION SHEET**

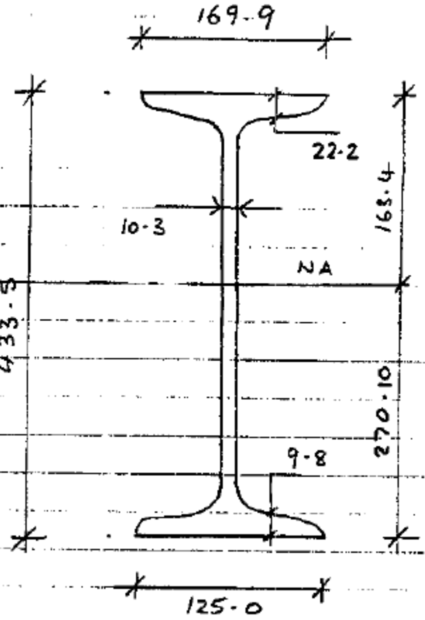
| OFFICE | MAIDSTONE | PAGE No. | 01/1 | CONT'N PAGE No. | |
|---|---|---|---|---|---|
| JOB No. & TITLE | 015528 : BE3/23 ASSESSMENTS | ORIGINATOR | [redacted] | DATE | Jan 00. |
| SECTION | ROWE GNQ4/14 : SECTION PROPERTIES | CHECKER | [redacted] | DATE | Feb 00 |

SECTION PROPERTIES

Section properties from the BD21/97 assessment:-

INTERNAL BEAM (MAX. CORROSION)

17.72" x 6.69" x 77.46 lbs/ft




$A = 91.25\ cm^2$

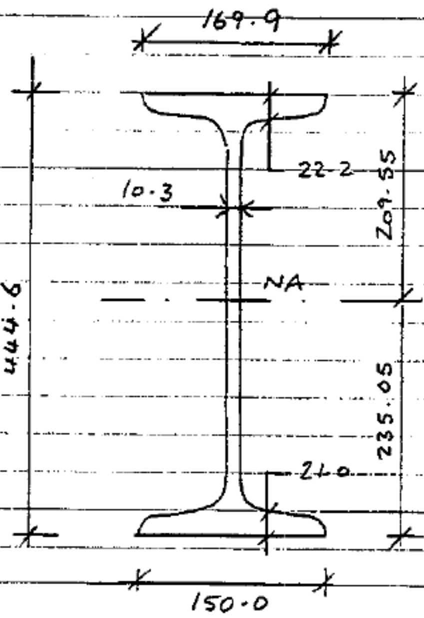
$I_{xx} = 24308\ cm^4$

$r_{xx} = 163.2\ mm$

$r_{yy} = 34.3\ mm$

$Z_t = \frac{I_{xx}}{y_t} = \frac{24308}{27.01} = 900\ cm^3$, $Z_c = \frac{I_{xx}}{y_c} = \frac{24308}{16.34} = 1488\ cm^3$

INTERNAL BEAM (GENERAL)




$A = 110.82\ cm^2$

$I_{xx} = 36858\ cm^4$

$r_{xx} = 182.4\ mm$

$r_{yy} = 36.8\ mm$

$Z_t = \frac{I_{xx}}{y_t} = \frac{36858}{23.505} = 1568\ cm^3$, $Z_c = \frac{I_{xx}}{y_c} = \frac{36858}{20.955} = 1759\ cm^3$

**BABTIE** **CALCULATION SHEET**

| | | | | | |
|---|---|---|---|---|---|
| OFFICE | MAIDSTONE | PAGE No. | 02/1 | CONT'N PAGE No. | 01/1 |
| JOB No. & TITLE | 015528 : BE3/73 ASSESSMENTS | ORIGINATOR | [redacted] | DATE | Jan 00. |
| SECTION | ROWE GNQ4/14 : INTERNAL BEAM | CHECKER | [redacted] | DATE | Feb 00 |

INTERNAL BEAM - DEAD LOADS

Use loads determined in BD21/97 assessment, removing partial safety factors as appropriate since BE3/73 assessment is based on working stresses. *BE3/73 ref (UNO).*

[$N/mm^2 \rightarrow ton/in^2 : \div 15.44$, $kNm \rightarrow tonsfeet : \times 0.329$]

*from BD21/97: $\gamma_{fL} = 1.05$, $\gamma_{f3} = 1.1$, $\gamma_{fL} = 1.15$*

self weight = $1.303/(1.05 \times 1.1)$ = 1.1 kN/m

jack arches = $9.557/(1.15 \times 1.1)$ = 7.6 kN/m

Fill = $0.914 \times ((24 \times 0.1) + (20 \times 0.223))$ = 6.3 kN/m

~~Total Σ 15.0 kN/m~~

max. DL effects

max. shear force $V = \dfrac{15.0 \times 9.029}{2} = 67.7\,kN$

max. moment $M = \dfrac{15.0 \times 9.029^2}{8} = 152.9\,kNm$

DL effects @ 1/4 Point

shear force $V = \dfrac{15.0 \times 9.029}{4} = 33.9\,kN$

Moment $M = \dfrac{15.0 \times 9.029^2}{8} - \dfrac{15 \times 9.029^2}{32} = 114.6\,kNm$

Note: Checks required at 1/4 point as this was considered to be in poor condition - capacity will be reduced to take account of corrosion.

**BABTIE** **CALCULATION SHEET**

| | | | | | |
|---|---|---|---|---|---|
| OFFICE | MAIDSTONE | PAGE No. | 02/2 | CONT'N PAGE No. | 02/1 |
| JOB No. & TITLE | C15528 : BE3/73 ASSESSMENT | ORIGINATOR | [redacted] | DATE | Jan 00. |
| SECTION | ROWE GNQ4/14 : INTERNAL BEAM | CHECKER | [redacted] | DATE | FEB 00 |

INTERNAL BEAM – CAPACITIES — BS153: Pt3B (UNO)

Bending

Basic permissible stress for steel which is assumed to be grade 43 ($f_y = 230N/mm^2$)

$\frac{d_1}{t} = \frac{401.5}{10.3} = 39 < 85$ ∴ refer to table 3, bending (2) — Table 3

∴ basic permissible stress = 142 N/mm²

However, require to check allowable working stresses from Section 10 of BS 153 : Pt 3B.

Since the internal beams are embedded within fill material / concrete jack arches the beams are considered to be fully restrained against lateral torsional buckling ∴ permissible stress of 142N/mm² can be adopted.

Table 1, Case II allows 25% increase in allowable stress ∴ permissible stress = 142 × 1.25 = 177.5 N/mm² — Table 1

Bending Capacity = stress × $Z_t$ = 177.5 × 900 × $10^{-3}$ (ref 01/1)

Bending capacity = 160 kNm

At general section the capacity is slightly greater; Capacity = 177.5 × 1568 × $10^{-3}$ (ref 01/1)

Bending capacity = 278 kNm

(permissible stress is the same throughout beam)

Capacities are based on elastic section properties since permissible stresses are used in BE3 Assessment

| **BABTIE** | | | | | CALCULATION SHEET |
|---|---|---|---|---|---|
| OFFICE MAIDSTONE | | PAGE No. | 02/3 | CONT'N PAGE No. | 02/2 |
| JOB No. & TITLE 015528 : BE3/73 ASSESSMENTS | | ORIGINATOR | [redacted] | DATE | Jan 00 |
| SECTION ROWE GNQ4/14 : INTERNAL BEAM | | CHECKER | [redacted] | DATE | FEB 00 |

Shear

Basic permissible stress taken from Table 3 :- — BS153:Pt3B (UNO) table 3

$= 85\ N/mm^2$ (average)

Now check allowable average shear stress from table 9

$\frac{d}{t} = \frac{401.5}{10.3} = 39 < 70$ ∴ from table 9.

$p_q = 85 N/mm^2$ ($f_y = 230\ N/mm^2$)

∴ $f_q = 85 N/mm^2$ — 11.1

Table 1, case II allows 25% increase in allowable stress ∴ permissible shear stress $= 85 \times 1.25$

$= 106.3\ N/mm^2$

∴ Shear Capacity $= 106.3 \times (433.5 \times 10.3) \times 10^{-3}$

$= 475 kN$ (max. Corrosion).

[effective shear area = D.t] — Pt 4 2.1.4.1

at general section Shear Capacity $= 106.3 \times 444.6 \times 10.3 \times 10^{-3}$

$= 487 kN$

| **BABTIE** | | | | **CALCULATION SHEET** | |
|---|---|---|---|---|---|
| OFFICE MAIDSTONE | PAGE No. | 02/4 | CONT'N PAGE No. | 02/3 |
| JOB No. & TITLE 01528 : BE 3/73 ASSESSMENTS | ORIGINATOR | [redacted] | DATE | Jan 00. |
| SECTION ROWE GNQ4/14 : INTERNAL BEAM | CHECKER | [redacted] | DATE | FEB 00 |

| | BE3/73 ref (UNO) |
|---|---|
| ASSESSMENT RATING | |
| Max. bending moment at Midspan | |
| Moment capacity = 278 kNm — 02/2 | |
| Dead Load moment = 152.9 kNm — 02/1 | |
| ∴ available LL moment = 278 − 152.9 = 125.1 kNm | |
| $= \frac{125.1}{9.966 \times 0.3048} = 41$ tons feet | |
| carriageway width = 2.9m (9.5ft) < 18ft ∴ SINGLE LANE | 302(a) |
| skew 29.9°, span = 9.029m (29.6 ft) | |
| girder spacing = 914mm (3 ft). | |
| From graph 1, Proportion factor k = 0.25 | Graph 1 |
| Available moment for one lane $= \frac{41}{0.25} = 164$ tons feet/lane | 301(a) |
| From graph 5, M = 164 tons feet/lane, span = 29.6 ft | Graph 5 |
| ⇒ 24 tons Gross Vehicle Weight (ie. no restriction required) | |
| PASS BE 3/73 | |
| Max. Shear at Support | |
| | 301(b) |
| Shear Capacity = 487 kN — 02/3 | |
| Dead Load Shear = 67.7 kN — 01/1 | |
| (i) available LL Shear = 487 − 67.7 = 419.3 kN | Part 2 Section 3 |
| $= \frac{419.3}{9.966} = 42$ tons > 12.5 tons ∴ OK (Table 1) | 301(b)(i) Table 1 |
| (ii) Subtract 0.625 × max. axle load (11 tons) | Part 2 Section 3 301(b)(ii) |
| = 42 − (0.625 × 11) = 35.125 tons | |

| **BABTIE** | | | | | CALCULATION SHEET |
|---|---|---|---|---|---|
| OFFICE MAIDSTONE | | PAGE No. | 02/5 | CONT'N PAGE No. | 02/4 |
| JOB No. & TITLE 015528 : BE3/73 ASSESSMENTS | | ORIGINATOR | [redacted] | DATE | Jan 00. |
| SECTION Rowe GNQ4/14 : INTERNAL BEAM | | CHECKER | [redacted] | DATE | FEB 00 |

(iii) Proportion factor K = 0.25, from graph 1 — BE3/73 ref 301(b)(iii) Graph 1.

$\frac{35.125}{0.25} = 140.5$ tons.

(iv) Add max. axle load, 140.5 + 11 = 151.5 tons — 301(b)(iv)

(v) From graph 12, Gross Shear = 2.5 — 301(b)(v) Graph 12

∴ axle weight = $\frac{151.5}{2.5}$ = 60.6 tons (permissible)

> 11 tons ∴ OK

No weight restriction required.

PASS BE 3/73

Check Bending at 1/4 Point

Moment Capacity = 160 kNm — 02/2

Dead Load Moment = 114.6 kNm — 02/1

∴ available LL moment = 160 − 114.6 = 45.4 kNm

= $\frac{45.4}{9.966 \times 0.3048}$ = 15 tons feet

From graph 1, proportion factor K = 0.25 — Graph 1

Bending moment due to one lane = $\frac{15}{0.25}$ = 60 tons feet/Lane

equivalent moment at midspan = $\frac{60}{0.75}$ = 80 tons feet/lane — Graph 5

(Moment at 1/4 pt is 75% of moment at midspan.)

⇒ 10 tons Gross Vehicle Weight

Check Shear at 1/4 Point

Shear Capacity = 475 kN — 02/3

DL shear = 33.9 kN — 02/1

∴ available LL shear = 475 − 33.9 = 441.1 kN > that available at support So OK/

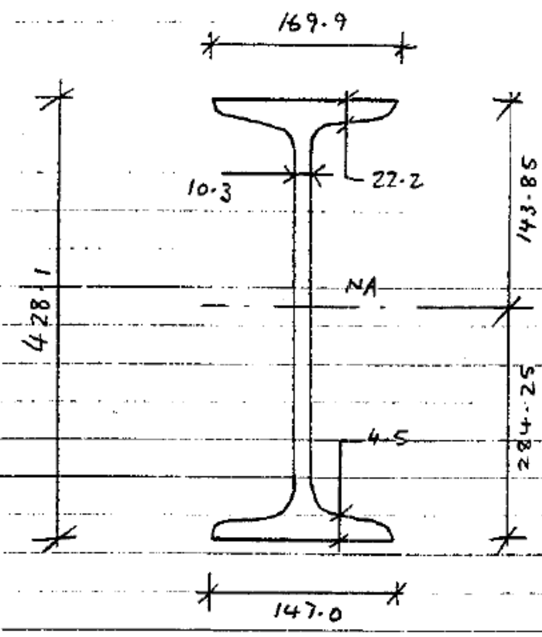
**BABTIE** **CALCULATION SHEET**

| | | | | | |
|---|---|---|---|---|---|
| OFFICE MAIDSTONE | | PAGE No. | 03/1 | CONT'N PAGE No. | 02/5 |
| JOB No. & TITLE 015528 : BE 3/73 ASSESSMENTS | | ORIGINATOR | [redacted] | DATE | Jan 00 |
| SECTION ROWE GNQ 4/14 : EDGE BEAM | | CHECKER | [redacted] | DATE | FEB 00 |

## SECTION PROPERTIES

Section properties from the BD21/97 assessment :-

EDGE BEAM (MAX. CORROSION)




$A = 85.93 \text{ cm}^2$

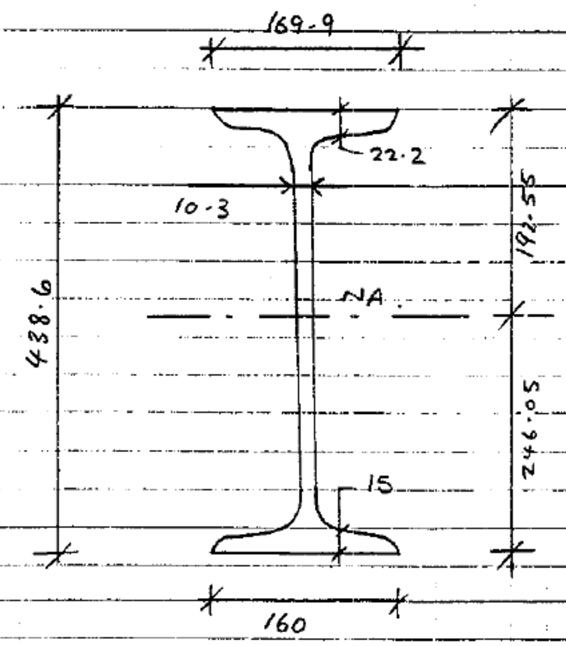
$I_{xx} = 20349 \text{ cm}^4$

$r_{xx} = 153.9 \text{ mm}$

$r_{yy} = 34.6 \text{ mm}$

$Z_t = \frac{I_{xx}}{y_t} = \frac{20349}{28.425} = 716 \text{ cm}^3$ , $Z_c = \frac{I_{xx}}{y_c} = \frac{20349}{14.385} = 1415 \text{ cm}^3$

EDGE BEAM (GENERAL)




$A = 103.32 \text{ cm}^2$

$I_{xx} = 32448 \text{ cm}^4$

$r_{xx} = 177.2 \text{ mm}$

$r_{yy} = 37.1 \text{ mm}$

$Z_t = \frac{I_{xx}}{y_t} = \frac{32448}{24.605} = 1319 \text{ cm}^3$ , $Z_c = \frac{I_{xx}}{y_c} = \frac{32448}{19.255} = 1685 \text{ cm}^3$

## EDGE BEAM - DEAD LOADS

BE3/73 ref (UNO)

Use loads determined in BD21/97 assessment, removing partial safety factors as appropriate since BE3/73 assessment is based on working stresses.

Self weight = 1.303 / (1.05 × 1.1) = 1.1 kN/m

jack arch = 7.6 / 2 (ref 01/1) = 3.8 "

Fill = 6.3 / 2 (ref 01/1) = 3.2 "

Σ 8.1 kN/m

**max DL effects**

max shear force $V = \frac{8.1 \times 9.029}{2} = 36.6$ kN

max. Moment $M = \frac{8.1 \times 9.029^2}{8} = 82.5$ kNm

**DL effects at ¼ Point**

Shear force $V = \frac{8.1 \times 9.029}{4} = 18.2$ kN

moment $M = \frac{8.1 \times 9.029^2}{8} - \frac{8.1 \times 9.029^2}{32} = 61.9$ kNm

| **BABTIE** | | | | | **CALCULATION SHEET** |
|---|---|---|---|---|---|
| OFFICE MAIDSTONE | PAGE No. | 03/3 | CONT'N PAGE No. | 03/2 | |
| JOB No. & TITLE 015528 : BE 3/73 ASSESSMENTS. | ORIGINATOR | [redacted] | DATE | Jan 00. | |
| SECTION ROWE GNQ 4/14 : EDGE BEAM | CHECKER | [redacted] | DATE | FEB 00 | |

EDGE BEAM – CAPACITIES — BS153: Pt3B (UNO)

Bending

Basic permissible stress for steel which is assumed to be grade 43 ($f_y = 230\,N/mm^2$)

$\frac{d_1}{t} = \frac{401.4}{10.3} = 39 < 85$ ∴ refer to table 3, bending (2) — Table 3

∴ basic permissible stress = $142\,N/mm^2$ (tens/comp)

However, require to check allowable working stresses from Section 10 of BS 153 : Pt 3B (Comp) — Section 10

Beam is effectively restrained by the concrete on one side and tie rods spaced at approx. 2.6m c/c

So $l_e = 2.6m$

$\frac{l_e}{r_{yy}} = \frac{2600}{37.1} = 70.1$, $\frac{D}{T} = \frac{438.6}{22.2} = 19.8$ — 10.2.1.1

From Table 7, A = 702 by interpolation = $C_s$

$C_s$ can be increased by 20% for rolled beams

∴ $C_s = 842$

From table 8 : $C_s = 842\,N/mm^2$, Grade 43 ($f_y = 230\,N/mm^2$) } $p_{bc} = 139\,N/mm^2$ — Table 8

Table 1, case II allows 25% increase in allowable stresses

∴ allowable tensile stress = $142 \times 1.25 = 177.5\,N/mm^2$

allowable comp. stress = $139 \times 1.25 = 173.8\,N/mm^2$

Beam will be checked for tensile stresses as these will be more critical as $Z_c >> Z_t$.

**BABTIE** **CALCULATION SHEET**

| | | | | | |
|---|---|---|---|---|---|
| OFFICE | MAIDSTONE | PAGE No. | 03/4 | CONT'N PAGE No. | 03/3 |
| JOB No. & TITLE | 015528 : BE3/73 ASSESSMENTS | ORIGINATOR | [redacted] | DATE | Jan 00 |
| SECTION | ROWE GNQ4/14 : EDGE BEAM | CHECKER | [redacted] | DATE | FEB 00 |

Bending capacity = Stress × $Z_t$ = $177.5 \times 716 \times 10^{-3}$

Bending Capacity = <u>127 kNm</u>

At the general section the bending capacity is greater = $177.5 \times 1319 \times 10^{-3}$

Bending Capacity = <u>234 kNm</u>

<u>Shear</u>

Basic permissible shear stress taken from Table 3:-

= 85 N/mm² (average). — Table 3

Now check allowable average shear stress from Table 9

$\frac{d}{t} = \frac{401.5}{10.3} = 39 < 70$ ∴ $p_q = 85$ N/mm² — Table 9

∴ $f_q = 85$ N/mm² — 11.1

Table 1, case II allows 25% increase in allowable stress ∴ permissible shear stress = 85 × 1.25

= 106.3 N/mm²

∴ Shear capacity = $106.3 \times 428.1 \times 10.3 \times 10^{-3}$

= <u>469 kN</u> (max corrosion)

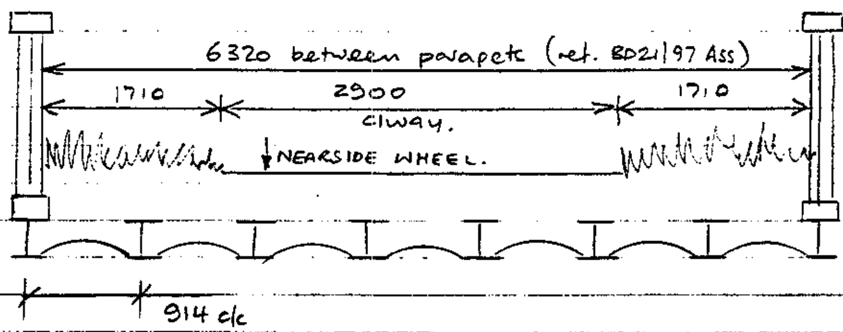
At general section, Shear capacity

= $106.3 \times 438.6 \times 10.3 \times 10^{-3}$

= <u>480 kN</u> (general)

| **BABTIE** | | | **CALCULATION SHEET** | | |
|---|---|---|---|---|---|
| OFFICE MAIDSTONE | PAGE No. | 03/5 | CONT'N PAGE No. | 03/4 |
| JOB No. & TITLE 015528 : BE3/73 ASSESSMENTS | ORIGINATOR | [redacted] | DATE | Feb 00. |
| SECTION Lowe GNQ4/14 : EDGE BEAMS | CHECKER | [redacted] | DATE | FEB 01 |

EDGE BEAM – ASSESSMENT RATING




By inspection of the above sketch it is clear that there is at least one internal beam between the edge beam and the nearside wheel so the edge beam does not need to be assessed for live loads – ref Cl 301

However, check for dead loads only :-

Max Bending at midspan

Moment Capacity = 234 kNm (03/4), DL mom = 82.5 kNm (03/2) ∴ OK ✓

Max. Shear at Support

Shear capacity = 480 kN (03/4), DL shear = 36.6 kN (03/2) ∴ OK ✓

Max. Bending at 1/4 point

Mom capacity = 127 kNm (03/4), DL mom = 61.9 kNm (03/2) ∴ OK ✓

Shear at 1/4 Point

Shear capacity = 469 kN (03/4), DL shear = 18.2 kN (03/2) ∴ OK ✓

∴ EDGE BEAM ADEQUATE FOR DEAD LOADS

**BABTIE** **CALCULATION SHEET**

| OFFICE | MAIDSTONE | PAGE No. | 04/1 | CONT'N PAGE No. | 03/5 |
|---|---|---|---|---|---|
| JOB No. & TITLE | 015528 : BE3/73 Assessments | ORIGINATOR | [redacted] | DATE | Jan 00. |
| SECTION | Rowe GNQ4/14 : JACK ARCHES | CHECKER | [redacted] | DATE | FEB 00 |

JACK ARCHES

Defects have been identified to the concrete jack arches - see BD21/97 Inspection Report However, the qualitative assessment of the jack arches is not considered to reduce the capacity of the bridge further than the capacity of the beams.

SUBSTRUCTURE

The wingwalls and abutments are in poor condition with mortar loss, missing brickwork and spalling noted. There is evidence of bulging to the wingwalls and abutments and cracking has been identified to the abutments. A qualitative assessment is not considered to reduce the capacity of the bridge further than the capacity of the beams.

**BABTIE** — **CALCULATION SHEET**

| OFFICE | MAIDSTONE | PAGE No. | 06/1 | CONT'N PAGE No. | 04/1 |
|---|---|---|---|---|---|
| JOB No. & TITLE | 015528 : BE3/73 ASSESSMENTS | ORIGINATOR | [redacted] | DATE | Jan 00. |
| SECTION | ROWE GNQ4/14 : SUMMARY | CHECKER | [redacted] | DATE | FEB 00 |

SUMMARY

INTERNAL BEAM

| | | |
|---|---|---|
| Bending Moment | - at Midspan | PASS |
| | - at 1/4 Point | FAIL (10tons) |
| Shear Force | - at Support | PASS |
| | - at 1/4 Point | PASS |

EDGE BEAM (DEAD LOAD ONLY)

| | | |
|---|---|---|
| Bending Moment | - at Midspan | PASS |
| | - at 1/4 Point | PASS |
| Shear Force | - at Support | PASS |
| | - at 1/4 Point | PASS |

FAIL BE 3/73 (10tons)

# APPENDIX C

FORM BA - Assessment and Check Certificate

**FORM BA (BRIDGES)**

**Babtie**

**CERTIFICATE FOR ASSESSMENT CHECK**

**STRUCTURE / LINE NAME** Rowe **CATEGORY OF CHECK** 1

**ELR / STRUCTURE No.** GNQ4/14

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

(1) It has been assessed in accordance with the Approval in Principle as recorded on Form AA signed by John Clarke on 8 February 2000.

(2) It has been checked for compliance with the following British Standards, Codes of Practice, BR Technical notes and Assessment standards:-

BE3/73
BS153 : Part 3B

**CATEGORY 1**

| **NAME** | **SIGNATURE** | | | |
|---|---|---|---|---|
| [redacted] | [redacted] | (ASSESSOR) | 22-02-00 | (DATE) |
| | | (ASSESSMENT CHECKER) | 22-02-00 | (DATE) |
| | | (TECHNICAL DIRECTOR BABTIE GROUP) | 25/02/00 | (DATE) |

**CATEGORY 2**

**(a) ASSESSMENT**

| **NAME** | **SIGNATURE** | | | |
|---|---|---|---|---|
| .............................. | .............................. | (ASSESSOR) | ...................... | (DATE) |
| .............................. | .............................. | (TECHNICAL DIRECTOR BABTIE GROUP) | ...................... | (DATE) |

**(b) CHECK**

| **NAME** | **SIGNATURE** | | | |
|---|---|---|---|---|
| .............................. | .............................. | (ASSESSMENT CHECKER)) | ...................... | (DATE) |
| .............................. | .............................. | (TECHNICAL DIRECTOR BABTIE GROUP) | ...................... | (DATE) |

**THE CERTIFICATE IS ACCEPTED BY**......[redacted]......................................

**FORM BAA (BRIDGES)**

Babtie

**CERTIFICATE FOR ASSESSMENT CHECK**

---

## NOTIFICATION OF ASSESSMENT CHECK

**STRUCTURE NAME** Rowe, Dismantled Whitchurch to Welshpool Line

**ELR / STRUCTURE No.** GNQ4/14

The above bridge has been assessed and checked in accordance with the 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

10 tons

Critical member Internal beams

### RECOMMENDED LOADING RESTRICTIONS

10 tons

### DESCRIPTION OF STRUCTURAL DEFICIENCIES AND RECOMMENDED STRENGTHENING

The limiting structural elements of the bridge are the internal steel beams which have a limiting assessment capacity of 10 tons at ¼ span location where there is significant corrosion and loss of section. It is not considered that other methods of analysis could be adopted to provide an improved result.

The edge beams were found to be adequate for dead loading alone. The edge beams did not require to be examined to BE3/73 since the position of the nearside wheels in the vehicle train was such that an internal beam was present between the wheels and the edge beam.

The structure is generally in poor/fair condition and it is recommended that minor remedial works are carried out on the superstructure and the substructure as a matter of urgency.

Propping of the beams would provide an interim solution for increasing the bridge capacity. However, in the long term, replacement of the deck should be considered. Due to the poor condition of the substructure some remedial/reconstruction work may be required to ensure that the substructure was capable of carrying any new deck.

In order to avoid the cost of constructing a replacement deck, in addition to carrying out extensive remedial works to the substructure, it is recommended that consideration be given to removing the structure. In order to maintain vehicular access over the disused railway the deck could be removed and the 'gap' filled to match the profile of the existing embankment behind each abutment allowing the continuation of the roadway. However, an investigation would be required to determine whether or not access under the bridge has to maintained. Alternatively, the entire structure and approach embankments could be removed and a new carriageway placed closer to the level of the surrounding land. Removal of the deck would also remove any future maintenance liability.

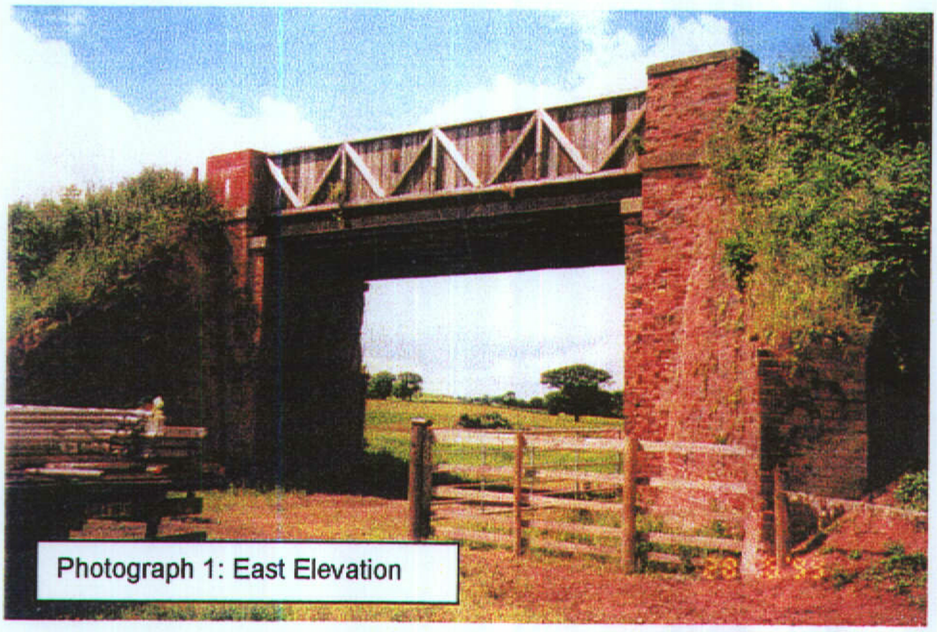
**Name:**[redacted]. **Signed**[redacted]. **Assessment Engineer**

**Name:**[redacted]. **Signed**[redacted] **Civil Engineer**

**Rail Property Ltd./ Shropshire County Council**

# Assessment and Inspection Report

| | |
|---|---|
| **Bridge Name:** | **Rowe** |
| **Bridge No.:** | **GNQ4/14** |
| **Road:** | **Unclassified** |
| **Location:** | **5 km East of Ellesmere** |
| **OS Ref:** | **SJ 450 357** |


Photograph 1: East Elevation

This report was commissioned by Rail Property Limited/ Shropshire County Council and is confidential. It is not to be passed to a third party without the permission of the Senior Civil Engineer or his delegated representative.

| | |
|---|---|
| **Copy No : 1** | **Issue No 1** |

**Report No : BPS/31417/GNQ4/14**

**July 1999**

**Babtie Group** Abbey Foregate Shrewsbury SY2 6BJ
Tel: 01743 253000 Fax 01743 253001

## Rail Property Ltd./ Shropshire County Council

# Assessment and Inspection Report

**Status of report:**

| | |
|---|---|
| **Interim Draft** | |
| **Final Draft** | ✓ |
| **Final** | |

**Bridge Name:** Rowe
**Bridge No.:** GNQ4/14
**Road:** Unclassified
**Location:** 5 km East of Ellesmere
**OS Ref:** SJ 450 357


**Report Prepared For and**
**On Behalf of Shropshire County Council:**
Babtie Group Limited
Abbey Foregate
Shrewsbury
Shropshire SY2 6BJ

Signed...[redacted]
[redacted] C.Eng., MICE
Technical Director, Structures

Signed..[redacted] 24/11/99
[redacted]
Assessing Engineer

**Client:**

Rail Property Limited
Room C5
Hudson House
York YO1 6HP

Signed.................

# Contents

# 1. General Description and Structural Details

## 1.1 Location

The bridge carries the road from Hampton Bank to Breaden Heath over a dismantled railway line. It is situated east of Ellesmere just north of Balmer Heath.

The Ordnance Survey National Grid Reference of the structure (to the nearest 100m) is **SJ 450 357.**

## 1.2 Construction

The bridge has a single span, with the deck consisting of 8 simply supported steel beams and transversely spanning intermediate concrete jack arches. The steel beams are seated on sandstone masonry coping stones, with no bearing evident. The abutments, wingwalls, and pilasters are all constructed in brickwork with a sandstone coping. However the sandstone coping on the east side of the bridge has been removed. The parapets are of a timber construction.

The clear skew span is 8.74 and the angle of skew is 29.9°. The width of the deck is 8.20m and the carriageway is 2.9m wide.

## 1.3 Statutory Undertakers

No evidence to indicate the presence of statutory undertakers equipment was found on site during the inspection.

Should however intrusive investigations or site work be considered information to confirm or otherwise the presence of statutory undertakers equipment should be sought from the relevant bodies.

## 1.4 General

A fence line to the east side of the bridge, between the North and South wingwalls, with a gate access point has been erected by the farmer to contain cattle.

The bridge is situated on a straight section of road at the brow of an embankment formed to traverse over the railway without the need of a level crossing. The gradient of the road over the bridge is such that the sight distances either side of the bridge are restricted.

The area beneath the bridge is largely free of obstruction apart from some vegetation growth.

The assessment records made available for review all date prior to 1959, additional information from 1959 to date has not been forthcoming.

The railway under the bridge has been dismantled, this was probably a consequence of the Beeching Report in the 1960's.

## 2. Condition Report

### 2.1 General

The structure was inspected as closely as possible with most elements being examined within arms length. Access to the deck soffit was obtained by means of a scaffold tower.

The bridge inspection was carried out by M G Walsh and B Schlebusch on 28 June 1999. The weather at the time of the inspection was dry and sunny with little cloud cover and a slight breeze.

Measurements were taken around the bridge as required to record corrosion evident in individual members. Principal dimensions critical to the assessment were also recorded to confirm information on previous survey drawings.

The top flange and web of the steel beams could not be inspected or measured since only the bottom flange of the beam is visible. A thorough visual inspection was nonetheless carried out on all the exposed parts of the structure and both the girders and jack arches were subjected to a hammer survey to assess their structural integrity.

No material testing has been carried out at this stage.

### 2.2 Main Girders

Beams are numbered 1 to 8 starting from the west side.

The longitudinally spanning steel beams have been recorded to be 450mm deep x 170mm wide. Reference to the BCSA Historic Steelwork Handbook gives a best match to a 17.72" x 6.69" x 77.46lbs/ft beam rolled in France by Longwy about 1900. Originally the 14° tapered flange thickness would be 24.33mm with a web of 16.2mm thick.

The bottom flanges of the beams show a consistent 3½° anticlockwise rotation when looking north in the direction of the span. It is considered that this rotation is probably long standing and may have occurred during construction.

The beams have all been repainted, directly over the old paint system, however the paint has broken down and rust is occurring beneath. Corrosion is worse at the north quarter point of the span, this being over the single-track line indicated on previous survey drawings. It is considered likely that this is due to the effects of steam from trains passing under the bridge during the single lines operating life. The corrosion evident at this quarter point is extensive with delamination of the steel member evident. This indicates that the corrosion has been long standing, and progressively the integrity of the beams is deteriorating.

Currently a maximum loss of section up to 45mm has been recorded in the flange width together with a loss up to 17mm in the flange thickness over a length of 1.2m in the north quarter span area.

**Beams 1 and 8 (External Beams)**

The general condition of the beams can be described as fair to poor. The beams are severely corroded at the north quarter point over a length of 1.0m with a reduction in the flange width from 170mm to 147mm. The flange thickness has been reduced in places to 3mm on the inner side and 6mm on the exposed side on the bridge. At beam 8 it is possible due to missing concrete to assess the web thickness, which was measured at approximately 11mm.

**Beams 2-7 (Internal Beams)**

The overall condition of the beams can be described as fair to poor. However the edges of the bottom flange show evidence of corrosion resulting in a minor loss of section over the whole length of the beams. Severe loss of section occurs locally at the northern quarter point, over a 1.2m length, to yield a flange width of 125mm and a thickness of 10mm. Moisture has also leached through the outer jack arches and is building up on the edges of beams 2 and 7.

## 2.3 Jack Arches

The concrete jack arches have a span of 902mm with an average rise at the crown of 140mm.

From the visual inspection the exposed surface appears to contain pebble like aggregate suggesting that it is a sea dredged or river bed aggregate. Leaching is very prominent all over the deck soffit, suggesting that leaching action within the concrete jack arches is taking place. This is considered to be due to carbon dioxide in the water dissolving calcium in the cement mix and extracting it to the surface. The calcium deposits formed at the surface, stalactites, are particularly prominent around the bottom of the arch and on the steel sections where the moisture will condense more readily.

**Archs 1 and 7 (Outer Bays)**

These two arches both sound hollow when struck with a hammer indicating partial debonding of the concrete. There is extensive spalling to a depth of 5mm on approx. 40% of these arches. The concrete has been softened due to the passage of water leaching out the lime from the cement mix and is in a state that it is relatively easy to extract throughout the arch with hand tools. At the void in the east outer arch it is possible to pick the concrete out by hand, although undisturbed the concrete doe not pose a hazard.

The severity in condition of the outer arches is attributed to rainwater running off the road and penetrating the verge, resulting in this accelerated deterioration.

**Archs 2 –6 (Inner Bays)**

These jack arches are in better condition than the outer bays and gave off a 'solid ring' when struck with a hammer. There is a slight loss of section due to leaching effects resulting in an exposed aggregate finish. There is extensive sooting to these inner arches, except in the area directly above the old line.

## 2.4 Abutments

The abutments are generally heavily weathered with spalling evident randomly over approximately 60% of the surface area. Due to the high skew angle of the bridge, the weathering effects are more prominent in the exposed areas due to the erosion of the soot deposits.

The abutments are bulging at mid-height, suggesting movement behind the abutment walls.

The abutments have been repointed with a cement based mortar to a depth between 15 and 20mm. The mortar behind this is clearly a lime mortar, which with the fullness of time has softened, resulting in mortar loss. This is likely to have led to the initial repointing work. However in exposed areas it is possible to see that the original mortar is still being washed out with voids forming behind the newer cement based mortar.

Moisture trapped within the abutments due to the cement based repointing work may contribute to the spalling effects through freeze-thaw action. As the depth of spalling is generally 20mm, the same depth to which repointing work was carried out this is considered to be a fair assumption.

Although spalling has not occurred all over the bridge it was noted that most of the brickwork surfaces sounded hollow when struck with a hammer indicating that separation of the surface layers may be taking place.

The high skew angle of the bridge has introduced high stresses at the acute corners of the bridge. A result of this is that vertical cracks have formed from the coping stone to ground level, effectively separating the outer beam support from the main part of the abutment structure. This is likely to have a significant effect of the load bearing capacity of the bridge.

Behind the abutments the fill appears to be a sandy material, and hence quite pervious. This is thought to be the principal reason that a majority of the abutment wall has been affected with dampness showing through most of the abutment faces. If any filter drain had been present behind the abutment wall it is likely that the fine sand particles have now clogged the drain, preventing it from functioning.

### North Abutment

At the west corner of the abutment an area of local damage, which appears to be long standing, has occurred. From the size (1.4m high x 1 brick deep) and location (0.5m-1.9m from ground level) it is likely to have occurred due to vehicle impact, possibly from a trailer.

At the top corner of the east side a section of brickwork has debonded (1m x 1m). This section completely supports the outer beam, giving the impression of a local shear/compression failure due to deterioration of the mortar joints.

Immediately below the coping, approximately on the centreline a section of brickwork has spalled away leaving a 200mm deep x 1200mm long x 300mm high recess in the wall. The coping stone, although incomplete, is likely to be under additional stresses due to its now cantilevered state.

### South Abutment

Bricks are missing from a section approximately 2m high beneath the east corner. Also in the same corner a vertical crack extends the full height of the abutment.

## 2.5 Wingwalls

### North West

This wingwall is in a poor condition. There are vertical and diagonal cracks from the top to the bottom of the wall and severe spalling and mortar loss visible. The majority of the wall sounds hollow when tapped, and vegetation is growing through the wall in places. The pilaster at the end of the wingwall has been extensively damaged, with the top 500mm being rotated and with many of the facing bricks missing. The coping stones are not seated properly.

### North East

This wingwall is in a poor condition. There are cracks throughout the brickwork and severe spalling. The majority of the wall sounds hollow when tapped. Vegetation is growing through the wall in a number of locations. The coping stones are no longer present on the wall, and are not visible on site.

### South West

This wingwall is in a very poor condition. There is severe brick and mortar loss. The majority of the wall sounds hollow when tapped. It is probable that the blue bricks present were part of a previous repair as there are more here than elsewhere in the structure.

### South East

This wingwall is in very poor condition, and sound hollow when tapped. There is a significant spalling and mortar loss is visible, with the end pilaster in a state of partial collapse. The coping stones are not located on the wall, nor are they visible anywhere on site.

## 2.6 Parapets

The parapets are of a timber construction, between two brickwork pilasters, at ¼ points additional fixing to the steel beams has been provided. The construction is itself in generally good condition, however in the event of a vehicular impact the parapets are unlikely to provide an adequate containment capacity.

### West Parapet

The west parapet appears to be in good condition, with the timber treatment still in good colour although it is beginning to fade.

### East Parapet

The east parapet appears to be in good condition, however the timber treatment is faded.

## 2.7 Road Surface

The bridge road surfacing is worn with loose stone in places. The vertical alignment of the bridge suggests that visibility is not good, and given the allowable traffic speed, 60mph, heavy braking may often be required, thus accounting for the surface wear.

The longitudinal profile of the bridge may also encourage axle lift off for multi-axle configurations.

Settlement of the fill either side of the bridge is evident with an abrupt change in level locally, and at the southern side of the bridge a transverse crack in the road surfacing is evident.

## 2.8 Old Railway Formation

The western side of the bridge between the abutments was fenced off to stop livestock from walking through.

The ground is sodden underfoot in places around the bridge, suggesting that the soil is fairly impervious or that the water table is quite high, as recent rainfall had been minimal.

# 3. Summary of Condition and Recommendations

## 3.1 Summary of Condition

The overall condition of the bridge can be described as poor, with the worst affected parts being the steel beams and the brickwork. The load bearing part of the structure (deck and abutments) shows no sign of subsidence, however deformation is noted in the abutments.

The visible parts of the beams are generally in a fair condition however the bottom flanges are corroded resulting in a loss of section for the member, particularly over a 1.2m length at the quarter point of the bridge where significant loss of section is noted.

The outer jack arches are in a particularly poor condition, with efflorescence and loss of section evident. The concrete in the outer arches is assumed to be weaker due to the ease in removing sections with hand tools. The internal jack arches (2 - 5) are generally in better condition, although they did exhibit severe efflorescence.

The abutments have evidence of movement, spalling, leaching and cracking underneath the coping stones. They are in poor condition.

The wingwalls are in very poor condition. There is evidence of movement, significant cracking, severe spalling and mortar loss. The walls have a fair amount of vegetation growing through them which will have an adverse effect of the stability of the walls.

## 3.2 Recommendations

In accordance with BD 21/97 and based on the approved form the "Approval In Principle" the bridge has a live load capacity of 3 tonnes.

It is considered that the structure has reached the end of its serviceable life and is now approaching a state where its continued use will expose users to a potential hazard.

To correct the faults of the wingwalls and abutments major construction work is needed. It is considered that in order to remedy the deteriorated state of the walls full reconstruction would be required.

The short term solution would be to prop the bridge deck and abutments with an appropriate system providing that it can be shown that the former will provide an acceptable load carrying capacity.

However it should be noted that the bridge is used by the farm to gain access between fields, the legal position should be considered with regard to any solution that may restrict access.

The parapet needs to be upgraded to provide a system capably of withstanding vehicular impact. This may be achieved with a safety barrier erected in the verge.

Alternative long term solutions would be:

- Fill the bridge in, remove deck and in-fill with engineered fill to road formation level followed by road construction.

  or

- as the sole purpose of the embankment is to enable vehicles to cross the bridge over the dismantled railway consider the removal of the bridge and the embankment.

A cost analysis study of any work can be carried out to indicate the most economic and viable solution, together with long term benefits.

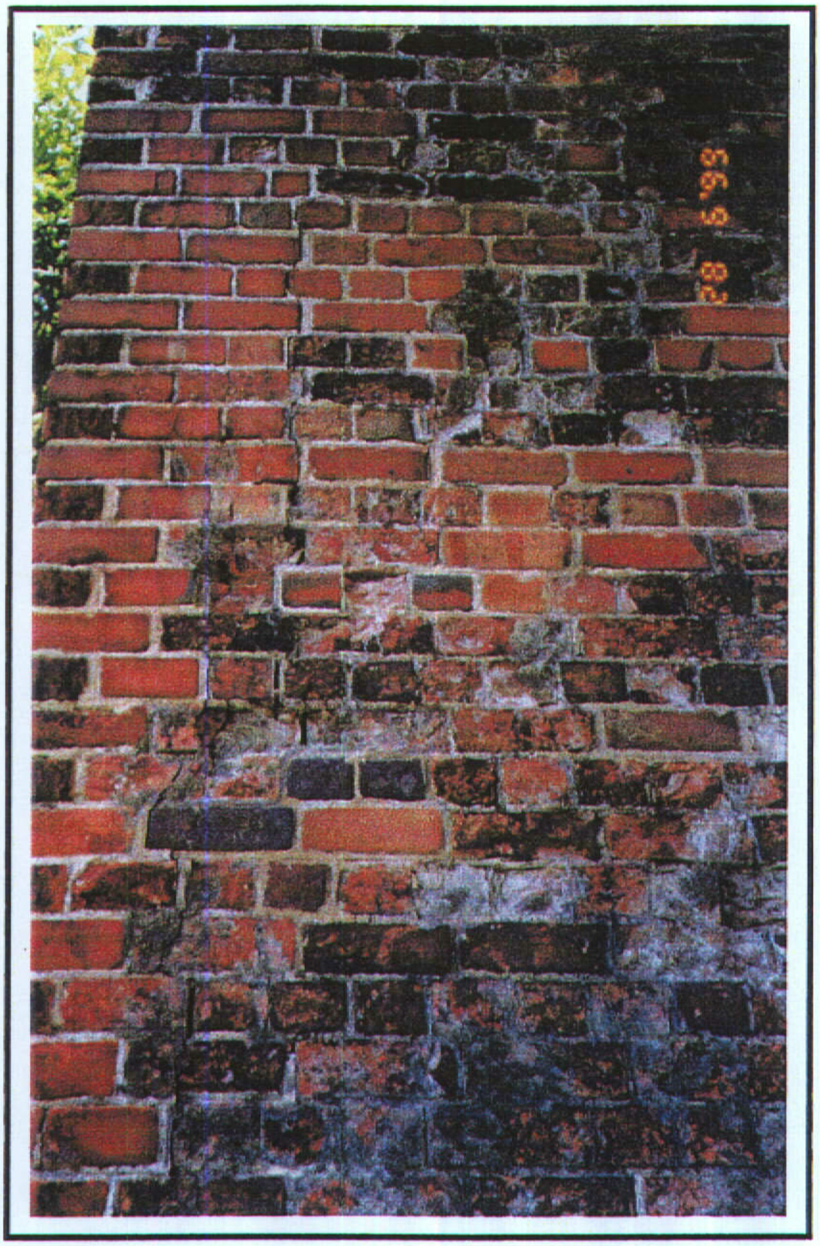
Given the severity of the bridge's condition it is recommended that the design and reconstruction works should be implemented as a matter of urgency.

# APPENDIX A

## Photographs and fold-out A3 Schedule located at the end of this Appendix


Photograph 2

Photograph 3



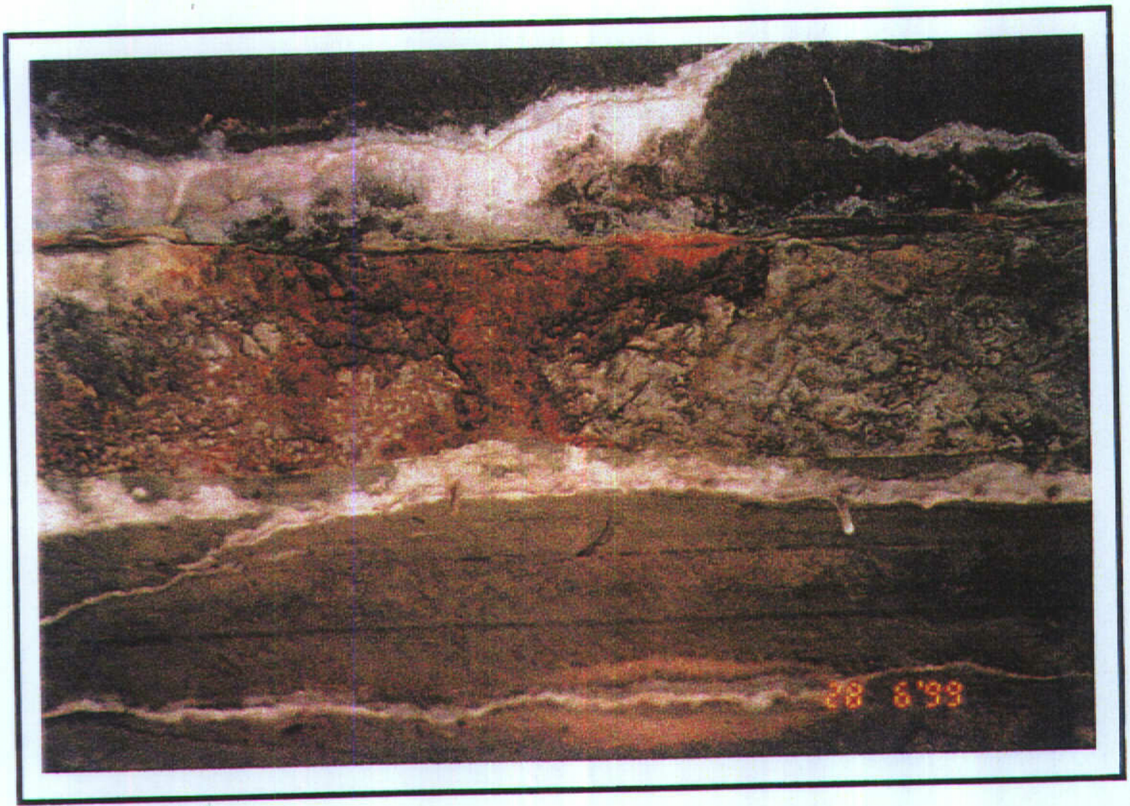

Photograph 4


Photograph 5


Photograph 6


Photograph 7

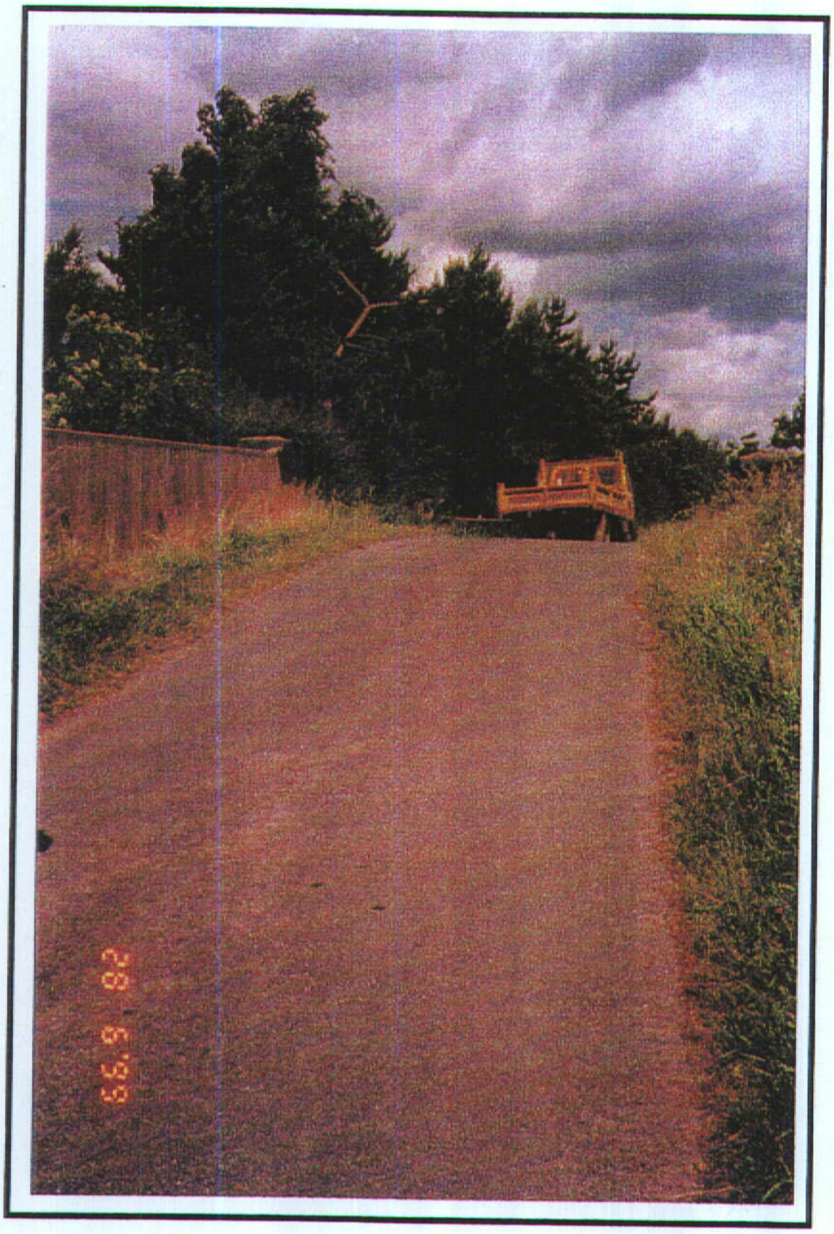

Photograph 8




Photograph 9


Photograph 10


Photograph 11



Photograph 12

Appendix A - Schedule Of Photographs

All photographs are landscape format, except where indicated with an *, these are portrait format and should be viewed from the right.

| Photograph No. | Description |
|---|---|
| 1 | East elevation (Front cover) |
| 2 | North-east wing wall |
| 3 | Top of north abutment, east side |
| 4 | North abutment, west side |
| 5 | South abutment |
| 6 | Severe spalling to north abutment under coping stone |
| 7 | View south on jack arches |
| 8 | Elevation on severe corrosion evident on an internal beam |
| 9 | Plan view on severe corrosion to a typical internal beam, white staining in the concrete due to leaching apparent |
| 10 | Rotation in the steel beams identified using level as reference |
| 11 | Timber parapets with brick pilasters either end of bridge` |
| 12 | View north over bridge |

# APPENDIX B

## Documents Relating to Approval in Principle for Assessment

## FORM AA (BRIDGES)

## APPROVAL IN PRINCIPLE FOR ASSESSMENT

| | |
|---|---|
| **STRUCTURE/LINE NAME** | Rowe<br>Dismantled Whitchurch to Welshpool Line |
| **ELR/STRUCTURE No.** | GNQ4/14 |

**BRIEF DESCRIPTION OF EXISTING BRIDGE:**

| | | |
|---|---|---|
| a) | Span Arrangement | 1 No 8.740 m clear span, 29° skew |
| b) | Superstructure Type | Steel Beams with concrete jack arches. |
| c) | Substructure Type | Brick abutments and wing walls |
| d) | Detail of any Special Features | None |

**ASSESSMENT CRITERIA:**

| | | | |
|---|---|---|---|
| a) | Loading and Speed | Assessment loading to BD 21/97 Speed 60 mph | |
| b) | Codes to be used | BA 16/97 (inc. amendment No 1) | Assessment of Highway Bridges and Structures |
| | | BA 61/96 | Assessment of Composite Highway Bridges and Structures) |
| | | BD 21/97 (inc. amendment No 1) | Assessment of Highway Bridges and Structures |
| | | BD 61/96 | Assessment of Composite Highway Bridges and Structures |
| c) | Proposed Method of Structural Analysis | Hand calculations for individual beams with no allowance made for transverse Distribution. | |
| d) | Details of Special Requirements | None | |

**STRUCTURAL ASSESSMENT ENGINEER'S COMMENTS**

**Superstructure**

Arches 1 and 7 (outer Bays) are in particularly poor condition as they sound hollow throughout and the concrete is loose and friable. There is on average 15-mm of concrete loss and the remaining concrete is soft. Both spalling and efflorescent are severe. Arches 2-6 (Inner Bays) are generally in a fair condition and give a solid ring when tapped. Efflorescence is still prominent throughout the arches.

The edges of the bottom flange of the girders have signs of severe corrosion at the north quarter point with a maximum of 45mm loss to the width of the flange and 17mm from the flange thickness, yielding a 3mm thick flange. This corrosion loss will be taken into account in the analysis. The general condition of the girders can be described as poor.

The parapets are of a timber construction and hence fall outside the scope of any assessment guidelines, as such a qualitative assessment was carried out. It is recognised that in the event of an errant vehicle on the bridge the containment capability of the bridge is likely to be insufficient.

## FORM AA (BRIDGES)

## APPROVAL IN PRINCIPLE FOR ASSESSMENT

**Substructure**

The wing walls are in poor condition. There is severe spalling and mortar loss visible. There is also evidence of bulging in the walls.

The abutments are in particularly poor condition with a great deal spalling evident throughout, often with whole bricks missing. Severe cracking has been identified at the four corners of the bridge. Much of the deterioration in the abutments may be attributed to the leaching action taking place in the mortar, which is likely to have the most significant effect on the capacity of the abutments. Movement within the abutment walls is evident with bulging at mid height particularly clear.

The poor condition of the abutments indicates that they are unlikely to be capable of carrying the full assessment live loading.

**CIVIL ENGINEER'S COMMENTS**

Distribution by simple statics will produce a conservative capacity for the longitudinal girders.
Further analysis may be required depending on the outcome.

**BRB WORKS GROUP COMMENTS – if applicable**

**PROPOSED CATEGORY FOR INDEPENDENT CHECK:**

| | |
|---|---|
| SUPERSTRUCTURE | **Category 1**<br>(Hand calculations for individual beams)<br>**Qualitative Assessment**<br>(Jack Arches) |
| SUBSTRUCTURE | (Qualitative Assessment) |

**CATEGORY 1**

**THE ABOVE ASSESSMENT, WITH AMENDMENTS SHOWN, IS APPROVED IN PRINCIPLE:**

SIGNED [redacted]

TITLE Senior Civil Engineer

DATE 14th September 1999

FORM AA/1 (BRIDGES)

## APPROVAL IN PRINCIPLE FOR ASSESSMENT

## ADDITIONAL INFORMATION REQUIRED FOR BRB OWNED PUBLIC ROAD OVER BRIDGE ASSESSMENT AS PART OF BRIDGE GUARD III.

| | |
|---|---|
| **STRUCTURE/LINE NAME** | Rowe, Unclassified |
| **ELR/STRUCTURE No.** | GNQ4/14 |
| **SCOPE OF ASSESSMENT** | Quantitative assessment of beams.<br>Qualitative assessment of jack arches.<br>Qualitative assessment of abutments and wing walls. |

## ASSESSMENT CRITERIA

a) **Standards and Codes of Practice to be used in assessment:**

| | |
|---|---|
| BA 16/97 (inc. amendment No 1) | Assessment of Highway Bridges and Structures |
| BA 61/96 | Assessment of Composite Highway Bridges and Structures |
| BD 21/97 (inc. amendment No 1) | Assessment of Highway Bridges and Structures |
| BD 61/96 | Assessment of Composite Highway Bridges and Structures |

b) **Proposed method of structural analysis:**
Hand calculations for individual beams with no allowance made for transverse distribution. Loss of section will be considered in the analysis.

c) **Planned Highway works/modification at this site:**
None known

d) **Road designation/class and whether classed as Heavy Load Route:**
Unclassified - not a Department Heavy Load Route

e) **Any other requirement:**
None

The above is agreed subject to the amendments and comments shown below:

SIGNED [redacted]

TITLE Technical Director, Babtie Group

DATE 10/9/99.

For and on behalf of Shropshire County Council

# APPENDIX C

## Assessment Calculations and Certification Documents

| Babtie<br>2 St George's House,<br>Vernon Gate,<br>Derby, DE1 1UQ.<br>Tel: 01332 285100 - Fax: 01332 285101 | Project<br>Rail Property / Shropshire CC Bridge Assessments | | | | Job Ref.<br>031417 | |
|---|---|---|---|---|---|---|
| | Part of Structure<br>Rowe Bridge GNQ4/14 - Jack Arch Bridge | | | | Sheet no./rev.<br>GNQ4/14 1 A | |
| | Calc. by<br>[redacted] | Date<br>06/09/99 | [redacted] | Date<br>22/9/99 | App'd by | Date |

| Ref. | Calculations | Output |
|---|---|---|

# Steel Jack Arch Bridge Assessment

## TABLE OF CONTENTS

**Section** **Page**

# 1. INTRODUCTION

**INTRODUCTION**

This bridge, referenced GNQ4/14 is a single span jack arch bridge carrying an unclassified road over a disused railway line.

The Ordnance Survey National Grid Reference for the bridge is SJ 450 357

Calculations for the analysis of the Jack Arch beam are to be performed using a system developed through the TEDDS v4.0 system.

The assessment will be carried out in accordance with BD21/97.

The steel beams are in particularly poor condition and hence the section properties have been reduced to represent measured section properties on site.

**Geometry**


The bridge is made up from 8 steel beams with jack arches formed in concrete between them.

| | | | | | |
|---|---|---|---|---|---|
| Clear distance between abutments is; $S_{clear}$ | | = | 8.740m | | |
| Depth of steel section; | Dp | = | 432.8 mm | | |
| Span to be used in calculations; | S | = | $(Dp \times 2/3) + S_{clear}$ | = | **9.029** m |

| Babtie<br>2 St George's House, Vernon Gate, Derby, DE1 1UQ.<br>Tel: 01332 285100 - Fax: 01332 285101 | Project: Rail Property / Shropshire CC Bridge Assessments | | | Job Ref.: 031417 | |
|---|---|---|---|---|---|
| | Part of Structure: Rowe Bridge GNQ4/14 - Jack Arch Bridge | | | Sheet no./rev.: GNQ4/14 3 A | |
| | Calc. by: [redacted] | Date: 06/09/99 | Chck'd by: [redacted] | Date: 22/9/99 | App'd by: | Date: |

| Ref. | Calculations | Output |
|---|---|---|

## 2. SUMMARY OF RESULTS

### Moment capacities for internal and external beams:

**SEVERELY CORRODED INTERNAL BEAM**

Moment capacity; $M_DINT_C$ = **250.209** kNm
Shear Capacity; $V_DINT_C$ = **493.299** kN

**GENERAL INTERNAL BEAM**

Moment capacity; $M_DINT_G$ = **361.000** kNm
Shear Capacity; $V_DINT_G$ = **509.598** kN

**SEVERELY CORRODED EXTERNAL BEAM**

Moment capacity; $M_DEXT_C$ = **215.420** kNm
Shear Capacity; $V_DEXT_C$ = **490.792** kN

**GENERAL EXTERNAL BEAM**

Moment capacity; $M_DEXT_G$ = **322.127** kNm
Shear Capacity; $V_DEXT_G$ = **502.759** kN

### Effects on internal and external beams:

**INTERNAL BEAM DUE TO HA(UDL & KEL) LOADING**

Maximum Moment; $M_{max}$ = **697.251** kNm
Moment at quarter span; $QM_{max}$ = **522.938** kNm
Maximum Shear; $V_{max}$ = **321.115** kN
Shear at quarter span; $VQ_{max}$ = **177.510** kN

**EXTERNAL BEAM DUE TO HA(UDL & KEL) LOADING**

Maximum Moment; $M_{max}X$ = **574.907** kNm
Moment at quarter span; $QM_{max}X$ = **431.180** kNm
Maximum Shear; $V_{max}X$ = **270.303** kN
Shear at quarter span; $VQ_{max}X$ = **152.103** kN

This results in a lower bound limiting capacity factor of **0.256**, i.e. 3 tonnes loading + Group 2 Fire Engines.

**INTERNAL BEAM DUE TO HA (SINGLE WHEEL) LOADING ( 3T + G2FE )**

Maximum Moment; $SWM_{max}$ = **316.561** kNm
Moment at quarter span; $SWQM_{max}$ = **237.420** kNm
Maximum Shear; $SWV_{max}$ = **140.249** kN

**INTERNAL BEAM DUE TO HA (SINGLE AXLE) LOADING ( 3T + G2FE )**

Maximum Moment; $SAM_{max}$ = **340.933** kNm
Moment at quarter span; $SAQM_{max}$ = **255.700** kNm
Maximum Shear; $SAV_{max}$ = **151.047** kN

**INTERNAL BEAM DUE TO HA (SINGLE AXLE) LOADING ( 3T )**

Maximum Moment; $SAM_{max}3$ = **317.709** kNm
Moment at quarter span; $SAQM_{max}3$ = **238.282** kNm
Maximum Shear; $SAV_{max}3$ = **140.758** kN

The effects calculated for the single wheel and single axle cases have resulted in a reduction in capacity from 3 tonnes + Group 2 fire engines to 3 tonnes.

The qualitative assessment for the jack arches, abutments and foundations identified no area that would warrant reducing the capacity of the bridge further.

THE CAPACITY OF THE BRIDGE STRUCTURE IS 3 TONNES.

| Babtie<br>2 St George's House,<br>Vernon Gate,<br>Derby, DE1 1UQ.<br>Tel: 01332 285100 - Fax: 01332 285101 | Project: Rail Property / Shropshire CC Bridge Assessments<br>Part of Structure: Rowe Bridge GNQ4/14 - Jack Arch Bridge | Job Ref.: 031417<br>Sheet no./rev.: GNQ4/14 4 A |
|---|---|---|
| | Calc. by: [redacted] · Date: 06/09/99 · Chck'd by: [redacted] · Date: 22/9/99 | App'd by: · Date: |
| Ref. | Calculations | Output |

# 3. BEAM CAPACITIES

## 3.1. Internal Beam Capacity at Critical Section

SEVERELY CORRODED SECTION PROPERTIES

To start the beam will be assessed using the critical section properties, following this the bridge will be looked at with the variance in properties along its length, particularly as the mid section of the beam has been subject to very minor corrosion. This consideration may improve to bridge capacity due to additional moment capacity obtained by doing this.

The section dimensions have been obtained from site measurement and previous assessment records.

INPUT

**Web of section**
; $T_1 =$ 10.3 mm
; $d_1 =$ 400.8 mm
**Top flange of section**
; $T_2 =$ 22.2 mm
; $d_2 =$ 169.9 mm
**Bottom flange of section**
; $T_3 =$ 9.8 mm
; $d_3 =$ 125 mm

; CALCULATION OF SECTION PROPERTIES;

AREA;

$A = 91.25$ cm$^2$

2nd Moment of Area

$I_{uu} = 24308$ cm$^4$ ; $I_{vv} = 1070$ cm$^4$ ; $I_{xx} = 24308$ cm$^4$ ; $I_{yy} = 1070$ cm$^4$

Radius of Gyration

$r_{uu} = 163.2$ mm ; $r_{vv} = 34.3$ mm ; $r_{xx} = 163.2$ mm ; $r_{yy} = 34.3$ mm

Plastic Section Modulus

$S_{xx} = 1305440$ mm$^3$ ; $S_{yy} = 209120$ mm$^3$

Distance to Combined Centroid

$X_e = 0.0$ mm ; $Y_e = 59.9$ mm ;

Distance to Equal Axis Area (only shapes with all rectangles at 90 degs)

$X_p = 0.0$ mm ; $Y_p = 123.7$ mm

Elastic Section Modulus

$Z_{xx} = 900090$ mm$^3$ ; $Z_{yy} = 126010$ mm$^3$

Vertical Distance from the extreme tensile fibre to the neutral axis

$NA_{xbar} = d_1/2 + T_3 + Y_e =$ **270.10 mm**

Vertical Distance from the extreme tensile fibre to the equal area axis.

$EA_{xbar} = d_1/2 + T_3 + Y_p =$ **333.90 mm**

| Babtie<br>2 St George's House,<br>Vernon Gate,<br>Derby, DE1 1UQ.<br>Tel: 01332 285100 - Fax: 01332 285101 | Project: Rail Property / Shropshire CC Bridge Assessments | | | Job Ref.: 031417 | |
|---|---|---|---|---|---|
| | Part of Structure: Rowe Bridge GNQ4/14 - Jack Arch Bridge | | | Sheet no./rev.: GNQ4/14 5 A | |
| | Calc. by: [redacted] | Date: 06/09/99 | Chck'd by: [redacted] | Date: 22/9/99 | App'd by: | Date: |

| Ref. | Calculations | Output |
|---|---|---|

## SECTION CAPACITY OF SEVERELY CORRODED BEAM.

The section capacity will be assessed based on the guidelines within BA56/96, BD21/97, and BD56/96.

As no information is available to provide a definite yield strength of the steel, a characteristic will be assumed as defined in BD21/97 for steel produced before 1955.

$f_y$ = 230 N/mm$^2$

**SECTION CLASSIFICATION:** **BD56/97 9.3.7**

**BD56/97: cl. 9.3.7.2: Webs.**

The depth between the elastic neutral axis of the section and the compressive edge of the web should not exceed: $28t_w\sqrt{\frac{355}{\sigma_{yw}}}$

where: $t_w$ is the thickness of the web plate
$\sigma_{yw}$ is the nominal yield stress of the web material.

$28 \times T_1 \times \sqrt{(355 / f_y)}$ = **358.299** mm

Where the depth between the elastic neutral axis of the section and the compressive edge of the web is:

$d_1 + T_3 - NA_{xbar}$ = **140.500** mm

Section **"passes"** cl. 9.3.7.2 check.

**BD56/97: cl. 9.3.7.3: Compression Flanges.**

The projection of the compression flange outstand, $b_{fo}$, should not exceed:

$$7t_{fo}\sqrt{\frac{355}{\sigma_{yf}}}$$

where: $t_{fo}$ is the compression flange thickness
$\sigma_{yf}$ is the nominal yield stress of the web material.

$7 \times T_2 \times \sqrt{(355 / f_y)}$ = **193.064** mm

Where the projection of the compression flange outstand is:

$(d_2 - T_1) / 2$ = **79.800** mm

Section **"passes"** cl. 9.3.7.3 check.

As the section passes both the checks the section may be described as compact.

**SLENDERNESS:** **BD56/97 cl.9.7**

**BD56/97: cl.9.7.1: Uniform I, channel, tee or angle sections.**

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

$\lambda_e$ = Effective length determined in accordance with 9.6.1
i.e As the beam is effectively restrained by the concrete, it is classed as stable against lateral torsional buckling the effective length is zero. Therefore:

$\lambda_{LT}$ = **0.00**

**LIMITING COMPRESSIVE STRESS:** **BD56/97 cl. 9.8**

**BD56/97: cl. 9.8.1: General.**

The value of $\sigma_{li} / \sigma_{yc}$ should be obtained from figure 10 according to the value of:

$$\lambda_{LT}\sqrt{\frac{\sigma_{yc}}{355}}$$

where: $\sigma_{yc}$ = Nominal yield stress of the web material.

As $\lambda_{LT}$ is zero then using figure 10 $\sigma_{li} / \sigma_{yc}$ yields 1.0, hence $\sigma_{li} = \sigma_{yc} = f_y =$ <u>230</u> N/mm$^2$

**BD56/97: cl. 9.8.1: Compact sections.**

The limiting compressive stress, $\sigma_{lc}$, should be taken as $\sigma_{li}$.

$\sigma_{lc}$ = $\sigma_{li}$ = **230** N/mm$^2$

**BEAMS WITHOUT LONGITUDINAL STIFFENERS:** **BD56/97 cl. 9.9**

**BD56/97: cl. 9.9.1: Bending resistance.**

**BD56/97: cl.9.9.1.2: Compact sections.**

The bending resistance, $M_D$, of a compact section should be taken as:

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

where:

$Z_{pe}$ = Plastic modulus of the section.
$\sigma_{lc}$ = Limiting compressive stress.
$\gamma_m$ = Partial safety factor for the material .
From Table 2, $\gamma_m$ = 1.2
$\gamma_{f3}$ = 1.1, But not used as incorporated into loading.

$M_D$ = $((S_{XX} \times \sigma_{lc}) / (\gamma_m))$ = <u>250.209</u> kNm

**The moment capacity of the section is <u>250.209</u> kNm**

| Babtie<br>2 St George's House, Vernon Gate, Derby, DE1 1UQ.<br>Tel: 01332 285100 - Fax: 01332 285101 | Project: Rail Property / Shropshire CC Bridge Assessments | | | Job Ref.: 031417 | |
|---|---|---|---|---|---|
| | Part of Structure: Rowe Bridge GNQ4/14 - Jack Arch Bridge | | | Sheet no./rev.: GNQ4/14 7 A | |
| | Calc. by: [redacted] | Date: 06/09/99 | Chck'd by: [redacted] Date: 22/9/99 | App'd by | Date |

| Ref. | Calculations | Output |
|---|---|---|

**BD56/97: cl. 9.9.2: Shear resistance.**

**BD56/97: cl. 9.9.2.2: Shear resistance under pure shear.**

The shear resistance, $V_D$, of a web panel under pure shear should be taken as:

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

Where:

| | | |
|---|---|---|
| $t_w$ | = | Thickness of the web |
| $d_w$ | = | The overall depth of a rolled section |
| $h_h$ | = | The height of the largest hole or cut out being considered<br>$h_h$ = 0 mm |
| $\gamma_m$ | = | Partial safety factor for the material .<br>From Table 2, $\gamma_m$ = 1.2 |
| $\gamma_{f3}$ | = | 1.1, But not used as incorporated into loading. |
| $\tau_l$ | = | Limiting shear strength of the web panel.<br>See notes below. |

The limiting shear strength, $\tau_l$, is given by:

$$\frac{\tau_l}{\tau_y} \propto \lambda = \frac{d_{we}}{t_w}\sqrt{\frac{\sigma_{yw}}{355}}$$

where:

| | | |
|---|---|---|
| $d_{we}$ | = | Depth of section between the flange plates |
| $t_w$ | = | Thickness of the web |
| $\sigma_{yw}$ | = | Nominal yield stress of the web material |

$\lambda$ = $(d_1 / T_1) \times \sqrt{(f_y / 355)}$ = **31.321**

Interpolation from one of figures 11 to 17 is dependant on a number of other factors, however as $\lambda$ is less than 50, $\tau_l / \tau_y$ =1.00

Where: $\tau_y = \dfrac{\sigma_{yw}}{\sqrt{3}}$; = **132.791** N/mm$^2$

Hence; $\tau_l = \tau_y$ = **132.791** N/mm$^2$

Shear Capacity, $V_D$:

$V_D$ = $((T_1 \times ((d_1 + T_2 + T_3) - h_h)) / (\gamma_m)) \times \tau_l$ = **493.299** kN

**RECAP OF CAPACITIES OF A SEVERELY CORRODED INTERNAL BEAM**

| | | | | | |
|---|---|---|---|---|---|
| **Moment capacity;** | $M_D INT_C$ | = | $M_D$ | = | **250.209 kNm** |
| **Shear Capacity;** | $V_D INT_C$ | = | $V_D$ | = | **493.299 kN** |

## 3.2. Internal Beam Capacity at General Section

**GENERAL BEAM SECTION PROPERTIES**

To start the beam will be assessed using the critical section properties, following this the bridge will be looked at with the variance in properties along its length, particularly as the mid section of the beam has been subject to very minor corrosion. This consideration may improve to bridge capacity due to additional moment capacity obtained by doing this.

The section dimensions have been obtained from site measurement and previous assessment records.

**INPUT**

**Web of section**

; $T_1$ = 10.3 mm
; $d_1$ = 403.9 mm

**Top flange of section**

; $T_2$ = 22.2 mm
; $d_2$ = 169.9 mm

**Bottom flange of section**

; $T_3$ = 21 mm
; $d_3$ = 150 mm

**; CALCULATION OF SECTION PROPERTIES;**

**AREA;**

A = **110.82** cm$^2$

**2$^{nd}$ Moment of Area**

$I_{uu}$ = **36858** cm$^4$ ; $I_{vv}$ = **1502** cm$^4$ ; $I_{xx}$ = **36858** cm$^4$ ; $I_{yy}$ = **1502** cm$^4$

**Radius of Gyration**

$r_{uu}$ = **182.4** mm ; $r_{vv}$ = **36.8** mm ; $r_{xx}$ = **182.4** mm ; $r_{yy}$ = **36.8** mm

**Plastic Section Modulus**

$S_{xx}$ = **1883480** mm$^3$ ; $S_{yy}$ = **289040** mm$^3$

**Distance to Combined Centroid**

$X_e$ = **0.0** mm ; $Y_e$ = **12.1** mm ;

**Distance to Equal Axis Area (only shapes with all rectangles at 90 degs)**

$X_p$ = **0.0** mm ; $Y_p$ = **30.2** mm

**Elastic Section Modulus**

$Z_{xx}$ = **1567910** mm$^3$ ; $Z_{yy}$ = **176760** mm$^3$

**Vertical Distance from the extreme tensile fibre to the neutral axis**

$NA_{xbar}$ = $d_1/2 + T_3 + Y_e$ = **235.05** mm

**Vertical Distance from the extreme tensile fibre to the equal area axis.**

$EA_{xbar}$ = $d_1/2 + T_3 + Y_p$ = **253.15** mm

## SECTION CAPACITY OF GENERAL INTERNAL BEAM.

The section capacity will be assessed based on the guidelines within BA56/96, BD21/97, and BD56/96.

As no information is available to provide a definite yield strength of the steel, a characteristic will be assumed as defined in BD21/97 for steel produced before 1955.

$f_y$ = 230 N/mm$^2$

**SECTION CLASSIFICATION:** **BD56/97 9.3.7**

**BD56/97: cl. 9.3.7.2: Webs.**

The depth between the elastic neutral axis of the section and the compressive edge of the web should not exceed: $28t_w\sqrt{\dfrac{355}{\sigma_{yw}}}$

where: $t_w$ is the thickness of the web plate
$\sigma_{yw}$ is the nominal yield stress of the web material.

$28 \times T_1 \times \sqrt{(355 / f_y)}$ = **358.299** mm

Where the depth between the elastic neutral axis of the section and the compressive edge of the web is:

$d_1 + T_3 - NA_{xbar}$ = **189.850** mm

Section **"passes"** cl. 9.3.7.2 check.

**BD56/97: cl. 9.3.7.3: Compression Flanges.**

The projection of the compression flange outstand, $b_{fo}$, should not exceed:

$$7t_{fo}\sqrt{\frac{355}{\sigma_{yf}}}$$

where: $t_{fo}$ is the compression flange thickness
$\sigma_{yf}$ is the nominal yield stress of the web material.

$7 \times T_2 \times \sqrt{(355 / f_y)}$ = **193.064** mm

Where the projection of the compression flange outstand is:

$(d_2 - T_1) / 2$ = **79.800** mm

Section **"passes"** cl. 9.3.7.3 check.

As the section passes both the checks the section may be described as compact.

| Babtie<br>2 St George's House,<br>Vernon Gate,<br>Derby, DE1 1UQ.<br>Tel: 01332 285100 - Fax: 01332 285101 | Project: Rail Property / Shropshire CC Bridge Assessments | | | Job Ref. 031417 | |
|---|---|---|---|---|---|
| | Part of Structure: Rowe Bridge GNQ4/14 - Jack Arch Bridge | | | Sheet no./rev. GNQ4/14 10 A | |
| | Calc. by [redacted] | Date 06/09/99 | Chck'd by [redacted] | Date 22/9/99 | App'd by | Date |
| Ref. | Calculations | | | Output | |

**SLENDERNESS:** **BD56/97 cl.9.7**

**BD56/97: cl.9.7.1: Uniform I, channel, tee or angle sections.**

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

$\lambda_e$ = Effective length determined in accordance with 9.6.1
i.e As the beam is effectively restrained by the concrete, it is classed as stable against lateral torsional buckling the effective length is zero. Therefore:

$\lambda_{LT}$ = **0.00**

**LIMITING COMPRESSIVE STRESS:** **BD56/97 cl. 9.8**

**BD56/97: cl. 9.8.1: General.**

The value of $\sigma_{li} / \sigma_{yc}$ should be obtained from figure 10 according to the value of:

$$\lambda_{LT}\sqrt{\frac{\sigma_{yc}}{355}}$$

where: $\sigma_{yc}$ = Nominal yield stress of the web material.

As $\lambda_{LT}$ is zero then using figure 10 $\sigma_{li} / \sigma_{yc}$ yields 1.0, hence $\sigma_{li} = \sigma_{yc} = f_y =$ 230 N/mm$^2$

**BD56/97: cl. 9.8.1: Compact sections.**

The limiting compressive stress, $\sigma_{lc}$, should be taken as $\sigma_{li}$.

$\sigma_{lc}$ = $\sigma_{li}$ = **230** N/mm$^2$

**BEAMS WITHOUT LONGITUDINAL STIFFENERS:** **BD56/97 cl. 9.9**

**BD56/97: cl. 9.9.1: Bending resistance.**

**BD56/97: cl.9.9.1.2: Bending resistance of Compact sections.**

The bending resistance, $M_D$, of a compact section should be taken as:

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

where:
$Z_{pe}$ = Plastic modulus of the section.
$\sigma_{lc}$ = Limiting compressive stress.
$\gamma_m$ = Partial safety factor for the material .
From Table 2, $\gamma_m = 1.2$
$\gamma_{f3}$ = 1.1, But not used as incorporated into loading.

$M_D$ = $((S_{xx} \times \sigma_{lc}) / (\gamma_m))$ = **361.000** kNm

**The moment capacity of the section is 361.000 kNm**

**BD56/97: cl. 9.9.2: Shear resistance.**

**BD56/97: cl. 9.9.2.2: Shear resistance under pure shear.**

The shear resistance, $V_D$, of a web panel under pure shear should be taken as:

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

Where:

| | | |
|---|---|---|
| $t_w$ | = | Thickness of the web |
| $d_w$ | = | The overall depth of a rolled section |
| $h_h$ | = | The height of the largest hole or cut out being considered<br>$h_h = 0$ mm |
| $\gamma_m$ | = | Partial safety factor for the material .<br>From Table 2, $\gamma_m = 1.2$ |
| $\gamma_{f3}$ | = | 1.1, But not used as incorporated into loading. |
| $\tau_l$ | = | Limiting shear strength of the web panel.<br>See notes below. |

The limiting shear strength, $\tau_l$, is given by:

$$\frac{\tau_l}{\tau_y} \propto \lambda = \frac{d_{we}}{t_w}\sqrt{\frac{\sigma_{yw}}{355}}$$

where:

| | | |
|---|---|---|
| $d_{we}$ | = | Depth of section between the flange plates |
| $t_w$ | = | Thickness of the web |
| $\sigma_{yw}$ | = | Nominal yield stress of the web material |

$\lambda$ = $(d_1 / T_1) \times \sqrt{(f_y / 355)}$ = **31.564**

Interpolation from one of figures 11 to 17 is dependant on a number of other factors, however as $\lambda$ is less than 50, $\tau_l / \tau_y = 1.00$

Where: $\tau_y = \frac{\sigma_{yw}}{\sqrt{3}}$; = **132.791** N/mm$^2$

Hence; $\tau_l = \tau_y$ = **132.791** N/mm$^2$

Shear Capacity, $V_D$:

$V_D$ = $((T_1 \times ((d_1 + T_2 + T_3) - h_h)) / (\gamma_m)) \times \tau_l$ = **509.598 kN**

**<u>RECAP OF CAPACITIES OF A GENERAL INTERNAL BEAM</u>**

| | | | | | |
|---|---|---|---|---|---|
| **Moment capacity;** | $M_D INT_G$ | = | $M_D$ | = | **361.000 kNm** |
| **Shear Capacity;** | $V_D INT_G$ | = | $V_D$ | = | **509.598 kN** |

| Babtie<br>2 St George's House,<br>Vernon Gate,<br>Derby, DE1 1UQ.<br>Tel: 01332 285100 - Fax: 01332 285101 | Project: Rail Property / Shropshire CC Bridge Assessments | | | | Job Ref.: 031417 | |
|---|---|---|---|---|---|---|
| | Part of Structure: Rowe Bridge GNQ4/14 - Jack Arch Bridge | | | | Sheet no./rev.: GNQ4/14 12 A | |
| | Calc. by: [redacted] | Date: 06/09/99 | Chck'd by: [redacted] | Date: 21/9/99 | App'd by | Date |

| Ref. | Calculations | Output |
|---|---|---|

## 3.3. External Beam Capacity at Critical Section

**SEVERELY CORRODED SECTION PROPERTIES**

To start the beam will be assessed using the critical section properties, following this the bridge will be looked at with the variance in properties along its length, particularly as the mid section of the beam has been subject to very minor corrosion. This consideration may improve to bridge capacity due to additional moment capacity obtained by doing this.

The section dimensions have been obtained from site measurement and previous assessment records.

**INPUT**

**Web of section**
; $T_1 =$ 10.3 mm
; $d_1 =$ 403.9 mm
**Top flange of section**
; $T_2 =$ 22.2 mm
; $d_2 =$ 169.9 mm
**Bottom flange of section**
; $T_3 =$ 4.5 mm
; $d_3 =$ 147 mm

**; CALCULATION OF SECTION PROPERTIES;**

**AREA;**

A = **85.93** $cm^2$

**2nd Moment of Area**

$I_{uu}$ = **20349** $cm^4$ ; $I_{vv}$ = **1030** $cm^4$ ; $I_{xx}$ = **20349** $cm^4$ ; $I_{yy}$ = **1030** $cm^4$

**Radius of Gyration**

$r_{uu}$ = **153.9** mm ; $r_{vv}$ = **34.6** mm ; $r_{xx}$ = **153.9** mm ; $r_{yy}$ = **34.6** mm

**Plastic Section Modulus**

$S_{xx}$ = **1123930** $mm^3$ ; $S_{yy}$ = **195230** $mm^3$

**Distance to Combined Centroid**

$X_e$ = **0.0** mm ; $Y_e$ = **77.8** mm ;

**Distance to Equal Axis Area (only shapes with all rectangles at 90 degs)**

$X_p$ = **0.0** mm ; $Y_p$ = **151.0** mm

**Elastic Section Modulus**

$Z_{xx}$ = **715920** $mm^3$ ; $Z_{yy}$ = **121260** $mm^3$

**Vertical Distance from the extreme tensile fibre to the neutral axis**

$NA_{xbar}$ = $d_1/2 + T_3 + Y_e$ = **284.25** mm

**Vertical Distance from the extreme tensile fibre to the equal area axis.**

$EA_{xbar}$ = $d_1/2 + T_3 + Y_p$ = **357.45** mm

| Ref. | Calculations | Output |
|---|---|---|

**SECTION CAPACITY OF SEVERELY CORRODED EXTERNAL BEAM.**

The section capacity will be assessed based on the guidelines within BA56/96, BD21/97, and BD56/96.

As no information is available to provide a definite yield strength of the steel, a characteristic will be assumed as defined in BD21/97 for steel produced before 1955.

$f_y$ = 230 N/mm$^2$

**SECTION CLASSIFICATION:** **BD56/97 9.3.7**

**BD56/97: cl. 9.3.7.2: Webs.**

The depth between the elastic neutral axis of the section and the compressive edge of the web should not exceed:

$$28t_w\sqrt{\frac{355}{\sigma_{yw}}}$$

where: $t_w$ is the thickness of the web plate
$\sigma_{yw}$ is the nominal yield stress of the web material.

$28 \times T_1 \times \surd(355 / f_y)$ = **358.299** mm

Where the depth between the elastic neutral axis of the section and the compressive edge of the web is:

$d_1 + T_3 - NA_{xbar}$ = **124.150** mm

Section **"passes"** cl. 9.3.7.2 check.

**BD56/97: cl. 9.3.7.3: Compression Flanges.**

The projection of the compression flange outstand, $b_{fo}$, should not exceed:

$$7t_{fo}\sqrt{\frac{355}{\sigma_{yf}}}$$

where: $t_{fo}$ is the compression flange thickness
$\sigma_{yf}$ is the nominal yield stress of the web material.

$7 \times T_2 \times \surd(355 / f_y)$ = **193.064** mm

Where the projection of the compression flange outstand is:

$(d_2 - T_1) / 2$ = **79.800** mm

Section **"passes"** cl. 9.3.7.3 check.

As the section passes both the checks the section may be described as compact.

| Babtie<br>2 St George's House,<br>Vernon Gate,<br>Derby, DE1 1UQ.<br>Tel: 01332 285100 - Fax: 01332 285101 | Project: Rail Property / Shropshire CC Bridge Assessments | | | | Job Ref.: 031417 | |
|---|---|---|---|---|---|---|
| | Part of Structure: Rowe Bridge GNQ4/14 - Jack Arch Bridge | | | | Sheet no./rev.: GNQ4/14 14 A | |
| | Calc. by: [redacted] | Date: 06/09/99 | Chck'd by: [redacted] | Date: 22/9/99 | App'd by | Date |

| Ref. | Calculations | Output |
|---|---|---|

**SLENDERNESS:** **BD56/97 cl.9.7**

**BD56/97: cl.9.7.1: Uniform I, channel, tee or angle sections.**

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

$\lambda_e$ = Effective length determined in accordance with 9.6.1
i.e As the beam is effectively restrained by the concrete on one side and tie rods at approximately 2.6m centres the effective length of the section will be taken as; $l_e$ = 2.6 m
$\eta$ = 1 Conservatively

$\nu$ is obtained from table 9 using the following parameters

$$\lambda_F = \frac{l_e}{r_y}\left(\frac{t_f}{D}\right) \quad and \quad i = \frac{I_c}{I_c + I_t}$$

$k_4$ is calculated from the expression in cl. 9.7.2 for beams symmetrical about the minor axis.

$$k_4 = \left[\frac{4(Z_{pe})^2\left(1-\frac{I_y}{I_x}\right)}{A^2h^2}\right]^{\frac{1}{4}}$$

**Calculations for $\nu$:**

| | | | | | |
|---|---|---|---|---|---|
| Mean thickness of flanges; | $t_f$ | = | $(T_2 + T_3)/2$ | = | **13.4** mm |
| Overall Depth of Section; | D | = | $d_1 + T_2 + T_3$ | = | **430.6** mm |
| Determine $\lambda_F$; | $\lambda_F$ | = | $(l_e / r_{yy}) \times (t_f / D)$ | = | **2.330** |
| Moment of Inertia Comp Flange; | $I_c$ | = | $(d_2 \times T_2^3) / 12$ | = | **15.491** $cm^4$ |
| Moment of Inertia Tens Flange; | $I_t$ | = | $(d_3 \times T_3^3) / 12$ | = | **0.112** $cm^4$ |
| | i | = | $I_c / (I_c + I_t)$ | = | **0.993** |

$\nu$ = $((4\times i\times(1-i)+0.05\times(\lambda_F)^2+(0.8\times(2\times i-1))^2)^{0.5}+(0.8\times(2\times i-1)))^{-0.5}$ = **0.756**

**Calculations for $k_4$:**

Dist. Between centroids of flanges; h = D - $t_f$ = **417.3** mm

$k_4$ = $((4 \times Z_{xx}^2 \times (1 - I_{yy}/I_{xx})) / (A^2 \times h^2))^{0.25}$ = **0.624**

$\lambda_{LT}$ = $((l_e / r_{yy}) \times k_4 \times \eta \times \nu)$ = **35.448**

| Babtie<br>2 St George's House,<br>Vernon Gate,<br>Derby, DE1 1UQ.<br>Tel: 01332 285100 - Fax: 01332 285101 | Project<br>Rail Property / Shropshire CC Bridge Assessments | | | Job Ref.<br>031417 | |
|---|---|---|---|---|---|
| | Part of Structure<br>Rowe Bridge GNQ4/14 - Jack Arch Bridge | | | Sheet no./rev.<br>GNQ4/14 15 A | |
| | Calc. by<br>[redacted] | Date<br>06/09/99 | Chck'd by<br>[redacted] | Date<br>22/9/99 | App'd by | Date |

| Ref. | Calculations | Output |
|---|---|---|

**LIMITING COMPRESSIVE STRESS:** **BD56/97 cl. 9.8**

**BD56/97: cl. 9.8.1: General.**

The value of $\sigma_{li} / \sigma_{yc}$ should be obtained from figure 10 according to the value of:

$$\lambda_{LT}\sqrt{\frac{\sigma_{yc}}{355}}$$

where: $\sigma_{yc}$ = Nominal yield stress of the web material.

$\sigma_{yc}$ = $f_y$ = **230** N/mm$^2$

$\lambda_{LT} \times \sqrt{(\sigma_{yc} / 355 \text{ N/mm}^2)}$ = **28.533**

As this is less than 45 it will be taken that; $\sigma_{yc}$ = $\sigma_{li}$

**BD56/97: cl. 9.8.1: Compact sections.**

The limiting compressive stress, $\sigma_{lc}$, should be taken as $\sigma_{li}$.

$\sigma_{lc}$ = $\sigma_{li}$ = **230** N/mm$^2$

**BEAMS WITHOUT LONGITUDINAL STIFFENERS:** **BD56/97 cl. 9.9**

**BD56/97: cl. 9.9.1: Bending resistance.**

**BD56/97: cl.9.9.1.2: Compact sections.**

The bending resistance, $M_D$, of a compact section should be taken as:

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

where:
$Z_{pe}$ = Plastic modulus of the section.
$\sigma_{lc}$ = Limiting compressive stress.
$\gamma_m$ = Partial safety factor for the material .
From Table 2, $\gamma_m$ = 1.2
$\gamma_{f3}$ = 1.1, But not used as incorporated into loading.

$M_D$ = $((S_{xx} \times \sigma_{lc}) / (\gamma_m))$ = **215.420** kNm

**The moment capacity of the section is 215.420 kNm**

| Babtie<br>2 St George's House,<br>Vernon Gate,<br>Derby, DE1 1UQ.<br>Tel: 01332 285100 - Fax: 01332 285101 | Project: Rail Property / Shropshire CC Bridge Assessments | | | | Job Ref. 031417 | |
|---|---|---|---|---|---|---|
| | Part of Structure: Rowe Bridge GNQ4/14 - Jack Arch Bridge | | | | Sheet no./rev. GNQ4/14 16 A | |
| | Calc. by: [redacted] | Date: 06/09/99 | Chck'd by: [redacted] | Date: 22/9/99 | App'd by | Date |

| Ref. | Calculations | Output |
|---|---|---|

**BD56/97: cl. 9.9.2: Shear resistance.**

**BD56/97: cl. 9.9.2.2: Shear resistance under pure shear.**

The shear resistance, $V_D$, of a web panel under pure shear should be taken as:

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

Where:
- $t_w$ = Thickness of the web
- $d_w$ = The overall depth of a rolled section
- $h_h$ = The height of the largest hole or cut out being considered. $h_h = 0$ mm
- $\gamma_m$ = Partial safety factor for the material. From Table 2, $\gamma_m = 1.2$
- $\gamma_{f3}$ = 1.1, But not used as incorporated into loading.
- $\tau_l$ = Limiting shear strength of the web panel. See notes below.

The limiting shear strength, $\tau_l$, is given by:

$$\frac{\tau_l}{\tau_y} \propto \lambda = \frac{d_{we}}{t_w}\sqrt{\frac{\sigma_{yw}}{355}}$$

where:
- $d_{we}$ = Depth of section between the flange plates
- $t_w$ = Thickness of the web
- $\sigma_{yw}$ = Nominal yield stress of the web material

$\lambda$ = $(d_1 / T_1) \times \sqrt{(f_y / 355)}$ = **31.564**

Interpolation from one of figures 11 to 17 is dependant on a number of other factors, however as $\lambda$ is less than 50, $\tau_l / \tau_y = 1.00$

Where: $\tau_y = \frac{\sigma_{yw}}{\sqrt{3}}$; = **132.791** N/mm$^2$

Hence; $\tau_l = \tau_y$ = **132.791** N/mm$^2$

Shear Capacity, $V_D$:

$V_D$ = $((T_1 \times ((d_1 + T_2 + T_3) - h_h)) / (\gamma_m)) \times \tau_l$ = **490.792 kN**

**<u>RECAP OF CAPACITIES OF A SEVERELY CORRODED EXTERNAL BEAM</u>**

| | | | | | |
|---|---|---|---|---|---|
| **Moment capacity;** | **$M_D EXT_C$** | **=** | **$M_D$** | **=** | **215.420 kNm** |
| **Shear Capacity;** | **$V_D EXT_C$** | **=** | **$V_D$** | **=** | **490.792 kN** |

## 3.4. External Beam Capacity at General Section

**GENERAL BEAM SECTION PROPERTIES**

To start the beam will be assessed using the critical section properties, following this the bridge will be looked at with the variance in properties along its length, particularly as the mid section of the beam has been subject to very minor corrosion. This consideration may improve to bridge capacity due to additional moment capacity obtained by doing this.

The section dimensions have been obtained from site measurement and previous assessment records.

**INPUT**

**Web of section**

; $T_1$ = 10.3 mm
; $d_1$ = 403.9 mm

**Top flange of section**

; $T_2$ = 22.2 mm
; $d_2$ = 169.9 mm

**Bottom flange of section**

; $T_3$ = 15 mm
; $d_3$ = 160 mm

**; CALCULATION OF SECTION PROPERTIES:**

**AREA:**

A = **103.32** $cm^2$

**$2^{nd}$ Moment of Area**

$I_{uu}$ = **32448** $cm^4$ ; $I_{vv}$ = **1423** $cm^4$ ; $I_{xx}$ = **32448** $cm^4$ ; $I_{yy}$ = **1423** $cm^4$

**Radius of Gyration**

$r_{uu}$ = **177.2** mm ; $r_{vv}$ = **37.1** mm ; $r_{xx}$ = **177.2** mm ; $r_{yy}$ = **37.1** mm

**Plastic Section Modulus**

$S_{xx}$ = **1680660** $mm^3$ ; $S_{yy}$ = **266920** $mm^3$

**Distance to Combined Centroid**

$X_e$ = **0.0** mm ; $Y_e$ = **29.1** mm ;

**Distance to Equal Axis Area (only shapes with all rectangles at 90 degs)**

$X_p$ = **0.0** mm ; $Y_p$ = **66.6** mm

**Elastic Section Modulus**

$Z_{xx}$ = **1318640** $mm^3$ ; $Z_{yy}$ = **167510** $mm^3$

**Vertical Distance from the extreme tensile fibre to the neutral axis**

$NA_{xbar}$ = $d_1/2 + T_3 + Y_e$ = **246.05** mm

**Vertical Distance from the extreme tensile fibre to the equal area axis.**

$EA_{xbar}$ = $d_1/2 + T_3 + Y_p$ = **283.55** mm

| **Babtie**<br>2 St George's House,<br>Vernon Gate,<br>Derby, DE1 1UQ.<br>Tel: 01332 285100 - Fax: 01332 285101 | Project<br>Rail Property / Shropshire CC Bridge Assessments | | | | Job Ref.<br>031417 | |
|---|---|---|---|---|---|---|
| | Part of Structure<br>Rowe Bridge GNQ4/14 - Jack Arch Bridge | | | | Sheet no./rev.<br>GNQ4/14 18 A | |
| | Calc. by<br>[redacted] | Date<br>06/09/99 | Chck'd by<br>[redacted] | Date<br>22/9/99 | App'd by | Date |
| Ref. | Calculations | | | | Output | |

**SECTION CAPACITY OF GENERAL BEAM.**

The section capacity will be assessed based on the guidelines within BA56/96, BD21/97, and BD56/96.

As no information is available to provide a definite yield strength of the steel, a characteristic will be assumed as defined in BD21/97 for steel produced before 1955.

$f_y$ = 230 N/mm$^2$

**SECTION CLASSIFICATION:** **BD56/97 9.3.7**

**BD56/97: cl. 9.3.7.2: Webs.**

The depth between the elastic neutral axis of the section and the compressive edge of the web should not exceed:

$$28t_w\sqrt{\frac{355}{\sigma_{yw}}}$$

where: $t_w$ is the thickness of the web plate
$\sigma_{yw}$ is the nominal yield stress of the web material.

$28 \times T_1 \times \sqrt{(355 / f_y)}$ = **358.299** mm

Where the depth between the elastic neutral axis of the section and the compressive edge of the web is:

$d_1 + T_3 - NA_{xbar}$ = **172.850** mm

Section **"passes"** cl. 9.3.7.2 check.

**BD56/97: cl. 9.3.7.3: Compression Flanges.**

The projection of the compression flange outstand, $b_{fo}$, should not exceed:

$$7t_{fo}\sqrt{\frac{355}{\sigma_{yf}}}$$

where: $t_{fo}$ is the compression flange thickness
$\sigma_{yf}$ is the nominal yield stress of the web material.

$7 \times T_2 \times \sqrt{(355 / f_y)}$ = **193.064** mm

Where the projection of the compression flange outstand is:

$(d_2 - T_1) / 2$ = **79.800** mm

Section **"passes"** cl. 9.3.7.3 check.

As the section passes both the checks the section may be described as compact.

**Babtie**
2 St George's House,
Vernon Gate,
Derby, DE1 1UQ.
Tel: 01332 285100 - Fax: 01332 285101

| Project | | | | Job Ref. | |
|---|---|---|---|---|---|
| Rail Property / Shropshire CC Bridge Assessments | | | | 031417 | |
| Part of Structure | | | | Sheet no./rev. | |
| Rowe Bridge GNQ4/14 - Jack Arch Bridge | | | | GNQ4/14 19 A | |
| Calc. by | Date | Chck'd by | Date | App'd by | Date |
| [redacted] | 06/09/99 | [redacted] | 22/9/99 | | |

| Ref. | Calculations | Output |
|---|---|---|

**SLENDERNESS:** **BD56/97 cl.9.7**

**BD56/97: cl.9.7.1: Uniform I, channel, tee or angle sections.**

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

$\lambda_e$ = Effective length determined in accordance with 9.6.1
i.e As the beam is effectively restrained by the concrete on one side and tie rods at approximately 2.6m centres the effective length of the section will be taken as; $l_e$ = 2.6 m
$\eta$ = 1 Conservatively

v is obtained from table 9 using the following parameters

$$\lambda_F = \frac{l_e}{r_y}\left(\frac{t_f}{D}\right) \quad and \quad i = \frac{I_c}{I_c + I_t}$$

$k_4$ is calculated from the expression in cl. 9.7.2 for beams symmetrical about the minor axis.

$$k_4 = \left[\frac{4(Z_{pe})^2\left(1 - \frac{I_y}{I_x}\right)}{A^2 h^2}\right]^{\frac{1}{4}}$$

**Calculations for v:**

| | | | | | |
|---|---|---|---|---|---|
| Mean thickness of flanges; | $t_f$ | = | $(T_2 + T_3)/2$ | = | **18.6** mm |
| Overall Depth of Section; | D | = | $d_1 + T_2 + T_3$ | = | **441.1** mm |
| Determine $\lambda_F$; | $\lambda_F$ | = | $(l_e / r_{yy}) \times (t_f / D)$ | = | **2.955** |
| Moment of Inertia Comp Flange; | $I_c$ | = | $(d_2 \times T_2^3) / 12$ | = | **15.491** $cm^4$ |
| Moment of Inertia Tens Flange; | $I_t$ | = | $(d_3 \times T_3^3) / 12$ | = | **4.500** $cm^4$ |
| | i | = | $I_c / (I_c + I_t)$ | = | **0.775** |

v = $((4 \times i \times (1-i) + 0.05 \times (\lambda_F)^2 + (0.8 \times (2 \times i - 1))^2)^{0.5} + (0.8 \times (2 \times i - 1)))^{-0.5}$ = **0.793**

**Calculations for $k_4$:**

Dist. Between centroids of flanges; h = $D - t_f$ = **422.5** mm

$k_4$ = $((4 \times Z_{xx}^2 \times (1 - I_{yy}/I_{xx})) / (A^2 \times h^2))^{0.25}$ = **0.769**

$\lambda_{LT}$ = $((l_e / r_{yy}) \times k_4 \times \eta \times v)$ = **42.689**

**LIMITING COMPRESSIVE STRESS:** **BD56/97 cl. 9.8**

**BD56/97: cl. 9.8.1: General.**

The value of $\sigma_{li}$ / $\sigma_{yc}$ should be obtained from figure 10 according to the value of:

$$\lambda_{LT}\sqrt{\frac{\sigma_{yc}}{355}}$$

where: $\sigma_{yc}$ = Nominal yield stress of the web material.

$\sigma_{yc}$ = $f_y$ = 230 N/mm$^2$

$\lambda_{LT} \times \sqrt{(\sigma_{yc} / 355 \text{ N/mm}^2)}$ = **34.361**

As this is less than 45 it will be taken that; $\sigma_{yc}$ = $\sigma_{li}$

**BD56/97: cl. 9.8.1: Compact sections.**

The limiting compressive stress, $\sigma_{lc}$, should be taken as $\sigma_{li}$.

$\sigma_{lc}$ = $\sigma_{li}$ = **230** N/mm$^2$

**BEAMS WITHOUT LONGITUDINAL STIFFENERS:** **BD56/97 cl. 9.9**

**BD56/97: cl. 9.9.1: Bending resistance.**

**BD56/97: cl.9.9.1.2: Bending resistance of Compact sections.**

The bending resistance, $M_D$, of a compact section should be taken as:

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

where:
$Z_{pe}$ = Plastic modulus of the section.
$\sigma_{lc}$ = Limiting compressive stress.
$\gamma_m$ = Partial safety factor for the material .
From Table 2, $\gamma_m$ = 1.2
$\gamma_{f3}$ = 1.1, But not used as incorporated into loading.

$M_D$ = $((S_{xx} \times \sigma_{lc}) / (\gamma_m))$ = **322.127** kNm

**The moment capacity of the section is 322.127 kNm**

| Ref. | Calculations | Output |
|---|---|---|

**BD56/97: cl. 9.9.2: Shear resistance.**

**BD56/97: cl. 9.9.2.2: Shear resistance under pure shear.**

The shear resistance, $V_D$, of a web panel under pure shear should be taken as:

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

Where:

| | | |
|---|---|---|
| $t_w$ | = | Thickness of the web |
| $d_w$ | = | The overall depth of a rolled section |
| $h_h$ | = | The height of the largest hole or cut out being considered<br>$h_h$ = 0 mm |
| $\gamma_m$ | = | Partial safety factor for the material .<br>From Table 2, $\gamma_m$ = 1.2 |
| $\gamma_{f3}$ | = | 1.1, But not used as incorporated into loading. |
| $\tau_l$ | = | Limiting shear strength of the web panel.<br>See notes below. |

The limiting shear strength, $\tau_l$, is given by:

$$\frac{\tau_l}{\tau_y} \propto \lambda = \frac{d_{we}}{t_w}\sqrt{\frac{\sigma_{yw}}{355}}$$

where:

| | | |
|---|---|---|
| $d_{we}$ | = | Depth of section between the flange plates |
| $t_w$ | = | Thickness of the web |
| $\sigma_{yw}$ | = | Nominal yield stress of the web material |

$\lambda$ = $(d_1 / T_1) \times \sqrt{(f_y / 355)}$ = **31.564**

Interpolation from one of figures 11 to 17 is dependant on a number of other factors, however as $\lambda$ is less than 50, $\tau_l / \tau_y$ =1.00

Where: $\tau_y = \dfrac{\sigma_{yw}}{\sqrt{3}}$; = **132.791** N/mm$^2$

Hence; $\tau_l = \tau_y$ = **132.791** N/mm$^2$

Shear Capacity, $V_D$:

$V_D$ = $((T_1 \times ((d_1 + T_2 + T_3) - h_h)) / (\gamma_m)) \times \tau_l$ = **502.759** kN

**RECAP OF CAPACITIES OF A GENERAL EXTERNAL BEAM**

| | | | | | |
|---|---|---|---|---|---|
| **Moment capacity;** | $M_DEXT_G$ | = | $M_D$ | = | **322.127 kNm** |
| **Shear Capacity;** | $V_DEXT_G$ | = | $V_D$ | = | **502.759 kN** |

# 4. LOADING

**Load Factors Adopted in Analysis ($\gamma_{FL}$)**

Dead Load Factors: BD21/97 Table 3.1

; Steel; $\gamma_{steel}$ = 1.05

; Concrete; $\gamma_{conc}$ = 1.15

; Surfacing; $\gamma_{surf}$ = 1.75

; Fill; $\gamma_{fill}$ = 1.20

Live Load Factors: BD21/97 Table 3.1

; HA Loading; $\gamma_{ha}$ = 1.50

Global Factors: BD21/97 cl. 3.10

; Partial factor; $\gamma_{f3}$ = 1.10

## 4.1. Dead Load on an Internal Beam

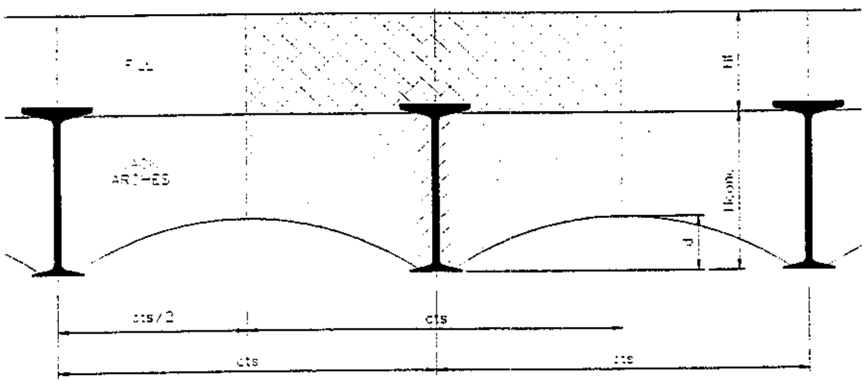


**DIAGRAM COMMENTS**

| | | | | |
|---|---|---|---|---|
| Beam Centres; | Cts | = | 914.4 mm | |
| Concrete Depth; | Hconc | = | 421.0 mm | |
| Rise in arc to crown; | d | = | 140.0 mm | |
| Depth of Fill; | Hf | = | 323.0 mm | |
| Additional Load Effects Factor; | $\gamma_{f3}$ | = | 1.1 | BD21/97 cl 3.10 |

**Steel Section:**

Steel mass per metre; Msteel = 115 kg/m

Applied Load; $D_s$ = Msteel × $g_{acc}$ × $\gamma_{steel}$ × $\gamma_{f3}$= **1.303** kN/m

| **Babtie**<br>2 St George's House,<br>Vernon Gate,<br>Derby, DE1 1UQ.<br>Tel: 01332 285100 - Fax: 01332 285101 | Project<br>Rail Property / Shropshire CC Bridge Assessments | | | | Job Ref.<br>031417 | |
|---|---|---|---|---|---|---|
| | Part of Structure<br>Rowe Bridge GNQ4/14 - Jack Arch Bridge | | | | Sheet no./rev.<br>GNQ4/14 23 A | |
| | Calc. by<br>[redacted] | Date<br>06/09/99 | Chck'd by<br>[redacted] | Date<br>22/9/99 | App'd by | Date |

| Ref. | Calculations | Output |
|---|---|---|

**Jack Arch:**

Density of Jack Arch; $\rho_{conc}$ = 23.54 kN/m$^3$

Area of Concete:

Aconc = Cts × ( Hconc - 0.5 × d ) **= 320954** mm$^2$

Applied Load; $D_a = \rho_{conc} \times Aconc \times \gamma_{conc} \times \gamma_{f3}$ **= 9.557** kN/m

**Fill:** (Allow for 100mm surfacing)

Density of Fill Material; $\rho_{fill}$ = 20 kN/m$^3$

Density of Surfacing; $\rho_{surf}$ = 24 kN/m$^3$

Applied Load; $D_f = Cts \times \gamma_{f3} \times (\rho_{surf} \times 0.1m \times \gamma_{surf} + (\rho_{fill} \times \gamma_{fill} \times (Hf - 0.1\ m)))$ **= 9.608** kN/m

**Total Dead Loads applied per beam;**

$\mathbf{D_{tot} = D_s + D_a + D_f}$ **= 20.468 kN/m**

**Maximum Dead Load Effects**

Maximum Shear Force, per single internal beam.

$D_{SF} = D_{tot} \times S / 2$ **= 92.399** kN

Maximum Bending Moment, per single internal beam.

$D_{MOM} = D_{tot} \times S^2 / 8$ **= 208.557** kNm

**Dead Load Effects at Quarter Span**

Shear Force, at ¼ span per metre width of deck.

$QD_{SF} = D_{tot} \times S / 4$ **= 46.199** kN

Maximum Bending Moment, at ¼ span per metre width of deck.

$QD_{MOM} = ( D_{tot} \times S^2 / 8 ) - ( D_{tot} \times S / 4 \times S / 8 )$ **= 156.418** kNm

## 4.2. Dead Load on an External Beam

**Steel Section**

$D_s$ = **1.303** kN/m

**Jack Arch**

$D_aX = D_a / 2$ = **4.779** kN/m

**Fill**

$D_fX = D_f / 2$ = **4.804** kN/m

**Timber Parapet**

$D_p$ = 0.5 kN/m

**Total Dead Loads applied per beam;**

$D_{tot}X = D_s + D_aX + D_fX + D_p$ **= 11.386 kN/m**

**Maximum Dead Load Effects**

Maximum Shear Force, per single internal beam.

$D_{SF}X = D_{tot}X \times S / 2$ = **51.398** kN

Maximum Bending Moment, per single internal beam.

$D_{MOM}X = D_{tot}X \times S^2 / 8$ = **116.011** kNm

**Dead Load Effects at Quarter Span**

Shear Force, at ¼ span per metre width of deck.

$QD_{SF}X = D_{tot}X \times S / 4$ = **25.699** kN

Maximum Bending Moment, at ¼ span per metre width of deck.

$QD_{MOM}X = ( D_{tot}X \times S^2 / 8 ) - ( D_{tot}X \times S / 4 \times S / 8 )$ = **87.009** kNm

## 4.3. Live HA UDL + HA KEL

**NOMINAL ASSESSMENT LIVE LOADS** **BD21/97 SECTION 5. LOADING**

Width of bridge deck between the parapets is 6.32 m

**BD21/97: cl. 5.6: Notional Lane Widths ($b_L$).**

From Table 5.1 determine the number of notional lanes:

No. of Notional Lanes: 2

**BA16/97: Chapter 2: Proportion Factor ($K_L$).**

For longitudinal beams the proportion factor is obtained using the following input data;

Span; S = **9.029** m

Beam Centres; Cts = **0.914** m

For internal beams interpolate from fig 2/2(b) to obtain the reduction factor:

$K_L I$ = 0.410

For external beams interpolate from fig 2/3(b) to obtain the reduction factor:

$K_L x$ = 0.385

For bending moments the gross moment is multiplied by the appropriate proportional factor.
The nominal shear is found by multiplying the HA-UDL by the appropriate proportional factor and the HA-KEL by 0.5.

**BD21/97: cl. 5.8 – cl.5.11: General**

The worst of the following cases should be considered.

(i) A UDL (which varies with loaded length) together with a KEL.
(ii) A single axle load.
(iii) A single wheel load.

**BD21/97: cl.5.19 – cl.5.21: Type HA Loading UDL and KEL**

**(i) A UDL (which varies with loaded length) together with a KEL.**

Loaded Length is equal to span; L = S = **9.029** m

HA Loading for loaded length between 2m and 50m is given by the following expression.

$W = 336 \times (1 / L)^{0.67}$ = **76.926** kN/m

HA-KEL; KEL = 120 kN

**BD21/97: cl.5.22 – cl.5.28: Reduction Factors for UDL and KEL**

Reduction factors may be incorporated into the expressions for loading, namely K and AF. The factor K will be omitted at the present as this can be used at a later stage to assess the capacity of the bridge using BD21/97 fig's 5/2 to 5/7 incl.

For $0 < L < 20$

AF is: a / 2.5

Where; a = 3.65 m ; and Notional Lane width; $N_L$ = 2.5 m

AF = $a/N_L$ = **1.460**

**Live Loading applied per single internal beam:**

HA: UDL
W1 = $W \times \gamma_{f3} \times \gamma_{ha} / AF$ = **86.937** kN/m

HA: KEL
KEL1 = $KEL \times \gamma_{f3} \times \gamma_{ha} / AF$ = **135.616** kN

**Evaluate Live Loading Effects for loading effective on a single internal beam:**

Shear at support
$L_{SF} = (((W1 \times L)/2) \times K_L I) + 0.5 \times KEL1$ = **228.716** kN

Shear at quarter span
$LQ_{SF} = (3 \times 0.5 \times KEL1 + W1 \times L \times K_L I)/4$ = 131.310 kN

Moment at midspan: KEL at mid-span
$L_{MOM} = (((W1 \times L2)/8) + (KEL1 \times L/4)) \times KLI$ = 488.694 kNm

Moment at ¼ span: KEL at ¼ span
$QL_{MOM} = (((((KEL1 \times 0.75 \times S) + (W1 \times S \times S/2))/S) \times S/4) - (W1 \times S/4 \times S/8)) \times K_L I$ = **366.520** kNm

**Total Effects due to HA Loading on a single internal beam:**

Shear at support
$V_{max} = D_{SF} + L_{SF}$ = **321.115** kN

Shear at quarter span
$VQ_{max} = QD_{SF} + LQ_{SF}$ = **177.510** kN

Moments at midspan
$M_{max} = D_{MOM} + L_{MOM}$ = **697.251** kNm

Moments at quarter span
$QM_{max} = QD_{MOM} + QL_{MOM}$ = **522.938** kNm

**Evaluate Live Loading Effects for loading effective on a single external beam:**

Shear at support
$L_{SF}X = (((W1 \times L)/2) \times K_L x) + 0.5 \times KEL1$ = **218.905** kN

Shear at quarter span
$LQ_{SF}X = (3 \times 0.5 \times KEL1 + W1 \times L \times K_L x)/4$ = **126.404** kN

Moment at midspan: KEL at mid-span
$L_{MOM}X = (((W1 \times L^2)/8) + (KEL1 \times L/4)) \times K_L x$ = **458.895** kNm

Moment at ¼ span: KEL at ¼ span
$QL_{MOM}X = (((((KEL1 \times 0.75 \times S) + (W1 \times S \times S/2))/S) \times S/4) - (W1 \times S/4 \times S/8)) \times K_L x$ = **344.172** kNm

**Total Effects due to HA Loading on a single external beam:**

Shear at support
$V_{max}X = D_{SF}X + L_{SF}X$ = **270.303** kN

Shear at quarter span
$VQ_{max}X = QD_{SF}X + LQ_{SF}X$ = **152.103** kN

Moments at midspan
$M_{max}X = D_{MOM}X + L_{MOM}X$ = **574.907** kNm

Moments at quarter span
$QM_{max}X = QD_{MOM}X + QL_{MOM}X$ = **431.180** kNm

Babtie
2 St George's House,
Vernon Gate,
Derby, DE1 1UQ.
Tel: 01332 285100 - Fax: 01332 285101

| Project | Job Ref. |
|---|---|
| Rail Property / Shropshire CC Bridge Assessments | 031417 |
| Part of Structure: Rowe Bridge GNQ4/14 - Jack Arch Bridge | Sheet no./rev.: GNQ4/14 27 A |
| Calc. by: [redacted] · Date: 06/09/99 · Chck'd by: [redacted] · Date: 22/9/99 | App'd by · Date |

| Ref. | Calculations | Output |
|---|---|---|

## Recap moment capacities for internal and external beams:

**CAPACITIES OF A SEVERELY CORRODED INTERNAL BEAM**

| | | | |
|---|---|---|---|
| Moment capacity; | $M_DINT_C$ | = | **250.209** kNm |
| Shear Capacity; | $V_DINT_C$ | = | **493.299** kN |

**CAPACITIES OF A GENERAL INTERNAL BEAM**

| | | | |
|---|---|---|---|
| Moment capacity; | $M_DINT_G$ | = | **361.000** kNm |
| Shear Capacity; | $V_DINT_G$ | = | **509.598** kN |

**CAPACITIES OF A SEVERELY CORRODED EXTERNAL BEAM**

| | | | |
|---|---|---|---|
| Moment capacity; | $M_DEXT_C$ | = | **215.420** kNm |
| Shear Capacity; | $V_DEXT_C$ | = | **490.792** kN |

**CAPACITIES OF A GENERAL EXTERNAL BEAM**

| | | | |
|---|---|---|---|
| Moment capacity; | $M_DEXT_G$ | = | **322.127** kNm |
| Shear Capacity; | $V_DEXT_G$ | = | **502.759** kN |

## Recap effects on internal and external beams:

**Maximum Dead Load Effects on an internal beam**

| | | | |
|---|---|---|---|
| Shear; | $D_{SF}$ | = | **92.399** kN |
| Bending; | $D_{MOM}$ | = | **208.557** kNm |

**Dead Load Effects at Quarter Span on an internal beam**

| | | | |
|---|---|---|---|
| Shear; | $QD_{SF}$ | = | **46.199** kN |
| Bending; | $QD_{MOM}$ | = | **156.418** kNm |

**Maximum Dead Load Effects on an external beam**

| | | | |
|---|---|---|---|
| Shear; | $D_{SF}X$ | = | **51.398** kN |
| Bending; | $D_{MOM}X$ | = | **116.011** kNm |

**Dead Load Effects at Quarter Span on an external beam**

| | | | |
|---|---|---|---|
| Shear; | $QD_{SF}X$ | = | **25.699** kN |
| Bending; | $QD_{MOM}X$ | = | **87.009** kNm |

**Maximum Live Load Effects due to HA & KEL on an internal beam**

| | | | |
|---|---|---|---|
| Shear; | $L_{SF}$ | = | **228.716** kN |
| Bending; | $L_{MOM}$ | = | **488.694** kNm |

**Live Load Effects due to HA & KEL at Quarter Span on an internal beam**

| | | | |
|---|---|---|---|
| Shear; | $LQ_{SF}$ | = | **131.310** kN |
| Bending; | $QL_{MOM}$ | = | **366.520** kNm |

**Maximum Live Load Effects due to HA & KEL on an external beam**

| | | | |
|---|---|---|---|
| Shear; | $L_{SF}X$ | = | **218.905** kN |
| Bending; | $L_{MOM}X$ | = | **458.895** kNm |

**Live Load Effects due to HA & KEL at Quarter Span on an external beam**

| | | | |
|---|---|---|---|
| Shear; | $LQ_{SF}X$ | = | **126.404** kN |
| Bending; | $QL_{MOM}X$ | = | **344.172** kNm |

| Babtie<br>2 St George's House,<br>Vernon Gate,<br>Derby, DE1 1UQ.<br>Tel: 01332 285100 - Fax: 01332 285101 | Project: Rail Property / Shropshire CC Bridge Assessments | | | | Job Ref.: 031417 | |
|---|---|---|---|---|---|---|
| | Part of Structure: Rowe Bridge GNQ4/14 - Jack Arch Bridge | | | | Sheet no./rev.: GNQ4/14 28 A | |
| | Calc. by: [redacted] | Date: 06/09/99 | Chck'd by: [redacted] | Date: 22/9/99 | App'd by | Date |

| Ref. | Calculations | Output |
|---|---|---|

**BD21/97: cl.5.28: C Factor.**

The live load capacity factor is to be determined and assessed against the highest value of K in the appropriate diagram (fig 5/2 to 5/7 incl.).

The value C is given by: $\frac{\textit{Available Live Load Cacacity}}{\textit{Live Load Capacity required for ADJUSTED HA Loading}}$

**Internal Beams:**

Midspan moments, with critical section:

$$= \frac{M_D - D_{MOM}}{L_{MOM}} \quad = \quad \mathbf{0.085}$$

Support shear, with critical section:

$$= \frac{V_D - D_{SF}1}{L_{SF}} \quad = \quad \mathbf{1.753}$$

It is obvious therefore that the capacity needs to be assessed in more detail considering critical section only at the quarter point.

Midspan moments, with general section; = **0.312**
Quarter span moments, with critical section = **0.256**

The moment C factor derived is such that the capacity of the bridge is 3 tonnes + GROUP 2 FE for all traffic intensities and road surface conditions.

**External Beams:**

Midspan moments, with critical section: = **0.217**
Support shear, with critical section: = **2.007**

It is obvious therefore that the capacity needs to be assessed in more detail considering critical section only at the quarter point.

Midspan moments, with general section; = **0.449**
Quarter span moments, with critical section = **0.373**

The moment C factor derived is such that the capacity of the bridge is 3 tonnes + Group 2 FE for all traffic intensities and poor road surface conditions.

Should the bridge be proved to have a good road surface then the capacity may be increased to 7½ tonnes + Group 2 FE for traffic intensities below high.

Given the above checks the analysis will proceed with a combined bending and shear check as the critical section was found to be at the quarter span.

| Babtie<br>2 St George's House,<br>Vernon Gate,<br>Derby, DE1 1UQ.<br>Tel: 01332 285100 - Fax: 01332 285101 | Project: Rail Property / Shropshire CC Bridge Assessments | | | | Job Ref.: 031417 | |
|---|---|---|---|---|---|---|
| | Part of Structure: Rowe Bridge GNQ4/14 - Jack Arch Bridge | | | | Sheet no./rev.: GNQ4/14 29 A | |
| | Calc. by: [redacted] | Date: 06/09/99 | Chck'd by: [redacted] | Date: 22/9/99 | App'd by | Date |

| Ref. | Calculations | Output |
|---|---|---|

## COMBINED BENDING AND SHEAR CHECK: INTERNAL BEAMS.

**BEAMS WITHOUT LONGITUDINAL STIFFENERS:** **BD56/97 cl. 9.9**

**BD56/97: cl. 9.9.1.2: Bending resistance of Compact sections.**

Moment capacity; $M_DINT_C$ = **250** kNm

**BD56/97: cl. 9.9.2.2: Shear resistance under pure shear.**

Shear Capacity; $V_DINT_C$ = **493** kN

**BD56/97: cl. 9.9.3: Combined Bending and Shear: Internal Critical Section.**

Using clause 9.9.3.2 the values of V and M used in the formulae given in clause 9.9.3.1 will be determined from the actual section considered, i.e. quarter span.

$\sigma_{yl}$ = $f_y$ = **230** N/mm$^2$

**BD56/97: cl. 9.3.2.1: Outstands in Compression.**

$$\textit{The ratio } \frac{b_{fo}}{t_{fo}} \textit{ should not exceed } 12\sqrt{355/\sigma_y} \textit{ or } 16, \textit{ whichever is the lesser.}$$

$12 \times \sqrt{(355 \text{ N/mm}^2 / f_y)}$ = **14.9**

( ( 169.9 - 10.3 ) / 2 ) / 22.2 = **3.6** ;SATIS

**BD56/97: cl. 9.3.2.2: Outstands in Tension.**

$$\textit{The ratio } \frac{b_{fo}}{t_{fo}} \textit{ should not exceed } 16.$$

( ( 125 - 10.3 ) / 2 ) / 9.8 = **5.9** ;SATIS

**BD56/97: cl. 9.4.2.4: Effective Compression Flange.**

As the two above conditions have been satisfied $K_c$ is taken as 1.

As there are no holes to consider $k_h$ is taken as 1.

$A_c$ = ( 169.9 - 10.3 ) × 22.2 = **3543.1** mm$^2$

$A_{fe}$ = $A_c$ = **3543.1** mm$^2$

$F_f$ = $A_c \times f_y$ = **814.9** kN

$d_f$ = 400.8 + ( 22.2 + 9.8 ) / 2 = **416.8** mm

$M_R$ = $( F_f \times d_f ) / \gamma_m$ = **283** kNm

As $M_R$ is greater than $M_D$ then the value $M_D$ will be used; $M_DINT_C$ = **250** kNm

When considering the shear, $V_R$ is the same as V as $m_{fw}$ set to zero has no effect.

| Ref. | Calculations | Output |
|---|---|---|

## COMBINED BENDING AND SHEAR CHECK: EXTERNAL BEAMS.

**BEAMS WITHOUT LONGITUDINAL STIFFENERS:** **BD56/97 cl. 9.9**

**BD56/97: cl. 9.9.1.2: Bending resistance of Compact sections.**

Moment capacity; $M_DEXT_C$ = **215** kNm

**BD56/97: cl. 9.9.2.2: Shear resistance under pure shear.**

Shear Capacity; $V_DEXT_C$ = **491** kN

**BD56/97: cl. 9.9.3: Combined Bending and Shear: Internal Critical Section.**

Using clause 9.9.3.2 the values of V and M used in the formulae given in clause 9.9.3.1 will be determined from the actual section considered, i.e. quarter span.

$\sigma_{yl}$ = $f_y$ = **230** N/mm$^2$

**BD56/97: cl. 9.3.2.1: Outstands in Compression.**

$$\text{The ratio } \frac{b_{fo}}{t_{fo}} \text{ should not exceed } 12\sqrt{355/\sigma_y} \text{ or } 16\text{, whichever is the lesser.}$$

12 × √ ( 355 N/mm$^2$ / $f_y$ ) = **14.9**

( ( 169.9 - 10.3 ) / 2 ) / 22.2 = **3.6** ;SATIS

**BD56/97: cl. 9.3.2.2: Outstands in Tension.**

$$\text{The ratio } \frac{b_{fo}}{t_{fo}} \text{ should not exceed } 16.$$

( ( 147 - 10.3 ) / 2 ) / 4.5 = **15.2** ;SATIS

**BD56/97: cl. 9.4.2.4: Effective Compression Flange.**

As the two above conditions have been satisfied $K_c$ is taken as 1.

As there are no holes to consider $k_h$ is taken as 1.

$A_c$ = ( 169.9 - 10.3 ) × 22.2 = **3543.1** mm$^2$

$A_{fe}$ = $A_c$ = **3543.1** mm$^2$

$F_f$ = $A_c \times f_y$ = **814.9** kN

$d_f$ = 403.9 + ( 22.2 + 4.5 ) / 2 = **417.3** mm

$M_R$ = ( $F_f \times d_f$ ) / $\gamma_m$ = **283** kNm

As $M_R$ is greater than $M_D$ then the value $M_D$ will be used; $M_DEXT_C$ = **215** kNm

When considering the shear, $V_R$ is the same as V as $m_{fw}$ set to zero has no effect.

| Ref. | Calculations | Output |
|---|---|---|

**Summary of the combined bending and shear checks on internal and external beams.**

As the values for MR and VR are not exceeded by the reduced loading considered providing that it can be demonstrated that in the event of full moment loading the shear effects are no more than 50% of the shear capacity, and vice-versa then the capacity will remain as noted.

As the shear is considered to be the strongest part of the section consideration will be given to the moment capacity, in the first instance and then to the shear capacity to ensure that the applied shear is less than 50% of the applied.

Internal Beam,

50% Shear Capacity is; $V_DINT_C / 2$ = **247** kN

Maximum shear force is; $V_{max}$ = **321** kN

Consider shear at quarter span.

Maximum shear is; $VQ_{max}$ = **178** kN

External Beam,

50% Shear Capacity is; $V_DEXT_C / 2$ = **245** kN

Maximum shear force is; $V_{max}X$ = **270** kN

Consider shear at quarter span.

Maximum shear is; $VQ_{max}X$ = **152** kN

In both cases it is noted that the combined capacity is acceptable as the shear force at the quarter point does not exceed 50% of the shear capacity, and the moment is within the full moment capacity.

The calculations may resume with assessment of the bridge under single axle and single wheel loading.

## 4.4. Live: Single Wheel

**(ii) A SINGLE WHEEL LOAD.**

As the HA and KEL loading 3 tonnes + G2 FE it is appropriate to apply the equivalent single wheel.

Although the road category has not been defined it would be fair to say that the heavy good vehicle use of the bridge would be medium, especially during the harvest season. The road condition is classified as poor, this being the only category applicable when no data is available (BD21/97 cl.5.214).

**BD21/97 Table 5/3/2: Nominal Single Wheel Load,**

NSWL = 29 kN

**BD21/97 cl. 5.34: Wheel Contact Area:**

$W_{ca}$ = 1.1 N/mm$^2$

Assuming a square contact area the length of each side is **162.369** mm

**BD21/97 cl. 6.7: Dispersal of loads through deck other than troughs:**

The dispersal is taken from the edge of the wheel at a ratio of 1 horizontally to 2 vertically through well compacted fill and surfacing materials. When considering jack arches it may be taken at a ratio of 1 to 1 to the mid-depth of the arch ring at the crown.

Dispersal is; Disp1 = Hf + ( Hconc - d ) + $\sqrt{(NSWL/W_{ca})}$ = **766.369** mm

As this is less than the beam centres the whole load may be applied to one internal beam.

**Effects of the single wheel load on an internal beam:**

Moments:

| | | | | |
|---|---|---|---|---|
| Mid-span; | $SW_{LIVE}M$ | = | $\gamma_{ha} \times \gamma_{f3} \times NSWL \times S / 4$ | = **108.004** kNm |
| ¼ span; | $SW_{LIVE}MQ$ | = | $\gamma_{ha} \times \gamma_{f3} \times NSWL \times 0.25 \times S \times 0.75 \times S / S$ | = **81.003** kNm |

Shear:

| | | | | |
|---|---|---|---|---|
| Support; | $SW_{LIVE}V$ | = | $\gamma_{ha} \times \gamma_{f3} \times NSWL$ | = **47.850** kN |
| ¼ span; | $SW_{LIVE}VQ$ | = | $\gamma_{ha} \times \gamma_{f3} \times NSWL \times 3 / 4$ | = **35.888** kN |

Totals:

Moments:

| | | | | |
|---|---|---|---|---|
| Mid-span; | $SWM_{max}$ | = | $SW_{LIVE}M + D_{MOM}$ | = **316.561** kNm |
| ¼ span; | $SWQM_{max}$ | = | $SW_{LIVE}MQ + QD_{MOM}$ | = **237.420** kNm |

Shear:

| | | | | |
|---|---|---|---|---|
| Support; | $SWV_{max}$ | = | $SW_{LIVE}V + D_{SF}$ | = **140.249** kN |
| ¼ span; | $SWV_{max}Q$ | = | $SW_{LIVE}VQ + QD_{SF}$ | = **82.087** kN |

**RECAP OF CAPACITIES OF AN INTERNAL BEAM**

| | | | |
|---|---|---|---|
| Moment capacity mid; | $M_DINT_G$ = | **361.000** kNm | SUFFICIENT |
| Moment capacity ¼ ; | $M_DINT_C$ = | **250.209** kNm | SUFFICIENT |
| Shear Capacity; | $V_DINT_C$ = | **493.299** kN | SUFFICIENT |

Combined shear capacity is 50% of above, the section is still sufficient.

As the moment capacity is sufficient proceed with single axle calculations.

Babtie
2 St George's House,
Vernon Gate,
Derby, DE1 1UQ.
Tel: 01332 285100 - Fax: 01332 285101



**Effects of the single wheel load on an external beam:**

Totals:

Moments:

| | | | | |
|---|---|---|---|---|
| Mid-span; | $SWM_{max}X$ | = | $SW_{LIVE}M + D_{MOM}X$ | = **224.015** kNm |
| ¼ span; | $SWQM_{max}X$ | = | $SW_{LIVE}MQ + QD_{MOM}X$ | = **168.011** kNm |

Shear:

| | | | | |
|---|---|---|---|---|
| Support; | $SWV_{max}X$ | = | $SW_{LIVE}V + D_{SF}X$ | = **99.248** kN |
| ¼ span; | $SWV_{max}QX$ | = | $SW_{LIVE}VQ + QD_{SF}X$ | = **61.586** kN |

**RECAP OF CAPACITIES OF AN EXTERNAL BEAM**

| | | | | |
|---|---|---|---|---|
| Moment capacity mid; | $M_DEXT_G$ | = | **322.127** kNm | SUFFICIENT |
| Moment capacity ¼ ; | $M_DEXT_C$ | = | **215.420** kNm | SUFFICIENT |
| Shear Capacity; | $V_DEXT_C$ | = | **490.792** kN | SUFFICIENT |

Combined shear capacity is 50% of above, the section is still sufficient.

As the moment capacity is sufficient proceed with single axle calculations.

## (iii) SINGLE AXLE LOAD.

Given that the bridge has satisfied the HA KEL loading and single wheel calculations for 3 tonne + G2 FE vehicles the following calculations will proceed on the same bases with 3 tonnes + G2 FE loading. There is no need to assess the external beam for single axle loading as such an arrangement is impossible.

**BD21/97 Table 5/3/2: Nominal Single Axle Load,**

NSAL = 57 kN

**BD21/97 cl. 5.34: Wheel Contact Area:**

$W_{ca}$ = 1.1 $N/mm^2$

Assuming a square contact area the length of each side is **160.963** mm

Given that the bridge is being assessed for 2 notional lanes the closest that two wheels may be assessed at is 0.7m centre to centre. For the dispersal noted earlier this gives a total dispersal with of;
0.7m + Disp1 = **1466.369** mm

For the given beam centres this results in **62.358** percent of the axle load acting on the beam, that is a total of; **35.544** kN

**Effects of the single axle load:**

Moments:

| | | | | |
|---|---|---|---|---|
| Mid-span; | $SA_{LIVE}M$ | = | $\gamma_{fa} \times \gamma_{f3} \times NSAL_{ap} \times S / 4$ | = **132.376** kNm |
| ¼ span; | $SA_{LIVE}MQ$ | = | $\gamma_{fa} \times \gamma_{f3} \times NSAL_{ap} \times 0.25 \times S \times 0.75 \times S / S$ | = **81.864** kNm |

Shear:

| | | | | |
|---|---|---|---|---|
| Support; | $SA_{LIVE}V$ | = | $\gamma_{fa} \times \gamma_{f3} \times NSAL_{ap}$ | = **58.648** kN |
| ¼ span; | $SA_{LIVE}VQ$ | = | $\gamma_{fa} \times \gamma_{f3} \times NSAL_{ap} \times 3 / 4$ | = **36.269** kN |

Totals:

Moments:

| | | | | |
|---|---|---|---|---|
| Mid-span; | $SAM_{max}$ | = | $SA_{LIVE}M + D_{MOM}$ | = **340.933** kNm |
| ¼ span; | $SAQM_{max}$ | = | $SA_{LIVE}MQ + QD_{MOM}$ | = **255.700** kNm |

Shear:

| | | | | |
|---|---|---|---|---|
| Support; | $SAV_{max}$ | = | $SA_{LIVE}V + D_{SF}$ | = **151.047** kN |
| ¼ span; | $SAV_{max}Q$ | = | $SA_{LIVE}VQ + QD_{SF}$ | = **82.469** kN |

**RECAP OF CAPACITIES OF AN INTERNAL BEAM**

| | | | |
|---|---|---|---|
| Moment capacity mid; | $M_DINT_G$ = | **361.000** kNm | SUFFICIENT |
| Moment capacity ¼ ; | $M_DINT_C$ = | **250.209** kNm | INSUFFICIENT |
| Shear Capacity; | $V_DINT_C$ = | **493.299** kN | SUFFICIENT |

Combined shear capacity is 50% of above, the section is still sufficient.

The moment capacity is insufficient for group 2 fire engine loading.

Try 3 tonne axle load.

| Babtie<br>2 St George's House,<br>Vernon Gate,<br>Derby, DE1 1UQ.<br>Tel: 01332 285100 - Fax: 01332 285101 | Project: Rail Property / Shropshire CC Bridge Assessments | | | | Job Ref.: 031417 | |
|---|---|---|---|---|---|---|
| | Part of Structure: Rowe Bridge GNQ4/14 - Jack Arch Bridge | | | | Sheet no./rev.: GNQ4/14 35 A | |
| | Calc. by: [redacted] | Date: 06/09/99 | Chck'd by: [redacted] | Date: 22/9/99 | App'd by: | Date: |

| Ref. | Calculations | Output |
|---|---|---|

Given that the bridge has failed to satisfy the single axle calculations for G2 FE vehicles the following calculations will proceed using 3 tonnes loading.

**BD21/97 Table 5/3/2: Nominal Single Axle Load,**

NSAL = 47 kN

**BD21/97 cl. 5.34: Wheel Contact Area:**

$W_{ca}$ = 1.1 $N/mm^2$

Assuming a square contact area the length of each side is **146.163** mm

Given that the bridge is being assessed for 2 notional lanes the closest that two wheels may be assessed at is 0.7m centre to centre. For the dispersal noted earlier this gives a total dispersal with of;
0.7m + Disp1 = **1466.369** mm

For the given beam centres this results in **62.358** percent of the axle load acting on the beam, that is a total of; **29.308** kN

**Effects of the single axle load:**

Moments:

| | | | | |
|---|---|---|---|---|
| Mid-span; | $SA_{LIVE}M$ | = | $\gamma_{ha} \times \gamma_{f3} \times NSAL_{ap} \times S / 4$ | = **109.152** kNm |
| ¼ span; | $SA_{LIVE}Q$ | = | $\gamma_{ha} \times \gamma_{f3} \times NSAL_{ap} \times 0.25 \times S \times 0.75 \times S / S$ | = **81.864** kNm |

Shear:

| | | | | |
|---|---|---|---|---|
| Support; | $SA_{LIVE}V$ | = | $\gamma_{ha} \times \gamma_{f3} \times NSAL_{ap}$ | = **48.359** kN |
| ¼ span; | $SA_{LIVE}Q$ | = | $\gamma_{ha} \times \gamma_{f3} \times NSAL_{ap} \times 3 / 4$ | = **36.269** kN |

Totals:

Moments:

| | | | | |
|---|---|---|---|---|
| Mid-span; | $SAM_{max}3$ | = | $SA_{LIVE}M + D_{MOM}$ | = **317.709** kNm |
| ¼ span; | $SAQM_{max}3$ | = | $SA_{LIVE}Q + QD_{MOM}$ | = **238.282** kNm |

Shear:

| | | | | |
|---|---|---|---|---|
| Support; | $SAV_{max}3$ | = | $SA_{LIVE}V + D_{SF}$ | = **140.758** kN |
| ¼ span; | $SAV_{max}Q$ | = | $SA_{LIVE}Q + QD_{SF}$ | = **82.469** kN |

**RECAP OF CAPACITIES OF AN INTERNAL BEAM**

| | | | |
|---|---|---|---|
| Moment capacity mid; | $M_DINT_G$ = | **361.000** kNm | SUFFICIENT |
| Moment capacity ¼ ; | $M_DINT_C$ = | **250.209** kNm | SUFFICIENT |
| Shear Capacity; | $V_DINT_C$ = | **493.299** kN | SUFFICIENT |

Combined shear capacity is 50% of above, the section is still sufficient.

The moment capacity is sufficient for 3 tonne single axle loading.

## 4.5. Summary of Loading vs. Capacity

**Dead**:

The bridge has shown by calculation that it has sufficient capacity to carry the structure dead weight.

**HA: UDL and KEL loading.**

Consideration of the internal and external beams has revealed that the capacity of the bridge can only be described as having sufficient capacity to be subject to 3 tonnes loading and Group 2 Fire Engines.

**HA: Single Wheel**

Consideration of the single wheel was carried out based on the previous results using Mp figures from table 5/3/2, and it was shown that the bridge provided a satisfactory level of resistance for both 3 tonne and group 2 fire engine loading.

**HA: Single Axle**

Consideration of the single axle was initially carried out using fire engine group 2 loading using Mp figures from figure 5/3/1 and it was shown that the bridge provided an unsatisfactory level of resistance. Further calculations were carried out using 3 tonnes axle loading upon which it was found that the bridge provided a satisfactory level of resistance.

**Conclusion:**

The capacity of the bridge is: **3 tonnes.**

# 5. QUALITATIVE ASSESSMENTS

## 5.1. Jack Arches

The jack arches were found to be in fair to poor condition, with the jack arches particularly on the outer arches as heavy leaching has resulted in the concrete becoming loose and friable. This is a condition which has developed over time and is likely to continue.

Given the current condition of the arches the capacity of the bridge assigned through numerical checking of the steel beams of 3 tonnes is seen as a fair limiting capacity. At this time no need to reduce the capacity of the bridge due to the condition of the jack arches is deemed appropriate.

## 5.2. Abutments

The abutments have suffered with erosion of the brickwork, and more extensively the mortar. The original mortar used in the abutments is thought to be a lime based mortar, which through time has suffered from the leaching process. The leaching process has resulted in the mortar becoming very loose and friable.

There is evidence when viewing the abutments parallel with the track under the bridge that bulging of the abutments has taken place. This displacement is likely to have occurred due to the weakened state of the mortar, and will have reduced the capacity of the abutment significantly.

Cracks have developed at various locations throughout the bridge abutments, particularly at the corners. This has been attributed to the high degree of skew in the bridge together with the effects of the mortar deterioration causing debonding on some of the joints.

At this time the capacity awarded to the steel beams of 3 tonnes is seen to be a fair representation of the capacity likely to have been retained by the abutments.

## 5.3. Foundations

There is no evidence to suggest that movement within the foundations is or has occurred. The ground around the bridge was noted to be sodden despite the general lack of rain in the recent time leading up to the inspection of the bridge. Should however there be a requirement to use the foundations for future work a full investigation would be warranted.

# 6. CONCLUSIONS

The capacity of the bridge is limited by numerical methods to that determined for the steel jack arch bridge beams.

That is, the capacity of the bridge is 3 tonnes, with no capacity for fire engines.

## FORM BA (BRIDGES)

## CERTIFICATION FOR ASSESSMENT CHECK

**STRUCTURE** ROWE BRIDGE , ELLESMERE - UNCLASSIFIED

**CATEGORY OF CHECK** 1

**STRUCTURE NO** GNQ 4/14

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

**(1)** It has been assessed in accordance with the Approval in Principle (where appropriate) as recorded on Form AA approved on 14 September 1999

**(2)** It has been checked for compliance with the following principal British Standards, Codes of Practice, BR Technical notes and Assessment standards.

BD 21/97 (The Assessment of Highway Bridges and Structures)
(incorporating Amendment No 1)
BA 16/97 (The Assessment of Highway Bridges and Structures)
(incorporating Amendment No 1)
BD 56/96 (The Assessment of Steel Highway Bridges and Structures)
BA 56/96 (The Assessment of Steel Highway Bridges and Structures)
BD 61/96 (The Assessment of Composite Highway Bridges and Structures)
BA 61/96 (The Assessment of Composite Highway Bridges and Structures

**CATEGORY 1**

**NAME** **SIGNATURE**

[redacted] [redacted]

(ASSESSOR) 5/11/99 (DATE)

(ASSESSMENT CHECKER) 5/11/99 (DATE)

(TECHNICAL DIRECTOR BABTIE GROUP) 1/12/99 (DATE)

**CATEGORY 2**

(a) **ASSESSMENT**

**NAME** **SIGNATURE**

(ASSESSOR) ______ (DATE)

(TECHNICAL DIRECTOR BABTIE GROUP) ______ (DATE)

(b) **CHECK**

**NAME** **SIGNATURE**

(ASSESSOR CHECKER) ______ (DATE)

(TECHNICAL DIRECTOR BABTIE GROUP) ______ (DATE)

**THE CERTIFICATE IS ACCEPTED BY .....**[redacted]**........................**

**FORM BAA (BRIDGES)**

## CERTIFICATION FOR ASSESSMENT CHECK

## NOTIFICATION OF ASSESSMENT CHECK

**STRUCTURE NAME** ROWE BRIDGE , ELLESMERE – UNCLASSIFIED

**STRUCTURE NO** GNQ 4/14

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**

3 Tonnes

Critical member: Internal Beam Of The Structure

**RECOMMENDED LOADING RESTRICTIONS**

3 tonnes

**DESCRIPTION OF STRUCTURAL DEFICIENCIES AND RECOMMENDED STRENGTHENING.**

The limiting structural elements of the bridge are the internal steel girders with an assessment capacity of 3 tonnes. The outer edge steel girders were assessed to have a capacity of 3 tonnes + Fire Engines Group 2. It is not considered that other methods of analysis would produce a significantly higher result

It is possible that propping would provide an interim short term solution for increasing the bridge strength. However the only practical long term method of strengthening would be redecking. It should be noted though that the condition of the substructure is poor and substantial repair works, probably involving rebuilding would be required.

However rather than undertaking the necessary extensive remedial works, and as the bridge appears to serve no useful purpose, it is recommended that serious consideration be given to demolition in order to remove a future maintenance liability.

The general condition of the bridge suggests that remedial works be considered as a matter of urgency.

**Name:** [redacted] **Signed:** [redacted] **Structural Assessment Engineer** 24/11/99

**Name:** [redacted] **Signed:** [redacted] **Senior Civil Engineer**

8

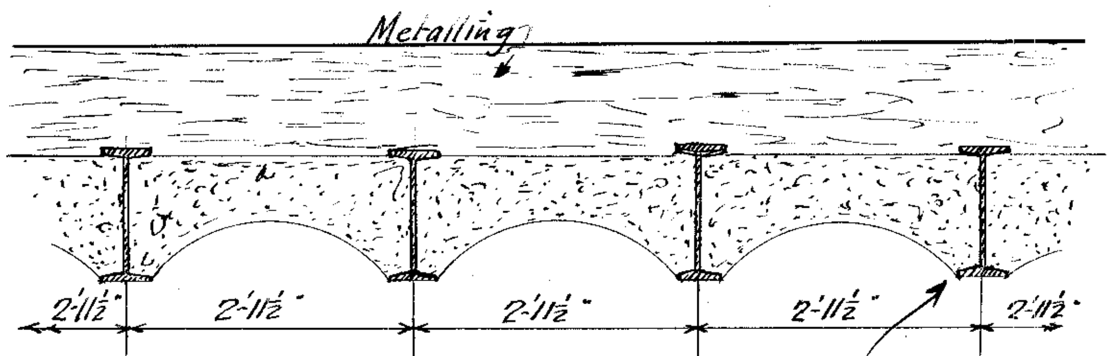
~~Central Wales Division~~
Section 1
Unclassified Road
Notice Plate 3T 5T 5T

County of Shropshire
Whitchurch to Aberystwyth 7.20 3/4
Nearest Stn Welshampton
Sketch in Cabinet
Corr: No —

Classification "Z"

Bridge Type:-
8 No Longitudinal Rolled Steel Joists, Concrete Jack Arches & backing, with Metalled Road Surface.

Skew Span 28'-8½"
Square do. 24'-8"
Clear Width between parapets 20'-6½"
2 No Grass Verges.
Date of Construction:- Probably about 1918




Inside Beams    Span 30'-6"

R.S.J. 17½" deep
6½ Flange
Flange 7/8" Mid thickness
3/4" at Edge
13/32" Web.

Note This section adopted for R.S.J's on this section of Line. Some slight variation of size actually exists.

Dead Load per Beam

| | |
|---|---|
| Beam | 61 |
| Concrete | 460 |
| Metalling (10") | 335 |
| | 856 lbs per foot run. |

Total. $\frac{856 \times 30.5}{2240} = 11.65$ Tons

Note. The section is not a British Standard.

Constants
Gross Area = 17.73 □"
M of I abt NA = 919 in⁴
Modulus (ten) = 105 in³

Dead Load Stress $\frac{11.65 \times 30.5 \times 12}{8 \times 105} = 5.08$ T/□"

Calculation form IA for stringers & longitudinal girders under the road and less than 2' deep (i.e. with Z stated in ins.$^3$).

## ABNORMAL LOADS

**BRIDGE No** 14, Whitchurch to Aberystwyth

**CAPACITY OF** STEEL LONGITUDINAL **GIRDER No** – , **SPAN No** –

From R. & R.T.A. Assessment
- Span (see rules, page 4) — 28.7 ft. — $L$
- Girder Spacing — 2.96 ft. — $S$
- Section Modulus — 105.0 ins.$^3$ — $Z$

(For C.I. only) $D/d$ = / ∴ Revised Modulus = / ins.$^3$ $Z'$

Permissible Stress — 12.0 tons/in.$^2$ $f_p$

Dead Load Stress (revised if span varies) $= 6.02 \times \left(\frac{28.7}{29.708}\right)^2 =$ 5.62 tons/in.$^2$ $f_d$

∴ Stress available for Live Load $(f_p - f_d)$ = 6.38 tons/in.$^2$ $f_\ell$

E.U.D.L. for Buses (Graph 1A) (Only if road is 25' wide, or over) / tons $W_b$

Proportion on Girder (Graph 1B) (Distribution*: None, 5/8) / $r_b$

Stress due to Buses $= \frac{1.5 \times W_b \times r_b \times L}{Z'} = \frac{1.5 \times \quad \times \quad \times}{}$ = / tons/in.$^2$ $f_b$

∴ Stress available for Abnormal Loads $(f_\ell - f_b)$ = 6.38 tons/in.$^2$ $f_a$

**LIGHT LOADING.** E.U.D.L. (Graph 2A or 2B) 1.235 tons $W_\ell$

Proportion on Girder (Graph 3A or 3B)

(Distribution*: ~~None~~, 5/6, ~~5/8, Due to depth of cover, C = . '~~) .246 $r_\ell$

(Which, if distribution is allowed must not be less than $\frac{1}{N-1}$ . )

Permissible ~~Axle~~* Trailer $= \frac{f_a \times Z'}{1.5 \times W_\ell \times r_\ell \times L} = \frac{6.38 \times 105}{1.5 \times 1.235 \times .246 \times 28.7} =$ **51.2 Tons**

**MEDIUM LOADING** (Only if L is 28' or over).

E.U.D.L. (Graph 2B) . tons $W_m$

Permissible Axle* / Trailer $= \frac{f_a \times Z'}{1.5 \times W_m \times r_\ell \times L} = \frac{\times}{1.5 \times \quad \times \quad \times} =$ . Tons

**HEAVY LOADING** E.U.D.L. (Graph 2A or 2B) 1.85 tons $W_h$

Proportion on Girder (Graph 3C or 3D)

(Distribution*: ~~None~~, 5/6, ~~5/8, Due to depth of cover~~, C = . ') .212 $r_h$

(Which, if distribution is allowed must not be less than $\frac{1}{N-1}$ . )

Permissible Bogie $= \frac{f_a \times Z'}{1.5 \times W_h \times r_h \times L} = \frac{6.38 \times 105}{1.5 \times 1.85 \times .212 \times 28.7} =$ **39.7 Tons**

Calculated: A.G.
Date: 11.11.63.
Checked: K.S
Date: 26-4-72

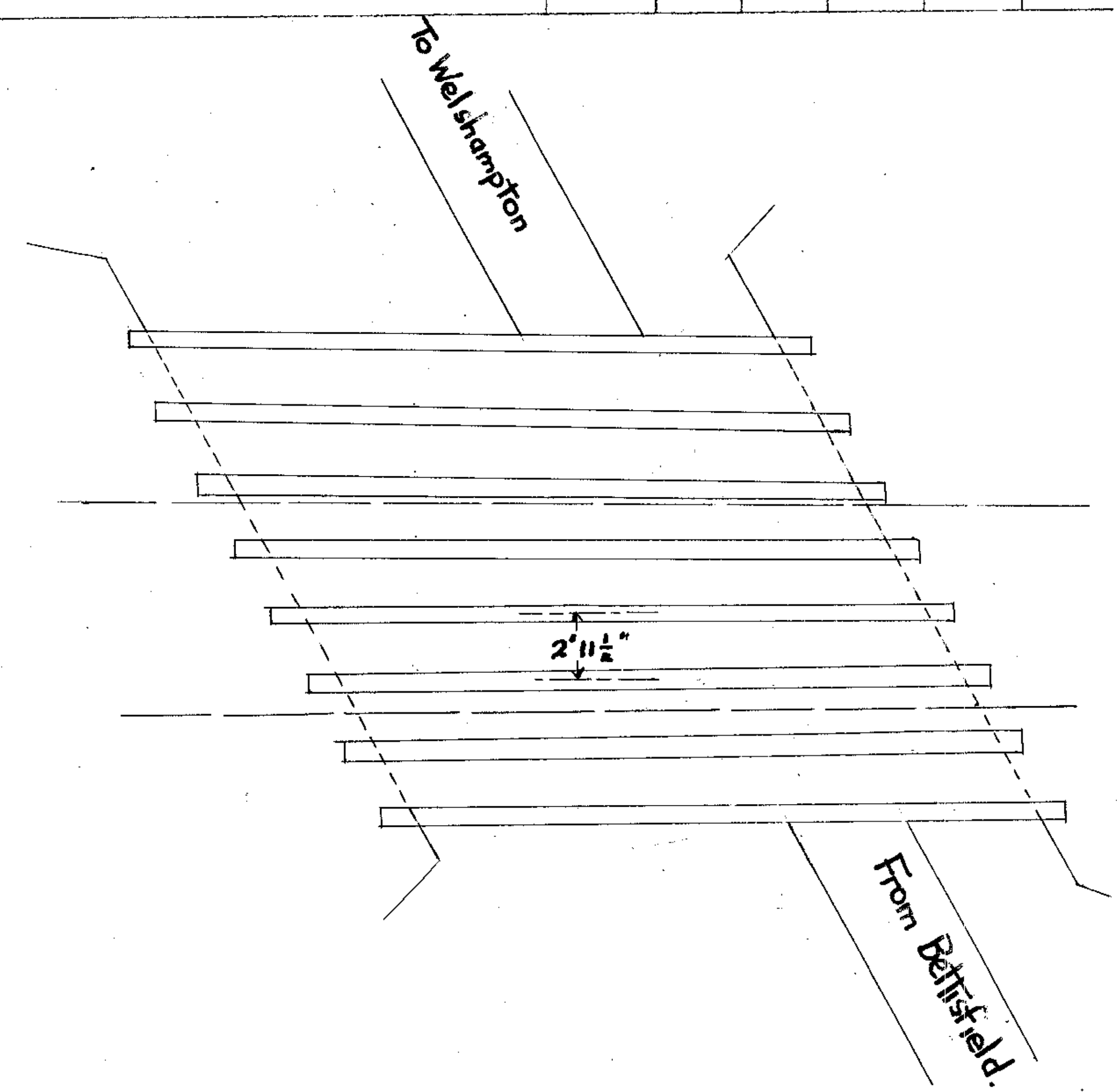
SUMMARY

| MEMBER | SPAN | CAPACITY |
|---|---|---|
| | | A |
| STEEL LONGITUDINAL | | B 40 |
| –"– –"– | | T 50 |

* Delete items which are not applic[able]

| | |
|---|---|
| LINE OR BRANCH.<br>Whitchurch to Aberystwyth. | REFERENCE.<br>53/7-20¾. |
| NEAREST STATION.<br>Welshampton | M.O.T. ROAD Nº & CLASSN.<br>Unclassified. |
| EXISTING NOTICE PLATES.<br>3I 5I 5I | ROAD NAME & LOCATION. |

| BRIDGE TYPE. | SPANS | BUILT | MAP FIGURE | 19 | 27 | 35 |
|---|---|---|---|---|---|---|
| 8 No. Longitudinal R.S.J.s<br>Jack Arching. | 29.708 | 1918 | CORRES. NºS. | | | |
| | | | RELEVANT DRG. NºS<br>31425 AY | | | |
| | | | LAST EXAM. | | | |




CAPACITY.

| MEMBER. | SPAN / SPACG | I | II | III |
|---|---|---|---|---|
| Longitudinal R.S.J. | 29.708 / 2.46 | 34.0 | 54.1 | 70 |
| | | | | |
| | | | | |
| | | | | |

## Strength of Steel Joists

| | |
|---|---|
| Span | 29.708 ft |
| Spacing | 2.958 ft |
| Modulus (R x RTA) | 105 ins$^3$ |
| D. L Stress | 6.02 tsi |
| Available for LL. (fp 12.00) | 5.98 tsi |

| Loading Class | I | II | III |
|---|---|---|---|
| Distribution factor | ·367 | ·327 | ·286 |
| Reduction factor | ·8 | ·8 | ·8 |
| Unit B.M | 4.55 | 3.7 | 3.3 |
| Effective B.M | 1.34 | ·966 | ·755 |

M. of R. $\frac{5.98 \times 105}{12}$ 52.3 ft

| | | | |
|---|---|---|---|
| P.V. | 39.0 | 54.1 | 69.3 tons |

Calc'd B Seathan

Ch'k'd J. W.

W20

MASONRY, BRICK & CONCRETE WORK

B.R. 12327/7

BRITISH RAILWAYS L.M. REGION SHREWSBURY DISTRICT

SHEET No. ........ OF ........

**BRIDGE CULVERT AND RETAINING WALL EXAMINATION REPORT**

NAME .................... AT 7 Ms. 20¾ Chs. BRIDGE No. 14

ON WHIT- AIBER LINE BETWEEN WHITCHURCH & ELLESMERE

TYPE OF ~~UNDER~~/OVER BRIDGE BRICK ABUTS + W.W'S R.S.J'S CONCRETE JACK ARCHES TIMBER PARAPETS CARRYING PUBLIC RD. OVER No TRACK

| Condition of Part | G = Good F = Fair P = Poor | Condition of Part | G = Good F = Fair P = Poor | Condition of Part | G = Good F = Fair P = Poor |
|---|---|---|---|---|---|
| 1 ARCH RING | | 14 STRUTS | | 27 CONCRETE DECK SLABS | |
| 2 SPANDRELS | | 15 BEARING STONES | G | 28 SPANDREL TIE BOLTS | |
| 3 PARAPETS TIMBER | G | 16 HAUNCHING TO GIRDER | | 29 JACK ARCH TIE BOLTS | |
| 4 ABUTMENTS | F | 17 TROUGH FILLING | | 30 SMOKE PLATES & FITTINGS | |
| 5 WING WALLS | F | 18 BALLAST WALLS | | 31 R.S.J'S | G |
| 6 PILASTERS | F | 19 JACK ARCHES | G | 32 | |
| 7 PIERS | | 20 GIRDER ENCASING | | 33 | |
| 8 CROSSHEADS | | 21 DRAINAGE | G | 34 | |
| 9 RELIEVING ARCHES | | 22 FIXINGS for PIPES & CABLES | | 35 | |
| 10 PILES | | 23 ROAD SURFACE | G | 36 | |
| 11 FOUNDATIONS | | 24 CONCRETE MAIN GIRDERS | | 37 PAINTING | F |
| 12 SCOUR | | 25 CONCRETE CROSS GIRDERS | | 38 POINTING | F |
| 13 INVERT | | 26 CONCRETE STRINGERS | | 39 LOAD RESTRICTION PLATES | G |

REMARKS (Refer to parts by above numbers)

4.5.6.+38.
Brickwork generally perished & decayed, heavy spalling in patches

37 BLACK. (BIT)

COMMENTS:— Fair

EXAMINED BY [redacted] (Examiner) ON 18.4.69 (Date)

RECOMMENDATIONS:— ✓

SIGNED [redacted] (Inspector, Supervisor or Technical Asst.) 17-10-69 (Date)

ACTION TO BE TAKEN:—

SIG[redacted] District Engineer Jan 70 (Date)

S3.

7m 20¾c (C.W.1.)



Stress available for Live Load. 8-5.08 - 2.92 T/□"

Max. permitted Weight of Vehicle.

At Max Speed.

E.U.D.L. No1 = 3.496 T/L

Proportion Carried = .5 (Girder Crs - 2'11½")

Distribution $\frac{2}{3} \times \frac{2.96}{4}$

Live Load Stress $3.496 \times .5 \times \frac{2}{3} \times \frac{2.96}{4} \times \frac{30.5 \times 12}{8 \times 105} = .375$ T/□"

$= \frac{2.92}{.375} = 7.78$

At 5mph.

E.U.D.L = 2.915 T/L

Stress = $\frac{2.915}{3.496} \times .375 = .312$ T/□"

$= \frac{2.92}{.312} = 9.34$

## Notice Plates

Prohibit all vehicles over 9 Tons.
& between 8T & 9T at Speeds over 5mph.

[redacted] Calculated by:- [redacted]
Checked by:-