CIS List December 2000

RAILTRACK CURRENT INFORMATION SHEETS

| Information Sheet No. | Subject | Date Issued | current? (R/T letter 6/12/00) |
|---|---|---|---|
| 1 | Application of HB Loading | 08/96 | |
| 2 | Analytical Assessment of Piers | 11/96 | |
| 3 | Use of BD34/90 and BA34/90 | 11/96 | |
| 4 | HB Loading | 11/96 | |
| 5 | BD 56 & 61 | 02/97 | |
| 6 | HB Loading | | |
| 7 | Earth Pressure Coefficient | 02/97 | ❄ |
| 8 | Bridges Constructed after 1975 | 02/97 | |
| 9 | Amendment to Minutes of Meeting | 03/97 | |
| 10 | Approvals in Connection with New Standards | | |
| 11 | Approval Procedures in Connection with BD21/97 | 07/97 | |
| 12 | Bridges Constructed after 1975 - revisions since C.I.S. 8 | 09/97 | |
| 13 | Bridges Constructed since 1975 - revisions since C.I.S. 12 | 10/97 | ❄ |
| 14 | BD21/97 Assessment, Traffic Flow and Road Surface Categories | 10/97 | ❄ |
| 15 | Accidental Wheel Loading and Footway Loading | 10/97 | ❄ |
| 16 | Technical Advice on Assessment of Piers | 01/99 | ❄ |
| 17 | British Rail Specifications | 08/98 | ❄ |
| 18 | Mechanism Analysis of Multi-span Arches | 03/99 | ❄ |
| 19 | Technical Advice on Condition Factors in Rigorous Arch Assessment | 01/99 | ❄ |
| 20 | Technical Advice on Assessment of Skew Arches | 01/99 | ❄ |
| 21 | Technical Advice on Single Span Arches with h > d | 01/99 | ❄ |
| 22 | Assessment of Jack Arches, Metal Plate Arches and Associated Ties in Metal Beam Bridge Decks | 00/01 | |
| 23 | Use of BD and BA 61 for Cased Filler Beam Bridges | 05/00 | ❄ |
| 24 | | | |
| 25 | | | |
| 26 | | | |
| 27 | HB Capacity with MEXE | 02/00 | ❄ |

# NETWORK RAIL

## BRIDGEGUARD 3 - LIST OF CURRENT INFORMATION SHEETS

| CIS No. | SUBJECT | STATUS | Date Authorised |
|---|---|---|---|
| 7 | Earth Pressure Co-efficient | Issue I | 16/02/00 |
| 13 | Bridges Constructed after 1975 | Issue I | 16/02/00 |
| 14 | BD21/97 Traffic Flow & Road Surface Categories | Issue I | 16/02/00 |
| 15 | AWL & Footway Loading | Issue I | 16/02/00 |
| 16 | Assessment of Piers | Issue I | 2/03/99 |
| 17 | British Rail Specifications | Issue I | 16/02/00 |
| 18 | Mechanism Analysis of Multi-Span Arches | Issue I | 10/03/99 |
| 19 | Condition Factors in Rigorous Arch Assessment | Issue I | 17/11/99 |
| 20 | Assessment of Skew Arches | Issue I | 2/03/99 |
| 21 | Technical Advice on Single Span Arches with h greater than D | Issue I | 6/04/99 |
| 22 | Assessment of Jack Arches, Metal Arch Plates & Ties in Metal Beam Bridge Decks | Final | 12/03/01 |
| 23 | Use of BD & BA61 For Cased & Filler Beam Bridges | Final | 23/06/00 |
| (24) | Limit State | Withdrawn | |
| (25) | Pedestrian Live Loading | Issued as letter | |
| (26) | Section 117 (BE4) Assessments | Issued as SE-TAN 211100/9 | 22/12/00 |
| 27 | HB Capacity with MEXE | Final | 7/05/00 |
| (28) | Lateral Torsional Buckling | Withdrawn | |
| 29 | Clarification of Interpretation of BD44/BA44 for Shear in Simply Supported Pre-tensioned Beam Decks | Final | 23/05/01 |
| 30 | Use of BD61 for Composite Bridges with Shear Connection | Final | 07/02/02 |
| 31 | Use of ARCHIE-M for the Analysis of single and Multi –Span Arches | Final | 05/11 01 |
| 32 | Strength of Rivets<br>Issue-2 | Final<br>Final | 21/05/02<br>Due April 04 |
| 33 | Con - Arches | Final | 27/06/03 |
| 34 | Condition Assessment of Post Tensioned Bridges<br>Appendix on Re-instatement to be added | Final | 25/04/03 |
| 35 | Assessment of Metal Hogging Plates<br>Extension of Parameters used | Final | 08/10/03 |
| 36 | Edge Girders in Jack Arches– Assessment of Torsional Buckling Strength | Draft | Feb. 04 |

CIS numbers shown in brackets have been withdrawn

→ JN → Corrigé

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# BRIDGEGUARD 3

## CURRENT INFORMATION SHEET No. 1

19 August 1996

**Distribution :**

**Originator :**

Railtrack Engineer : Tony Small

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**SUBJECT :** **Application of HA Loading**

It has been observed by some consultants that an apparent discrepancy exists between the HA loading requirements of BD21/93 and BD37/88.

It should be noted that the design loading of BD 37/88 shall be used to derive the assessment HA loading.

Please note that in all other respects the requirement of BD 21/93 loading shall be strictly achieved to during the assessment process.

Regards.

Tony Small

No. of Pages Following :

Project File

Central File

AS/BJJ/10732
19.08.96

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# BRIDGEGUARD 3

## CURRENT INFORMATION SHEET No. 2

S/S by No 16.

14 November 1996

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Originator :

Railtrack Engineer : Tony Small

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No of Pages to follow : 1

**SUBJECT :** **Analytical Assessment of Piers**

In some cases Railtrack's scope of services have required that piers should be considered to be part of the superstructure and assessed analytically. Railtrack do not now require that piers are assessed analytically unless they fulfill the relevant criteria described in Section 8 of BD 21/93.

Regards.

Tony Small

| | | | |
|---|---|---|---|
| Project File | ☑ | N.A.W. ☑ | H.K. ☑ |
| Central File | ☑ | ☐ | ☐ |

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01.11.99

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# BRIDGEGUARD 3

## CURRENT INFORMATION SHEET No. 3

14 November 1996

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No of Pages to follow : 1

SUBJECT : The Use of BD 34/90 and BA 34/90

current info sheet 3.

In some cases Railtrack's scope of services have required that BD 34/90 and BA 34/90 are to be included in the Technical Approval Schedule (TAS) in Form AA. Railtrack do not now require that BD 34/90 or BA 34/90 to be included in the TAS.

Should Form AA, which refers to BD 34/90 or BA 34/90 already have been issued then a relevant comment may be generated by the Reviewing Engineer or Railtrack's Technical Approval Authority (TAA). Due to the timing of this information sheet it may not be possible to include all comments on the standard commentary sheets. If this is so then a comment will be made directly on Form AA itself before being returned to the Assessment Engineer.

Regards.

Tony Small

Project File

Central File

AS/BJJ/11110
14.11.96

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29
4.30
27.11.96

**BRIDGEGUARD 3**

# CURRENT INFORMATION SHEET No. 4

**14 November 1996**

**Distribution:**

**Originator:**

Railtrack Engineer: Tony Small

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**No of Pages to follow: 1**

SUBJECT: HB Loading

The present check certificate form BA does not make provision for recording the number of HB units a structure can sustain

Would you please expand section (ii) after "The assessed capacity of the structure is as follows", to include:

*....... Tonnes*
*........ units of HB*
*(If units of HB are not available then this should be stated above)*

In order to facilitate future estimates of the capacity of the bridge to carry abnormal indivisible loads, it would be useful to include a summary table which identifies the number of units of HB which each critical element can carry. Additionally the base length of the adverse region of the influence line should also be identified in the summary table.

It should be noted that calculations with respect to the HB capacity of members/elements should generally only be assessed where the bridge is capable of carrying an assessed capacity of 40T.

Regards.

Tony Small

Project File ☐ ☐ ☐

Central File ☐ ☐ ☐

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22.11.96

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## BRIDGEGUARD 3
## CURRENT INFORMATION SHEET No. 5

10 February 1997

**Distribution:**

**Originator:**

Railtrack Engineer: Tony Small

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**No of Pages to follow: 1**

SUBJECT: BD 56 & BD 61

Railtrack have now approved the use of BD 56 and BD 61 for the assessment of metal bridges and composite bridges respectively.

The technical scope of the latest joint venture agreements now provides for the use of these documents for assessments. Most of the joint venture agreements for the 1995/96 programme were prepared prior to the introduction of BD 56 and BD 61 and therefore reference is not made to their use within the documentation.

Recent evidence suggests that both of these new codes can produce more realistic results than BS 5400 Part 3 and BS 5400 Part 5 when applied to existing structures. It is therefore to the benefit of all parties that where practical the new standards are adopted in favour of the old design standard.

It is appreciated that this may not always be possible, particularly where an assessment has already commenced on the basis of an agreed AIP.

Regards.

A Small

Tony Small

Project File

Central File

REF: D:/BG3/96-97jv/infosht5.doc

TOTAL P.07

6 pages total.

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## BRIDGEGUARD 3

## CURRENT INFORMATION SHEET No. 6

10 February 1997

**Distribution:**

**Originator:**

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**No of Pages to follow: 1**

SUBJECT: HB Loading

A partial safety factor ($\gamma_{fl}$) of 2.0 should be used with respect to the application of HB loading for masonry arch bridges, for all axles of the vehicle.

Regards.

A. Small

Tony Small

Project File

Central File

REF: BG3/96-97JV/INFCSHT6.DOC

12 FEB 1997 17:35 FROM TO 01920412965 P.03/06

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**BRIDGEGUARD 3**

**CURRENT INFORMATION SHEET No. 7**

**10 February 1997**

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**Originator:**

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**No of Pages to follow: 1**

**CURRENT INFORMATION SHEET NO. 7**

SUBJECT: Earth Pressure Co-efficient

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Under BD 21/93 an earth pressure co-efficient of up to 3 may be applied, provided that if this exceeds 50% of the passive pressure, a sensitivity analysis should be carried out. Should the assessment result prove to be sensitive to the value of K, the Reviewing Engineer and Railtrack Technical Approval Authority will decide whether the higher value of K is acceptable.

Regards.

A Small

Tony Small

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| | | | |
|---|---|---|---|
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REF: BG3/96-97JV/INFOSHT7.DOC

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## BRIDGEGUARD 3

## CURRENT INFORMATION SHEET No. 9

14 March 1997

**Distribution:**

**Originator:**

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**No of Pages to follow: 1**

SUBJECT: Amendment to Minutes Of Meeting

It should be noted that the minutes of the technical meeting held between yourselves and Pell Frischmann may contain an error in Section 4.2, Recommendations.

The minutes should reflect that recommendation should not generally be included in the Assessment Report. Occasionally a typographical error has occurred and the word 'not' has been omitted.

Can you please check through the minutes and make any alteration as necessary.

We apologise for any inconvenience caused.

Regards,

Tony Small

Project File

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## BRIDGEGUARD 3

## CURRENT INFORMATION SHEET No. 10

**6 May 1997**

**Distribution:**

**Originator:**

Railtrack Engineer: Tony Small

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**No of Pages to follow: 1**

SUBJECT: Approvals procedure in connection with new standards

Three new Assessment Standards have been recently been issued:

BD 56/96 The Assessment of Steel Highway Bridges and Structures.

BD 61/96 The Assessment of Composite Highway Bridges and Structures.

BD 21/97 The Assessment of Highway Bridges and Structures.

Some local authorities are seeking clarification where new standards have been implemented subsequent to the signing of the joint venture agreement.

Where the Assessing Engineers want to take advantage of the new standards then Railtrack have no objection to this.

In some cases Assessing Engineers who want to apply the new standards may have already submitted AIPs for approval based on the old standards.

As an interim measure Pell Frischmann have sometimes commented on the AIP at technical review stage to the effect that the new standards should be incorporated.

Notwithstanding the above, it will still be necessary to re-submit or more usually produce an Addendum to the AIP if any of the new Standards are to be adopted.

Please would you advise Pell Frischmann where you intend adopting the new standards if you have not already done so.

Regards.

A Small

Tony Small

Project File

Central File

REF: BG3/96/97JV/INFOSH10.DOC

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BRIDGEGUARD 3

CURRENT INFORMATION SHEET No. 11

1 JULY 1997

Distribution: Originator:

Railtrack Engineer: Tony Small

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No of Pages to follow: 1

REF: BG3/96/97TV/INFOSH11.DOC

DRAFT CURRENT INFORMATION SHEET NO. 11

SUBJECT: Approvals procedure in connection with BD 21/97

Currently there is a transition period from the use of BD 21/93 to BD 21/97. This information sheet provides a simplified procedure regarding the implementation of BD 21/97 for those structures which have previously been assessed (or are being assessed) to BD 21/93.

BD 21/97 may be substituted for BD 21/93 if desired by the Assessing Engineer without the need for submitting a revised AIP or an Addendum to the AIP. This supersedes Current Information Sheet No. 10 in this respect.

The use of BD 21/97 should be noted on the Form BA, CL 2 (ii).

Regards.

Tony Small

Project File

Central File

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BRIDGEGUARD 3

CURRENT INFORMATION SHEET No. 12

4 SEPTEMBER 1997

Distribution: Originator:

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No of Pages to follow: 1

DRAFT CURRENT INFORMATION SHEET NO. 12

SUBJECT: Bridges Constructed After 1975
Procedures Revised Since Issue of Current Information Sheet No. 8

---

There are a number of cases where Railtrack owned bridges have been completely or partially reconstructed after 1975.

Where the assessing engineer prefers to use the original design calculations for bridges constructed after 1975 instead of carrying out a full assessment the following procedure should be followed:

a. If the original design calculations are available, these maybe reviewed for validity as an alternative to carrying out assessment calculations. The nature and the extent of this review will depend on whether or not a valid "Certificate of Design and Checking" is available.

If such a certificate is available, signed on behalf of the designer and the checker and counter-signed in acceptance by British Rail, the review may consist simply of an overview to confirm that the calculations seem to be generally in order and that the design loading is clearly identified.

If such a certificate is not available, the review should be such as to establish with reasonable certainty that calculations are valid

In all cases a written record should be made of the nature and extent of the review.

b. All bridges will require inspection to confirm the accuracy of the drawings and to establish the present condition of the structure.

Regards.

Tony Small

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BRIDGEGUARD 3

CURRENT INFORMATION SHEET No. 13

2 OCTOBER 1997

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**No of Pages to follow: 1**

## CURRENT INFORMATION SHEET NO. 13

**SUBJECT:** **Bridges Constructed After 1975**
**Procedures Revised Since Issue of Current Information Sheet No. 12**

---

The procedures for assessing bridges constructed after 1975 have been the subject of much discussion within Railtrack. It has been decided to amend the procedures set out in the Current Information Sheet No. 12 which in itself supersedes the procedures in Current Sheet No. 8.

In general Railtrack will seek to remove from the normal assessment programme those bridges which are known to have been constructed after 1975. Where information relating to maintenance records and design calculations or certificates are available Pell Frischmann will pursue these documents and advise Railtrack whether a letter of confirmation of capacity should be issued on the basis of the available record information. Should the record information be unavailable or Pell Frischmann consider that an inspection and assessment is necessary then the local authority will be advised to carry out a full assessment in the normal way.

If in error a bridge which has been constructed after 1975 is included in the Joint Venture Agreement list of bridges then it would be appreciated if this was brought to the attention of Pell Frischmann London office.

Please note that this procedure now supersedes the procedures set out in Current Information Sheets No. 12 and Current Information Sheet No. 8.

Regards.

T Small

Tony Small

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Project File

Central File

REF: BG3/96/97JV/INFOSH13.DOC

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**BRIDGEGUARD 3**

**CURRENT INFORMATION SHEET No. 14**

**24 OCTOBER 1997**

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**No of Pages to follow: 1**

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**CURRENT INFORMATION SHEET NO. 14**

**SUBJECT: BD 21/97 Assessment - traffic flow and road surface categories**

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BD 21/97 has introduced relaxation of HA loading levels based on six categories of bridge situations in terms of road surface and traffic flows.

However due to uncertainties regarding possible future changes to both traffic flows and deterioration of road surface, Railtrack will generally require assessment to be based on a matrix of the six categories.

The Assessment Report shall state the results for the matrix. Clearly if the bridge passes the 40 tonnes Assessment Live Loading criteria when assessed using the worst category, (i.e. Hp), then the report could state this, and the full matrix would not need to be used.

The bridge capacities stated on Form BA should be as for the matrix, (unless bridge is adequate with Hp as noted above).

Alternatively a single value for the bridge capacity may be quoted on Form BA if agreement can be reached with the appropriate Railtrack Zone regarding a commitment by the Highway Authority to maintaining the surfacing at the stated 'good' level, and assumed traffic flows will not be exceeded. Railtrack's requirements for the control of road surfacing are given on the attached sheet.

Regards.

A Small

Tony Small

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Project File ☐ ☐ ☐

Central File ☐ ☐ ☐

REF: BG3/96/97JV/INFOSH14.DOC

ATTACHMENT TO IS No14

# RAILTRACK
*Civil Engineering*

## BRIDGEGUARD 3:
## ROAD SURFACE CATEGORIES (BD 21/97 SEC 5.24)

In order to accept a bridge assessment capacity based on a "Good" road surface category as defined in BD 21/97, Railtrack will need to be satisfied that the highway authority has a sufficient control system in place to ensure that the road is maintained within the specified surface deviation limits (or, if not, that suitable action is taken at the appropriate time).

Such a control system will need to consist of at least the following elements:-

(a) Measuring surface deviations at frequent enough intervals to ensure that significant exceedences do not occur without being detected. The actual measurement intervals will need to be justified, presumably by reference to recorded data on rates of deterioration in other similar circumstances.

(b) Transmitting advice that the surface deviations are approaching their limiting values to a person responsible for taking action and authorised to do so on behalf of the highway authority.

(c) Directing that suitable action be taken within a suitable timescale, e.g.:-

- resurfacing the road over the bridge;
- installing weight-restriction arrangements over the bridge and informing Railtrack.

(d) Confirming that the action has in fact been taken.

The details of the control system may of course be expected to vary considerably from authority to authority, but the system needs to be robust enough to ensure that:-

- it will be effective over a long period of time;
- it will be effective when people change or leave their jobs;
- it will be effective if highway maintenance is contracted out.

John Horsler

John Horsler

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**BRIDGEGUARD 3**

**CURRENT INFORMATION SHEET No. 15**

**OCTOBER 1997**

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**No of Pages to follow: 1**

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REF: BG3/96/97JV/INFOSH15.DOC

**CURRENT INFORMATION SHEET NO. 15**

**SUBJECT: Accidental Wheel & Vehicle Loading and Footway Loading**

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1. Many bridges have footways and verges which are unable to carry the current Accidental Wheel and Vehicle Loading requirements of BD 21/97, Cl 5.35.

   Therefore the assessment of a bridge is to differentiate between the adequacy of members carrying the carriageway and those supporting footways, verges and central reserves.

   The Assessment Report and Form BA shall state separately the superstructure results for:
   i) Carriageway members
   ii) Footway, verge etc, members
   iii) HB capacities, (if applicable).

Regards.

Tony Small

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| | | | |
|---|---|---|---|
| Project File | ☐ | ☐ | ☐ |
| Central File | ☐ | ☐ | ☐ |

REF: BG3/96/97JV/INFOSH15.DOC

# RAILTRACK
## BRIDGEGUARD 3

**CURRENT INFORMATION SHEET NO 16**
**SUBJECT: TECHNICAL ADVICE ON ASSESSMENT OF PIERS**
**DATE: January 1999**
**STATUS: ISSUE 1**

---

**1. Prepared by Gifford and Partners**

i) Author, Name: A.C. Holmes

Signature: ...................... Date 17.2.99

ii) Checker, Name: P D JANSEN

Signature: ...................... Date 16/2/99

**2. Endorsed by Pell Frischmann Consulting Engineers Ltd**

i) Endorsed by Senior Reviewing Engineer for Pell Frischmann Consulting Engineers Ltd.

Name A. G. SMALL

Signature: ...................... Date 2/3/99

**3. Approved for issue by Railtrack**

i) Approved by Railtrack Project Delivery (Gt Western Zone) for Bridgeguard 3

Name M. PALMER

Signature: ...................... Date 12-3-99

## CURRENT INFORMATION SHEET NO 16

## SUBJECT: TECHNICAL ADVICE ON ASSESSMENT OF PIERS

**This Current Information Sheet is issued for guidance purposes only: it is not mandatory. The Assessor must be satisfied that the advice given in this Information Sheet is appropriate to the structure in question.**

---

The following procedure shall be adopted with respect to the assessment of piers to multi span bridges:-

a) Masonry piers to multi span masonry arches.

The load capacity of this form of structure is very dependent on the global interaction of the arches and piers. Unless the piers can be deemed to be of sufficient stiffness to permit the structure to be considered as a series of independent arches, a quantitative assessment of the arches and piers shall be undertaken in a global analysis. Where the Assessment Consultant or Joint Venture consider that a global analysis is not necessary, they shall detail in the Approval in Principle document all information necessary to satisfy the Technical Approval Authority that their alternative approach is acceptable. It is accepted that a pier with height to width ratio of 2:1 or less can be considered as stocky where the pier height is defined as the distance between arch springing and top of the foundation.

The global analysis shall be undertaken using MULTI (as developed by the University of Dundee) except where otherwise approved by the Technical Approval Authority.

Except where agreed by the Technical Approval Authority, this procedure shall also apply to multi span structures containing at least one masonry arch span where the behaviour of the pier(s) could affect the global capacity of the structure. For example, the centre span of a three span arch replaced by a metal deck in order to achieve increased headroom.

b) Masonry piers to other structures.

A qualitative assessment shall always be undertaken to the masonry piers of a multi span structure other than the forms of structure detailed in (a) above, whatever the condition of the pier.

c) Metal piers

A quantitative assessment shall always be undertaken to a metal pier. The Assessment Consultant or Joint Venture shall ensure that they obtain sufficient information pertaining to the condition of all components of the pier, for example crossheads, columns, bracing, connectors, to permit a realistic assessment to be undertaken.

d) Concrete piers

Concrete piers exist in a number of forms and with varied aspect ratios: for example tall slender columns with a crosshead or a short wide leaf pier. As a result, it is considered that the form of the assessment (quantitative or qualitative) for a concrete pier shall be determined on an individual basis. The Assessment Consultant or Joint Venture shall justify their approach on engineering judgement with a quantitative assessment being proposed where it is considered that the assessment of the bridge could be dependent on the load capacity of the pier. The Assessment Consultant or Joint Venture shall also take into account the condition of the pier as determined from the inspection, when determining the proposed approach. They shall detail their reasoning in the Approval in Principle document for approval by the Technical Approval Authority.

e) Loading

For a quantitative assessment of a pier, the applied loading shall accord with the requirements of BD 21/97. The Assessment Consultant or Joint Venture shall especially note Clause 5.3 of BD 21/97 which states that 'When loading or principal combinations of loads other than those specified in this Standard are considered necessary for assessment purposes, these loadings shall comply with the requirements given in BD 37'. The Assessment Consultant or Joint Venture shall therefore consider the application of the appropriate loadings for the bridge in question. However, the piers do not have to be assessed for rail impact loading.

Where this procedure is adopted, there is no need to record this approach in Section 4.6 (Proposed Departures from Standard) or 4.7 (Aspects not covered by Standards) of the AIP.

Where this Current Information Sheet is used it shall be listed in Section 4.5 (List of Relevant Standards) of the AIP.

The approach for the assessment of piers detailed in this Information Sheet supersedes that contained in Information Sheet No 2.

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**BRIDGEGUARD 3**

**CURRENT INFORMATION SHEET No. 17**

**AUGUST 1998**

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**No of Pages to follow: 1**

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REF: BG3/JV/ADMIN/INFOSH17.DOC

**CURRENT INFORMATION SHEET NO. 17**

**SUBJECT: British Rail Specifications**

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Assessing Engineers and Checkers should be aware that British Rail Zones appear to have used specifications which varied from Zone to Zone.

In some cases the specifications are very different, and use of the wrong specification could lead to unsafe assessments, e.g.:

| Class of Concrete | Midland Zone Spec (1960s) (Lbs/sq in) | Southern Zone Spec (1968) (Lbs/sq in) |
|---|---|---|
| A | 4000 | 3000 |
| B | 3000 | 3750 |
| C | 1500 | 4500 |
| D | - | 6000 |
| E | - | 7500 |

Regards.

A Small

Tony Small

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Project File ☐ ☐ ☐

Central File ☐ ☐ ☐

REF: BG3/JV/ADMIN/INFOSH17.DOC

**CURRENT INFORMATION SHEET NO. 17**

**SUBJECT: British Rail Specifications**

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| E | - | 7500 |

Regards.

A Small

Tony Small

Project File ☐ ☐ ☐

Central File ☐ ☐ ☐

# RAILTRACK
## BRIDGEGUARD 3

CURRENT INFORMATION SHEET NO. 18
SUBJECT: 'MULTI' ANALYSIS
DATE: MARCH 1999
STATUS: ISSUE 1

---

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# CURRENT INFORMATION SHEET NO. 18

## SUBJECT: MECHANISM ANALYSIS OF MULTI - SPAN ARCHES

**This current information sheet is issued for guidance purposes only: it is not mandatory. The Assessor must be satisfiied that the advice given in this Information Sheet is appropriate to the structure in question.**

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## 1. Introduction

1.1 Current Information Sheet No 16 defines when global analysis using MULTI may be undertaken.

1.2 This present Current Information Sheet provides guidance regarding the parameters and assumptions to be used for MULTI analyses carried out in the Bridgeguard 3 Programme.

1.3 MULTI is a mechanism program which does not conform with the requirements of CL 6.26 of BD 21/97. However Railtrack confirm that this mechanism method is acceptable for the analysis of multi-span arches.

## 2.0 Modelling

2.1 In the absence of definite information the minimum level of backing over the piers shall be taken to be the level where the extrados crosses the vertical through the intrados at the springing point unless there are features or other structural evidence to the contrary The Assessment Report shall then state this basis for assessment.

2.2 No minimum backing level at abutments shall be assumed without some evidence of its presence.

2.3 Site investigation to confirm the level of backing need only be carried out if there is reason to doubt its presence. For example, the presence of extensive backing as indicated on record drawings shall be confirmed by supplementary evidence, (to be agreed with TAA), to indicate that such backing has been provided.

2.4 In the absence of definite information the top of each pier foundation may *normally* be assumed to be 0.5m below ground level.

2.5 The partial safety factor for material, ($\gamma_m$) may be taken as 1.0, in accordance with BD 21/97, CL 6.20; with condition factor ($F_c$) taking account of material deterioration. The use of $\gamma_m$ values as given in BS5628 relate to new work and are not relevant for Bridgeguard 3 assessments. Refer to Information Sheet No. 19 for guidance on the application of condition factors.

2.6 $\gamma_{f3}$ may be taken as 1.0, i.e. taking MULTI to be a validated method.

2.7 Realistic assumed values are to be taken for the soil properties in the absence of definite information. Very low or very high $\phi$ values shall be avoided. As MULTI uses an 'at rest' earth pressure coefficient, putting $\phi = 0^0$ may give unsafe results. Furthermore, ignoring lateral earth pressures (e.g.: by setting $\phi = 90^0$ in ARCHIE/MULTI) may also give unsafe results.

## 3.0 Loading

3.1 Proper account shall be taken for dispersion of axle loads (as per BD 21/97, CL 6.22), using relevant fill depth and position of axles relative to edge of structure. The use of the 2.5m dispersal/distribution width provided as a default value in MULTI is generally over-conservative.

3.2 Note that MULTI automatic routine to find the worst load position can in some cases give misleading information. It is understood the 'worst' load position is defined by MULTI as that giving the greatest eccentricity of thrust at a pier. However this occurs before the iterative manipulation of the thrust zones by the user takes place. We therefore suggest that each span of a multi-span arch shall be considered on a span by span basis for the application of loading and user manipulation.

## 4.0 Interpretation of Results

4.1 The adequacy of the piers may be considered to be satisfactory if the thrust zones can be shown to remain within the pier width. MULTI determines a satisfactory load path dependent on lines of thrust and material strength, with no restriction on the development of cracks in tensile areas. Additional checks on tensile and compressive stresses are therefore generally not required. In addition to the above analysis the piers will need to be considered qualitatively if serious defects have been observed.

## 5.0 Checking Category

5.1 Multi-span masonry arches assessed using MULTI will generally require a Category II Check, due to the greater complexity and engineering judgement required by this method of analysis.

## 6.0 Special Cases

6.1 Piers with cut-outs - The following aspects shall be taken into account in the MULTI analysis:

i) increased width of thrust zone (due to reduced areas at various sections up the height of the pier).

ii) lack of beneficial effect from self weight of masonry in cut-out zone (which when present pulls the resultant thrust back towards the centreline of the pier).

iii) The bridge may need to be analysed taking account of the total loading on the full width of the structure (i.e. DL + carriageway + footway loading).

6.2 Voided structure - Allowance shall be made for reduced weight over the piers or within the piers to allow for voids, (e.g. internal spandrel walls with vaulted construction, shell piers). The lack of weight may increase the eccentricity of the thrust zone in the pier.

6.3 Lateral loading on piers - Account shall be taken of any soil or surcharge loading which is applied to the face of piers, (e.g. a significant difference in levels between ground levels on each side of a pier)

## 7.0 Procedures

7.1 Where these procedures are adopted, there is no need to record this approach in Section 4.6 (proposed Departures from Standard) or 4.7 (Aspects not covered by Standards) of the AIP.

7.2 When this Current Information Sheet is used it shall be listed in Section 4.5 (List of Relevant Standards) of the AIP.

# RAILTRACK
## BRIDGEGUARD 3

**CURRENT INFORMATION SHEET NO 19**
**SUBJECT: TECHNICAL ADVICE ON CONDITION FACTORS IN RIGOROUS ARCH ASSESSMENT**
**DATE: January 1999**
**STATUS: ISSUE 1**

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## CURRENT INFORMATION SHEET NO 19

## SUBJECT: TECHNICAL ADVICE ON CONDITION FACTORS IN RIGOROUS ARCH ASSESSMENT

**This Current Information Sheet is issued for guidance purposes only: it is not mandatory. The Assessor must be satisfied that the advice given in this Information Sheet is appropriate to the structure in question.**

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The following procedure may be adopted with respect to the application of condition factors in the rigorous assessment of arch structures using a mechanism analysis (this is also deemed to include a modified mechanism analysis using programs such as 'ARCHIE' and 'MULTI'):-

a) For rigorous analyses using a mechanism method, the majority of defects can and should be modelled directly. For example, mortar loss can be modelled by reducing the barrel thickness; longitudinal cracking can be modelled by restricting the lateral distribution of live loads.

b) Where defects cannot be modelled directly, condition factors may be adopted. These should be based on the MEXE condition factors given in BD 21/97 and BA 16/97. When deducing appropriate condition factors, double counting of defects should be avoided.

c) The remaining condition factor should be split into two factors relating to material deterioration $F_c(m)$ and structural defects $F_c(s)$.

- $F_c(m)$ will take into account defects such as the condition and width of the mortar joints ($F_{mo}$ and $F_w$ in MEXE) and perhaps localised areas of deteriorated masonry. It should be applied to the masonry strength alone.

- $F_c(s)$ will take into account defects such as diagonal cracking (where this cracking is not deemed to be part of the mechanism forming). Where appropriate factors are taken from BA 16/97 Chapter 3 they should be applied to the live loads alone.

d) Where lateral or diagonal cracking is fine and is likely to close up in forming a mechanism, thus allowing the line of thrust to be transferred across it, no additional condition factor need be considered in a mechanism or modified mechanism analysis. Where lateral or diagonal cracking is wide and thus unlikely to close up and allow the thrust line to pass across it, this defect should be considered by reducing the barrel thickness where the depth of cracking is known, or by using an appropriate $F_c(s)$ factor.

e) Appropriate consideration shall be given to the condition of piers. Where possible defects should be modelled directly.

For multi-span structures, this Information Sheet should be read in conjunction with Current Information Sheet No. 18 covering the subject of multi-span arch analysis.

Where this procedure is adopted, there is no need to record this approach in Section 4.6 (Proposed Departures from Standard) or 4.7 (Aspects not covered by Standards) of the AIP.

Where this Current Information Sheet is used, it shall be listed in Section 4.5 (List of relevant standards) of the AIP.

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# RAILTRACK
# BRIDGEGUARD 3

**CURRENT INFORMATION SHEET NO 20**
**SUBJECT: TECHNICAL ADVICE ON ASSESSMENT OF SKEW ARCHES**
**DATE: January 1999**
**STATUS: ISSUE 1**

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## CURRENT INFORMATION SHEET NO 20

## SUBJECT: TECHNICAL ADVICE ON ASSESSMENT OF SKEW ARCHES

**This Current Information Sheet is issued for guidance purposes only: it is not mandatory. The Assessor must be satisfied that the advice given in this Information Sheet is appropriate to the structure in question.**

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The following procedure should be adopted for the assessment of skew arch structures, both single span and multi-span:-

a) For single span structures, the analysis shall generally be based on the skew span as recommended in BA16/97, Clause 3.5 for MEXE assessments. The only exception to this is where the applied live loads are located at a significant distance from the edge of the structure; for example, tunnels. For this exception, the analysis may be based on the square span.

b) For multi-span structures, the analysis should generally be based on the skew spans and skew pier widths.

c) The advice given in Annex G of BA16/97 relating to skew span enhanced load capacities is derived from research based on limited numerical analyses. Since this advice may not be safe for all skew bridges, it shall not be used in Bridgeguard 3 assessments.

d) Skew arch structures may exhibit defects resulting from out-of-balance lateral thrusts at supporting piers ("racking" effects) and torsional effects. It is difficult to quantitatively assess these effects in a 2-dimensional analysis, and testing of skewed arches to date has demonstrated the associated formation of 5 hinges at collapse (as opposed to the usual 4) and ring separation. Skew arches should, therefore, also be subject to an additional qualitative assessment where these defects are encountered.

   The qualitative assessment will normally consider the following:

   - whether monitoring of the structure is appropriate.
   - whether the quantitative assessment is at risk owing to unknown implications of the defects.

   The implications of both the quantitative and qualitative assessments should be considered in producing the final rating for the bridge.

e) Where a road is skewed to the bridge axis on which the arch is analysed (ie. usually the skew span) consideration should be give to skewed vehicles in the analysis.

f) The checking categories for skew arches using MEXE and "mechanism" analyses shall be as follows:

| | |
|---|---|
| Category I | Single span arch with any skew.<br>Multi-span arch structure with stocky piers and with any skew. |
| Category II | Multi-span arch structure with slender piers and with any skew. |

For multi-span structures, this Information Sheet should be read in conjunction with Current Information Sheet No. 18 covering the subject of 'MULTI' analysis.

Where this alternative procedure is adopted, there is no need to record this approach in Section 4.6 (Proposed Departures from Standard) or 4.7 (Aspects not covered by Standards) of the AIP.

Where this Current Information Sheet is used it shall be listed in Section 4.5 (List of relevant standards) of the AIP.

# RAILTRACK
# BRIDGEGUARD 3

**CURRENT INFORMATION SHEET NO 21**
**SUBJECT: TECHNICAL ADVICE ON SINGLE SPAN ARCHES WITH h GREATER THAN d**
**DATE: January 1999**
**STATUS: ISSUE 1**

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## CURRENT INFORMATION SHEET NO 21

**SUBJECT: TECHNICAL ADVICE ON SINGLE SPAN ARCHES WITH h GREATER THAN d**

**This Current Information Sheet is issued for guidance purposes only: it is not mandatory. The Assessor must be satisfied that the advice given in this Information Sheet is appropriate to the structure in question.**

BD 21/97 Clause 6.17 states that when the depth of fill at the crown of an arch is greater than the barrel thickness, the results from a modified MEXE analysis may be unconservative and should therefore be confirmed by an alternative method.

As an alternative to this restriction in BD 21/97, a modified MEXE method may be undertaken with the fill depth at the crown of the arch restricted to the barrel thickness. The appropriateness of this alternative should be considered carefully if there is a possibility of failure through crushing of the arch barrel.

Where this alternative procedure is adopted, there is no need to record this approach in Section 4.6 (Proposed Departures from Standard) or 4.7 (Aspects not covered by Standards) of the AIP.

Where this Current Information Sheet is used it shall be listed in Section 4.5 (List of Relevant Standards) of the AIP.

# RAILTRACK
# BRIDGEGUARD 3

**CURRENT INFORMATION SHEET NO 22**
**SUBJECT: ASSESSMENT OF JACK ARCHES, METAL ARCH PLATES AND ASSOCIATED TIES IN METAL BEAM BRIDGE DECKS**
**DATE: January 2001**
**STATUS: FINAL**

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Name JOHN BLAGROVE ..........

Signature: J. Blagrove .......... Date 14th March 2001

CURRENT INFORMATION SHEET NO 22

SUBJECT: ASSESSMENT OF JACK ARCHES, METAL ARCH PLATES AND ASSOCIATED TIES IN METAL BEAM BRIDGE DECKS

**This Current Information Sheet is issued for guidance purposes only: it is not mandatory. The Assessor must be satisfied that the advice given in this Information Sheet is appropriate to the structure in question.**

## 1 Introduction

This Current Information Sheet applies to the assessment of jack arches, metal arch plates and associated ties in metal beam bridge decks. It covers both transverse and longitudinal spanning jack arch decks.

## 2 Assessment of Jack Arch Bridge Decks

### 2.1 General

Jack arch bridge decks rely for their structural integrity on restraint to the edge bays and this is most commonly provided by ties (bars or straps) attached at or near the beam bottom flange level in the edge bays. Restraint can also be provided by service bays constructed so as to give in-plan rigidity to the edge bays (e.g. by a metal plate floor riveted to the beam bottom flanges).

Where the edge beams are sufficiently stocky, they may themselves be adequate to provide the necessary restraint. However, it is important to be aware that small lateral movements can be sufficient to destroy the composite behaviour of the deck. Therefore the untied edge beams would need to be rigid enough to prevent breakdown of composite action as well as strong enough to withstand the lateral loads. Composite action also requires the presence of structural backing above the jack arches.

Jack arch bridge decks may be assessed either quantitatively (i.e. by calculation) or by the empirical method described below.

### 2.2 Behaviour of Jack Arch Bridge Decks

The global behaviour of a jack arch bridge deck is a function of the complex interaction of many variable elements, namely beams, jack arches, backing, fill and surfacing. The elements generally combine to provide a composite construction, which determines both the load distribution characteristics of, and the stress distribution through the deck. Irrespective of whether it is necessary to assume composite action with the beams in order to justify the assessed beam capacity at ULS, composite action will modify the deflection characteristics over the area of the deck compared with that for a simplistic 'beam only' deck. Should this composite behaviour breakdown over part of the deck, there will be less distribution of load effects, therefore vertical and horizontal deflections will increase locally which in itself will cause further breakdown. It is considered essential that the transverse integrity is preserved to assure the ULS Capacity of the deck.

A study of such decks (see Section 2.4) has shown, for decks complying with the criteria listed in that Section, that their behaviour is not dissimilar to that of filler beam decks, ie there is both longitudinal and transverse composite action. As the jack arches behave not as true arches but partly as the 'filler' in the deck, it follows that conventional thrust force calculations are inappropriate and the lateral forces at the arch springings are less than those for true arches. Furthermore, the in plan deck transverse stiffness necessary for developing the internal forces to resist arch thrusts, is influenced significantly by the degree of longitudinal composite action. Lateral restraint (eg ties) is still required in order to preserve the structural integrity of the deck.

It can be argued that if the transverse composite action is to be durable and not breakdown, the backing to the arches should be structural. This was confirmed by the study that examined the correlation between damage, the degree of lateral restraint provided and the presence or otherwise of structural fill.

### 2.3 Quantitative Assessment Methods

Quantitative assessment methods should model the behaviour of the edge beams realistically with respect to lateral loading. Account should be taken of tie spacing and section, tie height above springing level, backing material type and height, considerations with respect to composite action (see below), the transverse and torsional stiffnesses of the external beams and the degree of restraint provided to the beams at the supports, as well as other standard arch/beam/fill parameters.

Where no effective ties are present, there are currently no generally accepted criteria for transverse deflection of longitudinal edge beams with respect to adverse implications for the integrity of jack arch decks. In such cases, proposed criteria should be identified fully in the Form AA submission. Due account should also be taken of the following:-

- the validity of assumptions about the existence of structural backing;
- any observed structural defects to the edge bay jack arches which may indicate movement or deterioration sufficient to reduce composite action.

Further guidance is given in Sections 2.4, 2.5 and 2.6.

### 2.4 Empirical Assessment Method

This empirical assessment method has been developed from a study of available record information for a large number of jack arch decks and also from an examination of historic loading standards. Subsequently the principles have been verified by a non-linear finite discrete element analysis.

Where the configuration of a brick, masonry or concrete jack arch deck complies with all the following criteria, the Capacity should be stated as *40 tonnes (no reported deficiencies which affect capacity)*.

The criteria are as follows:-

- a maximum clear span for a jack arch of 2.0 metres.
- the jack arch springs from the bottom flange of the adjoining beams.
- adequate horizontal restraint is provided to resist horizontal forces and to maintain composite action within the deck construction. This restraint should be assessed in accordance with Section 2.5.
- for any masonry or concrete jack arch the minimum combined thickness of the jack arch plus concrete fill above should be not less than 220 mm.
- There are no structural defects due to deterioration or distress that could affect the capacity of the jack arches. Defects should be assessed in accordance with Section 2.6.
- for all bridge decks, 'structural backing'* is provided to at least the height of the extrados of the arch, subject to a minimum effective shear depth** obtained from Figure 1.
- a maximum gross aspect ratio for a jack arch of 10 (beam spacing/rise of the arch).

The 'hidden' details used for the checking for compliance can be taken from the record drawings, provided there is no contrary evidence from the inspection. This contrary evidence may be dimensional and/or subsequent interpretation of defect(s)/deterioration.

**'Structural fill/backing' is classified as concrete, gypsum lime or mortared masonry.*

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

In the case of cast iron beams, however, a trial hole should be undertaken to confirm the existence of structural backing if there is any doubt. Also, for all beams, if the record drawings indicate a non-structural fill, then a trial hole should be undertaken.

The vast majority of jack arch bridge decks may be expected to comply with the above criteria. For the few that do not, it has been inferred from the study that they were not constructed to the standard practice or design of the time. In the case of cast iron bridge decks, it is considered that such jack arches should be classified as a 'failure'. This is because approximately 10% of cast iron jack arch bridge decks do not accord with the above criteria and these have generally been found to have structural defects. For wrought iron and steel bridge decks, each bridge that does not comply with the above criteria should be considered on an individual basis with careful examination of the various defects detailed in the Inspection Report.

### 2.5 Lateral Restraint to Jack Arches

As stated above, adequate lateral restraint is required to enable jack arches to perform structurally without distress; for discrete restraints (ties) their spacing/position affects the degree of lateral bending and/or twist in the supporting beams.

The study of jack arch decks has shown that significant structural defects can occur where insufficient, missing or damaged ties are present in the external bays of bridges with transverse spanning jack arches. Structural defects rarely occur to inner bays of these decks unless there is an absence of structural backing. Defects are also rare to bridge decks which have external service bays that have a form that can provide rigidity by in plan stiffening, unless there is some 'outside' contributor to the defect, e.g. differential settlement of abutments. It should be noted that it is not uncommon to have service bays that provide an inadequate restraint, e.g. cast iron plates sitting on the bottom flange of adjoining beams. In the absence of any other effective restraint, such service bays should be classified as 'soft edges' and, as such, should be discounted from providing lateral restraint.

For longitudinal spanning jack arches, the study has not identified any structural defects to the outer bays (i.e. end bays) that could be considered to be attributable to a lack of restraint arising from the omission of ties or indeed missing or corroded ties. Some of these bridges have slab construction to the end bays which act in similar fashion to service bays in transverse spanning jack arch structures, i.e. they may provide the necessary restraint. However, it has also been concluded from the study that there must be some restraint to lateral movement of the end transverse beams provided above the substructure, although details of this are not usually available, as damage is very rare. As a result, it can be concluded that horizontal restraint need not be provided to the end bays of longitudinally spanning jack arches unless there is visible evidence of distress. Where there is distress it will be necessary to undertake a very careful investigation in order to determine the structural deficiency (e.g. corrosion of hidden elements) pending a more detailed assessment.

From the study it has been possible to identify a minimum value of (specific) tie area for transverse spanning arches below which the integrity of the structure could be considered to be suspect. For cast iron bridge decks, this value is 260mm$^2$/m length of beam (for ties of wrought iron or steel). Additionally the ties should be spaced not more than 2.5m apart. Where ties are not provided at arch springing level but within the crown of the arch and the level of structural fill is such that the arch/tie/fill configuration could permit rotation of the edge beam, then the ties should be qualitatively 'failed'.

For wrought iron/steel bridge decks with transverse spanning jack arches, a similar appraisal has been undertaken and an acceptable level of tie area determined. Although the tie area is related to the arch aspect ratio, it is considered that a minimum (specific) tie area of 260mm$^2$/m length of the beam is appropriate for all such bridge decks. The ties should be spaced not more than 3.0m apart. Where ties are not provided at arch springing level but within the crown of the arch, then careful appraisal should be given to the stability of the arch/tie/fill configuration with respect to rotation of the edge beam, and hence to the acceptability of the ties, irrespective of the total area of ties. Where there is doubt, then the ties should be qualitatively 'failed'.

For both cast iron decks and wrought iron/steel decks, the ties should be regarded as ineffective if they are bowed/slack or not attached firmly and securely to the beams. Any hooked tie formed from bent rod should be deemed to be ineffective.

Ties to outer bays should therefore be subject to empirical assessment and where the 'design' ties comply with the minimum specific area and maximum spacings given above and there is no significant loss of section to the ties, then the ties can be deemed to have 'passed' and the bridge deck Capacity should be stated as *40 tonnes* with respect to the performance of the ties. Where the specific tie area provided is less than the minimum, or has been reduced to less than this figure by deterioration, or the tie spacing is excessive, then the ties should be 'failed' and the Capacity should be stated as *Dead Load only (inadequate ties to external bay).*

The specific tie area should be calculated using the following expression:-

> Specific Area of Tie ($mm^2/m$ length of the beam) = (No of ties +1) x (area of one tie)/clear skew span of beams supporting the arch.

*Notes:- (i) the addition of 1 to the number of ties reflects the restraint provided at the beam supports;*

*(ii) the area of the tie should be determined from calculation of the gross cross-sectional area of the tie (ignoring any section loss due to thread cut into the tie for connection purposes).*

## 2.6 Structural Defects to Jack Arches

From the study it is apparent that most of the structural defects to jack arch structures occur in the outer bays as a result of inadequate lateral restraint being provided. Where this is the case, the distress usually consists of either cracking or deformation of the arch barrel or more likely missing or loose brickwork at the crown of the arch and these are indicators of some lateral movement and/or rotation of the outer beam. It is essential that, if such defects resulting from the lack of transverse restraint are identified in the outer bay of a jack arch, work should be recommended to make them good to maintain the integrity of the structure. In such a case the jack arches should be assessed as a 'failure' and reported as *Dead Load only (repairs to jack arches recommended).* However, it is also important that the assessment of the bridge considers the cause of these structural defects, i.e. the inadequate lateral restraint; this should be reported as *Dead Load only (inadequate ties to external bay/adjacent arch bay).*

Other, general defects can be present in brick/masonry jack arches, such as loss of mortar, spalling of brickwork, missing bricks and the like which are a result of general decay of the fabric of the bridge arising from the penetration of water, train emissions, frost action, etc. With respect to this general deterioration, it is considered that professional judgement should be made with respect to whether the defect(s) could affect the capacity of the arch. For instance where concrete backing is present and there is no reason to doubt its integrity, the loss of some mortar or the odd brick is unlikely to affect the capacity of the jack arch. On the other hand, if the arch is deteriorating from the effects of water leakage at the crown or there are a number of bricks loose or missing and there is only non-structural fill above the arch it is likely that this should be considered as a 'failure' given the proximity of wheel loading and the accelerated deterioration that may be taking place as a result of repetitive loading. This is especially important in cast iron decks that have relatively shallow construction.

Less common structural defects which can lead to a 'failure' of a jack arch include supporting beams which are either bowed - reported under Horizontal Restraint as *Dead Load only (inadequate ties to .......)* - or have inadequate support to springings - reported under Jack Arches as *Dead Load only (repairs to supporting beams recommended).*

Also, there are some situations where there is either cracking or other movement to the substructure that has led to or could lead to cracks in the jack arches. For these bridges, recommendations should be made for the substructure to be monitored or for renovation works to be undertaken; this should be reported appropriately under *Substructure - Qualitative Assessment.........* In addition, the jack arches (in the absence of any other relevant deficiency/defect) should be reported as either (if cracked) *Dead Load only (repairs to jack arches recommended)* or (if uncracked) *40 tonnes (no reported deficiencies* *which affect capacity)* as appropriate.

It is important to distinguish between this cause/effect and a spreading arch leading to the cracking of the arch and induced cracking in the sub-structure; in this instance, the jack arches should be 'failed' and reported as *Dead Load only (inadequate horizontal restraint).*

The Table given below summarises the various deficiencies/defects that can affect the performance of the jack arches and ties to a bridge; the likely outcome of the empirical assessment is also stated.

**Summary Table for Empirical Assessment of Brick, Masonry and Concrete Jack Arches and Associated Ties**

The following table tabulates the various structural deficiencies/defects that can occur in a jack arch structure and the likely outcome of the empirical assessment.

| Type No | Deficiency/Defect | Empirical Assessment | |
|---|---|---|---|
| | | CI Decks | WI/Steel Decks |
| | **Deficiency** | | |
| 1 | No structural backing to crown level of extrados | Fail (1) (4) | Fail (4) |
| 2 | No ties in edge bay (if a jack arch) | Fail | Fail (5) |
| 3a (CI) | Ties in edge bay (if a jack arch) if < 260 $mm^2/m$ <u>or</u> if >2.5m apart | Fail | N/A (5) |
| 3b (WI/ST) | Ties in edge bay (if a jack arch) if < 260 $mm^2/m$ <u>or</u> if >3.0m apart | N/A | Fail (5) |
| 4 | Ties located within crown of external arch | Fail | Possible Fail (5) |
| 5 | Jack arch adjacent to 'soft' edge service bay treat as 2, 3a or 3b accordingly. | Fail | Fail (5) |
| | **Defect** | | |
| 6 | Rotation of supporting beam | Fail | Fail |
| 7 | Horizontal displacement of supporting beam | Fail | Fail |
| 8 | Inadequate support to springings eg corrosion of bottom flange of supporting beam over a significant length, missing bedding mortar | Possible Fail | Possible Fail |
| 9 | Transversely bowed bottom flange of supporting beam | Fail | Fail |
| 10 | Cracking at crown of arch owing to spreading of springings (other than 12, 13) | Fail | Fail |
| 11 | Distortion and any associated cracking of jack arch barrel | Fail | Fail |
| 12 | Arch crack resulting in substructure crack | Fail | Fail (5) |
| 13 | Substructure crack or other distress resulting in crack to jack arch | Possible Fail (3) | Possible Fail (3) (5) |

*Notes:*

*(1) Results also in loss of D/d (composite action).*
*(2) Not used.*
*(3) 'Substructure renovation' or 'Monitoring' as appropriate; 'Repair of arch' (if appropriate).*
*(4) If the record drawings indicate a non-structural backing and there is an absence of any defect which could be attributable to this, then a trial hole shall be undertaken prior to final certification.*
*(5) Not applicable in general to longitudinally spanning arches.*

## 3 Assessment of Metal Plate Arch Bridge Decks

The basic forms of metal plate arches can be described by three primary fields:-

| Field | Geometry | Type | Structural Action |
|---|---|---|---|
| Options | Single curvature<br>Double curvature | Un-stiffened<br>Stiffened | Arch (compressive)<br>Catenary (tensile) |

Arches Located on or Near the Top Flange

For metal plate arches located on or near the top flange, it is considered that the likely type of assessment should be as given below.

| Form | Comment | Assessment |
|---|---|---|
| Single curvature, stiffened or un-stiffened, arch | For external bays with ties and all internal bays; with structural infill | Quantitative - Tied arch |
| Single curvature, stiffened or un-stiffened, arch | For external bays with ties and all internal bays; with non-structural infill | Quantitative - Tied arch |
| Single curvature, stiffened or un-stiffened, arch | External bays, no ties, structural infill | Quantitative - Plate in simple bending (composite action?). |
| Single curvature, stiffened or un-stiffened, arch | External bays, no ties, no structural infill | Quantitative - Plate in simple bending. |
| Double curvature, stiffened or un-stiffened, arch | | Quantitative - as BD56. |
| Double curvature, stiffened or un-stiffened, catenary | | Quantitative - as BD56. |

Arches Located on or Near the Bottom Flange

For a metal plate arch springing from or near the bottom flange, consideration should be given as to whether it can be classified as 'permanent formwork' if there is concrete above. If the drawings show that there is concrete infill above the metal plate such that there is an equivalent masonry jack arch in both arch aspect and 'barrel' thickness (See Section 2.2), then the empirical jack arch assessment method may be used. However, the condition of the arch and any defect present in the road above would need to be allowed for in this assessment. If there is any indication to suggest that the structural infill may not be present or its condition is suspect, then confirmation should be obtained from a site investigation. If the infill is confirmed as being 'non-structural', then the metal plate should be assessed quantitatively (if ties are provided, then as a tied arch; if no ties are provided, then as a plate in simple bending) and reported accordingly.

## 4 Format of the Assessment Report and BA Certificate

It is very important that the text of the Assessment Report details the interpretation of the structural configuration of the jack arch structure and those deficiencies/defects that affect the empirical/quantitative assessment of the components detailed in this Information Sheet. The conclusions of the superstructure assessment should refer as appropriate to:-

- the quantitative assessment of the beams;
- the quantitative or the empirical assessment (as the case may be) of the jack arches and associated lateral restraint;
- the method of assessment of the metal plate arches (if any).

The Executive Summary and the BA Certificate should utilise identical wording.

Where the empirical method of assessment has been used for the jack arches, the format of the Conclusions to the Assessment of a jack arch bridge should in general be as follows:-

A. Where the jack arch bridge complies with the criteria given in Section 2.4 and there are no defects which can be considered to affect the capacity.

i) *Superstructure*

*Overall Capacity of the Deck ……………………..(the lowest rating of all components).*

a) *Quantitative*

*Deck Beam Capacity ……………………….*
*Plus any other elements assessed quantitatively ……………………….*

b) *Empirical*

| | | |
|---|---|---|
| *Horizontal Restraint* | - | *40 tonnes (no reported deficiencies which affect capacity). -* (Note: - this includes the case where external bays provide the necessary restraint irrespective of the presence/condition of ties in other bays.) |
| *Jack Arches* | | *40 tonnes (no reported deficiencies which affect capacity).* |

B. For a cast iron beam deck with no structural backing/inadequate effective shear depth

i) *Superstructure*

| | |
|---|---|
| *Overall Capacity of the Deck* | *Dead Load only* or *Inadequate for Dead Load* (as appropriate). |

a) *Quantitative*

*Deck Beam Capacity ……………………….*
*Plus any other elements assessed quantitatively ……………………….*

b) *Empirical*

*Deck Capacity Dead Load Only (no structural backing/inadequate effective shear depth to jack arches).*

C. For inadequate horizontal restraint and/or failure of the jack arches.

i) *Superstructure*

*Overall Capacity of the Deck* — *Dead Load only* or *Inadequate for Dead Load* (as appropriate).

a) *Quantitative*

*Deck Beam Capacity ………………………*
*Plus any other elements assessed quantitatively ………………………*

b) *Empirical*

*Horizontal restraint* — *40 tonne* *<u>or</u>*
*Dead Load only (inadequate ties to external bay/adjacent arch bay.*)*
* - delete as appropriate.

*Jack Arches* — *40 tonne* *<u>or</u>*
*Dead Load only (repairs to jack arches/supporting beams * recommended.)*
* - delete as appropriate.

## 5 Pro Forma for Empirical Assessment of Jack Arches and Associated Ties

The pro forma given on the following sheets has been developed to assist in the empirical assessment of jack arch decks and reflects the guidance and wording contained in this Information Sheet. Its use on such structures is recommended, as it is believed that it will contribute to both a consistency in approach and simplicity in reporting.

## 6 Reporting

### 6.1 Compliant Decks

Where a jack arch deck is compliant with the criteria given in Section 2.4, is not found to be deficient and has no defects which can be considered to affect the capacity, then the capacity should be reported as *40 tonnes*.

Otherwise, the capacity should be reported as *Dead Load only* followed by an explanation for this 'failure' in brackets.

### 6.2 Non-Compliant Decks

Where a deck is non-compliant with the criteria given in Section 2.4, is not found to be deficient and has no defects which can be considered to affect the capacity, it does not necessarily follow that the jack arches and ties (if any) will have a capacity less 40 tonnes. In such cases the proposed approach should be confirmed by the Assessor with the Technical Approval Authority prior to undertaking that aspect of the work.

## PRO FORMA FOR EMPIRICAL ASSESSMENT OF BRICK, MASONRY AND CONCRETE JACK ARCHES AND ASSOCIATED TIES

*(To be included with the Assessment Report Calculations)*

| | |
|---|---|
| **BRIDGE NAME:** | |
| **RAILTRACK NO:** | |

**Assessment should include completion of all three Sections even where Section 1 has shown the bridge deck to be non-compliant.**

### SECTION 1 CHECKS FOR COMPLIANCE WITH 40 T CONFIGURATION REQUIREMENTS

| | | | | **Compliant Yes/No** |
|---|---|---|---|---|
| What is maximum clear span of the arch<br>*Non-compliant if greater than 2.0m* | | | m | |
| Do jack arches spring from bottom flanges of beams?<br>*If not, non compliant* | | | | |
| What is the beam spacing?<br>What is the rise of the arch?<br>Gross aspect ratio<br>*Non -compliant if greater than 10* | b<br>$r_c$<br>$b/r_c$ | =<br>=<br>= | m<br>m | |
| What is the arch barrel thickness (including concrete fill above) and how is it derived ie from record drawings or site investigation?<br>*Non-compliant if thickness less than 220* | d | = | mm | |

## PRO FORMA FOR EMPIRICAL ASSESSMENT OF BRICK, MASONRY AND CONCRETE JACK ARCHES AND ASSOCIATED TIES

*(To be included with the Assessment Report Calculations)*

| BRIDGE NAME: |
| --- |
| RAILTRACK NO: |

### SECTION 2 CHECKS FOR DEFICIENCY

| Type No | Deficiency | Pass/ Fail |
| --- | --- | --- |
| 1 | What is the backing material? Is it structural?<br>Does the structural backing extend to at least the crown level of the arch extrados?<br>*If not, then fail (1) (4).*<br>What is effective shear depth of deck?<br>(= arch rise + barrel thickness + height of structural fill above crown of extrados) $D_s$ = mm<br>Is $D_s \geq$ minimum requirements of Fig 1. Fail if < Fig 1 | |
| 2 | Do jack arches span longitudinally (eg in half through girder construction) or transversely between longitudinal girders?<br>For longitudinal spanning jack arches, ignore following questions on ties/lateral restraint and state N/A.<br>Are ties provided in edge bays of transversely spanning jack arches?<br>*If yes, go to 3a/3b* *If not, fail unless edge bay is 'hard' (see 5)* | |
| 3a<br><br>CI | What is the cross sectional area of one tie? (allowing for corrosion losses) A = $mm^2$<br>What is number of ties per beam length? n = No<br>What is the clear skew span? L = m<br>Specific area of tie ($A_S$) = $\frac{(n+1) \times A}{L}$ $A_S$ = $mm^2/m$<br>*Non-compliant if less than $260mm^2/m$*<br>What is maximum tie spacing? S = m<br>*Non-compliant if greater than 2.5m for cast iron* | |
| 3b<br><br>WI/<br>ST | What is the cross sectional area of one tie? (allowing for corrosion losses) A = $mm^2$<br>What is number of ties per beam length? n = No<br>What is the clear skew span? L = m<br>Specific area of tie ($A_S$) = $\frac{(n+1) \times A}{L}$ $A_S$ = $mm^2/m$<br>*Non-compliant if less than $260mm^2/m$*<br>What is maximum tie spacing? S = m<br>*Non-compliant if greater than 3.0m for wrought iron/steel* | |
| 4 | Are ties located within crown of external arch?<br>*If so, then fail CI or possible fail for WI/ Steel* | |
| 5 | Does external bay construction provide alternative lateral restraint? (ie not soft edge)?<br>*If so, pass.*<br>*If not, are ties provided in first Jack Arch bay? If yes, treat as 3a (or 3b). Otherwise fail.* | |

*Notes:* (1) *Results also in loss of D/d (composite action) for cast iron beams*
(4) *A trial hole should be undertaken to confirm the existence of structural backing if there is any doubt.*

## PRO FORMA FOR EMPIRICAL ASSESSMENT OF BRICK, MASONRY AND CONCRETE JACK ARCHES AND ASSOCIATED TIES

*(To be included with the Assessment Report Calculations)*

| BRIDGE NAME: |
|---|
| RAILTRACK NO: |

### SECTION 3 CHECKS FOR DEFECTS

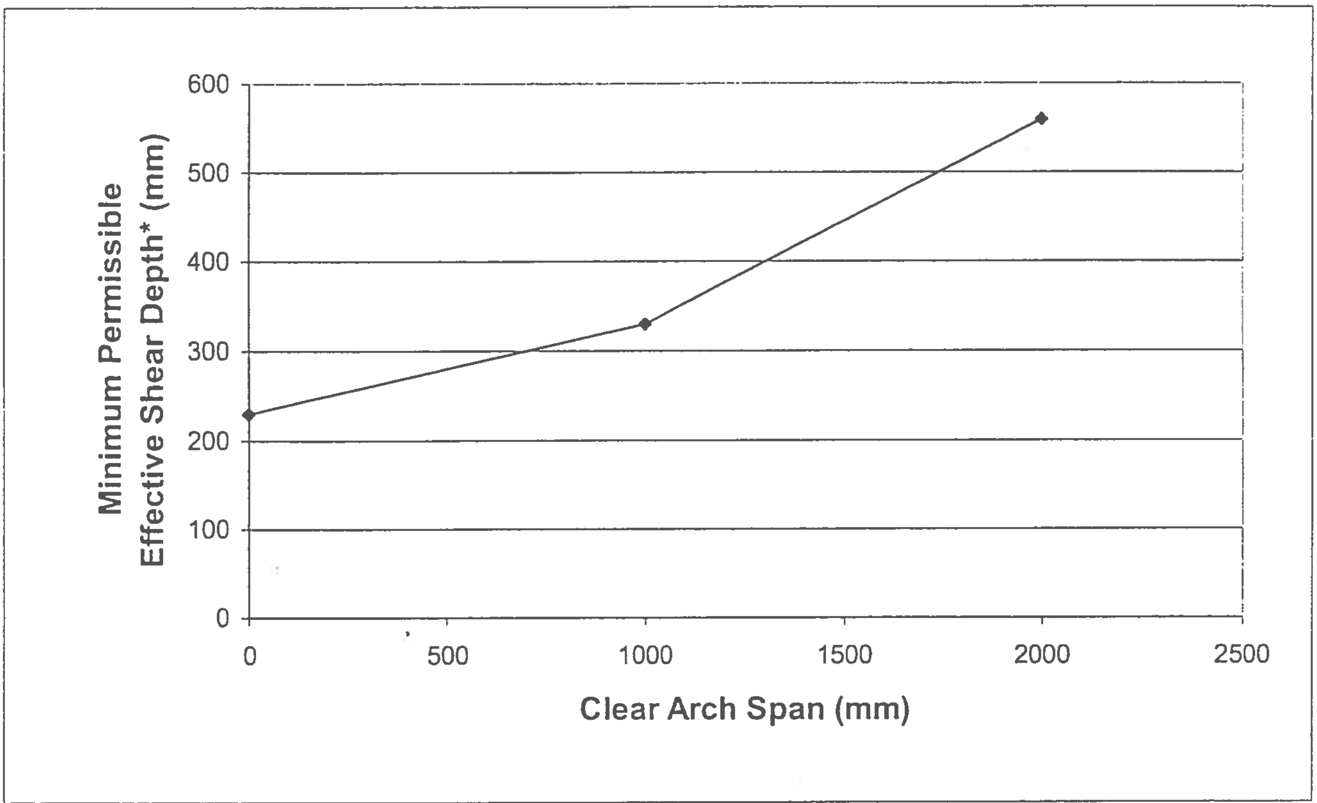
| Type No | Defect | Empirical Assessment | | Pass /Fail |
|---|---|---|---|---|
| | | CI Decks | WI/Steel Decks | |
| 6 | Rotation of supporting beam | Fail | Fail | |
| 7 | Horizontal displacement of supporting beam | Fail | Fail | |
| 8 | Inadequate support to springings eg corrosion of bottom flange of supporting beam over a significant length, missing bedding mortar | Possible Fail | Possible Fail | |
| 9 | Transversely bowed bottom flange of supporting beam | Fail | Fail | |
| 10 | Cracking at crown of arch owing to spreading of springings (other than 12, 13) | Fail | Fail | |
| 11 | Distortion and any associated cracking of jack arch barrel | Fail | Fail | |
| 12 | Arch crack resulting in substructure crack | Fail | Fail (5) | |
| 13 | Substructure crack or other distress resulting in crack to jack arch | Possible Fail (3) | Possible Fail (3) (5) | |

*Note:* (3) *'Substructure renovation' or 'Monitoring' as appropriate; 'Repair of arch' (if appropriate).*
(5) *Not applicable in general to longitudinally spanning arches.*

PRO FORMA FOR EMPIRICAL ASSESSMENT OF BRICK, MASONRY AND CONCRETE JACK ARCHES AND ASSOCIATED TIES

*(To be included with the Assessment Report Calculations)*

| | |
|---|---|
| BRIDGE NAME: | |
| RAILTRACK NO: | |




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

**Figure 1**

**CURRENT INFORMATION SHEET NO 22**
**SUBJECT: ASSESSMENT OF JACK ARCHES, METAL ARCH PLATES AND ASSOCIATED TIES IN METAL BEAM BRIDGE DECKS**
**DATE: February 2001**
**STATUS: FINAL**

**Part 1:** Originator

Organisation: Gifford and Partners

I certify that reasonable professional skill and care has been used in the compilation of this document.

| | |
|---|---|
| Signed: W. J. Newton | Title: Associate |
| Name: W J Newton | Date: 23.02.01 |
| To be signed by the document author. | |

I certify that the staff who have prepared the above documents are competent to carry out their duties and that (so far as I can reasonably ascertain) they have used reasonable professional skill and care.

| | |
|---|---|
| Signed: | Title: Technical Director |
| Name: T P Holmes | Date: 23. 2. 2001 |
| To be signed by the Director (or equivalent) to whom author is responsible. | |

**CURRENT INFORMATION SHEET NO 22**
**SUBJECT: ASSESSMENT OF JACK ARCHES, METAL ARCH PLATES AND ASSOCIATED TIES IN METAL BEAM BRIDGE DECKS**
**DATE: February 2001**
**STATUS: FINAL**

**Part 2:** Endorsement

Organisation: Pell Frischmann Consulting Engineers Ltd.

I certify that reasonable professional skill and care has been used in endorsing this document.

| | |
|---|---|
| Signed: A Small | Title: Technical Director |
| Name: A G SMALL CEng | Date: 28/02/01 |
| To be signed by the ~~Senior Reviewing Engineer~~. Technical Director. | |

**Part 3:** Acceptance on behalf of Railtrack

I approve the implementation of this Current Information Sheet with respect to the Bridgeguard 3 programme only.

| | |
|---|---|
| Signed: I Bucknall | Title: PRINCIPAL ASSET MANAGER |
| Name: I Bucknall | Date: 12 March 01. |
| To be signed by Railtrack Project Delivery Technical Service and Innovation Team | |

**CURRENT INFORMATION SHEET NO 23**
**SUBJECT: USE OF BD AND BA 61 FOR CASED AND FILLER BEAM BRIDGES**
**DATE: MAY 2000**
**STATUS: FINAL**

**1. Prepared by Gifford and Partners**

i) Author, Name: P.A. Jackson

Signature: P.A. Jack Date 31-5-00

ii) Checker, Name: T.P. Holmes

Signature: ................ Date 25.5.2000

**2. Endorsed by Pell Frischmann Consulting Engineers Ltd**

i) Endorsed by Senior Reviewing Engineer for Pell Frischmann Consulting Engineers Ltd.

Name D. J. MIDDLE

Signature: D. Middle Date 9/6/00

**3. Approved for issue by Railtrack**

i) Approved by Railtrack Project Delivery (~~London North Eastern~~ Great Western Zone) for Bridgeguard 3

Name JOHN BLAGROVE

Signature: John Blagrove Date 04.12.00.

**CURRENT INFORMATION SHEET NO 23**
**SUBJECT: USEOF BD AND BA 61 FOR CASED AND FILLER BEAM BRIDGES**
**DATE: MAY 2000**
**STATUS: FINAL**

**Part 1:** Originator

Organisation: Gifford and Partners

I certify* that reasonable professional skill and care has been used in the compilation of this document.

| | |
|---|---|
| Signed: P.A. Jackson | Title: Associate |
| Name: P.A. Jackson | Date: 31.5.00 |
| To be signed by the document author. | |

I certify* that the staff who have prepared the above documents are competent to carry out their duties and that (so far as I can reasonably ascertain) they have used reasonable professional skill and care.

| | |
|---|---|
| Signed: G Tilly | Title: DIRECTOR |
| Name: G P TILLY | Date: 2 — VI — 00 |
| To be signed by the Director (or equivalent) to whom author is responsible. | |

*This certification (including that detailed in the Introductory Sheets to this Information Sheet) is on the basis that Railtrack are in possession of draft documents (Revisions to BD 61 and BA 61 as detailed in Appendices A & B of this Information Sheet) from the Highway Agency and have recognised their relevance to their structures. As such Railtrack have asked their Reviewing Consultants to confirm that they have no reason to doubt the validity of these draft documents as prepared by another consultant for the Highway Agency and as such have prepared an introduction to provide some commentary on the use of these documents and how they should be utilised in the context of Bridgeguard 3.

**CURRENT INFORMATION SHEET NO 23**
**SUBJECT: USEOF BD AND BA 61 FOR CASED AND FILLER BEAM BRIDGES**
**DATE: MAY 2000**
**STATUS: FINAL**

**Part 2:** Endorsement Organisation: Pell Frischmann Consulting Engineers Ltd.

I certify* that reasonable professional skill and care has been used in endorsing this document.

| | |
|---|---|
| Signed: Dmiddle | Title: Principal Engineer |
| Name: D. J. MIDDLE | Date: 9/6/00 |
| To be signed by the Senior Reviewing Engineer. | |

**Part 3:** Acceptance on behalf of Railtrack

I approve the implementation of this Current Information Sheet with respect to the Bridgeguard 3 programme only.

| | |
|---|---|
| Signed: Ian Bucknall. | Title: PRINCIPAL ASSET MANAGER |
| Name: I Bucknall | Date: 23 Jun. 00. |
| To be signed by Railtrack Project Delivery Technical Service and Innovation Team | |

*This certification (including that detailed in the Introductory Sheets to this Information Sheet) is on the basis that Railtrack are in possession of draft documents (Revisions to BD 61 and BA 61 as detailed in Appendices A & B of this Information Sheet) from the Highway Agency and have recognised their relevance to their structures. As such Railtrack have asked their Reviewing Consultants to confirm that they have no reason to doubt the validity of these draft documents as prepared by another consultant for the Highway Agency and as such have prepared an introduction to provide some commentary on the use of these documents and how they should be utilised in the context of Bridgeguard 3.

Commercial-in-Confidence

Report No. B0395A/TM/38202
June 2000

# BRIDGEGUARD 3

# CURRENT INFORMATION SHEET NO 23

GIFFORD
AND PARTNERS

# BRIDGEGUARD 3

# CURRENT INFORMATION SHEET NO 23

## CONTENTS

## CURRENT INFORMATION SHEET NO 23

## SUBJECT: USE OF BD AND BA 61 FOR CASED AND FILLER BEAM BRIDGES

**This Current Information Sheet is issued for guidance purposes only: it is not mandatory. The Assessor must be satisfied that the advice given in this Information Sheet is appropriate to the structure in question.**

### 1 INTRODUCTION

The Highways Agency has recently produced BD 61 and corresponding advice note BA 61 on the assessment of composite bridges. BD 61 is the assessment equivalent of BD 16 and BS 5400: Part 5. It is the composite equivalent of BD 44 for concrete and is in the same format that is a very brief main text with an 'Annex A' which contains an assessment version of the design code. This keeps the original clause numbers adding new ones as required.

BS 5400: Part 5 is predominantly about bridges of concrete slab on steel girder form with stud shear connectors. This is the commonest modern form. In contrast, the bridges on the Bridgeguard 3 Contract are more typically of cased, filler beam or jack arch construction. BD and BA 61 cover these forms of construction, but a number of problems have been identified, relating to the sections covering these types of bridges, which have been raised with the Highways Agency. Largely as a result of this, the original drafters of the documents have undertaken further work. This has led to some revisions being proposed. Railtrack have received an advanced copy of the proposed revisions, which are currently subject to the Highways Agency approval process.

This Information Sheet details the revisions that affect the assessment of cased and filler beams and should forthwith be used on Bridgeguard 3 assessments. Please note that it does not cover jack arch structures. For jack arch structures, any proposal for the use of BD/BA 61 should be subject to discussion with the Technical Approval Authority.

The revisions to BD and BA 61 make some technical changes and also reduce the previous potential confusion and ambiguity. Appendix A of this Information Sheet contains extracts from the proposed revised BD 61 Annex A which includes all the significant changes highlighted. Sections with purely minor editorial corrections are not included in Appendix A. However, it should be noted that all references to 'characteristic strength' should be to 'characteristic or worst credible strength' and the references to 'steel' girders are changed to 'steel or iron'. Appendix B contains Chapter 8 of the proposed revision to BA 61 Annex A (which covers cased and filler beam construction) along with the appendix giving flow charts.

The clauses contained in Appendix A and B should be used for Bridgeguard 3. Most of the changes are clarifications: the revised text clarifies the intent of the original text.

### 2 LIMIT STATES

BD 21 normally considers only ULS although it does suggest that SLS checks will be required for more recent structures. However, BS 5400 Part 5 checks some things, notably interface shear in cased and filler beams, only at SLS. This was justified because the SLS check is more critical so the ULS checks would never govern design. Literal following of the text of BD 21 and BS 5400 Part 5 would result in this check being missed out completely. This is not justified and BD 61 resolves it by reintroducing SLS checks.

The nominal loading used for these checks will be the BD 21 load (ie with the reduction factor which is 0.91 for 40 tonne assessment live loading) but the load factors come from BD 37 with one exception for cast iron (see 3.1 below).

BD61 also introduces other SLS checks. However, these are a departure from the principle which BD 21 and Bridgeguard 3 follows, namely that only ULS is checked as serviceability failure, by definition, would be apparent at inspection. SLS checks would therefore not normally be required for Bridgeguard. This causes a problem in that it is necessary to identify which SLS checks are required to ensure safety. In general, the interface checks are required but the crack width check is not and the stress checks are only required for cast iron, see Section 5.

## 3 IRON GIRDERS

BD 61 covers bridges with cast and wrought iron as well as steel girders. Wrought iron girders are treated like steelwork with material strengths from BD 21. Cast iron is also covered but there are some particular points about it which deserve explanation:

### 3.1 Limit State for Cast Iron

BD 21 assesses cast iron using working stresses under nominal loads. BD 61, however, uses SLS and ULS checks. Although 'working stress' and 'service stress' approaches are often considered to be the same, 'working stress' in BD 21 implies that load factors are always 1.0 (except that BD 21 uses 1.5 for surfacing) whereas BS 5400 Combination 1 uses a factor of 1.2 for HA at SLS.

The proposed revisions to BD 61 (Clause 4.3.2b) deem the BD 21 stress limits to be a serviceability limit state but apply special load factors for highway load. The factor for HA load is indeed 1.0 but HB has a load factor of 0.92. The load factors for permanent load come from BD 21 table 3.1.

This gives the following approach.

1) The interface is checked at SLS using cracked elastic section properties. It is reasonable to suppose that the use of working stress checks means the section remains elastic at ULS which means ULS interface checks are not required to BA 61. The revisions confirm that only SLS checks are required.

If there are web stiffeners in the cast iron that are embedded in concrete, they obviously improve interface strength. The BD (Clause 5.3.3.8.1) gives a way of allowing for incidental shear connection.

2) The cast iron is checked to BD 21 stress limits under service loads with the load factors as detailed above. If the interface stress is OK, the cracked elastic composite section properties are used.

If the interface stress criteria are not met, the cast iron beam will have to be checked using the beam section alone.

3) The concrete compressive stress is checked against the BS 5400 SLS limit of $0.5f_{cu}$ ($0.38f_{cu}$ for sections in direct compression). It appears relatively unlikely that concrete stresses will be critical.

Although BD 61 says only SLS is checked, this would not always apply to all concrete in a cased beam and slab deck. The slab would still have to be checked to BD 44 in the normal way, including checking longitudinal shear stress in the concrete.

4) Since the analysis assumes linear elastic behaviour, construction history should be considered which usually means taking the dead load on the iron section alone.

## 3.2 Ductility and Global Analysis for Cast Iron

For ductile structures, the safe theorem of plastic design means that any elastic solution gives safe results even if the section properties used in determining the distribution of load effects throughout the structure are incorrect. However, cast iron is a brittle material and for such cases the safe theorem of plastic design is invalidated. It is therefore possible for a cast iron structure to fail due to overstress in a beam even when there is adequate strength in other beams to carry the 'excess' load. It is also possible, as a result of this, for either under or over-estimation of lateral load distribution to result in an over-estimation of capacity, depending on the structural configuration.

In a simple bridge consisting of reasonably similar beams, these problems are unlikely to be significant. Improved distribution will spread concentrated loads amongst more beams reducing the maximum stress in a beam. The usual assumption that under-estimating distribution properties (such as by using static or 'simple' distribution methods) is conservative is therefore valid. However, problems can arise when the edge beams are weaker than the interior beams. Simple load distribution assumptions may indicate that they experience no highway live load stress but, in fact, they can and this needs to be considered.

A more severe problem arises when bridges have a mixture of cast iron and steel or wrought iron beams. Normally, the cast iron beams reach peak load at much smaller deflections than the others. Any transverse stiffness of the deck therefore increases the load on them. With this type of deck, a simple supposedly 'conservative' distribution analysis can be unsafe and this problem should be considered in assessment. Such decks are quite common in reconstructed bridges where some of the original beams were reused.

# 4 CASED AND FILLER BEAMS

## 4.1 Location of Clauses

The BA contains rules for checking sections in cased and filler beams which are different from those in the BD and also different for different spans and types of construction. In the original documents, it was not clear when which applied. However, this potential confusion has now been resolved by changes to the text and by the inclusion of flow charts in BA 61. (These are included in Appendix B.) As a result of the way the clauses are spread through the documents, it will probably be found easier to refer to the flow charts first to identify which checks are required. The text then gives the details of the checks.

Basically the position is that provided the bridge complies with the detailing rules of BD and the concrete is at least grade 25, the BD rules can be used. For all other cases the BA has to be used. The documents state that you may also use the BA for cases covered by the BD, to obtain a more refined assessment.

## 4.2 Buckling

Clause 8.4 of the BD states that the cased part of the section can be considered as compact and Clause 8.1.6 of the BA notes that you can still assume this when the bond stresses are exceeded and the section is not considered as composite. The documents do not actually state that lateral torsional buckling can be neglected but it is considered unlikely that any fully cased or filler beam would ever suffer from this. Only for a very slender cased beam, which is not restrained by the slab, will this need to be considered.

## 4.3 Section Checks

There has always been an oddity in the assessment of the interface of this type of section in that BD 16 only requires it to be checked at SLS. Since BD 21 states that only ULS checks are required for older bridges, this could result in the check being missed completely. This is unsafe and a check is required which would normally be done at SLS.

BA 61 acknowledges and resolves another problem, relating to the use of plastic section analysis at ULS which gives a more severe requirement for interface strength than the normal elastic SLS check requires. This appears to imply that BD 61 and BD 16/BS 5400: Part 5 are unsafe. However, there is evidence that they are satisfactory for the type of structures they were intended for. This is why BD 61 rules can be used provided the structure complies with BD 61 detailing requirements and has at least grade 25 concrete. In the case of filler beam decks, it is necessary to have sufficient transverse reinforcement to resist the moments given by a conventional analysis to make it a 'complying' deck, see 5 below. BA 61 provides separate rules which cover a wider variety of structures.

Some of the application rules in BA 61 are different for filler and cased beams and also different for different spans and different types of cased beams. It was unclear in the original BA 61 which checks applied but the version in Appendix B is clearer. It uses two concepts which deserve some explanation:

i) Yield Moment

The expression 'yield moment' is used frequently in the BD but no corresponding definition given. The 'yield moment' should be considered as the moment at first yield according to elastic theory using full composite action with the ultimate $\gamma_m$ factors and with construction history considered. The conditions limit the design (i.e. assessment after $\gamma_m$ factors are applied) yield stress to 275N/mm$^2$. If this moment is not exceeded, bond stress is only checked at SLS using 8.5.1. Strictly, when elastic stress analysis is used at ULS a value for the limiting stress in concrete is required. However, BD 61 has not provided this limit. Neither has BD 44, which never used elastic section analysis at ULS. The only suitable limit is in the clauses of BD 56 dealing with non-compact sections. This limit is $\dfrac{0.75 f_{cu}}{\gamma_{mc}\gamma_{f3}}$. The concrete stress should be checked against this but it is unlikely to be critical in normal sections.

ii) Partial Shear Connection

The principle of these calculations is that the flexural strength is limited by the ability of the interface to transmit the force. The interface strength is assumed to be ductile and the force available is the interface strength times the length of interface between the considered section and the support. There are two approaches:

a) Linear Interpolation Method

In the linear interpolation method, the moment capacity with a given interface force is obtained by linear interpolation between the plastic moment capacity of the beam alone for a force of zero and the full composite plastic moment capacity with the force this requires. The interface force required for the full plastic moment capacity is equal to the force in the slab assumed in the plastic section analysis. The reference given (EC4) states that the interface force available in a simply supported span is equal to the length of the interface times stress times distance from the considered section to support. Strictly, the stated assumption of ductile interface used implies that the length to the end of the beam can be used. For the more unusual case of a continuous beam, the whole of the change in force between the critical hogging and sagging sections considered has to be transmitted across the interface.

b) Equilibrium Method

This is the same as a) except that, instead of using interpolation, a plastic section analysis is carried out with the compressive force in the slab and the tensile force in the steel limited to the calculated strength of the interface. This gives a higher moment capacity.

## 5. COMPLYING AND NON-COMPLYING FILLER BEAMS

### 5.1 Significance and Definitions

The difference between complying and non-complying filler beams is mainly relevant to the global analysis of the structure. However, it also affects the section checks since the BD 61 (as opposed to BA 61) rules are only allowed to be used for complying decks. There are also some extra restrictions on the section checks for non-complying decks which are otherwise as those given in BA 61 for complying decks.

If there is sufficient transverse reinforcement to resist the moments, obtained from an analysis using conventional grillage or finite element analysis, or the empirical rules in BD 61, it is a complying deck. Since it would be possible to use cracked transverse properties in this analysis, bridges that have very light or even no transverse steel could, in theory, be included. This may be an attractive option in the situations where there are problems with the rules for analysis of non-complying filler beams. However, it was not anticipated in the development of the BA and therefore the BA's longitudinal section checks for non-complying filler beams should be used for such cases.

### 5.2 Global Analysis of Non-Complying Filler Beams

BA 61 introduces analytical methods for filler beams without transverse reinforcement. Although it does not actually say so, they can be used for decks with some transverse steel but they give no way of gaining any advantage from the steel.

The rules do not appeal to purist analysts since they assume flexure and torsion in the slab are separate phenomena which they are not. The 'lateral distribution of reactions included' version, having disregarded the flexural strength of the concrete, then uses the flexural strength of the concrete to transmit the torsion moment. Some assessors therefore object to using them. However, they are based on the results of tests that showed the actual distribution to be better than the methods allow. They are therefore safe even though their theoretical basis is dubious and therefore can be used for Bridgeguard 3.

The methods described are essentially empirical methods and are based upon physical tests. As such, they suffer from the usual problems of empirical methods and their use should be restricted to bridges that are reasonably similar to those they were derived from. The commonest problem which comes up is what to do with large skews or other geometrically complicated structures. The view is that the methods can be used with big skews provided that a reasonably right mesh is used. A skew mesh may fail to detect genuine torsion problems in the slab.

Two different methods for global analysis are provided in BA 61, one considering the lateral distribution of reactions and the other not. The rules for allowing the former are quite severe but if flexure (rather than shear) is critical it should not make much difference which is used. Both methods work by considering the torsional stiffness and strength of the concrete. In the lateral distributions of reactions included method, half the torsional stiffness should be applied in each direction in the normal way. The torsional strength should then be checked by **adding the torsion per unit width from the elements in the two directions** before checking against the rule given.

When lateral distribution of reactions is not considered, torsional stiffness is only included in transverse elements and the full value should be used.

When the 'lateral distribution of reactions included' version is considered, a bending stiffness is used for the transverse elements but the flexural strength of the transverse members does not need to be checked, only the torsional strength.

In both cases, the area of concrete used for the torsion calculations should be restricted to a depth above the steel beam soffit of 1.5 times the spacing between the beam flanges. If this is done, the section analysis can be treated the same as for complying beams except that you are not allowed to use 8.1.2(ix) (which was 8.1.2(vii) in the original document). If this restriction is not complied with there is no guidance on how to check the section and in each case, the proposed methodology should be submitted to the Technical Approval Authority for acceptance.

## 6. PUNCHING SHEAR

There is a special rule for 'punching shear resistance' in 8.1.7 of the BA and it may be noted that:

1. Although not clear from the text, Figure 8.6 shows that the definition of the shear span, $a_v$, is different from that used in BD 44.

2. The rule often gives a lower strength than BD 44. This is because it covers flexure as well as shear, unlike that in BD 44. However, it is not valid for longer span to depth ratios. The amendments now define the limit of validity.

3. Although the position in the clause numbering system implies the rule also applies to cased beams, it appears it is only really meant for filler beam type bridges. In particular, the limiting span to depth ratio is defined in terms of spacing to web depth ratio which is unsafe for a bridge where the slab is shallower than the beams. It should therefore only be used for filler beam type bridges.

# APPENDIX A

## Extracts From Proposed Revisions to BD 61

## Annex A

### Note

1. In common with the published BD 61, all text which has been changed since BS 5400:Part 5 as modified by BD 16 is shown *in italics thus.*
2. In addition, text which has been changed since the published BD 61 is shaded thus. However, some additional minor changes to the text have been introduced, these may not have been shown as shaded.

The Assessor will need to compare the revised text contained in this Appendix with that of the published document to make himself aware of all the changes.

# 3 DEFINITIONS AND SYMBOLS

## 3.1 Definitions

For the purpose of this *Standard* the following definitions, and those given in *BS 5400* Part 1, apply.

**3.1.1** Cased composite beam. A beam composed of either rolled or built up *metal* sections, with a concrete encasement. *Normally the encased section is connected* to a concrete slab and the two elements are interconnected so as to form a composite section.

**3.1.2 Uncased composite beam.** A beam composed of either rolled or built-up *metal* sections,without a concrete encasement, which acts in conjunction with a concrete slab where the two elements are interconnected so as to form a composite section.

**3.1.3 Composite box beam.** A steel box girder acting compositely with a concrete slab.

NOTE: In a closed steel box the concrete is cast on the top steel flange, whereas in an open steel box the box is closed by the concrete slab.

**3.1.4 Composite column.** A column composed either of a hollow steel section with an infill of concrete or of a steel section cased in concrete so that in either case there is interaction between steel and concrete.

**3.1.5 Composite plate.** An in situ concrete slab cast upon, and acting compositely with, a structural steel plate.

**3.1.6 Concrete slab.** The structural concrete slab that forms part of the deck of the bridge and acts compositely with the steel beams. The slab may be of precast, cast in situ or composite construction.

**3.1.7 Composite slab.** An in situ concrete slab that acts compositely with structurally participating permanent formwork.

**3.1.8 Participating permanent formwork.** Formwork to in situ concrete, when the strength of the formwork is assumed to contribute to the strength of the composite slab.

**3.1.9 Non-participating permanent formwork.** Permanent formwork that *does*, or *does* not, act compositely with the in situ concrete, but where the formwork is neglected in calculating the strength of the slab.

***3.1.10 Filler beam*** deck. Rolled or built-up iron or steel sections that act in conjunction with a concrete slab and which are contained within the slab or *with slab surfaces flush with one or both flanges.*

***3.1.11 Cased beam*** deck. *Rolled or built-up* metal *sections, fully or partially encased in concrete, but not such that they are fully within the depth of the slab, such that composite action occurs.*

***3.1.12 Jack arch*** deck. *Rolled or built-up metal sections separated by concrete, stone or brick arches supported by the lower flanges, generally with loose fill or concrete fill above.*

**3.1.*13* Interaction**

**3.1.*13*.1 Complete interaction.** This implies that no significant slip occurs between the steel and the concrete slab or encasement.

**3.1.*13*.2 Partial interaction.** This implies that slip occurs at the interface between steel and concrete and a discontinuity in strain occurs *but that composite action is still capable of being generated.*

**3.1.*14* Shear connector.** A mechanical device to ensure interaction between concrete and steel.

**3.1.*15* Connector modulus.** The elastic shear stiffness of a shear connector.

**3.1.*16 Worst credible strength.*** *Worst credible strength at a location is the lower bound to the estimated strength. (see BD 44 for concrete and reinforcement).*

***3.1.17 Cross section redistribution class.*** *Criteria relating to the permitted redistribution of support moments.*

***3.1.18 Slip.*** *Movement of concrete along the steel/concrete interface.*

***3.1.19 Separation.*** *Movement of concrete perpendicular to the steel/concrete interface.*

**3.2 Symbols.**

The symbols used in this *Standard* are as follows:

*A* *Area of equivalent cracked transformed section*

$A_1$, $A_2$ *Projected areas of concrete resisting connector forces*

$A_b$ Cross-sectional area of transverse reinforcement in the bottom of the slab *effective in resisting bursting stresses in the concrete from the connector forces*

$A_{bs}$ Cross-sectional area of other transverse reinforcement in the bottom of the slab

$A_{bv}$ Cross-sectional area of additional transverse reinforcement

$A_c$ Cross-sectional area of concrete

$A_e$ Effective cross-sectional area of transverse reinforcement

$A_{ft}$ Cross-sectional area of top flange of steel section

$A_r$ Cross-sectional area of reinforcement

$A_s$ Cross-sectional area of steel section

*The value of the predicted mean resistance* $/\gamma_m\gamma_n$

*where* $\gamma_m$ *shall be replaced by the value of* $\gamma_m$ *calculated in accordance with Clause **4.3.3** of BD 56. For shear connectors in beams, the value of* $\gamma_m$ *so calculated shall be multiplied by an additional safety factor of 1.25 to allow for the brittle nature of failure along the shear connection.*

## 4.2 Material Properties

**4.2.1 General.** In analysing a structure to determine the load effects, the material properties associated with the unfactored characteristic, *or worst credible,* strength *shall* be used, irrespective of the limit state being considered. For analysis of sections, the appropriate value of the partial factor of safety $\gamma_m$, to be used in determining the design strength, *shall* be taken from *BD 56, BD 44 or below* depending on the materials and limit state. It should be noted that the stress limitations give in *BD 44* allow for $\gamma_m$. *The appropriate values of* $\gamma_m$ *are explicitly given in the expressions for assessment resistance in this Standard.*

*The values of* $\gamma_m$ *at the ultimate limit state are as given in table 4.1.*

***Table 4.1: Values of*** $\gamma_m$ ***at the ultimate limit state***

| *Structural component and behaviour* | $\gamma_m$ |
|---|---|
| *Shear connectors in isolation* | *1.10* |
| *Shear connectors in beam* | *1.375* |

*At the serviceability limit state,* $\gamma_m$ *for shear connectors in beams is replaced by* $\gamma_{slip} = 1.375$ *and a variable quantity given by **5.3.2.1** taking into account fatigue damage.*

**4.2.2 Structural steel *and iron.*** The characteristic, nominal *or worst credible* properties of structural steel *or iron shall* be determined in accordance with *BD 56 or BD 21 as appropriate.*

**4.2.3 Concrete, reinforcement and prestressing steels.** The characteristic *or worst credible* properties of concrete, reinforcement and prestressing steels *shall* be determined in accordance with *BD 44.* For sustained loading, it *is* sufficiently accurate to assume a modulus of elasticity of concrete equal to one half of the value used for short term loading.

## 4.3 Limit State Requirements.

**4.3.1 General.** *Except as specified in this Standard* all structural steelwork in composite beams *shall* be checked for compliance with the requirements of *BD 56* in relation to all limit states. *The* effects of creep, shrinkage and temperature *shall* be calculated in accordance with the recommendations of this *Standard,* for the relevant limit state.

The concrete and reinforcement in concrete slabs *shall* satisfy the limit state requirements of *BD 44* including the serviceability limit state stress limitations given in **4.1.1.3** of *BS 5400* Part 4 *as modified by **5.2.6.3** and **5.5** below. Deflection estimates may be disregarded unless specifically requested.* Where they are part of a composite beam section they *shall* also satisfy the limit state requirements of this *Standard.* The method of *assessing* crack widths at the serviceability limit state *shall* follow the recommendations of this *Standard.*

Shear connectors *shall* be *assessed* to meet the requirements of the serviceability limit state *and* the ultimate limit state given in this *Standard.*

Structural steelwork *shall* satisfy the fatigue requirements of *BS 5400* Part 10. *Reinforcement shall satisfy the fatigue requirements of BD 44.*

*When construction does not comply with the provisions of **5.3.3.3** and **6.3.3** composite action at the ultimate limit state shall be disregarded unless it can be shown to be effective at large deflections of the beam (see **6.1.3**).*

**4.3.2 Serviceability limit state.** A serviceability limit state is reached when any of the following conditions occur:

(a) The stress in *the* structural steel reaches $\sigma_{yc} / \gamma_m \gamma_{f3}$ *or* $\sigma_{yt} / \gamma_m \gamma_{f3}$, *where* $\sigma_{yc}$ *and* $\sigma_{yt}$ *are defined in BD 56. See also **5.2.1** below.*

*b) The stress in cast iron reaches the limits in BD21 with the values of $\gamma_{fl}$ for dead and superimposed load as in BD21 and*

*for ALL (Assessment Live Load) $\gamma_{fl} = 1.00$*
*for HB and associated 40^T ALL $\gamma_{fl} = 0.92$*
*for HB alone $\gamma_{fl} = 0.92$*

*Note: 0.92 is the ratio of the $\gamma_{fl}$ values in BD37 for the respective loading cases or 1.1/1.2.*

(c) The stress in concrete reaches the appropriate limit given in *BD 44* or the stress in the reinforcement reaches $0.80 f_{ry} / \gamma_{mr} \gamma_{f3}$. *See also **5.2.1** below.*

(d) The width of a crack in concrete, *assessed* in accordance with *Appendix A* reaches the appropriate limit given in *BS 5400* Part 4 *as modified by 5.2.6.3 below.*

(e) The vibration in a structure supporting a footway or cycle track reaches the appropriate limit given in *BD 37. See Advice Note for procedure.*

(f) The slip at the interface between steel concrete becomes excessive.

NOTE 1: In *deriving the rules* slip *has been* assumed to occur when the calculated load on a shear connector exceeds *0.55* times its nominal *initial mean* static strength *when the risk from fatigue is high and at 0.60 times its nominal initial mean static strength when the risk from fatigue is low. This criterion is implicitly taken into account in the safety factors and in the allowance for fatigue.*

*NOTE 2: There are no SLS stress limits for wrought iron in BD21.*

**4.3.3 Ultimate limit state.** General requirements for composite structures at the ultimate limit state are as given in *BS 5400* Part 1.

## 5.2 Analysis of Sections

**5.2.1 General.** The stresses in composite sections *shall* be determined in accordance with **5.2.2** to **5.2.5**. *When no moment is redistributed at the ultimate limit state, at cross sections assessed elastically at that limit state no stress checks are required at the serviceability limit state.* Crack widths *shall* be *assessed* in accordance with **5.2.6** *if so required.*

**5.2.2 Analysis.** Stresses due to bending moments and vertical shear forces *shall* be calculated by elastic theory using the appropriate elastic properties given in **4.2** and effective breadths as given in **5.2.3**, assuming that there is full interaction between the steel beam and the concrete in compression.

When *it is likely that* the cross section of a beam and the applied loading increase*d* by stages *and the actual construction sequence is unknown, worst credible construction sequences shall* be *assessed (see Advice Note) and agreed. These shall be assumed in assessing* the adequacy of the final condition. The bending stresses *shall* not exceed the appropriate limits given in **6.2.3** using the appropriate values of $\gamma_m$ and $\gamma_{f3}$ for the serviceability limit state except that the limiting tensile stress in the reinforcement *shall* be replaced by

$$0.80 f_{ry} / \gamma_{mr} \gamma_{f3}$$

### 5.2.3 Effective breadth of concrete flange

**5.2.3.1 General.** In calculating the stresses in a concrete flange, and in the absence of rigorous analysis, the effect of in-plane shear flexibility (ie shear lag) *shall* be allowed for by assuming an effective breadth of flange in accordance with **5.2.3.2**, **5.2.3.3** and *BD 56, except that for b/L values less than 0.05, a* $\psi$ *value of 1.0 may be assumed.*

**5.2.3.2 Effective cracked flange.** For a concrete flange in tension (which is assumed to be cracked), the effective breadth ratio $\psi$ *shall* be *replaced by the effective cracked flange factor, which is:*

$$(2\psi + 1)/3 \qquad (5.1)$$

where $\psi$ is the effective breadth ratio *for the uncracked concrete flange.*

**5.2.3.3 Width over which slab reinforcement is effective.** Only reinforcement within the effective breadth of the concrete slab *shall* be assumed to be effective in analysing cross sections. *The effective area of longitudinal reinforcement shall be taken as* $\Sigma(A_r \cos^4 \alpha_l)$, *where* $\alpha_l$ *is the angle between the bars and the web of the steel beam. When the reinforcement assumed to be at its design strength in tension produces a net transverse force on the steel beam this force shall be taken into account in the assessment or the effective areas adopted such that there is no net transverse force.*

### 5.2.4 Deck slabs forming flanges of composite beams

**5.2.4.1 Effects to be considered.** The slab *shall* be designed to resist:

(a) the effects of loading acting directly on it, and

### 5.3.2.4 Tests on shear connectors.

(a) **Nominal *initial mean static* strength.** The nominal *initial mean* static strength of a shear connector may be determined by push out tests. No fewer than three tests *are to be* made and the nominal *initial mean* static strength $P_{im}$ *is* taken as the lowest value of $f_{cu}P/f_c$ for any of the tests, where P is the failure load of the connectors at concrete strength $f_c$, and $f_{cu}$ *is the lower of* the specified characteristic *or worst credible* cube strength at 28 days. *For five or more tests the mean value is taken.* *Sometimes it may be permitted to justify a strength by calculation, when the effects of bending in the shear connector shall be included.*

(b) **Details of tests.** Suitable dimensions for the push-out specimens are given in figure *5.3*. Bond at the interfaces of the flanges of the steel beam and the concrete *shall* be prevented by greasing the flange or by other suitable means. The slab and reinforcement *shall* be either as given in figure *5.3* or as in the beams for which the test is designed.

The strength of the concrete $f_c$, at the time of testing, *shall* not differ from the specified *or worst credible* cube strength $f_{cu}$ of the concrete in the beams by more than ± 20%. The rate of application of load *shall* be uniform and such that failure is reached in not less than 10 minutes.

**Resistance to separation.** Where the connector is composed of two separate elements, one to resist longitudinal shear and the other to resist forces tending to separate the slab from the girder, the ties which resist the forces of separation may be assumed to be sufficiently stiff and strong if the separation measured in push-out tests does not exceed half of the longitudinal slip at the corresponding load level. Only load levels up to 80% of the nominal *initial mean* static strength of the connector need be considered.

5.3.3.4 *Assessment* **procedure: general.** Shear connectors *shall* be *assessed* at the serviceability limit state in accordance with **5.3.3.5**. *and* for fatigue in accordance with *BS 5400* Part 10 *as modified by **5.3.2.1**.*

Shear connectors *shall* be checked for static strength at the ultimate limit state *as* required by **5.3.3.6** or **6.1.3**, or when redistribution of stresses from the tension flange *or web panels* has been made in accordance with *BD 56.*

***5.3.3.5 Assessment resistance of shear connectors****. The assessment longitudinal shear resistance of shear connectors $P_a$ is:*

$$P_a = P_{am} / \gamma_{slip} \gamma_{f_3} \qquad (5.3)$$

*where $P_{am}$ is the nominal present mean static strength at assessment (**5.3.2.1**) and $\gamma_{slip}$ for shear connectors is given in 4.2.1.*

*5.3.3.6 Shear connector spacing and longitudinal shear resistance.*

*(1) Connector spacings not greater than 1000mm nor span/20*

*The size and spacing of the connectors at each end of each span under the maximum loading considered shall be such that the maximum longitudinal shear force per unit length q does not exceed the assessment longitudinal shear resistance $q_r$ per unit length by more than the margins in table 5.2, in which the fatigue vulnerability adopted shall be agreed. The size and spacing of connectors required shall extend:-*

*10% of the length of the span for $q/q_r \leq 1.1$*

*20% of the length of the span for $1.1 < q/q_r \leq 1.25$*

*33% of the length of the span for $q/q_r > 1.25$*

*but not greater than 5m.*

*Elsewhere the size and longitudinal spacing of connectors present may be constant over any length over which the total assessment shear force does not exceed the product of the number of connectors and the assessment static strength per connector as defined in **5.3.2.5**, provided the maximum shear force per unit length does not exceed the assessment shear resistance per unit length by more than the margin in table 5.2.*

*Where the connector spacing satisfies the above requirements except over regions not exceeding span/8 where the shear connectors do not comply with **5.3.3.7** then the shear connectors over this region shall comply with **5.3.3.8** and $q/q_r$ locally shall not exceed unity.*

*(2) Connector spacing exceeding 1000mm or span/20, but less than span/8.*

*The size and spacing of the connectors under the maximum loading considered shall be such that the maximum longitudinal shear force per unit length q does not exceed the*

# 6 *ASSESSMENT* OF SUPERSTRUCTURE FOR THE ULTIMATE LIMIT STATE

## 6.1 Analysis of Structure

**6.1.1 General.** Except where alternative methods are given in **6.1.2** elastic analysis *shall* be used to determine the distribution of bending moments, shear forces and axial loads due to the design ultimate *traffic* loadings specified in *BD 21, but with the load combinations in BD 37.* The use of alternative methods *shall* be in accordance with **8.2** of *BS 5400* Part 1 and *shall* only be undertaken where they can be shown to model adequately the combined effects of local and global loads due to combinations 1 - 5 *as given in BD37. Wrought iron and cast iron structures shall be assessed elastically as for steel using the material properties and strengths in BD21.*

**6.1.2**

**Deck slabs forming the flanges of composite beams.** The deck slab *shall* be *assessed* to resist separately the effects of loading given in **5.2.4.1**, but *assessment* loads relevant to the ultimate limit state *shall* be used. In general, the effects of local wheel loading on the slab *shall* be determined by elastic analysis. Alternatively, an inelastic method of analysis, e.g. yield line theory, *is permissible* where an appropriate solution exists subject to the requirements in **6.1.1**.

The resistance to global effects *shall* be determined in accordance with **6.2**. For local effects the assessment of the slab cross section *shall* be in accordance with *BD 44*. The combined effects of global bending and local wheel loading *shall* be taken into account in accordance with *BD 44*.

Proper account *shall* be taken of the interaction between longitudinal shear forces and transverse bending of the slab in the region of the shear connection. The methods given in **6.3** may be deemed to satisfy these recommendations.

**6.1.3 Composite action.** Where, for a beam built in stages, the entire load is assumed to act on the final cross section in accordance with **9.9.5** of *BD 56*, or where tensile stresses are redistributed from *the web or* the tension flange in accordance with ***9.5.4 or* 9.5.5** *respectively* of *BD 56*, the shear connectors and transverse reinforcement *shall* be assessed for the corresponding longitudinal shear in accordance with **6.3**.

*Composite action from shear connectors not complying with 5.3.3.3 shall be disregarded at the ultimate limit state, except:*

*(i) where the connectors can be shown to be strong enough to resist the bending including any additional bending caused by the calculated separation gap when lift-off occurs or*

*(ii) in checking for lateral-torsional buckling in accordance with Appendix C, or*

*(iii) when otherwise agreed.*

*Where (i) applies the contact area between the connector and the concrete shall be adjusted for any separation. When the construction cannot be justified by this procedure consideration shall be given to providing nominal ties to ensure the integrity of the construction generally, and particularly in sagging moment regions in the vicinity of hinges or the greatest sagging moment curvatures (see Advice Note).*

**6.2.2 Bending resistance of compact sections.** For a beam that is of compact section at the stage under consideration *with longitudinal shear connections satisfying **5.3.3.1** to **5.3.3.7**, **6.3.3** and **6.3.4*** the bending resistance *shall* be determined in accordance with **9.9.1.2** of *BD 56* assuming that the entire load acts on the cross section of the beam. The plastic modulus $Z_{pe}$ *shall* include the transformed area of the concrete in compression which *shall* be obtained from:

$$\text{The gross area of the concrete x } \frac{0.67\, f_{cu} / \gamma_{mc}}{\sigma_{yc} / \gamma_m}$$

where

$f_{cu}$ is the characteristic *or worst credible* concrete cube strength.

$\sigma_{yc}$ is the nominal *or worst credible* yield stress of the steel compression flange as defined in *BD 56*

$\gamma_m$ is the partial material factor for steel in accordance with *BD 56.*

$\gamma_{mc}$ *is the partial material factor for concrete in compression in accordance with BD 44.*

Concrete in tension *shall* be ignored but the transformed area of the reinforcement in concrete subject to tension *shall* be included and *shall* be obtained from:

$$\text{The gross area of reinforcement x } \frac{f_{ry} / \gamma_{mr}}{\sigma_{yc} / \gamma_m}$$

where

$f_{ry}$ is the characteristic or *worst credible* yield strength of the reinforcement.

$\gamma_{mr}$ *is the partial material factor for reinforcement in accordance with BD 44*

*Compact cross sections with shear connectors in accordance with **5.3.3.8** shall be designed for lateral torsional buckling according to **6.2.3.1(4)**.*

**6.2.3 Bending resistance of non-compact sections.**

***6.2.3.1 General.***

*(1) A steel flange that is attached to a concrete or composite slab by shear connection in accordance with **5.3.3.3** and **6.3.3** is assumed to be laterally stable, provided that the overall width of the slab is not less than the depth of the steel member.*

*(2) All other steel flanges in compression shall be checked for lateral stability.*

*(3) A calculation procedure for assessing lateral torsional buckling at the supports of composite beams is given in Appendix B.*

*(4) The calculation procedure for assessing the lateral-torsional buckling effect of composite beams with incidental shear connectors assuming U-frame action is inadmissible, but a calculation procedure assuming lateral restraint to the top flange is permitted. Suitable procedures for this type of restraint are given in Appendix C (this is developed from Appendix G of BS 5950 Part 1).*

*(5) Where the modulus of elasticity of the concrete has been reduced in accordance with* ***5.3.3.9*** *this should be taken into account in assessing the bending resistance.*

*(6) Lateral restraints to compression flanges not in contact with the concrete slab shall be assessed in accordance with Clause* ***9.12.1*** *of BD 56.*

***6.2.3.2 Bending resistance of non-compact sections with shear connectors satisfying 5.3.3.***
*Beams which have non-compact cross sections at supports shall satisfy the following rules.*
For a beam that is not of compact section at the stage under consideration the stresses *shall* be calculated at each stage of construction, using the appropriate loading and section properties based on transformed elastic section moduli. The transformed area of the concrete compression flange *shall* be obtained using either the short term or the long term modular ratio, as appropriate to the type of loading. Concrete in tension *shall* be ignored but the area of the reinforcement in concrete subject to tension *shall* be included. At the appropriate extreme fibres, the sum of these stresses at any stage *shall* not exceed:

*(a)* $\dfrac{\sigma_{lc}}{\gamma_m \gamma_{f3}}$ for steel compression flange

*(b)* $\dfrac{\sigma_{yt}}{\gamma_m \gamma_{f3}}$ for steel tension flange

*(c)* $\dfrac{0.75 f_{cu}}{\gamma_{mc} \gamma_{f3}}$ for concrete compression flange

(d) $\dfrac{f_{ry}}{\gamma_{mr} \gamma_{f3}}$ for reinforcement in tension

where

$\sigma_{lc}$ and $\sigma_{yt}$ are as defined in **9.9.1.3** of *BD 56* appropriate to the cross section at the stage under consideration, *where $\sigma_{lc}$ is determined from BD 56 using $\lambda_{LT}$*

*derived from Appendix B for beams without discrete U-frames. For beams satisfying* ***6.2.3.3****, $\sigma_{lc}$ shall be taken as $\sigma_{yc}$ in BD 56. For beams with discrete U-frames, $\sigma_{lc}$ shall be determined from BD 56.*

$f_{cu}$ is the characteristic *or worst credible* concrete cube strength

$f_{ry}$ is the characteristic *or worst credible* yield strength of the reinforcement

$\gamma_{f3}$ is the partial safety factor in accordance with *4.1.2 and BD 56.*

$f_{ry}$ is the characteristic *or worst credible* yield strength of the transverse reinforcement but not greater than 460 N/mm$^2$;

$f_{cu}$ is the characteristic cube strength of concrete *or worst credible* cube strength used in the assessment of the slab but not greater than 45 N/mm$^2$;

s is a constant stress of 1 N/mm$^2$ re-expressed where necessary in units consistent with those used for the other quantities.

(d) *When the spacing of shear connectors does not exceed 1000mm or span/20 the size and spacing of transverse reinforcement shall follow the recommendations relating the shear flow and the longitudinal shear resistance in* ***5.3.3.6(1)****. When this connector spacing is exceeded the connector force shall be assumed to be resisted over a distance equal to the lesser of 600mm, or three times the thickness of the slab on the compression side of the shear connectors. The total area of transverse reinforcement required in this zone to resist the local shear connector forces shall not be less than*

*(i) the calculated area in* ***6.3.3*** *if the conditions in* ***6.3.3.8*** *are not satisfied, or alternatively*

*(ii) half the calculated area in* ***6.3.3*** *if the conditions in* ***6.3.3.8*** *are satisfied.*

**6.3.3.2 Longitudinal shear.** The longitudinal shear force per unit length $q_p$ on any shear plane through the concrete *shall* not exceed the lesser of the following:

(a) $k_l f_{cu} L_s / \gamma_{mv} \gamma_{f3}$ *(6.4)*

(b) $v_l L_s / \gamma_{mv} \gamma_{f3} + 0.80\, A_e f_{ry} / \gamma_{mr} \gamma_{f3}$ *(6.5)*

where

$k_l$ is a constant equal to 0.23 for normal density concrete and 0.18 for lightweight aggregate concrete.

$v_l$ is the ultimate longitudinal shear stress in the concrete for the shear plane under consideration, to be taken as *1.35* N/mm$^2$ for normal density concrete and *1.05*N/mm$^2$ for lightweight aggregate concrete.

$\gamma_{mv}$ is *1.50 but may be reduced to 1.25 when the characteristic strength is ≥ 45 N/mm$^2$ or the worst credible strength ≥35N/mm$^2$.*

If $f_{cu}$ is taken to be less than 20 N/mm$^2$, the term $v_1L_s$ in (b) *shall* be replaced by $k_2f_{cu}L_s$ where $k_2$ is a constant equal to 0.060 for normal density concrete and 0.045 for lightweight aggregate concrete.

In haunched beams, not less than half the reinforcement required to satisfy (b) above in respect of shear planes through the haunch (planes 3-3 and 4-4 in figure *6.2*) *shall* be bottom reinforcement that complies with the definition of $A_{bv}$ in **6.3.3.1(b)**.

### 6.3.3.3 Interaction between longitudinal shear and transverse bending.

*(a)* ***Beams with transverse compression around shear connectors.*** Where the *assessment* loading at the ultimate limit state causes transverse compression in the region of the shear connectors, no account need be taken *of interaction between longitudinal shear and transverse bending* providing the recommendations of **6.3.3.2** are satisfied.

*(b)* **Beams with shear planes passing through the full depth of slab.** Where the shear plane passes through the full depth of the slab, no account need be taken of the interaction between longitudinal shear and transverse bending.

*(c)* **Unhaunched beams with shear planes passing round the connectors.** In unhaunched beams where the *assessment* loading at the ultimate limit state causes transverse tension in the slab in the region of the shear connectors, account *shall* be taken of the effect of this on the strength of shear planes that do not cross the whole depth of the slab (plane 2-2 in figure *6.2*) by replacing **6.3.3.2(b)** by

$$q_p \le v_1 L_s / \gamma_{mv} \gamma_{f3} + 1.60 A_{bv} f_{ry} / \gamma_{mr} \gamma_{f3} \qquad (6.6)$$

Where the *assessment* loads at the ultimate limit state can cause transverse compression in the slab in the region of the shear connectors account may be taken of the beneficial effect of this on the strength of shear planes that do not cross the whole depth of the slab (shear plane type 2-2 in figure *6.2*) by replacing **6.3.3.2(b)** by

$$q_p \le v_1 L_s / \gamma_{mv} \gamma_{f3} + 1.80 A_{bv} f_{ry} / \gamma_{mr} \gamma_{f3} + 1.60 F_T / \gamma_{f3} \qquad (6.7)$$

where

$F_T$ is the minimum tensile force per unit length of beam in the transverse reinforcement in the top of the slab due to transverse bending of the slab. Only loading that is of a permanent nature *shall* be considered when calculating $F_T$.

NOTE: For remaining symbols see **6.3.3.1**(a), (b) and (c).

*(d)* **Haunched beams.** In haunched beams, where the *assessment* loading at the ultimate limit state causes transverse tension in the slab in the vicinity of the shear connectors, no account of this need be taken, provided the reinforcement required to satisfy **6.3.3.3(a)** is reinforcement that satisfies the definition of $A_{bv}$ and the haunch dimensions satisfy the recommendations of **6.3.2.1**.

**6.3.3.4 Minimum transverse reinforcement.** The cross sectional area, per unit length of beam, of reinforcement in the slab transverse to the steel *or iron* beam *shall* be not less than

$$0.70\gamma_{mr}sh_c/f_{ry}$$

where

$h_c$ is the thickness of the concrete slab forming the flange of the composite beam.

Not less than 50% of this area of reinforcement *is required near the* bottom of the slab so that it satisfies the definition of $A_{bv}$ given in **6.3.3.1(b)**.

Where the length of a possible plane of shear failure around the connectors (shear plane 2-2 in figure *6.2*) is less than or equal to twice the thickness of the slab $h_c$, reinforcement in addition to that required for flexure *is required* in the bottom of the slab transverse to the steel *or iron* beam to prevent longitudinal splitting around the connectors. The cross sectional area of this additional reinforcement, per unit length of beam, $A_{bv}$ *shall* be not less than $0.70\gamma_{mr}sh_c/f_{ry}$. This additional reinforcement *is not required* if the minimum compressive force per unit length of beam, acting normal to and over the surface of the shear plane, is greater than 1.4 $sh_c$.

**6.3.3.5 Minimum transverse reinforcement in haunched beams.** The cross-sectional area of transverse reinforcement in a haunch per unit length of beam $A_{bv}$ as defined in **6.3.3.1(b)** *shall* not be less than

$$0.35\gamma_{mr}sL_s/f_{ry}$$

where

$L_s$ is the length of a possible plane of shear failure around the connectors (see shear plane type 3-3 or 4-4 in figure *6.2*).

**6.3.3.6 Curtailment of transverse reinforcement.** The transverse reinforcement provided to resist longitudinal shear *which is* curtailed *is acceptable* provided the *conditions* **6.3.3** are satisfied in all respects for the shear planes through the slab of type 1-1 in figure *6.2*. For this purpose the longitudinal shear force per unit, length $q_p$ for such a plane, *shall be* assumed to vary linearly from the calculated maximum force on the relevant plane, which is adjacent to the shear connectors, to zero mid-way between the centre line of the beam and that of an adjacent beam or to zero at an adjacent free edge.

**6.3.3.7 Detailing of transverse reinforcement.** The spacing of bottom transverse reinforcement bars, if *present and satisfying the conditions in* **6.3.3**, *shall* be not greater than four times the projection of the connectors (including any hoop which is an integral part of the connector) above the bars *nor* greater than 600mm.

**6.3.4 Shear Connectors.** The design of the shear connectors need not be considered at the ultimate state except as directed in **5.3.3.6**, **6.1.3** or where redistribution of stresses from *the web or* the tension flange is carried out in accordance with *BD 56*. Then the size and spacing of shear connectors *shall* be determined in accordance with **5.3.3.5** except that longitudinal shear per unit length *shall* be determined in accordance with **6.3.1** and the *assessed* static strength, per connector at the ultimate limit state, *shall* be taken as

$$P_{am} / \gamma_m \gamma_{f3}$$

where

$P_{am}$ is the nominal *present mean static* strength as defined in **5.3.2.1** or **5.3.2.2**, *but the 0.82 limit in the equation of 5.3.2.1(b) shall be disregarded.*

# 8 CASED BEAMS AND FILLER BEAM CONSTRUCTION

## 8.1 Scope

This Clause applies to simply supported filler beam decks, with or without the soffit *or the upper surface of the flanges* of the steel or *iron* member exposed, and to simply supported or continuous cased beams. The recommendations apply only where the encasement or filling is of normal density concrete (2300 kg/m³ or greater) *and the characteristic or worst credible strength of the concrete is not less than 25N/mm²*. The *Advice Note includes for use when agreed a method for assessing partially encased beams and an alternative method of assessing filler beams, which is generally more economical than the method described below and which can be used with lower strength concrete. In calculations the characteristic or worst credible strength shall not be taken in excess of 40N/mm².*

## 8.2 Limit State Requirements

Except where special requirements are given in the following clauses, cased beams and filler beam decks *shall* be assessed for the serviceability and ultimate limit states in accordance with Clauses **4**, **5** and **6**. *Construction with cast iron beams shall be assessed only at the serviceability limit state as specified in 4.3.2(b) and in this chapter. The properties of cast iron and wrought-iron shall be as specified in BD21.*

## 8.3 Analysis of Structure

### *8.3.1 General*

The distribution of bending moments and vertical shear forces, due to the *assessment* loadings at the serviceability and ultimate limit states, *shall* be determined by an elastic analysis in accordance with **5.1** and **6.1**, using an orthotropic plate or grillage analysis. Redistribution of moments at the ultimate limit state (see **6.1.4.2**) *is* not permitted in cased beams.

In simply supported filler beam decks transverse bending moments may be determined by the method given in **8.3.2**.

*Where there are no bearings the effective span shall be taken as the clear span plus, at each end, the least of:-*

- *half the depth of the metal section.*
- *the depth from the soffit to the elastic neutral axis of the metal section.*
- *half the projection of the metal beam past the face of the support.*

**8.3.2 Transverse moments in filler beam decks (approximate method).** This method is applicable to filler beam decks subject to the full *nominal assessment live loading (the UDL and KEL)* and/or up to 45 units of type HB loading where the following conditions are satisfied:

(a) the construction consists of simply supported *metal* beams solidly encased in normal density concrete;

(b) the span in the direction of the beams is not less than 6m and not greater than 18m and the angle of skew does not exceed 20°;

(c) the clear spacing between the tips of the flanges of the *metal* beams does not exceed two-thirds of their depth;

(d) the overall breadth of the deck does not exceed 14m;

(e) the amount of transverse reinforcement in the top of the slab is not less than $300mm^2/m$ if mild steel is used or $200mm^2/m$ if high yield steel is used.

The maximum transverse sagging moment per unit length of deck. $M_y$ due to either HA or HB loading, at any point not less than 2m from a free edge, is

$$M_y = (0.95 - 0.04\,l)\,M_x\,\alpha_L \qquad (8.1)$$

where

$M_x$ is the longitudinal bending moment per unit width of deck at the point considered due to the *full nominal assessment live* loading for the limit state considered

l is the span of the beams in metres

$\alpha_L$ is the ratio of the product of the partial safety factors $\gamma_{fL}\gamma_{f3}$ for the HB loading to the corresponding product for the full nominal *assessment live* loading for the limit state being considered.

Longitudinal bending moments per unit width of deck due to the full *assessment* loading are found by analysis of the deck as a set of separate longitudinal strips each of width not exceeding the width of one traffic lane.

It *is* assumed that there is a linear reduction in $M_y$ from the value at 2m from the free edge of the deck to zero at the edge.

The transverse hogging moment at any point may be taken as 0.1 $M_y$ per unit length of deck.

## 8.4 Analysis of Sections

The moments of resistance of cased and filler beams *shall* be assessed in accordance with **5.2** and **6.2** at the serviceability and ultimate limit states respectively. For this purpose a beam *shall* be considered as compact *(class 2 in Table 6.1)* provided any part of the steel *or iron* section not encased in concrete satisfies the criteria given in *BD 56 (DMBR 3.4.11) and BD21 (this assumes BD21 will be modified appropriately for buckling), except that any wrought iron or steel beam next to a cast iron beam shall be assessed as a class 3 cross-section and the moment of resistance of a cased beam with a slab width exceeding 1.5 times the depth of the metal section shall be restricted to 60% of the plastic moment of resistance of the metal section plus 40% of the plastic moment of resistance of the composite section*. Vertical shear *shall* be assumed to be resisted by the steel *or iron* section alone and the effects of shear lag in filler beam decks *shall be neglected. The stresses in cast iron shall not exceed the limits in Clause 4.3.2(b).*

## 8.5 Longitudinal Shear

**8.5.1 Serviceability limit state.** The longitudinal shear force per unit length between the concrete and steel beam *shall* be calculated by elastic theory, in accordance with **5.3.1** except that, in positive (sagging) moment regions of cased beams and in filler beams, concrete in tension *shall* be neglected. Shear lag effects shall be neglected in filler beam decks. The shear force to be transferred *shall* be that appropriate to the area of concrete and steel reinforcement in compression.

For highway bridges and footbridges, *providing there is no evidence of corrosion, fretting action or cracking sufficient to adversely affect the achievement of composite action*, the longitudinal shear force may be assumed to be resisted by bond between the steel *or iron* and concrete provided the local bond stress nowhere exceeds 0.5 N/mm$^2$ in cased beams or 0.7N/mm$^2$ in filler beams. The bond may be assumed to be developed uniformly only over both sides of the web and the upper surface of the top and bottom flanges of the steel beam where there is complete encasement and over both sides of the web and the upper surface of the top flange of the steel beam where the beam soffit is exposed. *Where both flanges of a filler beam are exposed, the bond may be assumed to be developed uniformly over both sides of the web provided that the filler beams are adequately tied together e.g. by reinforcement passing through the webs or through tie bars.* Where the local bond stress, calculated in the manner described, exceeds 0.5 N/mm$^2$ in cased beams or 0.7 N/mm$^2$ in filler beams the bond *shall* be ignored entirely.

*Where there are attachments present satisfying the requirements for incidental longitudinal shear connectors in **5.3.3.8.1** the resistance of these may be assumed in addition to the bond, but because of the (elastic) local bond criterion this is permitted only where connectors are well distributed.*

**8.5.2 Ultimate limit state.** The longitudinal shear force per unit length of beam *shall* be calculated in accordance with **8.5.1**, but for the *assessment* loading at the ultimate limit state. *Where there are no shear connectors* to transmit the longitudinal shear force due to vertical loading (see **8.5.1**), particular attention *shall* be given to shear planes of type 5-5 (figure 6.2(d)). *In the assessment of filler beams in which the beam spacing exceeds that in 8.3.2(c) possible failure planes through the concrete shall be examined, including planes partially on the steel/concrete interface, when the part on the interface shall be taken as β of that through the concrete.* The total cross sectional area per unit length of beam of fully anchored reinforcement intersecting the shear surface $A_e$ *shall* not be less than

$$\frac{\gamma_{mr}\left(q_p\gamma_3 - \nu_1 L_s / \gamma_{mv}\right)}{0.8 f_{ry}} \qquad (8.2)$$

where

$q_p$ is the longitudinal shear force per unit length at the ultimate limit state acting on that shear plane.

$L_s$ is the total length of shear plane minus one third $b_f$.

$\beta$ is the strength of interface relative to the concrete, taken as 0.67

*NOTE: The remaining terms are as defined in* ***6.3.3****.*

*In the assessment of the effects of incidental shear connectors at ULS isolated connectors within span/5 of the supports only may be considered and separation gaps (6.1.3) shall be disregarded.*

## 8.6 Temperature and Shrinkage Effects

**8.6.1 General.** Temperature and shrinkage effects need not be considered in filler beam construction. In cased beams, other than filler beams, consideration *shall* be given to the effects of temperature and shrinkage at the serviceability limit state. In the absence of more precise information the effects of temperature in cased beams *shall* be determined using the temperature effects given in *BD 37* for a similar reinforced concrete structure. The effects of shrinkage as modified by creep *shall* be assessed using the values of free shrinkage strain $E_{cs}$ and the reduction factor for creep $\phi_c$ as given in **5.4.3**.

**8.6.2 Longitudinal stresses and strains.** Longitudinal stresses and strains due to temperature effects and shrinkage modified by creep *shall* be calculated in accordance with **5.4.2** and **5.4.3**.

**8.6.3 Longitudinal shear.** *There shall* be shear connectors at the ends of cased beams, to transmit the longitudinal shear force Q, due to temperature effects and shrinkage modified by creep as described in **5.4.2.3** and **5.4.3**. The longitudinal shear force to be transmitted by the connectors *shall* be the net longitudinal force in the steel *or iron* beam due to temperature and shrinkage effects calculated on an elastic basis assuming full interaction. It may be assumed to be distributed at the ends of the beam in the manner described in **5.4.2.3**. The concrete *shall* be assumed to be uncracked. The effective breadth of the concrete flange *shall* be determined in accordance with **5.4.2.1**.

## 8.7 *Assessment* of Cracking

**8.7.1 General.** The methods given in **5.2.6**, *supplemented by the provisions in* **8.7.2** and **8.7.3**, may be used to *assess whether the degree of cracking* is not excessive at the serviceability limit state. Tensile reinforcement, *satisfying* the *provisions* of this Clause, may be assumed to contribute to the section properties of the composite beam.

**8.7.2 Cased beams.** Longitudinal bars placed in the side face of beams to control flexural cracking *shall* be of a diameter Φ such that:

$$\Phi / \sqrt{\frac{s_b s b}{f_{ry}}} \qquad (8.3)$$

where

$s_b$ is the spacing of bars in the side face of the beam.

b is the breadth of the section at the point where the crack width is being considered.

s is a constant stress of $\bar{1}$ N/mm$^2$, re-expressed where necessary in units consistent with those used for other quantities.

$f_{ry}$ is the characteristic yield stress of the reinforcement.

Where the overall depth of a cased beam exceeds 750mm *there shall be* longitudinal bars at 250mm spacing or closer in the side faces of the beam over a distance of two-thirds of the overall depth measured from the tension face, unless the *assessment* of crack widths (see **5.2.6**) shows that a greater spacing is acceptable.

**8.7.3 Filler beams.** The widths of cracks due to transverse bending of a filler beam deck *shall* be *assessed* in accordance with *BS 5400* Part 4, as for a reinforced concrete slab, neglecting any contribution from the steel beams to the control of cracking.

**8.8 Construction**

*The concrete cover to the metal beam on a surface assumed to transmit lonitudinal shear shall be nowhere less than 50mm.*

The soffit and upper surface of exposed flanges of filler beams *shall have been* protected against corrosion.

In cased beams, other than filler beams, *there shall* be stirrups formed by reinforcing bars *enclosing* the steel *or iron* beam and *longitudinal* reinforcement for control of cracking of the beam encasement. The spacing of *the* stirrups *shall* not exceed 600mm. The total cross-sectional area of stirrups (both legs) crossing a possible plane of shear failure of type 5-5 (figure 6.2(d)) *shall* be not less than

$$0.70\ \gamma_{mr}\ s\ L_s / f_{ry} \text{ per unit length of beam} \qquad (8.4)$$

where

$L_s$ is as defined in figure 6(d).

s is defined in **6.3.3.1**.

NOTE: Alternatively, mesh of equivalent area may be used.

Concrete cover to reinforcement *shall* be in accordance with the recommendations of *BD 44*.

# 11 COMPOSITE COLUMNS

## 11.1 General

**11.1.1 Scope.** This Clause gives *an assessment* method for concrete encased steel *or wrought iron* sections and concrete filled circular and rectangular hollow steel sections which takes account of the composite action between the various elements forming the cross section. *For axisymmetric columns, moments shall be resolved into the principal directions. For columns which are not axisymmetic* bending about the two principal axes of the column is considered separately for each axis. A method is given in **11.3.5.5** for determining the effect of interaction when bending about both axes occurs simultaneously. The column may be either statically determinate or rigidly connected to other members at one or both ends, in which case the loads and moments depend on the relative stiffnesses of adjoining members and cannot be obtained by statics alone. Members *are* assumed to be rigidly connected where, for example, the connection possesses the full rigidity that can be made possible by welding or by the use of high strength friction grip bolts.

*Where construction does not satisfy the requirements of the assessment methods of this Clause the method of BS 5400 Part 5 or a cased strut method, such as that in BS 5950 Part 1, may be employed with the agreement of the Overseeing Organisation.*

### 11.1.2 Materials

**11.1.2.1 Steel *or wrought iron.*** In columns formed from concrete encased steel or iron sections the structural steel *or iron* section *used in the assessment shall be one of the following:-*

(a) a rolled steel joist or universal section of grade 43 or 50 steel which complies with the requirements of BS 4: Part 1 and BS 4360; or

(b) a symmetrical I-section fabricated from grade 43 or 50 steel complying with BS 4360.

*(c) a symmetrical I-section of which the properties are taken from information on the drawings or in BD 21.*

Concrete filled hollow steel *or iron* sections *used in the assessment* may be either rectangular or circular and *shall*:

(1) be a symmetrical box section fabricated from grade 43 or 50 steel complying with BS 4360 *or iron complying with BD 21*; or

(2) *a structural hollow steel section complying* with BS 4360 and BS4: Part 2 or BS 4848: Part 2 as appropriate *or wrought iron complying with BD 21;*and

(3) have a wall thickness of not less than:

$b_s\sqrt{f_y/3E_s}$ for each wall in a rectangular section (RHS), or

$D_e\sqrt{f_y/8E_s}$ for circular hollow section (CHS) where

$b_s$ is the external dimension of the wall of the RHS

$D_e$ is the outside diameter of t he CHS

$E_s$ is the modulus of elasticity of steel *or wrought iron*

$f_y$ is the nominal yield strength of steel *or wrought iron*

The surface of the steel member in contact with the concrete filling or encasement *shall* be unpainted and free from deposits of oil, grease and loose scale or rust.

**11.1.2.2 Concrete.** The concrete *shall* be of normal density (not less than 2300 kg/m$^3$) with a characteristic 28 day cube strength *or lowest credible* strength of not less than 20 N/mm$^2$ for concrete filled tubes nor less than 25 N/mm$^2$ for concrete encased sections and a nominal maximum size of aggregate not exceeding 20mm.

**11.1.2.3 Reinforcement.** Steel reinforcement *shall* comply with the relevant clauses on strength of materials given in *BD 44.*

**11.1.3 Shear connection.** To use this assessment method provision is required for loads applied to the composite column to be distributed between the steel and concrete elements in such proportions that the shear stresses at the steel/concrete interface are nowhere excessive. Shear connectors *must be present* where these shear stresses, due to the assessment ultimate loads, would otherwise exceed 0.6 N/mm$^2$ for cased sections or 0.4 N/mm$^2$ for concrete filled hollow steel sections.

**11.1.4 Concrete contribution factor.** The method of analysis in **11.3** is restricted to composite cross sections where the concrete contribution factor $\alpha_c$, as given below, lies between the following limits:

for concrete encased steel *or wrought iron* sections $0.15 < \alpha_c < 0.8$.

for concrete filled hollow steel *or wrought iron* sections $0.10 < \alpha_c < 0.8$.

where

$$a_c = \frac{0.67 A_c f_{cu}}{N_{pl} \gamma_{mc} \gamma_{f3}}$$

and the squash load $N_u$ is given by:

$$N_{pl} = \frac{A_s f_y}{\gamma_m \gamma_{f3}} + \frac{A_r f_{ry}}{\gamma_{mr} \gamma_{f3}} + \frac{0.67 A_c f_{cu}}{\gamma_{mc} \gamma_{f3}}$$

except that for concrete filled circular hollow steel *or wrought iron* sections $\alpha_c$ and $N_{pl}$ *shall* be determined in accordance with **11.3.7**.

In the previous expressions,

$\gamma_m$ $= 1.05.$

$A_s$ is the cross-sectional area of the rolled or fabricated structural steel section.

$A_r$ is the cross-sectional area of reinforcement.

$A_c$ is the area of concrete in the cross section.

$f_y$ is the nominal yield strength *or worst credible strength* of the structural steel *or iron.*

$f_{ry}$ is the characteristic yield strength *or worst credible strength* of the reinforcement.

$f_{cu}$ is the characteristic 28 day cube strength *or worst credible cube strength* of the concrete.

***11.1.5 Steel contribution factor.*** *The steel contribution factor is*

$$\delta = \frac{A_s f_y}{N_{pl}\gamma_m \gamma_{f3}}$$

**11.1.*6* Limits on slenderness.** The ratio of the effective length, determined in accordance with **11.2.2.4** to the least lateral dimension of the composite column, *shall* not exceed:

(a) 55 for concrete filled circular hollow sections; or

(b) 65 for concrete filled rectangular hollow sections.

**11.2 Moments and Forces in Columns**

**11.2.1 General.** The loads and moments acting in the two principal planes of the column, due to loading at the ultimate limit state, *shall* be determined by an appropriate analysis in which the actual length of the column is taken as the distance between the centres of end restraints. Proper account *shall* be taken of the rotational and directional restraint afforded by adjoining members and the reduction in member stiffness due to inelasticity and axial compression. Alternatively, the method given in **11.2.2** may be used.

**11.2.2 Semi-empirical *assessment* method for restrained composite columns**

**11.2.2.1 Scope.** The semi-empirical method of analysis given in **11.2.2.2** to **11.2.2.6** is only applicable to isolated columns or columns forming part of a single storey frame provided that the restraining members attached to the ends of the column remain elastic under their *assessment* ultimate load; otherwise the stiffness of the restraining members *shall* be appropriately reduced in calculating the effective length of the column and the end moments. *The method is not applicable to cast iron sections.*

**11.2.2.2 Moments and forces on the restrained column.** End moments and forces acting in the two principal planes of the column *shall* be determined either by statics, where appropriate, or by . . . _

$b_s$     *is the breadth of the steel section,*

*The value of the yield strength of the structural steel to be used in calculations shall not exceed 355N/mm$^2$. The concrete cover to the structural steel shall be fully bonded to the steel and shall be unaffected by cracks likely to affect the composite action.*

***11.3.2 Major and minor axes.*** *For the methods of **11.3**, the major and minor axes of bending of the composite section are to be taken as the major and minor axes of the structural steel section.*

***11.3.3 Definition of slender columns.*** *For the methods of 11.3 a column length is defined as short when neither of the ratios $l_{ex}/h$ and $l_{ey}/b$ exceeds 12,*

*where*

*h is the overall depth of the composite section perpendicular to the major axis, and*

*b is the overall depth of the composite section perpendicular to the minor axis.*

$l_{ex}$ *and $l_{ey}$ are the effective lengths calculated in accordance with **11.2.2.4** in respect of the major axis and minor axis respectively.*

*It shall otherwise be considered as slender.*

***11.3.4 Slenderness limits for column lengths.*** *The effective length $l_{ex}$ shall not exceed the least of:*

20h, 250 $r_x$ and 100 $h_1^2/h_2$

*The effective length $l_{ey}$ shall not exceed the least of:*

20b, 250 $r_y$ and 100 $h_1^2/h_2$

*Where*

$h_1$     *is the lesser of h and b*

$h_2$     *is the greater of h and b*

*other symbols are as defined in **11.3.1** and **11.3.3**.*

***11.3.5 Short columns that resist combined compression and bending.***

***11.3.5.1 Scope.*** *Concrete-encased short columns may be assessed in accordance with **11.3.5.2** to **11.3.5.5** if the following conditions are satisfied.*

*(1) The steel contribution factor, $\delta$, defined in **11.2**, is not less than 0.50.*

*(2) The assessment eccentricities of the axial force, $e_x$ and $e_y$ satisfy*

$e_x \leq 1.5h$ *and* *(11.1a)*

*where*

$\lambda_{max}$ *is the greater of* $l_{ey}/b$ *and 12, when* $k_{1y} \leq k_{1x}$, *or*

$\lambda_{max}$ *is the greater of* $0.7\, l_{ex}/h$ *and 12, when* $k_{1y} > k_{1x}$

*other symbols are as defined elsewhere in 11.3.*

*The substitutions for* $k_2$ *and* $\overline{k}_3$ *are both applicable when the column length is slender about one axis only.*

**11.3.7 Ultimate strength of axially loaded concrete filled circular hollow sections.** In axially loaded columns formed from concrete filled circular hollow steel *or wrought iron* sections account *shall* be taken of the enhanced strength of triaxially contained concrete in the method given above by replacing the expressions for $\alpha_c$ and $N_{pl}$ given in **11.1.4** by the following:

$$\alpha_c = \frac{0.67 f_{cc} A_c}{N_{pl} \gamma_{mc} \gamma_{f3}} \qquad (11.6)$$

$$N_{pl} = \frac{f'_y A_s}{\gamma_m \gamma_{f3}} + \frac{0.67 f_{cc} A_c}{\gamma_{mc} \gamma_{f3}} \qquad (11.7)$$

where

$\gamma_m$ $= 1.05$

$f_{cc}$ is an enhanced characteristic strength of triaxially contained concrete under axial load, given by:

$$f_{cc} = f_{cu} + C_1 \frac{t}{D_e} f_y$$

(11.8)

$f'_y$ is a reduced nominal yield strength of the steel *or wrought iron* casing, given by:

$f'_y = C_2\, f_y$

$C_1$ and $C_2$ are constants given in table 11.1.

$D_e$ is the outside diameter of the tube.

t is the wall thickness of the steel or *wrought iron* casing and the remaining symbols are defined as in **11.1** and **11.2.2.4**.

Table 11.1
**Value of constants $C_1$ and $C_2$ for axially loaded concrete filled circular hollow sections.**

| $\frac{l_e}{D_e}$ | $C_1$ | $C_2$ |
|---|---|---|
| 0 | 9.47 | 0.76 |
| 5 | 6.40 | 0.80 |
| 10 | 3.81 | 0.85 |
| 15 | 1.80 | 0.90 |
| 20 | 0.48 | 0.95 |
| 25 | 0.00 | 1.00 |

## 14 JACK ARCH AND TROUGH CONSTRUCTION

*Methods for assessing jack arches and trough construction are given in the Advice Note.*

# APPENDIX B

## Proposed Revision to BA 61

## Annex A Chapter 8 – Cased Beams and Filler Beam Construction

### Note

It should be noted that the majority of the changes made since BA61 are shown *in italics*. However, some additional changes to the text and to some of the formulae have not been shown in italics.

The Assessor will need to compare the revised text contained in this Appendix with that of the published document to make himself aware of all the changes.

# 8 CASED BEAMS AND FILLER BEAM CONSTRUCTION

## 8.1 Scope

### 8.1.1 Introduction

*Since BS5400 Part 5, BD21 and BA16 were written certain aspects of the assessment of cased beams and filler beams have been re-examined. Less conservative methods have been developed for the assessment of longitudinal and transverse shear. New methods of analysis have been developed for old forms of construction which does not comply with BD61. New guidance is provided on incidental stiffening and strengthening, particularly in relation to the fill.*

*Sample flow charts summarising the use of the various methods for the assessment of cased beam decks and for filler beams are appended in Appendix I.*

There is a problem in the assessment *of cased beams* that, if plastic cross section analysis is permitted at ULS, the longitudinal shear strength of the steel/concrete interface is *usually* exceeded.

This *is also the situation for some filler beams, but* tests have been conducted on construction without transverse reinforcement, or with reinforcement inadequate to satisfy the requirements of **8.3.2**, which indicate the plastic design of the composite cross section at ULS is often achievable. In a form of filler beam construction developed in France [41], which includes special details and transverse prestressing, no check is required on the local bond stress at either SLS or ULS. However until such time as longitudinal shear in filler beams has been researched a check on the longitudinal shear on planes of type 6 and 7 in figure 8.3 is required as a safeguard against possible failure modes not yet observed.

The lower limit on the characteristic or worst credible strength (of 25N/mm$^2$) in clause 8.1 of BD61 does not apply to assessments in accordance with this chapter.

*Alternative procedures to those in Clauses 8.1.4, Clause 8.1.8 and Appendix H are permitted provided they are justified and provided they take into account differences in the performance of the structure at SLS and ULS.*

### 8.1.2 Cased beams

Cased beams exist which do not comply with the requirements of the Standard, and reduced interface shears are appropriate for less efficient casings and partial casings which do not have efficient confining reinforcement and may be entirely unreinforced. These are types B, C and D in figure 8.2. Permissible interface bond stresses at SLS for these beams are as follows (N/mm$^2$):

A. (As in Standard) $0.10\sqrt{f_{cu}}$ but $\leq 0.70$

| | | |
|---|---|---|
| B. | 20mm ≤ cover ≤ 50mm | $0.07\sqrt{f_{cu}}$ but ≤ 0.40 |
| C. | | $0.06\sqrt{f_{cu}}$ but ≤ 0.35 |
| D. | Cover ≥ 50mm | $0.10\sqrt{f_{cu}}$ but ≤ 0.50 |

*At ULS the same values apply but for partial shear connection calculations the longitudinal shear stress should not exceed 0.50N/mm$^2$.*

*The following conditions apply:*

*(i)* For cased beams of types A and C the checks required are as follows. Partial shear connection calculations should assume plastic methods of cross section design and a ductile shear connection. The equilibrium method is the method of Clause 5.5.2 of Ref. 28 or the 'plastic theory' method of Clause 6.2.1.2 of Ref. 4. The linear interpolation method is the more conservative method of Clause 6.2.1.2 of Ref. 4.

*(ii)* For spans exceeding 12m the *local* bond stress at SLS should not exceed the permissible bond stress given above nor 0.5N/mm$^2$. At ULS the moment of resistance should be taken as the yield moment.

*For spans exceeding 9m and up to 12m, the local bond stress at SLS should not exceed the permissible bond stress. At ULS the moment of resistance should be taken as the yield moment.*

For spans between 6m and 9m, the *local bond stress at SLS should not exceed the permissible bond stress.* At ULS the moment of resistance should be assessed as the greater of the yield moment and the flexural resistance assuming partial shear connection by the linear interpolation method.

For spans less than 6m, *at SLS the local bond stress should not exceed* the permissible bond stress. At ULS the moment of resistance should be assessed as the greater of the yield moment and the flexural resistance assuming partial shear connection by the equilibrium method.

*(iii)* Where the moment of resistance is taken as the yield moment the design strength of the metal should not be taken greater than 275N/mm$^2$.

*(iv)* The moment of resistance for cased beams type B should not be taken greater than the yield moment. *Checks on the shear connection at ULS are not required.*

*(v)* The moment of resistance for beams type D should not be taken greater than the lower of the moment at bond failure and the moment of resistance calculated in accordance with chapter 6 of the Standard assuming full composite action. A check at SLS is not required.

(vi) The moment of resistance for beams of types *A and C* may be assumed to be the yield moment where this is sufficient to satisfy the assessment loading.

(vii) These rules do not apply to cased beams where the soffit of the slab is above the top flange of the steel beam by more than 25mm. *This is because, despite the rules, cased beams rely disproportional upon the bond to the top flange, which is reduced by raising the soffit of the slab. There is no relevant experimental work on such construction, but clearly lower interface bond stress would be needed, possibly 20% lower for beams type A and C and 40% lower for beam type B and zero for beam type D.*

(viii) In beams with abnormal concrete depths above the metal beam the depth of concrete used in calculation should be restricted such that the elastic neutral axis lies within the depth of the steel section *for both elastic and plastic cross section analysis.*

(ix) As an alternative to *(viii)* the full depth of concrete may be assumed and the metal beam concentrated at the centroid of the steel section.

(x) *The yield moment and flexural resistance are calculated using the assessment strengths of materials appropriate to the ULS.*

(xi) *Generally the same rules apply to cased beams not attached to a slab, when they are subject to the same slenderness limitations as RC beams.*

### 8.1.3 Complying filler beams

This clause considers filler beam construction which complies with the Standard and may be used when the construction or the grade of concrete does not satisfy clause **8.1** of BD61. This section considers only filler beam construction in which the transverse reinforcement is adequate to resist the moments obtained analytically. For this situation it is recommended that orthotropic plate analysis is employed, but grillage representations, allowing for the different stiffness transversely and longitudinally, are an acceptable alternative. The transverse distribution rules of BA 16 are inappropriate for this form of construction.

The method in **8.3.2** of the Standard, which gives transverse distribution rules, should now be regarded as a Category A alternative (see Foreword). However providing the conditions in **8.3.2(b)** and **(d)** are satisfied:

*(i) the transverse hogging moment may be taken as 10% of the maximum sagging moment, and*

*(ii) it may be assumed that there is a linear reduction in the transverse sagging moment in the 2m side strips as specified in the Standard.*

Filler beams may be assessed assuming the principles in **8.1.2**, but the stress on the shear connection at ULS taken as 1.4 times the values for cased beams type A and:

For spans exceeding 8m the local bond stress at SLS should not exceed the permissible bond stress and the flexural capacity at ULS should be assessed as the greater of the yield moment and the flexural resistance assuming partial shear connection by the linear interpolation method.

For spans up to 8m the local bond stresses at SLS should not exceed the permissible bond stress and the flexural capacity at ULS should be assessed as the greater of the yield moment and the flexural resistance assuming partial shear connection by the equilibrium method.

Alternatively, *irrespective of the span*, for beams satisfying **8.3.2(c)** of the Standard the moment of resistance may be assumed to be the full plastic moment of resistance without a check on the bond stress at ULS, provided the *characteristic or worst credible* strength of the metal beam used in the calculation is not greater than 275N/mm$^2$ and provided **8.1.2(viii)** is satisfied or provided there is not more than 75mm of concrete above the top flange of the steel section. Where of these conditions only **8.3.2(c)** of the Standard is not satisfied the moment of resistance may be taken as the yield moment.

### 8.1.4 Non-complying filler beams

(a) Filler beams with concrete encasement/infills

For beams not complying with **8.3.2** of the Standard, providing there is no evidence of excessive corrosion, fretting action or cracking (in the case of cemented materials) sufficient to adversely affect the achievement of composite action, Clauses **8.1, 8.3.1, 8.4, 8.5.1** and **8.5.2** apply.

In filler beams without transverse reinforcement, the lateral distribution of load is greater than that suggested by grillage analysis with pinned transverse members. To reproduce the actual distribution the flexural stiffness of beams should be based on the composite cross section, and the torsional stiffness of internal beams taken as that of the rectangle of concrete horizontally between mid span of adjacent infill elements and vertically *by a height above the soffit of the metal beam no greater than 1.5 times the vertical distance between the flanges.* Suitable grillages for analysing filler beam decks are shown in figure 8.4, where the member properties to be assumed are shown in Table 8.1. In using the grillage in figure 8.4(b) the torsion per unit width is to be taken as the sum of the torsions per unit width in the two directions. The two grillages give similar distributions of bending moments and torsions, but that in figure 8.4(b) allows lateral distribution of reactions, whereas the grillage in figure 8.4(a) allows none. *In skewed bridges the transverse beams should be approximately perpendicular to the longitudinal beams.*

**Table 8.1 Cross Section Properties for Global Analysis of Non-complying Filler Beams**

| | Lateral Distribution of Reactions: Disregarded | Included |
|---|---|---|
| Main beams | | |
| I | $I_{composite}$ | $I_{composite}$ |
| J | $J_{steel}$ | $J_{steel}$ |
| A* | $A_{composite}$ or $\infty$ | $A_{composite}$ or $\infty$ |
| Intermediate beams | | |
| I | - | 0 |
| J | - | $J_{concrete} / 2$ |
| A* | - | $A_{concrete}$ or $\infty$ |
| Transverse beams | | |
| I | 0 | $I_{concrete}$ |
| J | $J_{concrete}$ | $J_{concrete} / 2$ |
| A* | $A_{concrete}$ or $\infty$ | $A_{concrete}$ or $\infty$ |

* Analysis is insensitive to A

The grillage in figure 8.4(b) should only be used where the abutments are sufficient to resist the torsions, a condition which may be assumed providing:

(a) *The abutments are in good condition and not sufficiently cracked so as to relieve torsion moments, and*

(b) the slab should project a distance equal to the depth of the metal beam past the end of the beam, or

(c) the concrete in the deck should be monolithic with a substantial abutment beam of depth not less than 50% greater than the depth of the filler deck, or

(d) the concrete in the deck should be monolithic with a concrete abutment.

In assessing the results of the analysis, the flexural resistance and bond checks may be assumed to be the same as for complying filler beams providing the effective concrete is restricted for torsion to that within a depth above the steel beam soffit of 1.5 times the depth between the flanges. For flexure *8.1.2(viii)* applies (the alternative in *8.1.2(ix)* does not apply). The torsional strength of the concrete may be taken as

$$\frac{0.50(f_{cu} / \gamma_{mc})^{0.5}}{\gamma_{f3}}$$ *when the beam spacing/depth ratio of the metal beams exceeds 2.0*

*or otherwise*

$$\frac{0.58(f_{cu} / \gamma_{mc})^{0.5}}{\gamma_{f3}}$$

When this *stress is exceeded the reduced stiffness given by the expression H1 in Appendix H, may be used, when no check on the stress is required.*

*(b) Filler beams with masonry infills*

Dense brickwork filler beams with the mortar fully bonded to the bricks and the *metal* beams shall be assessed in accordance with the above provisions *and those in clause 5.4 (of BD61),* except the bond stress in **8.5.1** and the strength of the shear planes through the masonry shall be taken not greater than *0.35*N/mm$^2$, the resistance of attachments shall not be taken greater than 60% of the value in **5.3.3.8.1** and moment of resistance should not be taken in excess of the yield moment of *the composite section as defined above in* ***8.1.2(x).***

For the analysis two methods are permitted is as follows: An analysis using comparable section properties as those in Table 8.1, for which the stiffness is calculated *as 1000 times the compressive strength in accordance with Appendix H (Section H1), when the torsional stresses in the masonry should not exceed:*

$$\frac{0.65(f_{mk} / \gamma_{m})^{0.5}}{\gamma_{f3}}$$

*when the beam spacing/depth ratio of the metal beams exceeds 2.0, or otherwise*

$$\frac{0.75(f_{mk}/\gamma_m)^{0.5}}{\gamma_{f3}}$$

Where this is exceeded the stiffness may be reduced to that given by expression H1 in Appendix H, when no check on the stress is required.

When the brickwork is not bonded to the steel beams, similar provisions apply except that the bond shall be taken not greater than 0.30N/mm$^2$ and the resistance of attachments shall be taken no greater than 40% of the values in **5.3.3.8.1**.

Where the *soffit between the beam flanges is of sound structural material and the material above is weaker but complies with **8.1.8**, then providing the total depth of the deck less the top 75 mm of surfacing is not less than 20% thicker than the depth of the metal beams, the transverse stiffness per unit length may be taken as 2% of the longitudinal stiffness, per unit width. No checks are then required on the stresses in the elements orthogonal to the girders in the analysis.*

### 8.1.5 Vertical shear resistance

For cased beams and filler beams the shear resistance of cemented material up to 250mm above the level of the steelwork, and for a width on either side as shown in figure 8.5, may be added to that of the steelwork assuming a shear strength of concrete $v_c$ based on the concrete strength $f_{cu}$ as given in BD 44. Strictly this value applies only when there is a small amount of longitudinal reinforcement and for this purpose the steelwork is deemed to be effective as reinforcement. The shear assumed to be carried by the concrete should not exceed 15% of the total shear in cased beams and 30% of the total shear in filler beams.

For dense brickwork filler beams the provisions of clause 8.4 (of BD61) apply:

### 8.1.6 Procedure when longitudinal shear resistance is inadequate

When the longitudinal shear exceeds the permissible *interface bond stress at either SLS or ULS* composite action should be disregarded and all beams with fill on both sides should be considered to be compact, irrespective of the cross section slenderness.

### 8.1.7 Punching shear resistance

The punching shear resistance to a wheel load may be assessed assuming the load is replaced by two strip loads, each of which has the same width and centroid of the part of the load which would be carried by statics to the supporting beam. The shear may be assumed to be carried over a width equal to the loaded width plus $a_v$, assuming a concrete strength of $3v_c d/a_v$, where $a_v$ is the distance from the strip to the face of the web of the neighbouring metal beam and d is the depth from the surface of the concrete to the lower web/flange intersection of the metal beam. *For dense brickwork the shear strength should be taken as* $3 f_v / a_v \gamma_{mv}$, *where* $f_v$ *is from BS5628 and* $\gamma_{mv}$ *is taken as 2.5.*

For a wheel load on a bay adjacent to an edge beam the resistance should be taken as 70% of that for an internal bay of similar dimensions, unless it can be shown that the horizontal thrust resulting from the arching action shown in Fig 8.6 can be adequately resisted.

*Flexural checks under local loading are not required providing the beam spacing to web depth ration does not exceed 4.0 for internal bays and 2.5 for external bays. Where checks are necessary arching action may be assumed and horizontal composite action between the lower part of the metal beam and the concrete on either side of the web equal to the least of:*

- *the half of the distance between the webs of the metal beams*
- *the position of the centreline of the nearest load*
- *the edge of the construction.*

### 8.1.8 Effect of end restraints and of finishings and infill material not satisfying BD 44

Where tests with vehicles of weight not less than 70% of the assessment vehicle suggest there are significant incidental strengthening effects under four passages of the vehicle, or *where these effects can confidently be regarded of comparable or better characteristics to those demonstrated to have substantial stiffness in the literature*, an increase in strength may be taken into account as follows:

(i) Effect of end restraint

The effect of end restraint from friction in resisting the resolved longitudinal and transverse forces (if any are assumed) may be taken into account calculated from the dead loads above the level of the soffit, including that in the abutment beam and a coefficient of friction of:

0.35 for concrete on masonry or masonry on masonry

0.50 for concrete cast on concrete with an unprepared surface

0.60 for concrete cast on concrete with a prepared surface, or monolithic concrete of strength not exceeding 20N/mm$^2$

0.75 for monolithic concrete of strength exceeding 20N/mm$^2$

It should be noted that where the abutments are thicker than the slab there may be significant end moments from continuity with the support, but this should be discounted due to the likely loss of this effect when the concrete cracks, unless there is flexural reinforcement present satisfying BD 44.

(ii) Effect of finishings

The contribution of the concrete which would not normally be regarded of structural quality *(here described as "weak concrete")*, masonry and well compacted (cohesive or weakly cemented cohesive) material, between and above the steelwork, may be taken into account in the assessment of the effect of live loads on the longitudinal bending (for stiffness and strength) of filler beam decks, where it can be shown that the material is in contact with the full depth of the web or on flat rough concrete surfaces of construction satisfying **8.1.2**, **8.1.3** or **8.1.4**. *The following guidance is restricted to metals for which the characteristic or worst credible strength does not exceed 275 N/mm$^2$ and relates to checks on load levels up to SLS loading. The effective cross section to be used in the calculations is as defined in Appendix H, Section H5. The method assumes that the better traffic compaction of the fill in the older bridges offset the probably greater variability of the properties in the original materials. The effect of any finishings above a sprayed-on waterproofing systems should be disregarded.*

An initial elastic cross section analysis is required in which such material may be taken into account by assuming *combined modular ratios for the fill and weak and structural concrete of:*

$\alpha_e$ = 15 where there is at least 150mm of structural concrete above the top flange of the steel beam, or otherwise

$\alpha_e$ = 30

The strain at the surface of the carriageway determines the method by which the finishings may be taken into account in carrying the live load moments as follows:

| Multiple passage of vehicles (as at SLS) | Single passage of vehicles (as at ULS) | Method of taking finishings into account |
|---|---|---|
| $\leq 350\mu\varepsilon$ | $\leq 500\mu\varepsilon$ | by the analysis just described |
| $\leq 700\mu\varepsilon$ | $\leq 1000\mu\varepsilon$ | by increasing the elastic section modulus of the steel by $(h_c + h_s)/h_s$ |

In the calculations the dead load should be assumed to be carried by the bare steel section, but the superimposed dead load may be carried on a composite cross section satisfying **8.1.2, 8.1.3, or 8.1.4** ***and Appendix H.***

The calculation may be used for any purposes for which the results are more economical than the methods of **8.1.2, 8.1.3, 8.1.4** ***and Appendix H.*** No checks on the stresses in the finishings are required.

In *structures* in which shear is critical the effect of finishings should only be taken into account when approved.

It is difficult to offer guidance in regard to construction in which it is suspected that the parapets may carry a significant portion of the bending moment

- A load test might damage the parapet.
- When the deck is less strong than the theory suggests brittle failure could result.
  - Where the parapet is well connected to the deck it could sometimes contribute to the strength.
- Substantial parapets can span independently of the deck, thereby removing heavy dead loads from the deck.

(iii) Effect of infill material not satisfying BD 44

Suggestions for inclusion at the ULS of the effects of infill material not satisfying BD 44 are presented in Appendix H, which may be used when approved.

(iv) Combined effects of end restraint, finishes and infill material not satisfying BD 44.

End restraints and either finishes or, where permitted, infill material not satisfying BD 44 may be considered to act simultaneously, but account should be taken of the fact that end moments may reduce deflections disproportionately to their increase in resistance to the load. The effect of finishes not satisfying Appendix H and infill material not satisfying BD 44 may not be combined.

(v) Testing

As an alternative to (i) to (iv), where due to non uniformity of material, doubt as to its quality or where these procedures are believed to underestimate the strength, other agreed procedures are permitted. A procedure such as that in the second sentence of **5.3.3.8.3** above may be valid, but the increase in strength attributable to increase in stiffness needs to be established.

**Figures 8.1, 8.2 and 8.3**

**Figure 8.4**

**Figures 8.5 and 8.6**

## *APPENDIX I: SAMPLE FLOW CHARTS ILLUSTRATING THE ASSESSMENT PROCEDURES FOR BRIDGE DECKS CONSIDERED IN CHAPTER 8*

*Due to the large variation in the bridge types covered by Chapter 8 it is not possible to produce flow charts covering all the situations likely to be encountered and the different checks necessary for cast iron, wrought iron and steel.*

*Sample flow charts are included for:*

*Cased beams*
*Filler beam decks with concrete infills*

*In these are summarised the options for the analysis and the assessment procedures.*

***Notes for the assessment of Cased Beams and Cased Beam Decks***

1. *BA61 considers construction beyond the scope of BD61 and gives rules for construction complying with BD61 which, overall, generally justifies an increase in capacity.*

2. *No specific methods of analysis are suggested for cased beams, either in BD61 or BA61. The methods normally employed are grillage analysis, analysis with ribbed thin plate FE elements, orthotropic plate analyses (not common) and the method in Chapter 2 of BA16.*

3. *BA61 shows higher local bond stresses at SLS than BD61 and thus using BD61 is normally found to be the critical condition. However in BA61 the flexural resistance is subject to a more rigorous check at ULS which often gives a lower capacity than the simple check in BD61. Therefore it is inadmissible to check flexure/interface bond stresses at SLS using BA61 and at ULS using BD61.*

4. *The effect of incidental effects in clause **8.1.8** on cased beam construction has not been assessed, but there is no reason why they should not be considered in the assessment of cased beams, provided the assessed strength of the metal beams does not exceed 275N/mm$^2$ and provided sprayed on waterproofing systems have not been used. Normally the effects should be considered only when they are appreciable.*

***Notes for Assessment of Filler Beams***

1. *BA61 considers construction beyond the scope of BD61, which includes construction in which there is insufficient reinforcement to resist transverse moments. It also gives rules for construction complying with BD61 which overall generally justifies an increase in capacity.*

2. *BA61 contains different rules from those in BD61 for bond stresses, shear on longitudinal planes through the concrete and vertical shear, it has a new clause on punching shear from point loads.*

3. *BA61 shows higher local bond stresses at SLS than BD61 and thus using BD61 is normally found to be the critical condition. However in BA61 the flexural resistance is subject to a more rigorous check at ULS which often gives a lower capacity than the simple check in BD61. Therefore it is inadmissible to check flexure/interface bond stresses at SLS using BA61 and at ULS using BD61.*

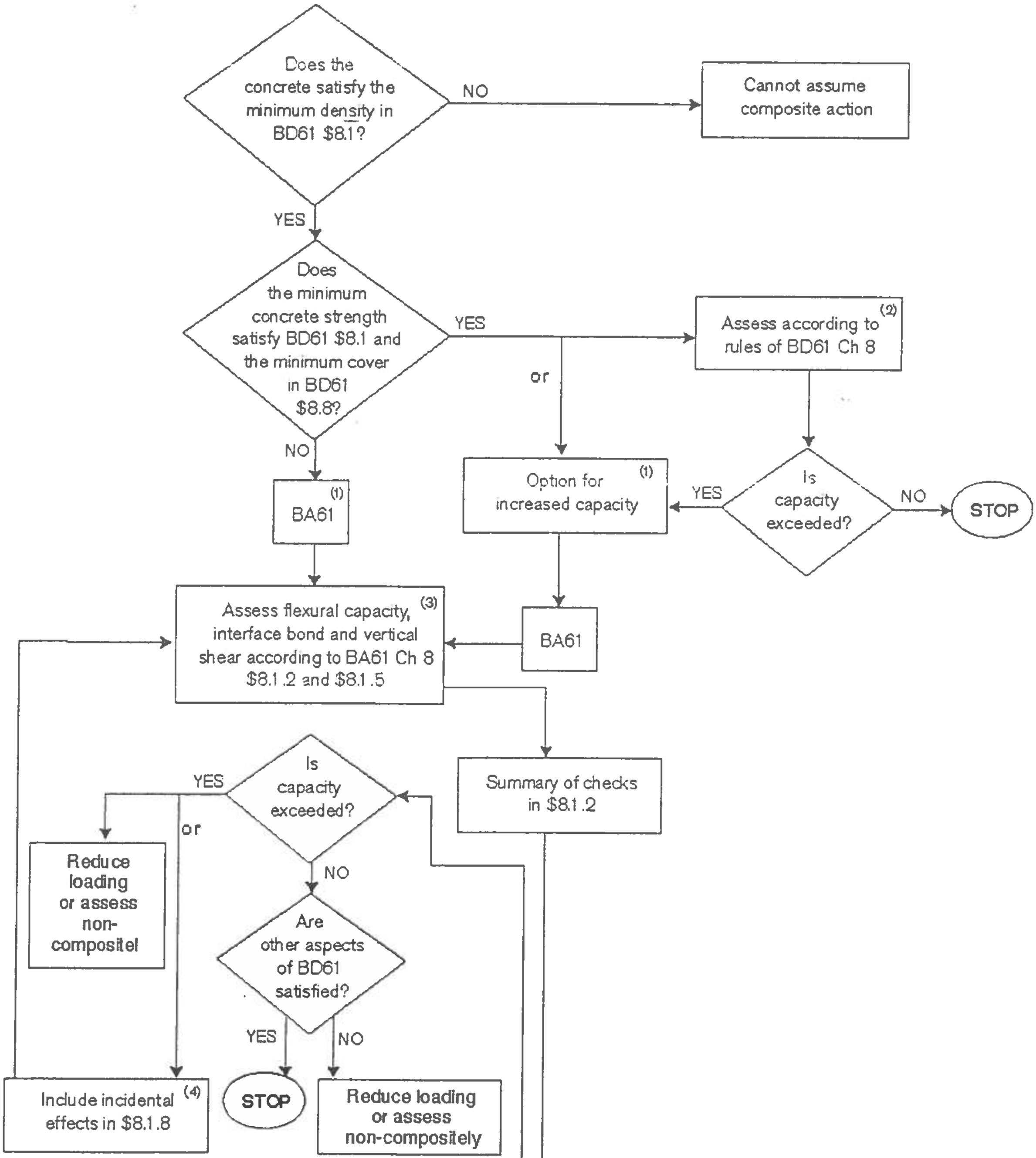
4. *BA61 gives higher shear resistance.*

5. *Besides advice in the Foreword and that implicit in the flow chart it is beyond the scope of BA61 to advise on procedures to best suit an individual project.*

6. *Where there are masonry infills the assessment procedure is similar, but the stiffnesses and limiting stresses are appropriately modified.*

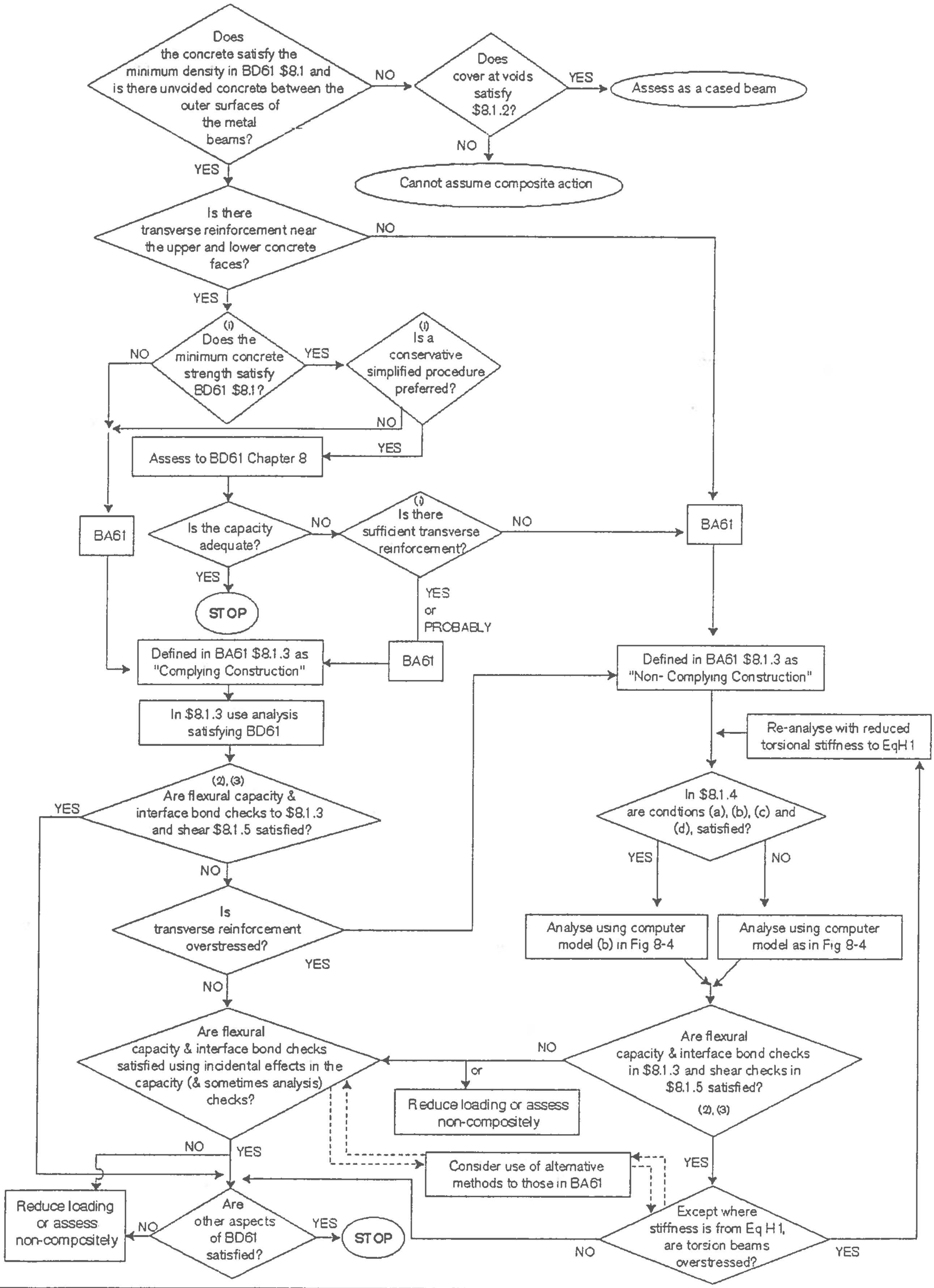
7. *Where incidental effects are included it is always necessary to ascertain the capacity without these effects since the permitted increase in capacity for these effects is related to the capacity of the deck without them.*

# Assessment of Cased Beams and Cased Beam Decks




| Beam Type | SLS Checks | ULS Rigorous Check | ULS Simplified Check |
|---|---|---|---|
| A and C | $8.1.2(3) | $8.1.2 (i) to (iii), (vii) to (ix) | $8.1.2 (i) to (iii), (vi) to (ix) |
| B | $8.1.2 | $8.1.2 (ii), (iii), (iv), (vii) to (xi) | None |
| D | None | $8.1.2 (ii), (iii), (v), (vii) to (xi) | None |

# Assessment of Filler Beam Decks with Concrete Infills



**METHODOLOGY FOR CARRYING OUT SECTION 117 ASSESSMENTS**

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**This methodology may be issued to organisations responsible for carrying out Section 117 assessments.**

## 1 Introduction and Background

1.1 A "Section 117 assessment" has nothing to do with determining an actual load carrying capacity of a bridge. It is an exercise carried out to ascertain Railtrack's legal liability under Section 117 of the Transport Act 1968 as regards the load-bearing capacity of a bridge carrying a public highway.

1.2 Section 117 of the above Act requires (inter alia) that Railtrack maintain its highway-carrying bridges such that they have "the required load-bearing capacity". Section 117 provides further that "the appropriate Minister" may prescribe particular load-bearing standards for these bridges. If no Standards are prescribed, bridges must be capable of bearing "the weight of the traffic which ordinarily uses, or may reasonably be expected to use, the highway carried by the bridge" at the time Section 117 came into force.

1.3 For bridges in England and Wales, the Railway Bridges (Load Bearing Standards) (England and Wales) Order 1972 sets out the required load-bearing standards. This Order, often known as "SI 1705" (Statutory Instrument 1972 No. 1705) specifies the loads to be carried by various categories of bridge, mainly determined by the date of construction (or date of last reconstruction if reconstruction has occurred).

1.4 For bridges constructed before 1 January 1955, SI 1705 specifies the loading as that due to the heaviest vehicles permitted under the 1969 Construction and Use Regulations. SI 1705 also specifies that these bridges are to be assessed in accordance with the Ministry of Transport Technical Memorandum (Bridges) No BE4 1967 as amended up to 11 November 1970 ("BE4").

1.5 For bridges constructed after 1 January 1955, the loading is specified in terms of HA and/or HB loading, generally according to date and class of road carried. No method of assessment is specified for bridges constructed after 1 January 1955. It is considered reasonable, however, that bridges constructed after this date and up to 1973 should be assessed generally in accordance with BE4 because this represents the best assessment practice at the time Section 117 came into force. For bridges constructed after 1973, assessment should be in accordance with the BE standard current at the time of construction.

1.6 SI 1705 does not apply to Scotland and no equivalent Scottish Order was ever made. Therefore "the required load-bearing capacity" for bridges in Scotland defaults to the weight of the ordinary traffic of the day. It is considered reasonable that the ordinary traffic of the day for any given bridge in Scotland should be taken as equal to the specified loading for a bridge in England or Wales of the same date and carrying an equivalent class of road.

1.7 Because SI 1705 does not apply to Scotland, no assessment methods are specified for Scottish bridges. However, it is considered reasonable that bridges in Scotland constructed up to 1973 should be assessed generally in accordance with the Scottish equivalent of BE4, that is the Scottish Development Department Technical Memorandum (Bridges) No SB9/74 ("SB9/74"). (This is a revised version of a document originally issued in 1967. The technical content of SB9/74 is virtually identical to that of the 1967 version, and both are very similar to BE4.)

1.8 Certain forms of construction (e.g. prestressed concrete) are not covered at all in BE4. Other forms (e.g. multi-span arches, jack arches) are given incomplete coverage. Recommendations are given below for assessment of bridges with construction form not adequately covered in BE4.

1.9 The recommendations given below include a table (derived from SI 1705) giving the loading which should be used for Section 117 assessments in various circumstances.

Note: the term "BE4 assessment" has commonly been used to indicate the process of determining whether or not a bridge has the required load-bearing capacity under Section 117 of the Transport Act 1968. As explained above, the process does not necessarily include use of BE4, so the term can be misleading. The term "Section 117 Assessment" is therefore considered preferable.

## 2 Procedure for Undertaking a Section 117 Assessment

2.1 A Section 117 assessment should be undertaken in a very similar way to a BD 21 assessment in that an Approval in Principle (AIP), Assessment Report and Certificate (Form BA) should be produced. However, there is no need to produce an Inspection Report as the report produced for the BD 21 assessment may be utilised and should generally be referenced in the AIP for the Section 117 assessment. It should be noted that some of the conclusions in the Inspection Report may not be totally applicable to the Section 117 assessment because of the limited nature of this assessment and also because there are fundamental differences between earlier assessment methods and BD 21.

2.2 The Technical Approval Authority for all Section 117 assessments is the appropriate Railtrack Zone. A model form AA has been developed for use and is available from Railtrack or from the Reviewing Consultant. This follows a very similar format to the BD 21 Form AA, but has been amended to reflect the requirements of the Section 117 assessment such as the loading and relevant standards. It also includes sections which detail the results of the BD 21 assessment and identify the relevant BD 21 reports.

## 3 General Provisions – Bridges Built After 1922 but Before 1955

3.1 The majority of Section 117 assessments will be carried out in accordance with BE4. Many such assessments are covered by the provisions of Clause 301(a) of BE4 (SB9/74 in Scotland); where reasonable evidence can be provided to show that all the following are true in respect of a bridge:

- it was constructed after 1922 and prior to 1955 (or if the superstructure was completely reconstructed between these dates);
- it carried a classified or trunk road when it was constructed (as advised by Railtrack);
- its overall condition is not "bad"*;

then the bridge should be taken to meet the requirements of BE4 (SB9/74 in Scotland) and the assessment does not require calculations.

(*Note: where the BD 21 assessment inspection report does not give an overall condition rating, this should be determined from Railtrack's latest Detailed Examination report – if the overall rating is given as "good" or "fair" the bridge should be taken as being "not in bad condition".)

3.2 In such cases the Section 117 Assessment AIP should state that the bridge is to be assessed in accordance with the provisions of BE4 (or SB9/74) clause 301(a). The Section 117 Assessment Report should state that the bridge has been assessed as capable of carrying Construction & Use traffic in accordance with BE4 (or SB9/74). The back-up supporting information should refer to the specific provisions of clause 301(a).

**4 General Provisions – Other Bridges**

4.1 In the case of bridges where quantitative methods are required, it is widely accepted by highway authorities that a Section 117 assessment need consider only those components which have failed to achieve 25 tonne capacity to BD 21 (for bridges constructed before 1955) or 38 tonne to BD 21 (for bridges constructed in 1955 or later).

4.2 With regard to the superstructure, a quantitative assessment should be undertaken of each component which has been subject to a quantitative BD 21 assessment and which could possibly "fail" a Section 117 assessment. (Note, however, that this does not apply to components which fall outside the scope of the Section 117 assessment or to components for which the Section 117 assessment is qualitative – see later in this document for details.) Where the assessor can demonstrate without recourse to calculations that a component will "pass" BE4 (or SB9/74), a suitable statement should be made to this effect.

4.3 It is important to identify and report separately each "failed" component and the reason for "failure" (e.g "cross girders 3 and 7 fail because corroded bottom flange overstressed in bending at midspan"), rather than simply reporting the whole bridge as a "Section 117 assessment failure". The actual assessed live load capacity of the "failed" component should also be stated where this can be identified from the level of analysis undertaken. Such information may be necessary in determining the nature of and liability for any subsequent action.

4.4 Components should not be identified as "conditional passes", even if they have been (incorrectly) reported as such in the BD 21 assessment. If, for example, the BD 21 assessment capacity of a component has been made conditional on some action, e.g. "40 tonne capacity provided that the rivets are replaced", the Section 117 assessment should state that the component "fails because of inadequate rivet shear capacity but would pass if some of the corroded rivets were replaced" (or wording to similar effect).

4.5 BE4 and SB9/74 state in clause 301(c) that “in arriving at an assessment of the bridge, the foundations, substructure and superstructure should all be considered”. Nothing further is stated as regards foundations and substructure, nor as regards arch spandrel walls; it is thus reasonable to assume that these components are intended to be assessed qualitatively. Where such a component has “failed” a qualitative BD 21 assessment (this does not include a component that requires monitoring), a qualitative assessment of the component (taking into account the loading criteria of BE4) should therefore also be included in the Section 117 assessment.

4.6 It can be seen from the above that there are some significant differences with respect to the content and conclusions of a quantitative Section 117 Assessment Report as compared with a BD 21 Assessment Report. A model for a quantitative Section 117 Assessment Report has been produced for use and is available. The main difference is that there are no Results Summary Sheets and obviously, the result is a “pass” or “fail” for each component examined. As stated above, for each “failed” component the reason for “failure” should be given, together with the assessed live load capacity where this can
be identified from the level of analysis undertaken. Only one section of text is required (Summary of Results), as there is no need to repeat the information which is given in the BD 21 Assessment Report.

4.7 The various documents for a Section 117 assessment should include sufficient cross-referencing to the relevant BD 21 documents to ensure that a technical audit trail is fully established.

4.8 A Certificate (Form BA) is also required and a model for a quantitative Section 117 assessment is available.

## 5 Bridges of More than One Span

5.1 SI 1705 states that “where a . . . bridge consists of more than one separately supported span, each span, together with its supports and its superstructure, shall . . . be treated as a separate . . . bridge”. Therefore if a multi-span bridge has simply-supported spans which were constructed or reconstructed at different times, each span should be considered separately for Section 117 assessment purposes.

## 6 Approval Process

6.1 As stated above, the relevant Railtrack Zone is the Technical Approval Authority (TAA) for both the Approval in Principle (Form AA) and Certification (Form BA) of Section 117 assessments. This is different from the process for BD 21 assessments. The Zone, however, will generally require both the Form AA and the assessment to be reviewed by the appropriate Reviewing Consultant before giving approval. As is the case with BD 21 assessments, the standard procedure requires the Form AA to be approved prior to commencement of the assessment.

## 7 Analysis and Methods of Calculation

### 7.1 General – Pre-1955 Bridges

7.1.1 For pre-1955 bridges in England and Wales, the analysis should be undertaken in accordance with Ministry of Transport Technical Memorandum (Bridges) No. BE4 *The Assessment of Highway Bridges for Construction and Use Vehicles* 1967 (as amended up to 11
November 1970). For pre-1955 bridges in Scotland, Scottish Development Department Technical Memorandum (Bridges) No SB9/74 should be used. The technical content of SB9/74 is very close to that of BE4 and references to BE4 below may generally be taken as applying equally to SB9/74 (although clause numbering may differ).

7.1.2 BE4 utilises working stresses and makes reference to other Codes of Practice which were current at the time. The Codes current at the time of the Order and which should be used as appropriate, depending on the form of construction of the bridge, are as follows:

a) BS 153 Parts 3 and 4 – Specification for Steel Girder Bridges: Part 3B Stresses and Part 4 Design and Construction (reset and reprinted April 1966) and subsequent amendments up to and including Amendment No 8 (AMD 93 September 1968);
b) CP 114: Part 2: 1969 – The Structural Use of Reinforced Concrete in Buildings;
c) CP 115: 1969 – The Structural Use of Prestressed Concrete in Buildings;
d) CP 116: 1969 – The Structural Use of Precast Concrete;
e) CP 117: 1967: Part II – Code of Practice for Composite Beams for Bridges.

7.1.3 BE4 identifies for various forms of construction the different material stresses that should be utilised and also the approach which should be adopted for the analysis, in some cases utilising the codes detailed above. Some forms of construction are not covered and there are also some important differences with respect to modern day codes which need to be highlighted. These are detailed below by discussing each common form of bridge construction in turn. It is important to grasp that where a method is not covered in BE4, the approach should be in accordance with that reasonably held to have been normally available in 1972. The method must be logically defensible, not necessarily defensible in terms of modern engineering knowledge.

### 7.2 Single Span Masonry Arches

7.2.1 BE4 details clearly the method of analysis for single span arches which is adapted from the method set out in "Military Load Classification by the Reconnaissance and Correlation Methods", MEXE May 1963. However, BE4 does not give any quantitative guidance on what condition factor should be used for the structure; it only discusses the various cracks and other defects which should be taken into account. It cannot at present be established with any certainty how practising engineers in the early 1970s in fact derived condition factors. Until more information is available, the condition factor used for a Section 117 assessment should provisionally be taken as the same as that derived for the relevant BD 21 assessment. However, if this results in a section 117 "failure" in any particular case, further advice should be sought from Railtrack.

7.2.2 BE4 makes no reference to the result being possibly unconservative for cases where the depth of fill is greater than the thickness of the arch barrel as is stated in BD 21 Clause 6.17. It is therefore considered that for a BE4 analysis the full depth of fill can be used without recourse to any additional checks. There is no requirement in BE4 to consider axle lift off or any longitudinal gradient to the road.

7.2.3 As in BD 21, there is a limitation on the span of 60 ft (18.2m). The approach to be adopted for spans greater than this should be agreed with the TAA. BE4 is very clear that the skew span should be utilised in the analysis.

7.3 <u>Multi Span Masonry Arches</u>

7.3.1 BE4 is rather ambiguous with regard to multi span arches. There are two places where reference is made to this form of construction. Part III clause 2 states, with respect to the method of analysis for arches, that "it is intended to be applied primarily to single span arches and in the case of multiple spans, particular attention is drawn to clause 8(e)(vii)". This latter clause is included in the list of factors that need to be considered in the determination of the condition factor for an arch; it states that "where the bridge consists of multi-span arches and the strength of intermediate piers is in doubt, the structure should be closely examined for cracks or deformation arising from this cause".

7.3.2 The above statements do not clearly define how a multi span arch should be assessed. However, it might be read that in the vast majority of cases multi span arches should be considered as individual single span arches and that only when damage is apparent to the pier should the effect of the pier be taken into account. In such case, a condition factor should be incorporated. However, in line with other structural defects, no value for the condition factor is given in BE4. This approach can be compared with that given in "Military Load Classification of Civil Bridges by the Reconnaissance and Correlation Methods (Solug Study B.38.), MEXE 1963" from which BE4 was developed, in which the approach for multi span arches is much clearer. In the MEXE document, it is clearly the case that a reduction should be made in the capacity of an arch structure where piers are present irrespective of whether there are defects. It does not define the dimensions for a pier, although it is clear that it is trying to relate a pier to an abutment, which it defines as sufficiently massive to resist the full thrust of an arch. No information is provided on what should be done if the pier exhibits significant defects.

7.3.3 It is not known why BE4 does not reflect MEXE in this matter. It may be that the authors of BE4 thought that the determination of the adequate strength of a pier could best be achieved by a qualitative inspection to remove any question regarding whether a pier is stocky enough to be considered an abutment. This is supported by the fact that BE4 has completely removed the whole concept of "abutment factor" for arches, thus indicating that a qualitative assessment of the adequacy of the substructures is all that is required. For abutments, this is also the present day approach.

7.3.4 The current Railtrack Line Code of Practice RT/CE/C/015 The Assessment of Underbridge Capacity (issue 1) utilises the factors contained in MEXE: namely a factor of 0.9 for an arch supported on one abutment and one pier and 0.8 for an arch supported on two piers. It is therefore considered reasonable that, to avoid any further debate on the issue, the Section 117 assessment of multi span arches should utilise these factors. However, it should be noted that these factors apply to multi span arches where the piers do not show any signs of significant distress. To cater for defects that may be present in the piers, it is considered that a qualitative assessment of the piers should be included to supplement the capacity obtained from the individual arch analyses.

7.3.5 For multi span arch assessment, account should also be taken of the points given above for single span arches.

7.4 Reinforced Concrete Bridges

7.4.1 Reinforced concrete bridges should be analysed to CP 114: Part II: 1969 The Structural Use of Reinforced Concrete in Buildings – the current Code at that time. However, the stress limitations should be as stated in BE4 Part 1 clause 304(d). These stresses apply irrespective of the age of the bridge as stated in the Code by reference to clause 301(a). Comparison with CP114 shows that these stresses appear to reflect a concrete with a 28-day strength of 15N/mm$^2$. This reflects the approach adopted in BD 21 in cases where the strength of the concrete is not known. However, where there is evidence to support the use of a higher strength concrete, then the appropriate strength should be utilised. In general, this should reflect the strength utilised in the BD 21 analysis.

7.5 Metallic Bridges

7.5.1 BE4 is most detailed with respect to metallic bridges as these form a significant proportion of the bridge stock constructed prior to 1922. As in BD 21, distribution factors are provided for determining the loading dispersal/distribution for standard beam decks and for troughing decks. For beam decks, BE4 also details the approach that should be adopted when the distribution curves are not applicable.

7.5.2 Permissible stresses should be in accordance with those detailed in BE4 Part I clause 304. It should be noted that where reference is made to BS 153: Part 3B, stresses are based on Table 1, case II. For most steel or wrought iron elements, this results in an increase of the permissible stresses by 25%.

7.5.3 The method of analysis of the metal members should be as stated in BE4 Part I clause 305. Methods of calculation for steel and wrought iron are specified in clause 305(b)(i), that is generally in accordance with the version of BS153 current in 1967. However, where bridge details do not comply with BS153, “the variations shall be taken into consideration and due allowance made in assessment”.

7.5.4 Thus, where half-through girder bridges have construction details not covered by BS153 (e.g. cross girder locations not coincident with main girder stiffeners), it is considered appropriate to determine girder top flange effective length in accordance with the British Rail internal document *1963 Addendum to BS 153*, because this represents best assessment practice at the time. The intention to use the *1963 Addendum* should be stated in the AIP. Where details of the bridge do comply with BS153, the *1963 Addendum* should not be used because clause 305 requires calculations to be based on BS153 in such cases.

7.5.5 A common type of superstructure consists of metallic beams supporting jack arches. BE4 is evidently intended to cover this form of construction since jack arches are referred to in Part I clause 303, subclauses (b)(ii) and (c)(i). Nothing further, however, is stated as regards assessment methods for jack arches; in particular, tie bars are not mentioned at all. It is therefore considered that jack arches and their associated tie bars (if any) should be assessed against the general requirements of clause 301(b) that “the structure shall be examined for possible faults . . . and allowance made for its condition when the carrying capacity is assessed”.

7.5.6 In accordance with the above, lack of edge-bay tie bars or equivalent lateral restraint should not in itself be taken as warranting a Section 117 assessment “failure”. The approach to be taken for the jack arches and the tie bars should be as follows:

a) Jack arches which do not carry highway loading as specified in BE4 should be assessed only for self-weight (plus footway loading if crowds are expected – see clause 302(c)).

b) If a jack arch supports highway loading and is in good condition, the arch and its supports should generally be deemed to “pass” BE4 qualitatively.

c) If a jack arch is not in good condition, this should be taken into account in the assessment of the arch and its supports, but it should not necessarily be “failed”. Signs of significant distress due to traffic loading may warrant a “failure”, but a modest degree of deterioration due to weathering or water penetration should generally not.

d) If edge-bay tie bars are present and in good or fair condition, they should generally be deemed to “pass” BE4 qualitatively (regardless of whether or not they comply with the empirical criteria given in Current Information Sheet (CIS) 22).

e) If there are no tie bars and there is no evidence that they were ever provided, tie bars do not form a structural element of the bridge and so their absence is immaterial to a qualitative assessment.

f) If there are no tie bars but there is evidence of their presence in the past, or if tie bars are present but in poor condition, consideration should be given to justifying a BE4 “pass” on the basis of one or both of the following:
   (i) the presence of an unloaded edge bay beyond the outermost bay carrying highway loading – it may be justifiable to deem from inspection that this provides adequate restraint to the adjacent loaded bay, or the degree of support could be investigated by doing simple hand calculations of the type likely to have been done in the late 1960s;
   (ii) the presence of a “stocky” edge beam which has sufficient strength (in conjunction with the residual strength of the tie bars if any) to resist lateral thrust from the jack arch – this should be determined from simple hand calculations of the type likely to have been done in the late 1960s and should thus generally ignore lateral deflection of the beam or any possible deterioration of the structure under repeated loading.

## 7.6 Composite (Metal/Concrete) Bridges

7.6.1 BE4 makes no reference to composite action between metal and concrete. At the time of SI 1705, there was a Code of Practice for Composite Beams for Bridges – CP 117: Part 2: 1967. However, it should be recognised that most “composite” bridges were probably not designed as such, as no positive shear connection was provided. Therefore use of CP 117 may not be appropriate for the majority of cases. In addition, the condition of the bridge may be such as to preclude composite action; this should have been appraised in the BD 21 assessment. It should also be noted that CP 117 does not apply to filler beam decks.

7.6.2 In view of the above, all proposals for Section 117 assessments of composite metal/concrete bridges should be agreed with Railtrack at an early stage, before completion of the AIP documentation.

7.7 <u>Pre-tensioned Prestressed Concrete Bridges (with and without infill concrete)</u>

7.7.1 No reference is made to prestressed concrete in BE4. However, at the time of SI 1705, prestressed concrete bridges were normally designed in accordance with CP 115: 1969 *The Structural Use of Prestressed Concrete in Buildings* or CP 116: 1969 *The Structural Use of Precast Concrete*, as appropriate. Section 117 assessments of pretensioned concrete bridges should therefore be undertaken in accordance with the relevant one of these documents. The points given below should be read in conjunction with the appropriate Code as they clarify the provisions of the Code and reflect the practice at that time.

7.7.2 Permissible bending tensile stresses should accord with Table 5 for "maximum working load often occurring and/or of long duration", i.e. tensile stresses are permitted in the prestressed concrete.

7.7.3 In the transmission zone, shear should be checked either by treating the section as reinforced concrete and therefore utilising CP114 or considering it as a prestressed concrete section with a reduced prestress force whose value will depend on the section's distance from the end of the beam. It is considered reasonable to utilise whichever gives the greater strength.

7.7.4 Shear should be checked at critical sections, ie points of maximum shear and changes of section including changes of links. Shear should be checked under working loads with erection history considered. The principal tensile stress in the beam should be calculated at different positions down the beam in order to determine the critical location. The following formula should be used:

Maximum principal tensile stress = $f_{cd}/2 - \sqrt{[(f_{cd}/2)^2 + f_{cv}^2]}$

where

$f_{cd}$ = direct stress in concrete at level considered (value after losses)

$f_{cv}$ = shear stress at level considered = Vay/Ib (i.e. as used for longitudinal shear)

where

V = shear force

a = area of concrete outside the point considered

y = distance from centroid of area a to centroid of section considered

I = second moment of area of section about neutral axis

b = width of section at point considered. This will be web width when the beam alone is checked but for infill decks will normally be beam spacing when imposed load is considered.

This maximum principal tensile stress should be calculated separately for the self-weight plus wet concrete load taken on the beam alone and the other loads taken on the composite (for infill decks) section and the resulting stresses should then be added. The infill should be checked to CP 114.

7.7.5 With bending stresses limited as detailed in 4.9.2 above, sections can be treated as "uncracked" for the purpose of shear calculations of simply supported beams.

7.8 Post-tensioned Concrete Bridges, Timber Bridges

7.8.1 The method of assessment for post-tensioned concrete bridges and timber bridges (or bridges with timber components) should be agreed with Railtrack.

## 8 Loading for Section 117 Assessments

8.1 The live loading to be applied for Section 117 assessments is defined in SI 1705 (for bridges in England and Wales) and depends on a number of factors, principally the date of construction (or date of last reconstruction) and class of road carried. The information given in the Table below is derived from SI 1705. These loadings should be used for
bridges in Scotland as well as those in England and Wales. For multi-span bridges, the loading (and the method of assessment) appropriate to the date of each span should be applied to that span.

| Date of construction (or date of last reconstruction) | Road class in 1972, as given in the Local Govt Act 1966 Sec 27(2) | Relevant Schedule/Part of SI 1705 | Live loading (see Note 1) |
|---|---|---|---|
| Before Jan 1955 | All | Part I of Schedule 1 | BE4 C&U vehicles (See Note 2) |
| Jan 1955 - Jan 1962 + SI 1705 Schedule 2 list (1962 - 1972) | All | Part II of Schedule 1 | BS 153 Pt 3A, HA only (see Note 3) |
| Jan 1962 → (except Schedule 2 list) | Principal: classified as 45HB unit road | Part IV of Schedule 1 | BS153 Pt 3A, HA + 45HB (see Note 3) |
| | Principal: not classified as 45HB unit road | Part III of Schedule 1 | BS 153 Pt 3A, HA + 37½HB (see Note 3) |
| | Not principal | Part II of Schedule 1 | BS 153, Pt 3A, HA only (see Note 3) |

Note 1: These loadings do not apply where the road carried by the bridge is subject to a traffic weight prohibition under section 1, 6, 12 or 17 of the Road Traffic Regulation Act 1967(b) in place or pending at the time SI 1705 came into force.

Note 2: "C&U vehicles" loading is as given in BE4 Part I clauses 201, 202 and 302. If using the 1967 version of BE4, the dimension shown in clause 202 as 4ft 6in should be changed to 4ft 0in (September 1970 amendment to BE4).

Note 3: HA and HB loadings are to BS 153: Part 3A (amended to 1968), modified by MoT Memo 771 (reprinted 1968) (except paragraphs 4i, 4ii and 4iii).

8.2 Referring to Note 1, it should be noted that some weight limits on or near bridges are imposed for environmental reasons rather than bridge strength reasons and it is important to ascertain if this is the case when a bridge is reported as having a weight limit. Further advice should be sought from Railtrack in all cases where a weight limit notice is displayed on or near a bridge.

8.3 For some bridges an assessment of the HB capacity will be required. This depends on the classification of the road carried at the time of SI 1705. Railtrack will normally undertake the determination of this classification.

8.4 Regarding footway loading, in the case of pre-1955 bridges BE4 clause 302(a) makes clear in the text and diagram that Construction and Use vehicles loading is intended to be applied to the road carriageway only, not to the footways or verges. The following should therefore apply for all assessments to BE4:

- vehicle loading should be taken as applied only to the carriageway of the road;
- accidental wheel loading on footways/verges should not be taken into account;
- footway loading should not be taken into account unless notified by Railtrack that crowd loading could reasonably have been expected at the time of SI 1705;

8.5 BE4 makes no mention of secondary loading or load effects such as those due to wind, temperature, settlement, centrifugal forces, traction and braking, accidental impact or parapet containment. Such loading or load effects should therefore not be taken into account for assessments to BE4.

8.6 It is not anticipated that very many post-1955 bridges will need to be subject to Section 117 assessment. For those which do, footway loading and secondary loading effects should be taken into account as given in BS 153 (amended as specified in SI 1705).

## 9 Substructures, Foundations, Spandrel Walls, Columns and Piers

9.1 BE4 Part 1 (and SB9/74 Part 1) state that "in arriving at an assessment of a bridge, the foundations, substructure and superstructure should all be considered" (clause 301(c)). However, it is only in relation to the superstructure that "the constituent parts should all be investigated for the loading they carry". This implies that all other parts of the bridge should be subject only to a qualitative assessment (essentially in line with the BD 21 approach).

9.2 However, it should be noted that quantitative assessment is explicitly required for mild steel and wrought iron columns (clause 305(b)(i) and for cast iron columns (clause 305(b)(ii)(2)). Slender reinforced concrete columns (i.e. those where buckling failure is a possibility) should also be assessed quantitatively, but stocky concrete columns or piers should be assessed as if they were masonry structures.

9.3 Masonry piers of multi-span arches should be assessed qualiitatively. (The effect of the piers on the quantitative assessment of the arch capacity should be taken into account as given in 6.3.4 above.)

9.4 Abutments, wingwalls, spandrel walls and foundations should be assessed qualitatively.

## 10 Defects

10.1 Each bridge should be assessed on the basis of the inspection results obtained for the BD 21 assessment. BE4 states that "the structure shall be examined for possible faults, e.g. corrosion, settlement, faulty material, and allowance made for its condition when the carrying capacity is assessed" (Cl 301(b)). Any reduction in section of components used for the BD 21 assessment should be applied to the Section 117 assessment where quantitative methods are required.

10.2 In many cases, qualitative results for the Section 117 assessment are likely to mirror those obtained for the BD 21 assessment. However, there may be situations where it is appropriate to take into account that the Section 117 assessment loading is significantly less than the actual loading a bridge is likely to have experienced in the recent past (say 38 ton), and that this may be used to justify a favourable qualitative result.

## 11 Computer Analysis Methods

It is considered reasonable that computer methods of analysis can be used if they reflect methods which were generally available and used in practice in 1972. This would include two-dimensional grillage analysis etc., which in principle could be achieved by hand calculation and/or by using published charts or graphical methods. However, finite element analysis and other "advanced" numerical methods are not considered appropriate. Nor are techniques such as yield line analysis considered in general appropriate because they are "ultimate" analyses which do not accord with the "elastic" analysis approach of BE4 and other Codes of that time.

## 12 Category of Check

For Section 117 assessments the category of check should normally be Category 1.

# RAILTRACK
# BRIDGEGUARD 3

**CURRENT INFORMATION SHEET NO 27**
**SUBJECT: HB CAPACITY WITH MEXE**
**DATE: February 2000**
**STATUS: FINAL**

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**3. Approved for issue by Railtrack**

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## CURRENT INFORMATION SHEET NO 27

## SUBJECT: HB CAPACITY WITH MEXE

**This Current Information Sheet is issued for guidance purposes only: it is not mandatory. The Assessor must be satisfied that the advice given in this Information Sheet is appropriate to the structure in question.**

To determine the allowable number of units of HB that can be carried by a single span masonry arch bridge the following approach may be adopted.

**Allowable No of HB Units = Modified Axle Loads determined using the Modified MEXE Method x 1.6**

Where Axle Lift Off or Centrifugal Effects apply to the determination of the Assessment Live Load Capacity in accordance with Clauses 3.2.7 and 3.2.9 of BA 16/97, then the Modified Axle Load shall be suitably amended prior to application of the 1.6 factor.

The Modified MEXE Method shall be applied in accordance with BD 21/97 and BA 16/97 with due consideration of Current Information Sheets No 20 and 21.

The 1.6 factor takes into account a comparison between HA and HB vehicles with respect to vehicle widths, load dispersal and distribution, axle spacings, overloading factors and impact factors. This factor is deemed to represent a realistic lower bound value.

# APPENDIX A - SUPPORTING JUSTIFICATION

## 1. INTRODUCTION

Current Information Sheet No 27 identifies a method for determining the HB capacity of a single span masonry arch bridge using the Modified MEXE Method. This assumes the application of a 1.6 factor to the Modified Axle Loads. This factor is derived by comparing the geometry and load factors associated with HA (C & U vehicles) and HB vehicles as detailed below.

## 2. VEHICLE WIDTH, LOAD DISPERSAL AND LOAD DISTRIBUTION

Although it is known that Pippard bases the Modified MEXE Method on theoretical work, the MEXE monogram is a gross simplification of Pippard's equation. Developed with the aid of real load tests, the Modified MEXE Method is an empirical method. It is therefore not known exactly how the MEXE Method has treated the effects of load dispersal, load distribution or the presence of adjacent vehicles.

From Pippard's research[1] it can be inferred that both load dispersal and load distribution were considered but there is no explicit reference to adjacent vehicles. In comparing HB with HA vehicles, width conversion factors can be determined which are dependent on the consideration of these three parameters (load dispersal, load distribution and adjacent vehicles) as shown in Figure 1.

A realistic lower bound conversion factor of 1.2 is proposed which assumes a reasonable upper limit of 2.5m depth of fill.

## 3. AXLE SPACING

BA 16/97 Annex B describes how the critical loading of different numbers and spacing of axles on arches can be derived by comparing their effects on a simply supported beam with a span equal to half the arch 'span'. The basic axle loads given are based on a two-axle bogie but it omits to say what the spacing is. However, BA 16/97 Figure 3/5a shows that the factor for a single axle becomes one at an arch span of 4m which corresponds to an equivalent beam span of 2m. The critical load case for two point loads on a simply supported beam arises with the loads off centre by a quarter of their spacing; you can check this by writing an expression for the moment and differentiating to get the worst case. If, however the span is less than 1.7 times the spacing the maximum moment for the two-load case is less than the moment under a single load on centre. It follows that the fact that BA 16/97 gives the factor for a single axle as becoming 1 at an arch span of 4 implies that the axle spacing it uses is 4 divided by 2 to get the equivalent beam and by 1.7 to get the spacing. This comes to 1.176m. This suggests it is based on 1.21m (4ft) and rounded. This corresponds closely to the typical spacing in BD 21/93 Appendix D Table D1 of 1.2m.

By comparing the effect of a bogie with wheels at 1.2m centres on a beam of half the arch span, a correction factor for any bogie can be derived. The factor for the HB spacing of 1.8m is shown in Figure 2.

A lower bound conversion factor of 1.07 is proposed which is based on the maximum span of 18m allowable in MEXE.

## 4. OVERLOADING AND IMPACT

To acknowledge the reduced probability of overloading between HA and HB vehicles and the reduced frequency of abnormal loads, a conversion factor based on $\gamma_{fl}$ factors can be developed. It may be argued that this may not be applicable since MEXE is not strictly a limit state approach. However, working stress codes generally allow some overstress and, as this amounts to the same thing as using load factors, this approach is probably reasonable.

Using the BD 37/88 values for $\gamma_{fl}$ of 1.5 for HA and 1.3 for HB, produces a conversion factor of 1.154. However, using the BD 21/97 factors for the rigorous analysis of arches, 1.9 for HA, ignoring impact, and 2.0 for HB (Clause 6.20) produces a conversion factor of 0.95.

The reduced speed of HB vehicles is also recognised in BD 21/97 (Appendix H and Clause 6.2) by the absence of an impact factor. This compares with the need to apply a 1.8 impact factor to one axle of an HA vehicle. However, MEXE is derived from the application of equal axles spaced at approximately 1.2m spacings which is not consistent with HA Loads (C & U vehicles) for which BD 21/97 Appendix D would give the following axle loads:

| | | | | |
|---|---|---|---|---|
| 1.8 x 10.17t | ← | 1.85m spacing | → | 10.17t or |
| 1.8 x 11.5 t | ← | 1.3m spacing | → | 7.5t |

Consequently, it is not apparent whether impact factors were considered at all in the derivation of MEXE. However, it would appear reasonable to consider some benefit from the reduced impact generally associated with HB Loading. Assuming an equal spread of the impact factor to the two HA axles and assuming no impact on HB loads, a conversion factor of 1.4 is reached.

Combining both overloading and impact factors, a total conversion factor between 1.154 x 1.4 = 1.62 and 0.95 x 1.4 = 1.33 is possible. However it is understood that an internal Department of Transport Document AHB16 'Assessment of Masonry Arch Bridges under Abnormal Loads' states in Paragraph 2.2 that, because abnormal loads are so infrequent, a load conversion factor of 1.25 can be allowed in calculating the allowable abnormal axle load from that given by the MEXE method. It is therefore proposed to adopt this latter conversion factor as a conservative approach.

## 5. OVERAL FACTOR – CONCLUSIONS

By combining the above factors, a global conversion factor can be derived, equal to:

$$\mathbf{1.2 \times 1.07 \times 1.25 = 1.6}$$

References:

(1) *The Approximate Estimation of Safe Loads on Masonry Bridges* – Prof AJS Pippard, Civil Engineer in War, Vol 1, 365, Institution of Civil Engineers, London, 1948.

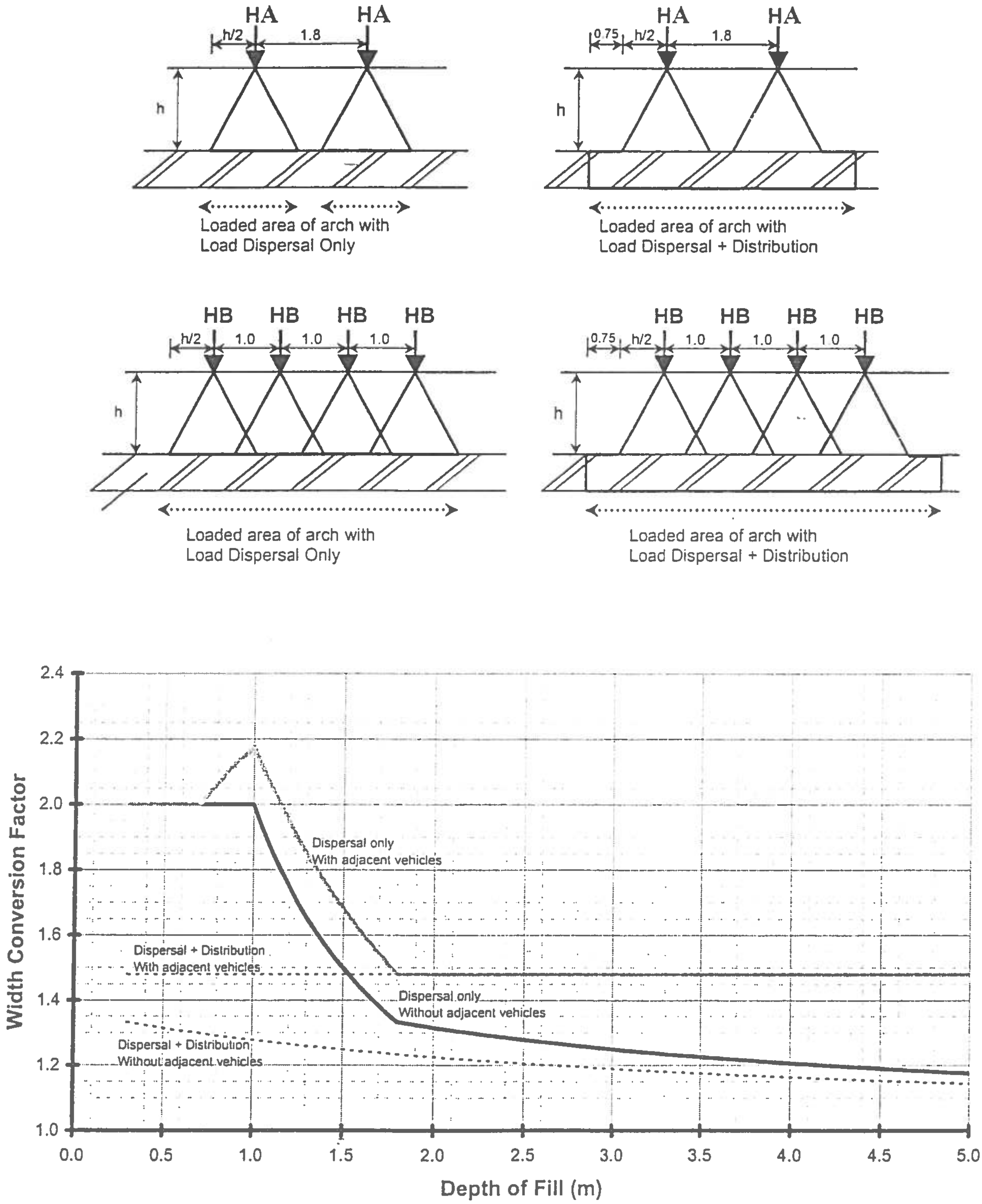


Figure 1: HB/HA Conversion Factor for Vehicle Width, Load Dispersal and Distribution

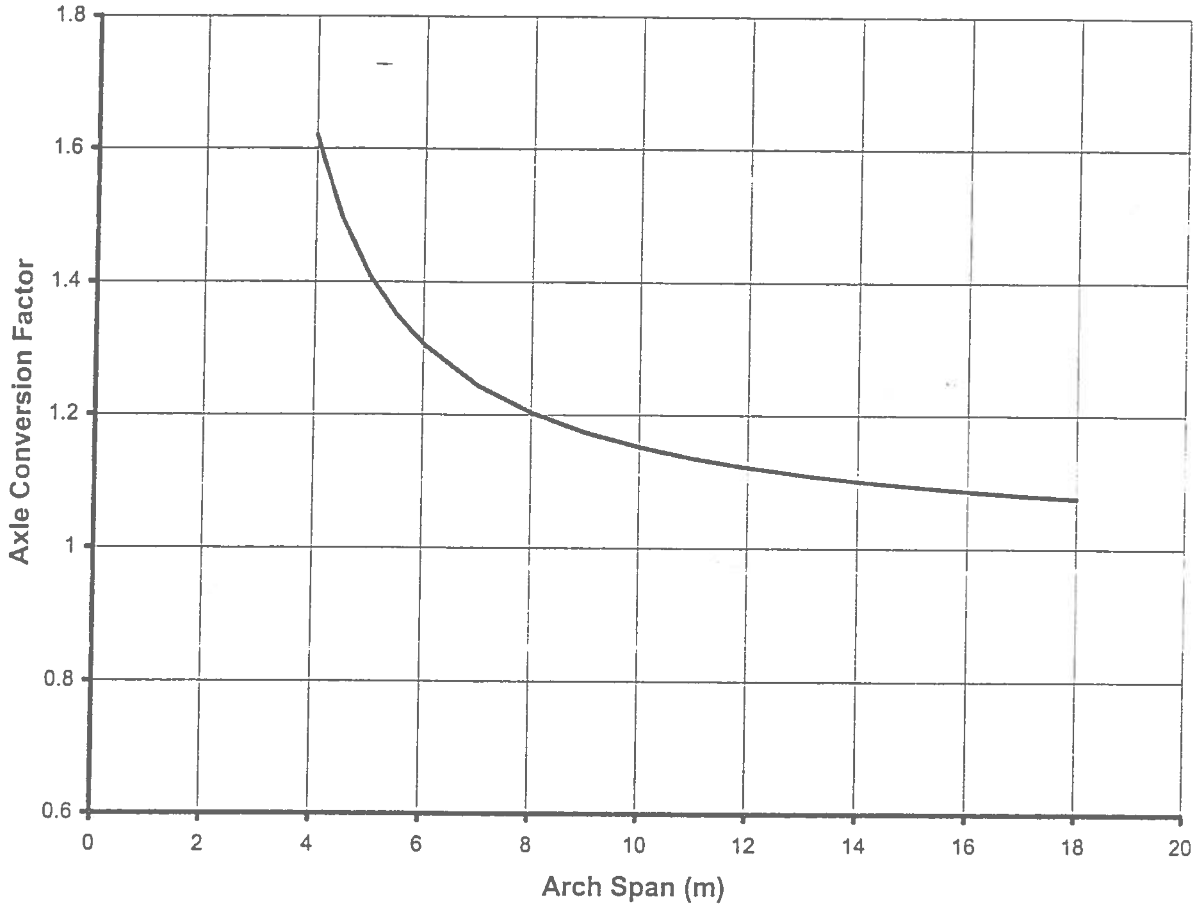


**Figure 2: HB/HA Conversion Factor for Axle Spacing**

## APPENDIX B

## ALTERNATIVE PROPOSALS

The following proposals for undertaking HB assessments using MEXE have been prepared by:

Lancashire County Council
Aspen Burrow Crocker (effectively refers to LCC proposal)
Mouchel (effectively refers to LCC proposal)
Mott Macdonald
Bullen Consultants
W S Atkins

These are included for information only.

From: CHIEF ENGINEER (BRIDGES)

HPB/3145/2/ERLC

To: BRIDGES STAFF IN HP SECTION (Copy to GE/PE)

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## LANCASHIRE COUNTY COUNCIL
## BRIDGE CLIENT INSTRUCTION NO. BCI 9.1
## ASSESSMENT OF ARCH BRIDGES FOR ABNORMAL LOADS

1. The Modified MEXE Method given in paragraph 5 of DTp Advice Note BA16/84 (with Amendment No. 1) shall be used for assessing arch bridges for abnormal loads provided the span of the arch does not exceed 18m, and the arch is not flat or appreciably deformed. Otherwise the alternative methods given in paragraph 8.7.2 of DTp Departmental Standard BD 21/84 (with Amendment No. 1) shall be used.

2. In using the Modified MEXE Method for abnormal loads the following assumptions shall be used (for background see Note on file HPB/3050 dated 11 February 1992):-

   (a) The modified axle load shall be multiplied by two factors:-

      (i) a factor of 1.25 to allow for the infrequent nature of abnormal loads.

      (ii) a factor to allow for the different lateral dispersion of abnormal loads and C & U vehicles equal to:-

$$\frac{\text{(Overall width across tyres of abnormal load (m) + 0.5)}}{3.0}$$

   (b) In using the simple method given in Appendix AN/B of DTp Advice Note BA16/84 (with Amendment No. 1) to allow for the effects of multiple axles the two axle bogie with an axle spacing of 1.83m shall be used for comparing bending moments.

3. The allowable number of units of Type HB loading obtained by using the above assumptions is equivalent to the Modified Axle Load multiplied by a factor of 1.6.

Signed (Author)........E. R. L. Cole........ Date........6 March 1992........

Name (Capital letters)........DR E. R. L. COLE........

Initialled by PE/GE........—........



BCIABD

LANCASHIRE COUNTY COUNCIL

COUNTY BRIDGES

DESIGN SHEET

| Scheme / Structure | Designed By | Checked By: Theory | Checked By: Arithmetic | Date | Sheet No. of sheets |
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| Scheme: ASSESSMENT OF ARCH<br>Structure: BRIDGES FOR ABNORMAL LOADS. | Elu | | | 11-2-92 | |

## Lateral Distribution of Abnormal Loads

In using any assessment method for arches it is important to decide what width of arch is carrying the load. The previous Assessment Code, Technical Memorandum BE 3/73, states in Part III paragraph 8(e)(xi) that the MEXE Method was based on an 8ft (2.44m) width of arch. The current Assessment Code, Departmental Standard BD 21/84, states in paragraph 8.7.2.3 that in using alternative methods to the Modified MEXE Method axles of C&U vehicles shall be assumed to have a 1.8m track and shall be located within 2.5m transverse width with a 0.7m minimum spacing between the track width of adjacent vehicles.

Departmental Standard BD 37/88 states in paragraph 6.3.1 that HB vehicles shall be used to have a 3.0m overall track and 3.5m overall width.

Paragraph 6.3.2 gives contact areas for the wheel loads.

$$25 \text{ Units HB} - \text{Diameter} = \sqrt{\frac{2500 \times 25}{1.1} \times \frac{4}{\pi}} = 269 \text{mm}$$

$$45 \text{ Units HB} - \text{Diameter} = \sqrt{\frac{2500 \times 45}{1.1} \times \frac{4}{\pi}} = 361 \text{ mm}$$

An internal DTp Document AHB 16 'Assessment of Masonry Arch Bridges under Abnormal Vehicles' states in paragraph 4.5 that when using the MEXE Method the allowable axle load can be increased by a factor equal to width over abnormal load wheels divided by 8ft (2.44m). Calculate this factor for HB vehicle :-

$$\text{Factor for 25 units} = \frac{3.269}{2.44} = 1.34$$

$$\text{Factor for 45 units} = \frac{3.361}{2.44} = 1.38$$

However the MEXE Method is based on work by Pippard and Heyman in this paper 'The estimation of the strength of masonry arches', Proc ICE Part 2 Dec 1980 states that the effective arch rib supporting the load will depend on the cover to the arch ie. dispersion of load through the fill is being allowed. Therefore as the width of arch supporting each lane is likely to be significantly greater than 2.5m and HB vehicles will have clearance to parapet or other vehicles it is suggested that it would be more appropriate to calculate the above factors assuming additional dispersal say 250mm on each side.

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COUNTY BRIDGES

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Revised factor for 25 units $= \frac{3.769}{2.94} = 1.28$

Revised factor for 45 units $= \frac{3.861}{2.94} = 1.31$

Frequency of Abnormal Loads

The above internal DTp Document AHB16 also states in paragraph 2.2 that a factor of 1.25 can be allowed in calculating the allowable abnormal axle load from that given by the MEXE Method because abnormal loads are so infrequent.

Combined Factor

As the whole basis of the MEXE Method is empirical there does not seem any justification for using different factors for different no of units of HB, so use the lowest.

Combined factor $= 1.28 \times 1.25 = 1.60$

Suggest wording in TAL form should be :-

'In determining allowable no of HB units the modified axle load determined by the Modified MEXE Method shall be multiplied by a factor of 1.6 to allow for different lateral dispersion and frequency of occurrence.

Effects of Multiple Axles on Arches

In using the simple method given in Appendix AN/B of Departmental Advice Note BA 16/84 the modified axle load determined by the Modified MEXE Method shall be used assuming that the double axle bogie has an axle spacing of 4 ft (1.22m) (see paragraph 2.2 of internal DTp Document AHB 16 which refer to C&U Regulations. However this assumption may need to be modified in light of the change of the C&U Regulation (cf. Appendix DS/A of BD 21/84 with paragraph 2.2 of internal DTp Document AHB 16). Whilst the Motor Vehicles (Construction and Use) Regulations 1978 Schedule 7 Part IV allows a gross bogie weight of 18.3 tonnes for a minimum axle spread of 1.2m it is not one of the load cases used in the assessment of arches by BD 21/84 (see Appendix DS/A of BD 21/84). Also as Table 5.6 of BA 16/84 has allowable axle

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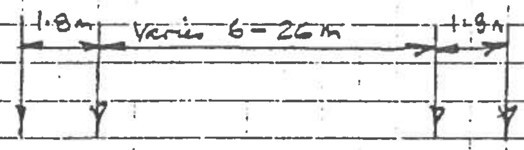
loads of greater than 9 tonnes (which are associated with minimum axle spacings of 1.85m) if the weight restriction on the bridge is greater than 25 tonnes and abnormal loads are unlikely to be travelling on roads with bridges which are weight restricted below this value, then the 1.85m axle spacing is possibly more appropriate. Calculations show that if a 1.85m axle spacing is used instead of a 1.2m axle spacing then the permissible abnormal load is likely to be reduced by 7 to 14 %. as the span reduces from 18 to 10 m.

For HB vehicles the axle spacing is as follows (see BD 37/88 Fig 12) :-




Hence for spans up to 18.0m (the max allowed for MEXE) the second bogie will not be on the arch, so the HB load can be considered purely as a double axle. As the axle spacing of 1.8m is so close to the axle spacing of 1.85m used in the MEXE analysis the permissible no of HB units can be obtained simply by multiplying the modified axle load by 1.6 with no correction for longitudinal spacing.

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COUNTY BRIDGES

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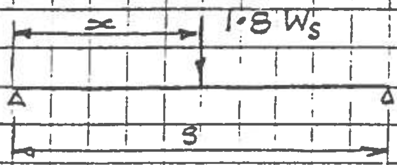
Consider single double and triple axle from Appendix DS/A of BD21/84 applied to a simply supported span equal to half the arch span (see Appendix AN/B of BA 16/84. Use Nominal axle weight conversion factor from Table 8.2 of BD 21/84

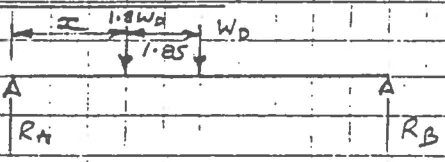
Single Axles.




Max moment arises when $x = S/2$ and occurs under load.

$$M_S = 1.8\,W_S \cdot \frac{S}{4}$$

Double Axles




$$R_A = 2.8\,W_D - \frac{x}{S}\,1.8W_D - \frac{(x+1.85)}{S}\,W_D$$

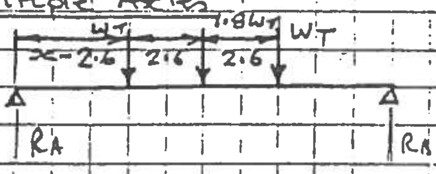
$$= W_D\left(2.8 - 2.8\frac{x}{S} - \frac{1.85}{S}\right)$$

$\therefore$ Moment under $1.8W_D$ $(M_D)$ $= R_A x = W_D x\left(2.8 - 2.8\frac{x}{S} - \frac{1.85}{S}\right)$

$$\frac{dM_D}{dx} = 2.8W_D - 5.6\frac{x}{S}W_D - 1.85\frac{W_D}{S} = 0 \text{ (for max)}$$

$$x = \frac{2.8S - 1.85}{5.6} = \frac{S}{2} - 0.37$$

Triple Axles




Max moment arises when central axle is at midspan and occurs at midspan.

$$R_A = W_T + \frac{1.8W_T}{2} = 1.9W_T$$

$$M_T = 1.9W_T\,x - W_T\,2.6 = W_T(0.95S - 2.6)$$

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COUNTY BRIDGES

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Compare bending moments on 18m span arch (longest span for which MEXE Method is appropriate) for 40 tonne lorry from Table 5.6 of BA 16/84.

1. $M_S = 1.8 \, W_S \, \frac{S}{4} = 1.8 \, . \, 11.5 \, . \, \frac{9}{4} = 46.6$ tonne-m

2. For double axles $x = \frac{S}{2} - 0.37 = \frac{9}{2} - 0.37 = 4.13$ m

$M_D = W_D x \left(2.8 - 2.8\frac{x}{S} - \frac{1.85}{S}\right) = 10 \, . \, 4.13 \left(2.8 - 2.8 \, . \, \frac{4.13}{9} - \frac{1.85}{4.13}\right)$

$= 44.1$ tonne-m

3. $M_T = W_T (0.95S - 2.6) = 8 (0.95 \, . \, 9 - 2.6) = 47.6$ tonne-m

For a 10.4m span arch (shortest span for which triple bogie is appropriate)

1. $M_S = 1.8 \, W_S \, \frac{S}{4} = 1.8 \, . \, 11.5 \, . \, \frac{5.2}{4} = 26.9$ tonne-m

2. For double axles $x = \frac{S}{2} - 0.37 = \frac{5.2}{2} - 0.37 = 2.23$ m

$M_D = W_D x \left(2.8 - 2.8\frac{x}{S} - \frac{1.85}{S}\right) = 10 \, . \, 2.23 \left(2.8 - 2.8 \, . \, \frac{2.23}{5.2} - \frac{1.85}{5.2}\right)$

$= 27.7$ tonne-m

3. $M_T = W_T (0.95S - 2.6) = 8 (0.95 \, . \, 5.2 - 2.6) = 18.7$ tonne-m

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Alternatively compute bending moments on 18 m span arch for any modified axle load $W_m$ using Fig 5.5a of BA 16/84.

1. $M_S = 1.8 W_S \frac{S}{4} = 1.8 \cdot 1.75 W_m \cdot \frac{9}{4} = 7.1 W_m$ tonne-m

2. $M_D = W_D x \left(2.8 - 2.8\frac{x}{s} - \frac{1.85}{s}\right) = 1.0 W_m \cdot 4.13 \left(2.8 - 2.8 \cdot \frac{4.13}{9} - \frac{1.85}{4.13}\right)$

   $= 4.4 W_m$ tonne-m

3. $M_T = W_T (0.95 s - 2.6) = 0.75 W_m (0.95 \cdot 9 - 2.6) = 4.5 W_m$ tonne-m

For 10.4 m span :-

1. $M_S = 1.8 W_S \frac{S}{4} = 1.8 \cdot 1.6 \cdot \frac{5.2}{4} = 3.7 W_m$ tonne-m

2. $M_D = W_D x \left(2.8 - 2.8\frac{x}{s} - \frac{1.85}{s}\right) = 1.0 W_m \, 2.23 \left(2.8 - 2.8 \cdot \frac{2.23}{5.2} - \frac{1.85}{5.2}\right)$

   $= 2.8 W_m$ tonne-m

3. $M_T = W_T (0.95 s - 2.6) = 0.82 W_m (0.95 \cdot 5.2 - 2.6) = 1.9 W_m$ tonne-m

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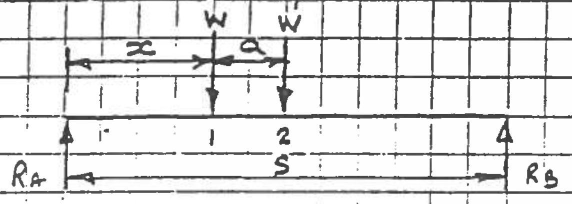
Compare bending moments produced by double axles at 1.85m c/c to double axles at 1.2m c/c.




$R_B = \dfrac{W(2x+a)}{S}$ $\qquad R_A = \dfrac{W(2S-2x-a)}{S}$

Moment under 1st load $(M_1) = R_A x$

$= \dfrac{Wx(2S-2x-a)}{S}$

$\dfrac{\partial M_1}{\partial x} = \dfrac{W}{S}(2S-4x-a) = 0$ (for max)

$\therefore x = \dfrac{2S-a}{4}$

Now for max span of 18m $\quad S = 9.0$

For $a = 1.2m$

$x = \dfrac{18.0-1.2}{4} = 4.2m$

$M_1 = \dfrac{W.\,4.2.\,(18.0-8.4-1.2)}{9.0} = 3.92\,W$

For $a = 1.85m$

$x = \dfrac{18.0-1.85}{4} = 4.04m$

$M_1 = \dfrac{W.\,4.04\,(18.0-8.08-1.85)}{9.0} = 3.62W$ $\qquad$ ie. 7.6% less

Now for a span of 10m $\quad S = 5.0$

For $a = 1.2m$

$x = \dfrac{10.0-1.2}{4} = 2.2m$

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$$M_1 = \frac{W.\ 2.2(10.0 - 4.4 - 1.2)}{5.0} = 1.94\,W$$

For $x = 1.85$ m

$$x = \frac{10.0 - 1.85}{4} = 2.04\text{ m}$$

$$M_1 = \frac{W.\ 2.04(10.0 - 4.08 - 1.85)}{5.0} = 1.66\,W$$

ie. 14.4% less

# CURRENT INFORMATION SHEET NO 29

**SUBJECT: CLARIFICATION OF INTERPRETATION OF BD 44/BA 44 FOR SHEAR IN SIMPLY SUPPORTED PRETENSIONED BEAM DECKS**

## 1. INTRODUCTION

It is apparent from the review of assessment calculations that approaches for assessing shear strength in pretensioned beams using BD 44/BA44 vary between assessing engineers. This variation has been considered and clarification of the key issues is given below.

## 2. PRETENSIONED BEAMS

BD 44/BA 44 allow shear to be checked at 1d from the support and no closer (BD 44 Clause 6.3.4.4). It is confirmed that in pretensioned beams when this clause is used the prestress at 1d from the support can be used. Although the prestress may be reduced closer to the support because of the transmission length this does **not** have to be considered. This approach is safe because of the enhancing effect of the short shear span and would only be invalid if more than 50% of the shear force at the support is due to load applied within 1d.

## 3. INFILL

Where the shear strength of the infill concrete is added to that of the pretensioned beam in accordance with Clause 7.4.2.2 of BA 44 and the steel area $A_s$ used to calculate its strength is taken as zero, short shear span enhancement given in BD 44 Clause 5.3.3.3 can be used in the infill.

## 4. ALTERNATIVE APPROACH

As an alternative, Clause 6.3.4.1 of BD 44 allows the section to be considered as ordinary reinforced concrete but this would not normally be beneficial unless additional secondary reinforcement was provided.

# APPENDIX A

# AUDIT TRAIL

# SHEAR IN PRETENSIONED BEAM DECKS

## 1. INTRODUCTION

It is apparent from assessment calculations reviewed by Gifford and Partners that approaches for assessing shear strength in pretensioned beam decks using BD 44/BA44 vary between assessing engineers.

The following provides background information to the recommended approach in the Current Information Sheet.

## 2. SHEAR CAPACITY OF PRESTRESSED BEAMS

Within a prestressed beam the end zone is treated in the same way as any other prestressed section except that the prestress force is reduced in proportion to the transmission length. For example, if the section considered were 400mm from the end and the transmission length were 600mm, two thirds of the normal prestress would be used.

This calculation suggests that the basic shear strength reduces as you move towards the end of the beam. When BS 5400 was being implemented in the early 1980s there was concern that this would interfere with short shear span enhancement. It was therefore conservatively decided not to allow the "check at 1d only" rule for prestressed sections. This was known to be a very conservative decision but was considered appropriate for design. BD 44 reverses the decision stating that shear need not be checked closer than 1d out providing any shear reinforcement calculated for the section at a distance d is continued up to the support. Thus BD 44 allows shear to be checked at 1d out which means the prestress at 1d out can be used. This interpretation of BD 44 has been confirmed by Clark[1] who was largely responsible for drafting BD 44 under contract to the Highways Agency.

BA 44 does not give any background to the 1d rule for prestressed concrete but for RC it says "tests indicate that, where $a_v < d$ the load is transferred to the support by direct strut action and the ultimate shear strength of the concrete rises sharply". The same applies to prestressed concrete. It applies whether or not there are any links and indeed it is not clear that the requirement for links not to reduce going into the support is justified as links so close to the support are ineffective as will be seen by considering strut and tie action.

The 1d rule is actually more important for prestressed concrete because the code does not give short shear span enhancement for this. However, there is ample evidence that short shear span enhancement does arise in prestressed concrete. Whilst for RC BS 5400 uses short shear span from 2d and BD 44 uses it from 3d, it can be shown that, unless more than 50% of the shear force at the support is due to loads applied within 1d of the support, short shear span enhancement from 1d would easily be sufficient to justify the1d rule in pretensioned beams. This is considered in Annex A.

## 3. SHEAR CAPACITY OF INFILL CONCRETE

Many of the types of beams under consideration form part of infill inverted T beam type bridges. For these, BA 44 allows the shear strength of the infill, considered as RC, to be added to that of the beams. As there is normally no reinforcement in the infill, the shear strength of this is calculated taking As as zero. This makes the requirements for reinforcement anchorage in Clause 5.8.7 (referred to in Clause 5.3.3.2) and also that in Clause 5.3.3.2 (short shear span enhancement) meaningless. Short shear span enhancement can therefore always be used.

Recent research suggests[2] that the strength of these types of beams is higher than the code rules indicate. The strength of the pretensioned beams themselves is also affected by short shear span enhancement which the code does not allow to be used. The assessed strength is therefore still likely to be conservative when enhancement is used on the infill strength.

## 4. REFERENCES

1. Clark L A. Private Communication. 20.09.00.

2. Cullington D W. Private Communications. 2000.

## ANNEX A

## JUSTIFICATION FOR 1d RULE USING SHORT SHEAR SPAN ENHANCEMENT

Say short shear span enhancement works from Kd.
Say transmission length is >1d (worst case) and beam does not extend significantly beyond support (conservative).

Ignoring tensile strength of concrete (conservative) basic shear strength of section uncracked in flexure is proportional to prestress and therefore, within transmission length, is proportional to distance from support.

| | | |
|---|---|---|
| If basic shear strength at d from support (with no short shear span enhancement) | = | V |
| Basic shear strength at x < d from support | = | xV/d |
| Corrected for short shear span enhancement | = | (Kd/x)(xV/d) |
| | = | KV |
| | > | V (provided K>1) |

Test results suggest that short shear span enhancement is similar to that in RC, ie it applies from about 3d and therefore K is approximately 3.

**BRIDGEGUARD 3**
**CURRENT INFORMATION SHEET NO 29**
**SUBJECT:CLARIFICATION OF INTERPRETATION OF BD 44/BA 44 FOR SHEAR IN SIMPLY SUPPORTED PRETENSIONED BEAM DECKS**
**DATE: OCTOBER 2000**
**STATUS: FINAL**

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**Part 2:** Endorsement Organisation: Pell Frischmann Consulting Engineers Ltd.

I certify that reasonable professional skill and care has been used in endorsing this document.

| | |
|---|---|
| Signed: | Title: |
| Name: | Date: |
| To be signed by the Senior Reviewing Engineer. | |

**Part 3:** Acceptance on behalf of Railtrack

I approve the implementation of this Current Information Sheet with respect to the Bridgeguard 3 programme only.

| | |
|---|---|
| Signed: | Title: |
| Name: I Bucknall | Date: |
| To be signed by Railtrack Project Delivery Technical Service and Innovation Team | |

**CURRENT INFORMATION SHEET NO 30**

**SUBJECT: USE OF BD 61 FOR COMPOSITE BRIDGE WITH SHEAR CONNECTION**

BD 61 shall be used with the following amendments:-

**1. 4.3 Limit State Requirements**

Delete all except the last paragraph and insert before it:

Shear connectors shall be assessed to meet the requirements of the serviceability limit state and, where required by 6.3.4 of this standard, the ultimate limit state as specified in this standard. Other aspects of composite construction will be checked at the ultimate limit state only.

Except as specified above, all structural steelwork shall be checked for compliance with BD 56 (DMRB 3.4.11) at the ultimate limit state only and all concrete shall be checked for compliance with BD 44 (DMRB 3.4.14) at the ultimate limit state only.

**2. 5.3 Longitudinal Shear**

**2.1. 5.3.2.1 Nominal Strengths of Shear Connectors Embedded in Normal Density Concrete**

Add to definition of *N* in (b)

In the absence of more accurate information the present annual number of heavy goods vehicles may be assumed to be that given by BS 5400: Part 10 Table 1. The total number of vehicles may be obtained from this and the age of the bridge with the reductions given in BA 61 Annex A.

Alternatively, where traffic data is available, it may be used with interpretation from BD 21 5.24 and TRL Report SR802. In general, it will be conservative to assume the present traffic flow has existed from construction and the reductions given in BA 61 Annex A may be applied. However, the history of particular bridges should be considered to see if they are likely to be exceptions. These arise particularly where the road network has changed since the bridge was built, as with bridges which are on routes which have been bypassed. The possibility that a bridge was subject to heavy goods traffic from factories, ports or whole industries which have closed or reduced substantially since the bridge was built should also be considered.

Add: at the beginning of (c)

Nominal static strengths of bolt and rivet heads may be calculated from Equation C1. Means of preventing separation of the concrete should be present such as by encasement or other mechanical devices, unless tests are carried out to demonstrate that adequate means of preventing separation are present.

$$\text{Nominal Static Strength} = \frac{2.0 A_1 f_{cu}}{\gamma_{mc} \gamma_b} \qquad \text{Equation C1}$$

where:

$\gamma_b$ may be taken as 1.25 for bolts/bolt heads or other connectors with predominantly vertical surfaces resisting the horizontal shear, and as 2.0 for rivet heads or for other connections with predominantly inclined surfaces resisting the horizontal shear;

$A_1$ is the face area of the connector in the direction of horizontal shear.

**BRIDGEGUARD 3**
**CURRENT INFORMATION SHEET NO 30**
**SUBJECT:USE OF BD 61 FOR COMPOSITE BRIDGES WITH SHEAR CONNECTION**
**DATE: MARCH 2001**
**STATUS: FINAL**

**Part 1:** Originator

Organisation: Gifford and Partners

I certify that reasonable professional skill and care has been used in the compilation of this document.

| | |
|---|---|
| Signed: | Title: |
| Name: | Date: |
| To be signed by the document author. | |

I certify that the staff who have prepared the above documents are competent to carry out their duties and that (so far as I can reasonably ascertain) they have used reasonable professional skill and care.

| | |
|---|---|
| Signed: | Title: |
| Name: | Date: |
| To be signed by the Director (or equivalent) to whom author is responsible. | |

**BRIDGEGUARD 3**
**CURRENT INFORMATION SHEET NO 30**
**SUBJECT:USE OF BD 61 FOR COMPOSITE BRIDGES WITH SHEAR CONNECTION**
**DATE: MARCH 2001**
**STATUS: FINAL**

**Part 2:** Endorsement Organisation: Pell Frischmann Consulting Engineers Ltd.

I certify that reasonable professional skill and care has been used in endorsing this document.

| | |
|---|---|
| Signed: | Title: |
| Name: | Date: |
| To be signed by the Senior Reviewing Engineer. | |

**Part 3:** Acceptance on behalf of Railtrack

I approve the implementation of this Current Information Sheet with respect to the Bridgeguard 3 programme only.

| | |
|---|---|
| Signed: | Title: |
| Name: I Bucknall | Date: |
| To be signed by Railtrack Project Delivery Technical Service and Innovation Team | |

**CURRENT INFORMATION SHEET NO 31**

# SUBJECT: USE OF 'ARCHIE-M' FOR THE ANALYSIS OF SINGLE AND MULTI-SPAN ARCHES

## 1. Introduction

A number of Current Information Sheets have been issued for the Bridgeguard 3 programme covering the assessment of single and multi-span masonry arch bridges in which the use of the ARCHIE/MULTI programs as developed by the University of Dundee have been adopted as acceptable methods of analysis. These Current Information Sheets are as follows: -

a) Current Information Sheet No 16 details the procedure to be adopted for the assessment of multi-span arches and approves the use of MULTI for bridges where a global analysis is necessary.

b) Current Information Sheet No 18 provides guidance regarding the parameters and assumptions to be used for MULTI analyses.

c) Current Information Sheet No 19 details the procedure that can be adopted with respect to the application of condition factors in the rigorous assessment of arch structures using a mechanism analysis (this is also deemed to include a modified mechanism analysis using programs such as 'ARCHIE' and 'MULTI').

d) Current Information Sheet No 20 details the procedure that can be adopted for the assessment of skew arches and makes reference to Current Information Sheet No 18.

Since the issue of these Current Information Sheets, the program ARCHIE-M (developed by Obvis Ltd) has been issued which covers the assessment of both single and multi-span masonry arches. It is understood that the ARCHIE/MULTI programs are no longer available. This Current Information Sheet details the procedure that shall be adopted for the use of ARCHIE-M on Bridgeguard 3 assessments.

## 2. ARCHIE-M

ARCHIE-M is considered to be an acceptable program for the assessment of single and multi-span arches with the following provisos: -

i) Version 2-1-0 of the program is adopted or any subsequent version subject to the approval of the Technical Approval Authority.

ii) The 'default' setting is used for the longitudinal dispersal of axle loads on the arch barrel.

With reference to Current Information Sheets 16, 18, 19 and 20 all references to 'ARCHIE' or 'MULTI' can be taken to be equally applicable to 'ARCHIE-M'.

The continued use of 'ARCHIE' and 'MULTI' is not precluded by this Current Information Sheet.

## CURRENT INFORMATION SHEET NO 32

## SUBJECT: STRENGTH OF RIVETS

This Current Information Sheet is issued for guidance purposes only: it is not mandatory. The Assessor must be satisfied that the advice given in this Information Sheet is appropriate to the structure in question.

---

### 1. Introduction

This Current Information Sheet applies to the assessment of shear in riveted connections in metal beam bridge decks. It is concerned with shear in the rivets; bearing stress, which is not normally critical, should continue to be assessed in accordance with BD 21 and BD 56.

Experience with bridge assessment has shown that static rivet strength as calculated using the above assessment standards frequently governs the capacity of girders resisting horizontal shear using the $\frac{VA\bar{y}}{I}$ approach.

Rivet shear is sometimes also found to govern the capacity at flange and web splices, yet rivet failures are almost unknown.

This Current Information Sheet describes a revised method of calculating rivet shear based on studies into the shear and rivet capacity of metal girders carried out on behalf of Network Rail by Cass Hayward and Partners. This is the approach adopted in Network Rail Company Code of Practice RT/CE/C/025 The Structural Assessment of Underbridges, and is less onerous than the requirements of the standards BD 21 and BD 56.

As Technical Approval Authority for its own bridges, Network Rail is able to grant a derogation of the standards BD 21 and BD 56 although this must be referenced in Form AA.

### 2. Rivets in Shear

In a fastener subjected to shear only the average ultimate shear stress should not exceed $\frac{\sigma_q}{\gamma_m \gamma_{f3} \sqrt{2}}$

Where, $\sigma_q =$ $0.9\sigma_{ult}$
$\gamma_m =$ 1.20 for web/flange rivets,
1.33 for all other rivets.
$\gamma_{f3} =$ 1.1
$\sigma_{ult} =$ ultimate tensile strength of the rivet material, to be taken as: -
350 N/mm$^2$ for wrought iron,
430 N/mm$^2$ for pre 1905 steel,
450 N/mm$^2$ for post 1905 steel.

Riveted web to flange connections made using flange angles (both side of web), with the rivet size and spacing configuration shown in Table 1, have at least equivalent capacities to webs up to the tabulated thickness. No explicit calculation of shear or bearing capacity is required for these configurations, provided the web is adequate.

| | Maximum Web Thickness | | |
|---|---|---|---|
| | 3/4" rivets<br>4" centres | 7/8" rivets<br>4" centres | 1" rivets<br>4" centres |
| WROUGHT IRON<br>*characteristic yield stress of web plate material = 220N/mm$^2$*<br>*uts of rivet material = 350N/mm$^2$*<br>*$\gamma_m$ (web shear) = 1.2* | 7/16" | 9/16" | 3/4" |
| STEEL PRE 1905<br>*characteristic yield stress of web plate material = 230N/mm$^2$*<br>*uts of rivet material = 430N/mm$^2$*<br>*$\gamma_m$ (web shear) = 1.05* | 7/16" | 9/16" | 3/4" |
| STEEL POST 1905<br>*characteristic yield stress of web plate material = 230N/mm$^2$*<br>*uts of rivet material = 450N/mm$^2$*<br>*$\gamma_m$ (web shear) = 1.05* | 7/16" | 5/8" | 13/16" |
| This table only applies for girders and rivets with the assumed characteristic yield stress and uts shown in the above table. For all other material property combinations the connection capacity should be determined by calculation. | | | |

*Table 1*

## 3. Rivets in Bearing

The bearing pressure between a fastener and each of the connected parts should continue to be assessed in accordance with BD 21 and BD 56.

## Part 1: Originator Organisation: Cass Hayward and Partners

I certify that reasonable professional skill and care has been used in the compilation of this document.

| | |
|---|---|
| C M Booth | Title: Sub Consultant |
| Signed: | Date: |
| To be signed by the document author. | |

I certify that the staff who have prepared the above documents are competent to carry out their duties and that (so far as I can reasonably ascertain) they have used reasonable professional skill and care.

| | |
|---|---|
| A Monnickendam | Title: Partner |
| Signed: | Date: |
| To be signed by the Director (or equivalent) to whom author is responsible. | |

**Part 2: Endorsement Organisation: Flint and Neill Partnership**

I certify that reasonable professional skill and care has been used in endorsing this document.

| | |
|---|---|
| Dr A R Flint | Title: |
| Signed: | Date: |
| To be signed by the Senior Reviewing Engineer. | |

**Part 3: Acceptance on behalf of Network Rail**

I approve the implementation of this Current Information Sheet with respect to the Bridgeguard 3 programme only.

| | |
|---|---|
| I K Bucknall | Title: |
| Signed: | Date: |
| To be signed on behalf of the Head of Structures Engineering. | |

# APPENDIX A

# AUDIT TRAIL

## AUDIT TRAIL

A report by Railtrack Southern Zone in October 1997 (Reference 1) showed that a high percentage of existing underbridges theoretically fail the criteria in the Railtrack Line Code of Practice RT/CE/C/015 "The Assessment of Underbridge Capacity".

A subsequent Report by Cass Hayward and Partners in August 1998 (Reference 2) on behalf of Railtrack investigated the strength of rivets related to the assessment of underbridge capacity. This included a series of tests on two (post 1905) steel riveted main girders which had been removed from an existing underbridge. The conclusions from this Report were included in the Railtrack Line Code of Practice RT/CE/C/025: Issue 1 "The Structural Assessment of Underbridges", which supersedes the earlier RT/CE/C/015.

In BS5400 the shear capacity of a rivet at ULS is related to the yield stress of the rivet material, with $\tau = \sigma_y /(\gamma_m \gamma_{f3} \sqrt{2})$, and $\gamma_m = 1.1$. A reduction is applied for hand driven rivets.

RT/CE/C/025 modified this approach and relates the shear capacity to the UTS of the rivet material (which was felt to be a more consistent property), with a modified $\gamma_m$ intended to give suitable behaviour at SLS/working loads; with $\tau = 0.9\sigma_{ult} /(\gamma_m \gamma_{f3} \sqrt{2})$, and $\gamma_m = 1.33$. No reduction is applied for hand driven rivets. Values of $\sigma_{ult}$ are provided for wrought iron, pre 1905 steel, and post 1905 steel rivets.

Subsequent studies into the "Shear and Rivet Capacity of Metal Girders" (References 3 & 4) were carried out by Cass Hayward and Partners on behalf of Network Rail. These include an extensive literature search, finite element analysis, theoretical studies, and a number of material/load tests on redundant girders.

These studies confirmed that the behaviour of riveted joints at SLS was primarily governed by the yield stress of the rivet material, and that the behaviour at ULS was primarily governed by the UTS of the rivet material. It concluded that the behaviour of actual joints at ULS was of primary significance, that small amounts of slip and yield at SLS could be tolerated, and that the value of $\gamma_m$ for underbridges may be set to give suitable reliability at ULS, without any additional adjustment for behaviour at SLS. Improved values for the material properties of wrought iron and pre 1905 steel were recommended, resulting from the testing programme and literature search.

The study concluded that a reduced value of $\gamma_m = 1.2$ at ULS could be adopted for rivet shear in web/flange connections. This value allows for uncertainties in the relationship between rivet shear failure and rivet UTS, and possible gaps between rivets and holes. It also allows for the reduced statistical risk of material variability in larger rivet groups. The study confirmed that a value of $\gamma_m = 1.33$ at ULS was appropriate for all other rivet groups.

As part of the study a simple tabular method for rivet assessment in web/flange connections was developed. This gives maximum web thicknesses for which given configurations of rivet diameter and spacing have at least equivalent strength. For connections complying with the tabulated values no explicit calculation of rivet shear or bearing is required, provided the web has adequate capacity.

The above recommendations regarding improved material properties, reduced $\gamma_m$, and simplified tabular assessment, have been incorporated into Issue 2 of RT/CE/C/025.

Consideration has been given to application of the above principles to assessments carried out to BD21. Whilst it is clear that the physical behaviour of materials and girders will be similar, it should be appreciated that there are some differences in the application of partial ($\gamma$)

factors between the codes.

RT/CE/C/025 allows a reduced value of $\gamma_{f3}$ = 1.0 at ULS (otherwise 1.1) for certain categories of bridges. BD21 recommends an increased value of $\gamma_m$ = 1.2 for wrought iron plate in shear, which effectively counteracts the higher yield stress for wrought iron suggested by this code. Typical "average" values of $\gamma_{fl}$ are around 1.25 at ULS for underbridges assessed to RT/CE/C/025, whereas they are probably around 1.3 for overbridges assessed to BD21.

It may be seen that there is likely to be a greater margin between applied loading at ULS and SLS on overbridges than on underbridges. Therefore it may be concluded that there are unlikely to be any adverse effects at SLS on overbridges compared to those on underbridges to which the study into "Shear and Rivet Capacity of Metal Girders" refers. It is therefore proposed that the recommendations may be applied to overbridges assessed to BD21.

**References:**

1. An investigation of rivet stresses in metallic railway underline bridges of Southern Zone.
   Railtrack Report Reference NNK, 10.10.97
2. The Assessment of Underbridge Capacity: Report on Rivet Strength.
   Cass Hayward Report, Issue 2, 20.8.98.
3. Study of Shear Capacity of Metallic Girders: Phase 1 – Final Report
   Cass Hayward and Partners, Revision 02, 06.09.02
4. Study of Shear and Rivet Capacity of Metal Girders: Phase 2 – Draft Final Report.
   Cass Hayward and Partners, Revision 02, 07.10.03.

CURRENT INFORMATION SHEET NO 33

SUBJECT: CON-ARCHES

**This Current Information Sheet is issued for guidance purposes only: It is not mandatory. The assessor must be satisfied that the advice given in this Information Sheet is appropriate to the structure in question.**

## 1. Introduction

A number of technical issues have arisen concerning the assessment of Con-Arch Bridges. These bridges are formed from precast portal frame units with sloping side members, which were placed side by side on bearings and made monolithic by cast in situ reinforced concrete infill (shear keys).

This Current Information Sheet details the actions to be taken with respect to a Bridgeguard 3 assessment of a Con-Arch structure.

## 2. Implications Arising from the Construction Sequence

An elastic analysis using bases which are fixed in position but free to rotate will produce a safe capacity. In such an analysis, the effects of the construction sequence on the ultimate capacity can be ignored.

The effects of axial loading on flexural strength should be considered in the assessment.

## 3. Effects of Temperature, Creep and Shrinkage

The effects of temperature, creep and shrinkage can be ignored in the determination of the ultimate capacity of this form of structure.

## 4. Effects of Soil Pressure

The capacity of the Con-Arch should be tested against an appropriate range of soil pressures. In general, equal soil pressures should be utilised on both sides of the portal. The range of soil pressures should be stated in the Approval in Principle (Form AA).

## 5. Possible Implications of a Serviceability Failure

No serviceability calculations need to be undertaken.

The possibility of progressive collapse due to yielding of reinforcement and cracking of concrete can be ignored.

Where cracking is present, at mid span it can be considered purely as a Serviceability issue, i.e. it should be reported on as part of an inspection, and does not need to be considered in the quantitative assessment.

## 6. Shear Capacity of the Top Member of a Con-Arch

The shear link spacing in the top member of a Con-Arch is often too great for them to be taken into account according to BD 44 so the shear capacity is likely to be restricted. It should be noted that the shear capacity at the root of the haunch is unlikely to be the critical section for shear in this member.

As a result of the above, an alternative form of analysis can be utilised which is based on BS 8110 which allows an enhancement in shear capacity by considering the coexistent axial load. The shear capacity can be determined from the following formula: -

$$V_u = b_w d\,[\xi_s v_c + 0.6(N/A_c)\,(Vh/M)]$$

Where:

| | | |
|---|---|---|
| $V$ | = | shear force |
| $M$ | = | coexistent moment, not to be taken as less than Vh |
| $N$ | = | coexistent axial force |
| $A_c$ | = | area of concrete |
| | = | bh for a rectangular section |

All other variables are as in BD 44 clause 5.3.3 except that the enhancement contained in 5.3.3.3 may not be used in combination with the above.

The assessor should investigate which combinations of axial load and bending moment give the critical case for shear. A conservative approach which could be used initially would be to consider the maximum moment in combination with the maximum shear force and the minimum axial force from the corresponding load cases.

## 7. Potential Implications of Buckling given the Slender Nature of a Con-Arch

Con-Arch structures are comparatively slender and if treated as slender columns to BD 44, this could result in a possible failure mode.

For the legs of Con-Arches with standard rail headroom, it is considered implausible that buckling could be a significant factor.

Where a conventional elastic analysis is used in the assessment with no redistribution, top member buckling can also be ignored where the ratio of the effective length (0.7 x length of top member between the legs) to the minimum thickness of the member is less than or equal to 19. Where it is greater than 19, the procedure to be adopted should be agreed with the Technical Approval Authority.

For a Con-Arch with distinct legs and top members with an angle between their outside faces greater than 45°, the length of the top member for this purpose may be taken as the distance between the nodes corresponding to the intersection of the medians of the two members.

## 8. The Effects of Axle and Wheel Loading

Simple calculations have shown that for Con-Arches with shallow fill depth the single wheel, or more correctly two adjacent single axle loads can be the critical load case, rather than HA UDL and KEL, when it is assumed that there is no distribution between the precast units.

However, transverse distribution of load would make it unlikely that wheel loading would be critical. Where necessary the assessor should give due consideration to making a realistic allowance for transverse distribution of loading through the shear keys. The assessor should substantiate the capacity of the shear key to sustain the required load distribution. This should avoid the need for a computer aided 3-D distribution analysis in most cases.

Where necessary, the assessor should also give due consideration to making a realistic allowance for distribution of the effects of Accidental Wheel Loading and parapet self weight loading.

## 9. Approval In Principle (Form AA)

The Approval in Principle document (or the Form BA where the Form AA has already been approved) should make reference to this Current Information Sheet.

## CURRENT INFORMATION SHEET NO 34

## SUBJECT: CONDITION ASSESSMENT OF POST TENSIONED ELEMENTS

This Current Information Sheet is issued for guidance purposes only: It is not mandatory. The assessor must be satisfied that the advice given in this Information Sheet is appropriate to the structure in question.

### 1. Introduction

Network Rail own a number of post-tensioned overbridges that have been inspected and assessed under the Bridgeguard 3 assessment programme. These assessments have been on an 'As built' basis. It has been assumed in all cases that the post tensioning systems employed are in good condition and that they are fully functional. The assessment certificate (Form BA) has been accepted by Network Rail as 'Provisional' pending confirmation that either there is sufficient evidence that the 'As Built' assessment can be accepted as the 'Assessed' capacity or intrusive investigations are undertaken from the results of which an appropriate capacity can be determined.

The purpose of this document is to give guidance to organisations that are to undertake this condition assessment and thus conclude the Bridgeguard 3 assessment of a post-tensioned bridge.

The three stages of the process are as follows: -

| | |
|---|---|
| Stage 1 – | Information Collection and Recommendations (including comment on 'As Built' assessment). |
| Stage 2 – | Development of Site Investigation and Testing Plan |
| Stage 3 – | Review of Findings and Final Assessment Conclusions |

The methodology for the intrusive investigation will generally be along the lines of those detailed in BA50/93 – 'Post-Tensioned Concrete Bridges. Planning, Organisation and Methods for Carrying Out Special Inspections' and the Interim Advice Note IA3 – 'Post-Tensioned Concrete Bridges, BA 50/93'. The phasing of the works is however not as stated in these documents. It is considered that the Bridgeguard 3 inspection and assessment should be adequate to confirm the available construction records and provide a record of the condition of the structure such that vulnerable areas can be identified without a further 'preliminary' inspection.

### Stage 1 –Information Collection and Recommendations

This is a desktop study in which all pertinent information is reviewed to ascertain whether intrusive investigation is necessary and if so, the objectives of the site work. This desktop study should give consideration to matters such as: -

**i) The age of the bridge and the form of the post-tensioning system.**

The Network Rail stock of post-tensioned bridges includes a number of early bridges; some have been in service for 40 to 50 years. These bridges contain early forms of post-tensioning systems. The design of these systems and how they are utilised in the structure often have features that would nowadays be considered undesirable and their durability may be in question.

The vulnerability of the system to deterioration can be assessed from manufacturers literature of the time and consideration of the methods/sequence of construction and quality of construction of the period. Information on the system should give details of ducting, spacers, grouting etc and the drawings should provide data on the sequence of construction, location of construction joints etc.

### ii) Significant Defects

The type, location and extent of defects or other visible signs such as leakage could be indicators of problems with the integrity of the post-tensioning system. It is important to establish the likely cause(s) of any such defects. For instance, if there are longitudinal cracks on the line of a tendon then this could be a sign that the tendon has severed and re-anchored.

### iii) 'As Built' Assessment

Structures can contain longitudinal only, transverse only or both longitudinal and transverse stressing elements. In some assessments, the transverse stressing may not have been utilised because an analysis using a line beam only has been undertaken and the capacity is not conditional on these elements. However, if the longitudinal post-tensioning is defective there may be a need to use the transverse stressing in a further more rigorous analysis. In these cases, intrusive investigation may be warranted.

### iv) Sensitivity of the Post-tensioned Elements to Loss of Prestress

Some additional assessment should establish the sensitivity of the deck capacity to the loss of prestress. It could be that there is a considerable margin of safety which would preclude the need for intrusive investigations. Alternatively, the failure of an anchorage could have a significant effect on the shear capacity of a post-tensioned element, especially in some early structures where there was very little shear reinforcement in the form of links.

Once all the relevant data has been collated, it should be possible to determine whether the 'As Built' assessment can be considered as a realistic capacity or whether intrusive investigations are required.

A pro forma for undertaking the Stage 1 – Information Collection and Recommendations (including comment on 'As Built' Assessment) is included in Appendix A.

Sections 1 and 2 of this pro forma require the organisation to include all relevant details about the form of construction, prestressing systems etc and the form, location and extent of defects. All the pertinent information from the 'As Built' assessment can also be included.

Section 3 – Permits the organisation to detail their recommendations on whether an Intrusive Investigation is required and the reasons for their decision. Where an investigation is considered necessary, a summary of the proposed investigative actions should be detailed, including information on the proposed Specialist Subcontractor and Supervisor.

Section 4 – Details the Checks and Approvals necessary to complete Stage I. If Intrusive Investigations are not necessary then following approval from Network Rail, it should be possible to finalise the Bridgeguard 3 BD21 assessment.

Where Intrusive Investigations are considered necessary, then the process should move to Stage 2 as detailed below: -

**Stage 2 - Development of Site Investigation and Testing Plan**

**General**

In Stage 1, a study of the available records and findings of the initial Bridgeguard 3 inspection will have been undertaken and it will have identified each of the critical elements of the post-tensioning system and the implications for the capacity of the structure. In this Stage, assessment of critical elements should be developed to determine the extent of the investigatory works from which a site investigation/testing plan can be drawn up. A Pro forma for producing a Stage 2 Site Investigation and Testing Plan is included in Appendix B.

These assessments need to consider any observed defects; their implications on the post-tensioned elements and also deficiencies that may be inherently present within the post tensioning system that has been used.

Targeting of specific items and/or locations for the intrusive investigations should be considered such that following the completion of the site investigation works a robust conclusion can be made.

Where a sensitivity analysis indicates that loss or failure will not significantly affect the load carrying capacity then the levels of investigation may be reduced. Conversely, if the structure appears to be unduly sensitive then additional testing may need to be considered in the development of the testing plan. It should be noted that a number of early post-tensioned bridges have minimal shear reinforcement and are reliant on the post-tensioning.

General Requirements for Undertaking an Intrusive Investigation are contained in Appendix C.

**The Site Investigation and Testing Plan.**

Following a thorough review of all the available information, records and the findings of any risk assessments a plan should be developed to identify the scope of intrusive investigation and testing required.

The testing plan should provide a schedule of investigation and testing for the post-tensioned elements such that on completion of these works a robust judgement can be made regarding the condition of the post-tensioned elements that are contained within the structure.

Should this initial testing identify issues of concern with respect to the assessed capacity, durability or other matters of concern then additional site investigation and testing might be required. The schedule should therefore seek to obtain as much information as possible in the initial visit.

The schedule should indicate but is not limited to consideration of: -

**Post Tensioning Anchorages**

Anchorages may need to be exposed to confirm the prestressing system used including the number and types of wire/strands/bars forming each tendon. Their exposure may also be necessary to ascertain the condition of the tendon and the integrity of the grouting.

Where an anchorage is exposed any tubes and grout holes should be probed. Details of wedge types may assist in identifying the prestressing system.

### Post-tensioning Cables or Bars

Intrusive investigation to inspect the cables or bars at critical sections should be undertaken. The methodology should be as given in BA50/93 and IA3.

Test locations should generally be located where tendons are easily accessible. In the case of draped tendons locations will be within the central portion of the soffit or possibly from above at the beam-ends. Where beams are continuous investigation of tendons over intermediate piers should be considered.

### Internal Visual Examination of Voided Decks

Where decks are of voided construction, it may be possible to visually inspect these voids using a boroscope. Web faces of post-tensioned beams can be inspected for cracks and the deck slab checked for leakage using this method.

### Edge Beams

Investigation of tendons can be undertaken where these are close to the face of the elevation. Where tendons are draped, it is possible to investigate these in more detail than in other areas of the structure.

### Material Testing

Proposals for collection of grout and concrete samples and on site material testing should be considered to support any recommendations. Testing of grout and water samples taken from ducts should be carried out, in particular testing of these for chlorides to confirm whether contamination due to road de-icing salts has occurred. Early beams may have added chlorides to both the concrete and (occasionally) grout. Chloride additives may have been used in the mastic to seal anchorages prior to grouting.

### Concrete Strengths

Where details of the concrete strengths are not known and the assessment has utilised "assumed" values or is based on a sensitivity analysis, cores for crushing tests should be considered.

### Location Plans for Proposed Intrusive Investigation

Trial holes, core-holes and any proposed test sample locations are required to be submitted on (preferably) A3 sheets along with the schedule.

Location plans should indicate which testing is to be undertaken from within a Railway possession and testing that can be undertaken from the highway and does not require a possession.

### Other Tests

Where defects or concerns have been identified from the earlier inspection, additional testing may need to be specified, e.g. samples to confirm ASR. Details of any additional testing shall be identified and agreed with Network Rail.

**Other Information**

Test locations should be chosen so as to minimise interference with the railway or the highway. Methods of working should be adopted, wherever possible which avoid the need for a possession of the track or a road closure.

Once the proposed investigation and testing plan has been fully developed the effects of the proposals on the structural integrity of the structure should be assessed. This assessment should include an outline methodology and supporting calculations to demonstrate that the proposals will have no adverse affect on the structure and should be submitted with the completed plan for approval by Network Rail.

In approving the proposed Investigatory Works, Network Rail will also require all work to be undertaken in accordance with their Health and Safety procedures.

Where reinstatement will be necessary, details of the materials and methods to be used will need to be provided to Network Rail for approval.

The organisation will need to prepare suitable contract documentation including Specification, Drawings, Schedules etc for the appointment of the Investigation Contractor.

**Reporting**

On the completion of the site investigation works, an 'Interpretative Report' shall be produced. The report should review and discuss whether the objectives stated in the testing plan have been met. If the inspection indicates, for any reason, that the objectives were not achieved, then the next course of action will need to be agreed with Network Rail. This action may include the need to carry out additional or different types of testing. These findings should be reported to Network Rail and the next course of action agreed.

The assessments of critical elements produced in the development of the testing plan should be revised in the light of the findings of the site investigations. The revised assessments should be included in the Interpretative Report along with a discussion of any issues where 'risk' has been identified that may need to be considered further by Network Rail.

Network Rail should be advised immediately if any element of the structure or the entire structure is identified as being 'At risk' of collapse or partial failure as soon as practicable following the conclusion of the on site testing and/or reappraisal of the Risk Assessments.

**Stage 3 – Review of Findings and Final Assessment Conclusions**

The conclusions of the Interpretative Report and revised assessments of critical elements shall be considered in this next stage – i.e. the completion of the Bridgeguard 3 Assessment Report.

If the site investigation has indicated that the post-tensioning is in good condition and that no risks were identified that would affect the assessed load carrying capacity then a 'Final' Form BA should be produced for endorsement. This should include a definitive statement regarding the findings of the intrusive investigations.

If deficiencies or risks are identified by the 'Interpretative Report' then these need to be carried through into a final assessment of the bridge. Where defects have been identified, these shall be detailed and their implications on the strength and durability discussed. If defects are considered to affect the assessment an Addendum Form AA (Approval in Principle) will be required to be submitted detailing how a re-assessment will be undertaken to allow for deficiencies in the Post-Tensioning. In some instances, the defects/deficiencies/risks that have been identified may not affect the assessment of the load carrying capacity and a 'Final' Form BA may be agreed, e.g. defects in transverse stressing, where this has not been used in the assessment. A statement should be included identifying the deficiencies and raising any issues that may need to be considered by Network Rail.

Where defects are to be allowed for in the assessment then following agreement of an Addendum AIP, a revised assessment report and calculations shall be submitted through the normal Bridgeguard reviewing process.

**Submission**

Completed Pro Forma in Appendix A and Appendix B are to be submitted in electronic format to Network Rail.

# APPENDIX A

## INFORMATION COLLECTION AND RECOMMENDATIONS PRO FORMA

BRIDGEGUARD 3 – POST-TENSIONED BRIDGES

CONDITION ASSESSMENT OF POST-TENSIONED ELEMENTS

STAGE 1 - INFORMATION COLLECTION AND RECOMMENDATIONS

| | |
|---|---|
| **Name of Bridge** | **National Grid Ref:** |
| **ELR/Bridge Number** | **Mileage** |
| **Highway Authority** | **Date of Design:** |
| **Zone** | |

## 1. BRIDGE DETAILS

**(i) Form of Construction**
(brief description From AR – if applicable reference other structures with similar details)

**(ii) Span Details**

| | |
|---|---|
| Number: | Road Carried/Type: |
| Length: | Carriageway Width: |
| Angle of Skew: | Footway/Verge Width: |

**(iii) Post-tensioned Elements**

| | | | |
|---|---|---|---|
| Longitudinal Stressing | Y/N | | |
| Transverse Stressing | Y/N | If Yes was it used in the assessment? | Y/N |
| Are Stressing Records Available? | Y/N | If Yes State Location | |
| Are Grouting Records Available? | Y/N | If Yes State Location | |

**(iv) Date of Construction:**

**(v) Date of Bridgeguard 3 Inspection:**

**(vi) Summary of Significant Defects:**

**(vii) Date of Bridgeguard 3 Assessment**

| | | | | |
|---|---|---|---|---|
| Assessment Results | | Line Beam/Grillage | *(Delete as appropriate)* | |
| Quantitative | Y/N/NA | If Yes State Capacity | Carriageway in bending | Carriageway in Shear |
| | | | Footway in bending | Footway in Shear |

List any Assumptions made in the Assessment

Qualitative Y/N/NA If Yes What Elements

## 2 CONSTRUCTION INFORMATION

**(i) Documents Referred to in Undertaking this Review**

Tender/Construction Issue/As-Built
*(delete as appropriate and attach drawing list)*

**(ii) Construction Sequence/ Method of Construction**

Precast/In situ/Segmental/Sequential/Other (describe)

**(iii) Stressing Systems**

Name of System/Sub Contractor:

Longitudinal; Cable/Bar System

Factory/Site Stressed

Factory/Site Grouted

Transverse: Cable/Bar System *(Delete as appropriate)*

**(iv) General Information**

| | | Y | N | Don't know |
|---|---|---|---|---|
| (a) | Is the deck waterproofed? | ☐ | ☐ | ☐ |
| | If yes state type | | | |
| (b) | Are the anchorages recesses filled with concrete/grout? if yes state which | ☐ | ☐ | ☐ |
| (c) | Were any additives used in the grout or anchorage protection? eg $CaCl_2$/High Alumina. | ☐ | ☐ | ☐ |
| | If yes state type | | | |
| (d) | Does the deck have positive drainage? | ☐ | ☐ | ☐ |
| (e) | Are there any buried/built in services? | ☐ | ☐ | ☐ |
| | List providers if known | | | |
| (f) | Has the assessment determined whether the loss of prestress would significantly alter the assessed capacity? | ☐ | ☐ | ☐ |
| | If yes state the conclusions | | | |

**(v) Other Information**

## 3. RECOMMENDATIONS

Intrusive Investigation Required? Y N

Reasons for decision

Proposed Specialist Subcontractor

Summary of Investigative Actions

| | | |
|---|---|---|
| Proposed SI Supervision | Name | Title |
| Proposed SI Report Compiler | Name | Title |

## 4. CHECKS AND APPROVALS

| | | |
|---|---|---|
| Prepared by | Name | Title |
| Checked by | Name | Title |
| Approved by | Name | Title |
| External Review by | Name | Title |
| For and on behalf of | | |
| Accepted by Network Rail | Name | Title |

# APPENDIX B

# SITE INSPECTION AND TESTING PLAN

**BRIDGEGUARD 3 – POST TENSIONED BRIDGES**

**STAGE 2 – SITE INVESTIGATION AND TESTING PLAN**

**1.0 Details of Prestressing and Reinforcement**

Longitudinal Prestress

| Beam Type | Location | Drg Ref | P/S Type | No of Tendons | Duct/Sheath Details | Anchorage Positions | Stressing Sequence |
|---|---|---|---|---|---|---|---|
| | | | | | | | |

Transverse Prestress

| | Location | Drg Ref | P/S Type | No of Tendons | Duct/Sheath Details | Anchorage Positions | Stressing Sequence |
|---|---|---|---|---|---|---|---|
| | | | | | | | |

Secondary Reinforcement

Flexural - Longitudinal

- Transverse

Shear

| 2.0 Site Investigation Check List | | | Date of Inspection: | |
|---|---|---|---|---|
| Location | Defect/Deficiency Identified from BG3 Site Inspection | Does this Defect/Deficiency affect the Assessed Capacity/Structural Integrity | Investigation Required? Yes/No | Proposed Investigation or reason why not considered appropriate |
| | | | | |

# APPENDIX C

# GENERAL REQUIREMENTS FOR UNDERTAKING AN INTRUSIVE INVESTIGATION

## 1. BRITISH STANDARDS AND CODES OF PRACTICE

All editions of British Standards and Codes of Practice quoted in this Specification and Appendices shall be those current on the date of investigation.

## 2. DEFINITIONS

These definitions refer to the Terms of Appointment, Specification (including Schedules) and Schedule of Rates in this Investigation.

(a) 'Exploratory Work' shall mean a specific investigation undertaken at a specified location in an element of a bridge to ascertain the condition of a particular component.

(b) 'Routine Testing and Sampling' shall mean general concrete testing carried out to specific test areas of an element of a bridge.

(c) 'Bridge' shall mean a bridge being examined as part of the Investigation.

(d) 'Element' shall mean a part of a bridge such as an abutment, pier, span, etc.

(e) 'Component' shall mean the reinforcement or prestressing system or the like in an element.

(f) 'Contractor' shall mean the Testing Firm appointed to undertake the Works.

## 3. PROFESSIONAL ATTENDANCE ON SITE

The Contractor shall provide Network Rail with full information regarding the supervisory staff he proposes to employ on the Investigation for approval. This information shall include the following:

(a) management structure and responsibilities;

(b) curriculum vitae;

(c) full details of their relevant experience in the investigation of post-tensioned concrete structures, including the methods and techniques relevant to the Investigation, the hazards that could be encountered and measures that should be adopted to counter these and also the testing techniques to be employed;

(d) Qualifications;

(e) The names and similar details shall be provided for all specialist sub-contractors.

Approval by Network Rail shall not relieve the Contractor of any of his responsibilities under the Contract.

## 4. SETTING OUT FOR EXPLORATORY WORK AND ROUTINE SAMPLING AND TESTING

The Contractor shall set out the location of exploratory work and the test areas for routine sampling and testing in accordance with Site Investigation and Testing Plan and the requirements of the Specification.

## 5. CONTROL OF NOISE, DUST AND MUD

5.1 The Contractor shall comply with the recommendations for practical measures to reduce noise set out in BS 5228: Parts 1, 2 and 4.

5.2 Compliance with Sub-Clause 1 of this Clause does not confer immunity from relevant and legal requirements.

5.3 The Contractor shall take all necessary steps to avoid creating a nuisance from dust and mud.

## 6. LABORATORY ACCREDITATION

6.1 All laboratory tests required under the Contract shall be undertaken only by testing laboratories accredited in accordance with BS 7502 by the National Measurement Accreditation Service (NAMAS) for such tests.

6.2 Where testing is carried out in another member state of the European Communities such tests shall be undertaken by an appropriate organisation offering suitable and satisfactory evidence of technical and professional competence and independence. This condition shall be satisfied if the organisation is accredited in a member state of the European Communities in accordance with the relevant parts of the EN45000 Series of Standards for the tests carried out.

6.3 The Contractor shall provide all necessary documentation required by Network Rail to verify the approval of a testing laboratory for any type of testing required under this Contract.

6.4 The Contractor shall ensure that each laboratory proposed to undertake works for this Contract shall admit Network Rail, or its representative, to his premises during normal working hours for the purposes of inspecting and witnessing the testing.

## 7. PRIVATE AND PUBLICLY OWNED SERVICES OR SUPPLIES

7.1 The Contractor shall satisfy himself as to the exact position of Statutory Undertakers and other publicly and privately owned services or supplies affected by the Investigation.

7.2 The Contractor shall, during the progress of the Investigation take all measures required by any Statutory Undertaker or the management of other publicly or privately owned services or supplies, for the support and full protection of all such services or supplies but subject to any instructions or contrary directions by Network Rail. No such services or supplies shall be interrupted without the written consent of the appropriate authority or owner.

## 8. SUBSTANCES HAZARDOUS TO HEALTH

8.1 In this Clause `substance hazardous to health', has the same meaning as:

(a) Regulation 2 of the Control of Substances Hazardous to Health Regulations 1988 (COSHH);

(b) Regulation 2 of the Control of Lead at Work Regulations 1980 (CLAW);

(c) Regulation 2 of the Control of Asbestos at Work Regulations 1987 (CAW).

8.2 A substance hazardous to health shall only be used or generated in or about the Investigation where specified in the Contract or with the consent of Network Rail.

8.3 Where any substance hazardous to health is so used or generated the Contractor shall provide Network Rail with the method statement:

(a) a copy of the assessment of the risks created by the use of that substance as required by Regulation 6 of the COSHH Regulations, Regulation 4 of the CLAW Regulations or Regulation 5 of the CAW Regulations as appropriate;

(b) details of the measures to be taken to prevent or adequately control the exposure of those working with or those who may be affected by the substance as required by Regulation 7 of the COSHH Regulations, Regulations 5-16 of the CLAW Regulations or Regulations 7-18 of the CAW Regulations as appropriate.

## 9. REPORT OF THE INVESTIGATION

The Report for each bridge shall include the results of all exploratory works and routine sampling and testing, including all laboratory results.

## 10. COPIES

The Contractor shall provide two copies of any draft report and two copies of the final Report.

## 11. REPORT CONTENT

The Reports shall include the following:

i). location plan/elevations for all testing, investigations and samplings;

ii). all exploratory works undertaken and the results of these works, including all in situ and laboratory test results;

iii). all routine testing and sampling undertaken, including all in situ and laboratory test results, photographs, drawings, etc.

iv). any other information relevant to the investigation.

The report shall be factual except where interpretation and diagnosis are required to explain the findings of the investigation.

## 12. REINSTATEMENT OF AREAS OF INVESTIGATION

12.1 Reinstatement of core holes, drill holes, broken out areas of deck slab shall be undertaken in accordance with Railtrack Company Standards with particular reference to RT/CE/C/008.

| | |
|---|---|
| Section 80 | Structural Concrete |
| Section 83 | Structural Concrete Repairs |
| Section 85 | Concrete for Ancillary Purposes |
| Section 175 | Concrete |

12.2 Reinstatement of the carriageway and footway to be to the approval of the Highway Authority.

# APPENDIX D

## POST TENSIONING SYSTEMS – REFERENCES

## REFERENCES

BA50/93 – "Post-Tensioned Concrete Bridges. Planning, Organisation and Methods for Carrying Out Special Inspections"

Interim Advice Note IA3 – "Post-Tensioned Concrete Bridges, BA 50/93".

CIRIA Guide 106 – Post Tensioning Systems for Concrete in the UK: 1940 - 1985

FIP/5/3 Report on Prestressing Steel 1. Types and Properties

**CURRENT INFORMATION SHEET NO 35**
**SUBJECT: ASSESSMENT OF METAL HOGGING PLATES IN METAL BEAM BRIDGE DECKS**

This Current Information Sheet is issued for guidance purposes only: it is not mandatory. The Assessor must be satisfied that the advice given in this Information Sheet is appropriate to the structural arrangement in question.

## 1 Introduction

Some outmoded forms of construction in wrought iron and steel bridges have presented difficulties for assessors. Hogging metal plates (commonly called buckle plates) are an example of one of these forms of construction, and are commonly found within metal overbridges.

This Current Information Sheet applies to the assessment of metal hogging plates in metal beam bridge decks. It covers both longitudinal and transverse spanning hogging plates.

## 2 Scope

2.1 This document deals with the assessment of the strength of metal hogging plates and includes both longitudinal and transverse spanning hogging plates.

2.2 Plates span between the top flanges of either longitudinal or cross beams within a metal bridge deck. Alternatively, they span between the bottom flanges. The plates are formed from square or rectangular panels, which are curved upwards (hogging). The plates are normally either riveted or bolted down to the supporting beams, as well as being connected by riveted splice plates to each of the adjoining buckle plates. These plates support fill material, either structural or non-structural, which acts as a medium for the dispersal of wheel loadings.

2.3 This document details a method for the quantitative assessment of metal plates, which has been determined from a series of sophisticated analyses covering the range of typical hogging plates that can be found in overbridges. A summary of the investigative work carried out is contained Appendix A. The method of assessment is similar to that of the Modified MEXE Assessment for arches in that a Provisional Wheel Load value is first determined based solely on the plate thickness. The result is then modified by a series of Modifying Factors to account for the influences of span, plate rise, fill depth, any in-plane stiffening and plate position. The resulting value is then adjusted for the material safety factors to produce an Allowable Wheel Load to be compared against BD 21 wheel loading levels.

2.4 This Current Information Sheet covers the assessment of hogging metal plates with the following restrictions. For general details of a hogging plate refer to Figure 1.

- Plate thickness between 8mm and 16mm
- Clear span of plate between 900mm and 2300mm
- Plate rise between 60mm and 105mm
- Fill depth between 300mm and 900mm
- All the hogging plates should either span between top flanges or bottom flanges of the beams.
- A double curvature plate can be conservatively assessed as a single curvature plate.
- The rivets connecting the plate to the girder should be a minimum of 5/8 inch (16mm) diameter and at a maximum pitch of 4 inch (100mm) or equivalent.

# 3 Assessment of Hogging Plates

## 3.1 General Background Information

The assessment method has been developed from a study of the available record information for a large number of metal bridges incorporating hogging plates and the loading that they experience. In particular, the following background information should be noted: -

Live Loading

Critical vehicle loading is applied through a single wheel acting vertically on the road surface. A 300mm square wheel footprint is applied, giving a uniform pressure of approximately 1.1 $N/mm^2$ at the road surface and transferred through the surfacing and fill using a 2:1 dispersion with depth in accordance with BD 21. The distributed wheel load has been investigated in both the central and eccentric arrangements in order to determine the critical wheel load position.

Hogging Plate Dimensions

Figure 1 details the dimensions that are required for the assessment of a hogging plate.

In Plane Stiffening

The study of the different forms of hogging plates has shown that various forms of stiffening have been incorporated in the fabrication of a hogging plate to facilitate the connection to adjoining hogging plates and the supporting girders and also to provide edge stiffening to external hogging plates. These stiffening types are shown in Figure 2.

Generally, it has been found that the addition of plate stiffening significantly increases the capacity of the hogging plates for all support conditions considered. It is also noted that the in-plane stiffening of buckle plates is a more effective method of strengthening than the alternative of providing only ties or cross braces between the supporting girders.

## 3.2 Method of Assessment

The method of assessment is based on the findings of a number of sophisticated analyses undertaken on hogging plates of different dimensions and in plane stiffening with varying depth of cover. The assessment rules are applicable to both wrought iron and steel and also to longitudinal and transverse spanning hogging plates.

The initial assessment is in terms of a maximum allowable axle loading based on an un-stiffened buckle plate and is representative of the most common type of hogging plate detail. This is deemed the Provisional Wheel Load. The effect of the plate support condition is incorporated in the parameters.

This Provisional Wheel Load is then modified by a series of Modifying Factors whose values are related to the bridge specific parameters of span, plate rise, fill depth, in plane stiffening and plate position.

In service corrosion of hogging plates can lead to a global or local reduction of plate thickness. The effects of global corrosion of the plate can be incorporated in the numerical assessment directly by using a reduced thickness in the equation of Section 3.3. Local corrosion can consist of holes in the plates. Where the holes have a diameter not more than six times the plates thickness and they are more than 60 times the plate thickness apart, their effect can be ignored. The effect of holes with a diameter more than this limit or at closer spacing should be considered using a non-linear analysis approach similar to that used in the study as detailed in Appendix A.

3.3 Provisional Wheel Load

The Provisional Wheel Load *PWL* (kN), is obtained from the following equation which is a function of plate thickness $t_p$ (mm):

$$PWL = 618000 \times (t_p/1000)^{1.865} \text{------------------------------(1)}$$

*Where* 8mm $\leq t_p \leq$ 16mm

Extrapolation outside the range shown is not permitted. For ease of use, the following Table gives the commonly found thicknesses of metal hogging plates, together with the associated *PWL*.

| **Plate Thickness (mm)** | ***PWL*** |
|---|---|
| 9.525 (3/8in) | 105 kN |
| 11.1125 (7/16in) | 140 kN |
| 12.7 (½in) | 180 kN |
| 15.875 (5/8in) | 272 kN |

This provisional Assessment is based on the following parameters:-

- Plate span — $S$ = 1200mm (4 ft)
- Plate rise — $R_x$ = 76.2mm (3 in)
- Fill depth — $d_f$ = 350mm (14 in)
- Surfacing constant — 100mm thick

The provisional wheel load obtained is then adjusted by application of the modifying factors in Section 3.4.

3.4 Modifying Factors

The following Modifying Factors are used to adjust the Provisional Wheel Load. Extrapolations outside the ranges shown in each of the Tables below are not permitted.

i) <u>Factor for variation in plate span ($f_{span}$)</u>

| **Plate span (mm)** | **Factor $f_{span}$** |
|---|---|
| 900 | 1.62 |
| 1200 | 1.00 |
| 1500 | 0.69 |
| 1800 | 0.51 |
| 2300 | 0.34 |

$$f_{span} = 1.3574 \times (S/1000)^{-1.6761} \text{-------------------------(2)}$$

*Where* 900mm $\leq S \leq$ 2300mm

ii) Factor for variation in plate rise $f_{rise}$

| Plate rise (mm) | Factor $f_{rise}$ |
|---|---|
| 63.5 (2.5in) | 0.84 |
| 76.2 (3in) | 1.00 |
| 88.9 (3.5in) | 1.106 |
| 101.6 (4in) | 1.213 |

$f_{rise} = 8.373 \times R_x/1000 + 0.362$---------------------------(3)

*Where* 60mm $\leq R_x \leq$105mm

iii) Factor for variation in fill depth $f_{fill}$

| Fill depth (mm) | Factor $f_{fill}$ |
|---|---|
| 300 | 0.95 |
| 350 | 1.00 |
| 400 | 1.05 |
| 450 | 1.10 |
| 500 | 1.15 |
| 550 | 1.20 |
| 600 | 1.25 |
| 750 | 1.40 |
| 900 | 1.55 |

$f_{fill} = d_f/1000 + 0.65$-------------------------------------(4)

*Where* 300mm $\leq d_f \leq$900mm

iv) Factor for plate stiffening effect $f_{stiff}$

| Plate stiffening type (see Figure 2) | | Factor $f_{stiff}$ |
|---|---|---|
| Type | Stiffening arrangement | |
| SI | Single un-stiffened plate (generic model) | 1.00 |
| S2 | Single plate with Tee stiffener(1) | 1.25 |
| S3 | Joined plates with splice plate(2) | 1.40 |
| S4 | Joined plates with Tee stiffener(1) | 2.50 |
| S5 | Vertical stiffener to buckle plate(3) | 3.50 |

(1) Tee stiffener – minimum size 5" x 3" x ⅜"
(2) Splice plate – minimum size 5" x ⅜"
(3) Vertical stiffener – minimum size 2 No 5" x 3" x ⅜" Tees and ⅜" web (12" min depth)

v) Plate Position effect $f_{pos}$

The position of the plate within a deck also affects its load carrying capacity. An edge plate confined by an external and an internal girder has about 50% the capacity of an internal plate.

| **Plate Position** | **Factor $f_{pos}$** |
|---|---|
| Internal Plate | 1.00 |
| Edge Plate | 0.50 |

3.5 Allowable Wheel Load

The Provisional Wheel Load (*PWL)* is first obtained from equation (1) depending on the plate thickness of the structural model to be assessed making suitable allowance for any global corrosion.

The actual ultimate Wheel Load $W_u$ *(kN)* is obtained from the following equation:

$$W_u = 0.9 \times PWL \times f_{span} \times f_{rise} \times f_{fill} \times f_{stiff} \times f_{pos} \text{------------------------} (5)$$

Note that this formula makes an allowance for the dead load and superimposed dead loading applied to the structural arrangement by the insertion of 0.9 in equation (5). This factor gives a conservative value to the ultimate wheel load $W_u$ because it is found that for increasing depths of fill the capacity of applied wheel load also increases in proportion.

The ultimate wheel load obtained should then be further modified to take account of the applied safety factors, $\gamma_m$, $\gamma_{Fl}$ and $\gamma_{F3}$. The allowable wheel load value ($W_a$) thus calculated may then be compared with the appropriate load tables in BD 21 to obtain the assessed live load capacity of the hogging plate.

## 4 Additional information

A typical form of assessment calculation is provided in Appendix B.

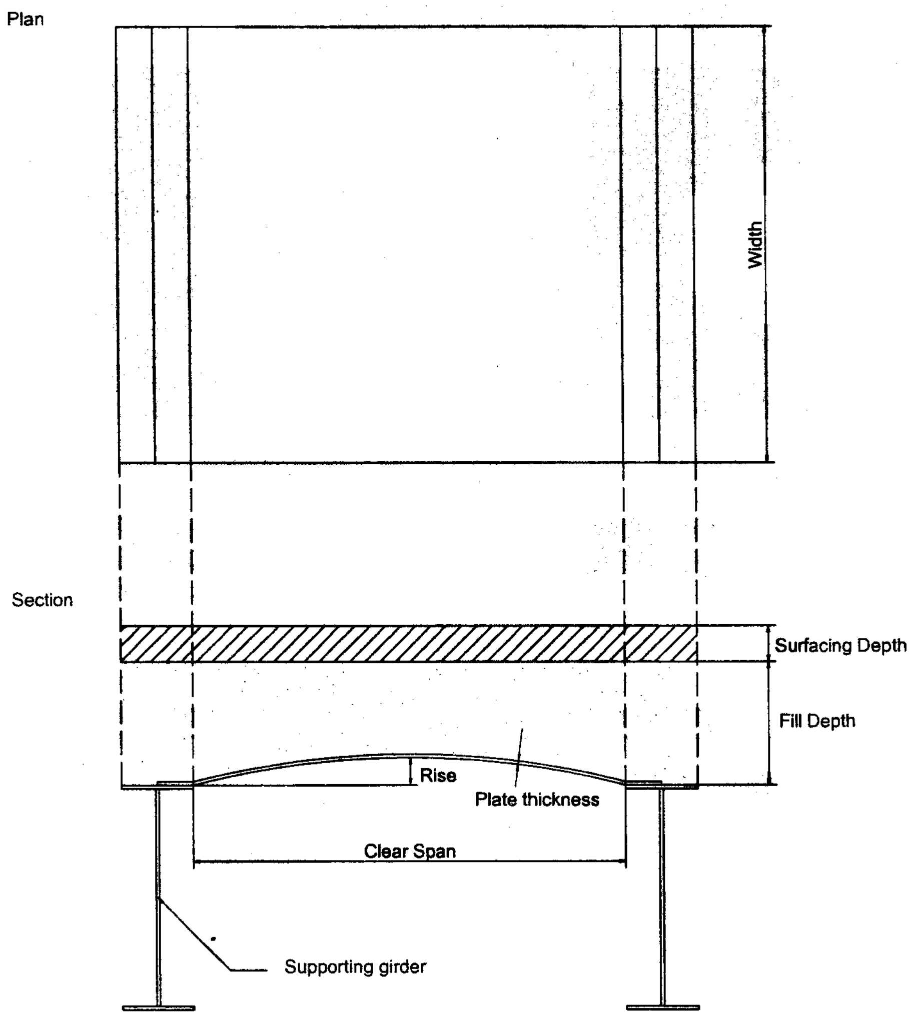
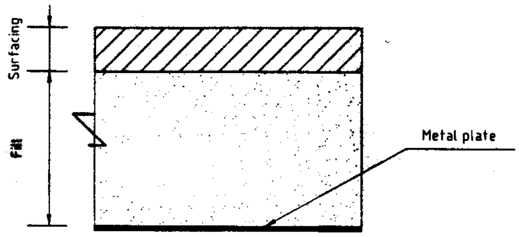


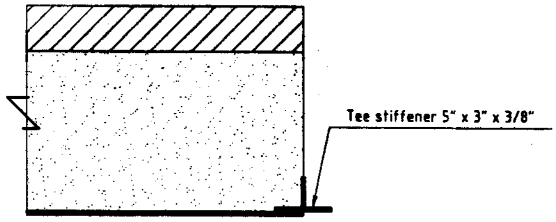

**Figure 1 Typical Hogging Plate Detail**

Note: For hogging plates sitting on top of the bottom flanges of the girders, the fill depth should be measured from the top of the bottom flange.

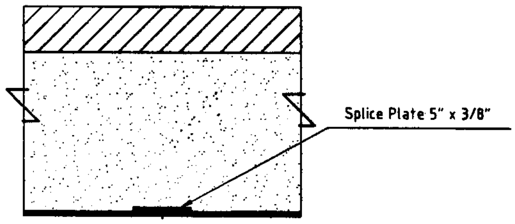
**Type S1 (single un-stiffened plate)**



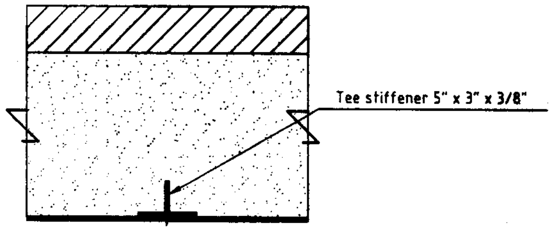

**Type S2 (single plate with Tee stiffener)**




**Type S3 (Joined plates with splice plate)**




**Type S4 (Joined plates with Tee stiffener)**




**Type S5 (Vertical stiffener to hogging plate)**

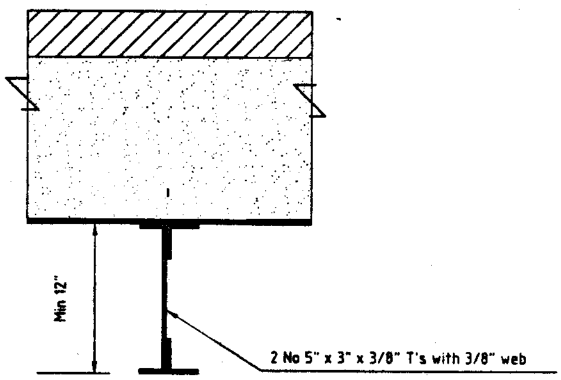



**Figure 2 Types of Metal Plate Stiffeners**
(Minimum stiffener sizes shown)

# APPENDIX A

## SUMMARY OF INVESTIGATIVE WORK

A technical appraisal of current assessment codes and relevant literature was undertaken to determine the potential benefits of carrying out more sophisticated analyses to identify the perceived likely modes of hidden strength of metal floor plates. The Bridgeguard 3 database was also reviewed to compile an index of applicable bridges and to identify common forms of structural types. The largest population of bridge types were found to contain either single curvature arched plates or flat plates supported on the top flanges of the girders. Typical ranges of potential variations for these structural arrangements of metal floor plate and support structures were examined and noted.

A localised parametric finite element (FE) model of a single curvature arched plate was developed with consideration given to idealised flexural simple and fixed boundary conditions supported with both rigid and sprung supports for horizontal movements.

The characteristics of the generic plate model were as follows:

- Span – 1200mm
- Rise – 75mm
- Plate thickness – 9.525mm (3/8in)
- Depth of fill – 350mm
- Depth of surfacing – 100mm

The loading was based on a single wheel, applied over a square contact area, using the dispersal through both the fill and the surfacing as defined in BD21/01 (one horizontal to two vertical). The wheel load was considered to act either centrally over the metal plate, or eccentrically such that the dispersed loading was still fully applied to the plate. It should be noted that the fill material was modelled to act solely as a dispersal medium. It neither acted compositely with the metal plate nor assisted with preventing its deformation. The lateral stability introduced by the dead load of the fill was included but the stiffening effect was ignored.

Geometric and material non-linear analyses were undertaken, and subsequently the parameters of span, geometry and plate thickness were each varied in turn. These variations were:

- Span – 900mm, 1200mm 1500mm, 1800mm, 2300mm
- Rise – 69.85mm (2.75in), 76.2mm (3in), 101.6mm (4in)
- Plate thickness – 9.525mm (3/8in), 11.113mm (7/16in), 12.7mm (1/2in), 15.875mm (5/8in)
- Depth of fill – 350mm, 500mm, 600mm, 900mm

A family of ultimate capacity curves for critical loading conditions was obtained from the analysis results.

In addition global finite element models with single curvature arched buckle plate were set up and validated. These models represented bridges containing longitudinal girders with the metal plates spanning transversely and bridges with cross girders spanning between longitudinal edge girders and supporting metal plates spanning longitudinally. Critical load positions were investigated with geometric and material non-linear solutions.

The work carried out in this phase clearly indicated that the stiffness of the support beams had quite an effect on the capacity of the buckle plates. The load carrying capacity of buckle plates improved when the loading was applied to the inner bays. This was carried out for the buckle plates supported on longitudinal beams.

The effect on load carrying capacity of plates due to effects of stiffeners was investigated for the following types of stiffeners.

- Plate stiffening (T-stiffeners, Plate Stiffeners)
- Plated Support (Lateral vertical plates at intervals)
- Cross Bracing (Angles and T-sections at intervals)

# APPENDIX B

## FORM OF ASSESSMENT CALCULATION

**Example of calculation to assess capacity of metal hogging plate**

## Introduction

The calculation provides a suggested method of obtaining the actual applied wheel loading for a typical metal hogging plate. The structural arrangement is shown in Figure A1.

The following key parameters, which are those having the greatest effect on the capacity of the metal plate, are recorded:

- Plate thickness $t_p$ (mm)
- Plate span S (mm)
- Plate rise $R_x$ (mm)
- Fill depth $d_f$ (mm)
- Position of Plate (internal or edge)

Additional strengthening of the metal plate by various types of stiffening has also to be carefully considered. Five types of stiffening are available for use in the capacity assessment.

After calculation of the Provisional Wheel Loading Assessment *PWL*, using equation (1), this value is adjusted (see Section 3.4) by using the various modifying factors for the parameters noted above.

## Calculation

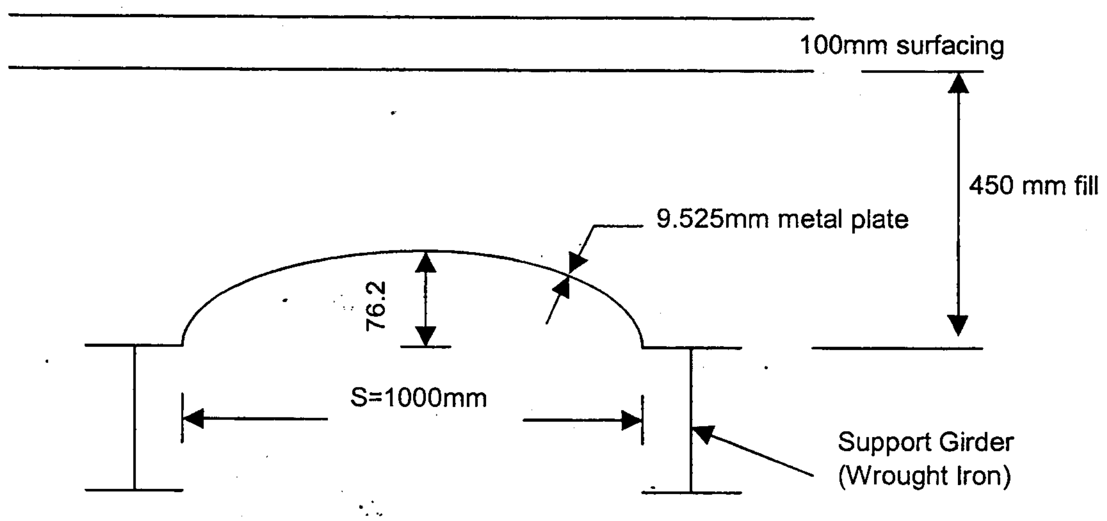



**Figure A1 Structural arrangement of metal hogging plate**

1. Record the following data:

| | | |
|---|---|---|
| i) | Plate thickness------------------- | $t_p$ = 9.525 mm |
| ii) | Plate span------------------------- | S = 1000 mm |
| iii) | Plate rise-------------------------- | $R_x$ = 76.2 mm |
| iv) | Fill depth-------------------------- | $d_f$ = 450 mm |
| v) | Internal Plate | |

2. Obtain the Provisional Wheel Loading Assessment *PWL*

From Section 3.3, using equation (1),

$$PWL = 618000 \text{ x } (t_p / 1000)^{1.865}, \text{ where } t_p = 9.525 \text{ mm}$$
$$\therefore\ PWL = 618000 \text{ x } (9.525 / 1000)^{1.865} = 105 \text{ kN}$$

Note that the above calculation allows for the support conditions of the metal plate. This is because, in the analysis work, the variation between each of the support conditions has been compared, and the weighted average for each of the four plate thicknesses analysed has been entered into equation (1).

3. Obtain values of the required modification factors *f*

Refer to Section 3.4 equations (2) to (4)

i) Plate span $f_{span} = 1.3574 \text{ x } (S/1000)^{-1.6761}$----------------------(2)

For $S$ = 1000 mm,

$$f_{span} = 1.3574 \text{ x } (1000/1000)^{-1.6761}$$

$$\therefore\ f_{span} = 1.36$$

ii) Plate rise $f_{rise} = 8.373 \text{ x } R_x/1000 + 0.362$----------------------(3)

For $R_x$ = 76.2 mm,

$$f_{rise} = 8.373 \text{ x } 76.2/1000 + 0.362$$

$$\therefore\ f_{rise} = 1.00$$

iii) Fill depth $f_{fill} = d_f/1000 + 0.65$-----------------------------------(4)

For $d_f$ = 450 mm,

$$f_{fill} = 450/1000 + 0.65$$

$$\therefore\ f_{fill} = 1.10$$

iv) Plate stiffening effect

The metal plate is connected to its adjoining plates with riveted splice plates.
Therefore, by reference to the Table in Section 3.4, the stiffening type is Type S3.

$$\therefore\ f_{stiff} = 1.40$$

v) Position of Plate

The factor for an internal plate is 1.00.

$$\therefore\ f_{pos} = 1.00$$

4. Obtain the actual wheel loading assessment $W$

Refer to Section 3.5 equation (5)

$$W_u = 0.9 \times PWL \times f_{span} \times f_{rise} \times f_{fill} \times f_{stiff} \times f_{pos} \text{-----------------} (5)$$

$$\therefore \quad W_u = 0.9 \times 105 \times 1.36 \times 1.00 \times 1.10 \times 1.40 \times 1.00$$

$$\therefore \quad W_u = 198 \text{ kN}$$

Since this value is an ultimate (factored) wheel load, it is necessary to adjust this value by dividing the load by the appropriate partial safety factors, $\gamma_m$, $\gamma_{Fl}$ and $\gamma_{F3}$.
In this case, for wrought iron $\gamma_m = 1.20$. Also $\gamma_{Fl} = 1.5$, and $\gamma_{F3} = 1.1$.

The actual allowable wheel loading that can be applied is given by:

$$W_a = 198/(1.20 \times 1.5 \times 1.1)$$

$\therefore$ $\underline{W = 100 \text{ kN}}$ Plate has a capacity of 40 tonne ALL

**CURRENT INFORMATION SHEET NO 36**

**SUBJECT: ASSESSMENT OF THE TORSIONAL BUCKLING STRENGTH OF LONGITUDINAL EDGE GIRDERS IN JACK ARCH BRIDGES**

**This Current Information Sheet is issued for guidance purposes only: it is not mandatory. The Assessor must be satisfied that the advice given in this Information Sheet is appropriate to the structural arrangement in question.**

## 1 Introduction

This Current Information Sheet applies to the assessment of the ultimate capacity of longitudinal edge girders in jack arch bridges which are found to have an inadequate resistance because of the apparent lack of effective lateral restraints to prevent their instability caused by torsional buckling. It only applies to steel or wrought iron edge girders where the form of the bridge comprises longitudinal beams and brick jack arches which support the overlying fill and road construction.

## 2 Scope

2.1 This document details a method for the quantitative assessment of longitudinal edge girders, in jack arch bridges. Figure 1 shows a cross section of the typical structural arrangement considered; structural parts and key parameters have been annotated.

2.2 The method of assessment covers edge girders ranging from 7.5m to 15m span with girder spacing varying between 1.25m and 2.05m and takes account of lateral tie location and area of tie.

2.3 For this method of assessment girders shall consist of flange and web plates joined together by the use of continuous L-angles (cleats). The rivets joining these components have been assumed to provide complete fixity between the components.

2.4 The assessment of brick jack arches and their associated tie configurations should be carried out in accordance with the requirements of Current Information Sheet No 22 to ensure their compliance as load carrying elements.

2.5 The fill has been used in a semi-structural manner, thus this CIS does not cover for waterloged fills and in those adjacent to service bay areas.

2.6 The parapet (masonry or any other form) has not been used structurally in this CIS.

2.7 The edge beams should be of riveted construction with typical uncorroded stiffeners.

## 3 Method of assessment

### 3.1 General Background Information

The assessment method has been developed from a study of available record information for a large number of brick jack arch bridges with longitudinal girders, the loading they experience and a series of non-linear analyses to account for the influences of span, girder spring tie location and area of tie. In particular, the following background information should be noted: -

Wheel Load Distribution

The majority of bridges have footways located adjacent to the edge girder. In these situations the behaviour of the edge girder is evaluated under Accidental Wheel Live loading. The loading has been converted to an equivalent HA udl and kel load representations. This equivalent loading has been applied in a line along the position of the outer wheels using half the overall intensity. It has been assumed that the other half of the loading and line of wheels have an insignificant effect on the edge girder.

Two critical load positions were investigated; one for wheel loads over the crown of the jack arch giving maximum lateral thrust, and one with wheel loads as close to the edge girder as possible.

Girder geometry has been determined from the study of record information. The depth of girder utilised in the analysis (see Fig 1) has been set at Span/12. Other girder dimensions are as shown in Table 3.1 (see Fig.1). Typical arrangements of stiffeners have been modelled with the spacing of joggled T-stiffeners set at span/8.

| Span [m] | 7.5 | 10.0 | 12.5 | 15.0 |
|---|---|---|---|---|
| Flange Width [mm] | 350 | 400 | 450 | 500 |
| Flange Thickness [mm] | 12.7 | 12.7 | 25.4 | 25.4 |
| Web Thickness [mm] | 9.5 | 9.5 | 9.5 generally<br>12.7 at support | 9.5 generally<br>12.7 at support |

**Table 3.1 Span, Girder Flange and Web Parameters Used**

Girder spacings of 1.25m, 1.65m and 2.05m are considered for each girder. The rise of the jack arch has been kept constant at 360mm throughout the study as has the structures of the jack arch.

A total of seven tie configurations are considered for each combination of girder span and girder spacing. Table 3.2 shows the configurations investigated; the cross sectional area of tie per metre length of span and the height of ties above girder soffit. Ties have been located at the stiffener positions. Tie rods are perpendicular to the edge beam.

| **Configuration** | **Cross sectional area of tie [mm$^2$/m]** | **Height of ties above girder soffit [mm]** |
|---|---|---|
| a | 0 | N/A |
| b | 200 | 0 |
| c | 600 | 0 |
| d | 200 | 150 |
| e | 600 | 150 |
| f | 200 | 300 |
| g | 600 | 300 |

**Table 3.2 Tie Configurations Considered**

A number of parameters relating to the fill, surfacing and parapet wall are kept constant and are as follows: -

- Depth of fill and surfacing – 350mm
- Height of parapet wall – 1800mm
- Width of parapet wall – 450mm

Material Properties and Partial Safety Factors

The material properties and partial safety factors are in accordance with BD 21/01 and BD 56/96 and are detailed in Table 3.3. Factors $\gamma_{FL}$ and $\gamma_{F3}$ were applied to the loads in accordance with BD 21/01. The partial factor for material strength $\gamma_m$ was applied to the yield strength of wrought iron both for the girder and the ties.

| Description | Material Parameter | Value | Partial Safety Factors |
|---|---|---|---|
| Wrought Iron | Density | 7700 kg/m$^3$ | $\gamma_{fL}$ 1.05 $\gamma_{f3}$ 1.1 |
| | Young's Modulus | 200 kN/mm$^2$ | |
| | Poisson's Ratio | 0.3 | |
| | Yield Stress | 220 N/mm$^2$ | $\gamma_m$ 1.2 |
| Jack Arch | Density | 22 kN/m$^3$ | $\gamma_{fL}$ 1.2 $\gamma_{f3}$ 1.1 |
| Fill | Density | 22 kN/m$^3$ | $\gamma_{fL}$ 1.2 $\gamma_{f3}$ 1.1 |
| Surfacing | Density | 22 kN/m$^3$ | $\gamma_{fL}$ 1.2 $\gamma_{f3}$ 1.1 |
| Parapet Wall | Density | 22 kN/m$^3$ | $\gamma_{fL}$ 1.2 $\gamma_{f3}$ 1.1 |

**Table 3.3 Material Properties**

Failure Modes

The non-linear analyses, which were carried out as part the study, took into account the following effects in determining the ultimate capacity: -

- Geometric non-linear behaviour due to premature buckling of the edge girder
- Material non-linear behaviour of both the edge girder and tie elements
- Friction/slip at the bearings due to horizontal thrust from the jack arch

Examination of the results of these analyses has shown that: -

i) Without ties and with permanent loading due to the self-weight of a masonry parapet all girder arrangements considered do not slide at the bearing supports. However, without parapets and without ties it is expected that the edge girders would generally slide under live loading.

ii) Without ties the failure mode corresponding to the ultimate girder capacity is one of lateral torsional buckling.

iii) With ties the failure associated with ultimate girder capacity varied according to the girder spacing and the transverse position of the live load. In some cases the girder buckled, whilst in other cases the ties yielded first, which then triggered a sudden failure of the girder.

iv) No single transverse position of the live load, to produce either maximum vertical loading on the girder or to produce maximum horizontal loading on the girder, was found to be critical for all girder arrangements investigated. Hence both extremes have been considered to determine the critical live load condition.

### 3.2 Development of Assessment Method

In all, 168 non-linear analyses have been undertaken to account for the following: -

- Four bridge spans
- Three girder spacings
- Two load positions
- Seven arrangements of ties.

From each ultimate capacity determined, an equivalent ultimate UDL (kN/m) was then determined; ie the ultimate load that the girder could carry.

Using BD 56/96 it was then possible to determine the equivalent effective length for each girder.

### 3.3 Assessment method

The ultimate bending capacity of a longitudinal edge girder in a jack arch bridge complying with the limitations set out in Section 2 should be derived in accordance with the requirements of the Highway Bridges and Structures Assessment Code BD 56/96 (based on the Steel Bridge design code BS 5400: Part 3). However, the determination of the effective length of the girder for lateral torsional buckling should be determined from Table 3.4.

Since the effective length is influenced by the tie configuration the assessor is required to obtain the appropriate value from Table 3.4 and 3.5 for the correct combination of girder span, girder spacing, and tie configuration. Interpolation is allowed for both these tables.

| | | Effective lengths $l_e$ | | | | | | |
|---|---|---|---|---|---|---|---|---|
| **Girder span** | **Girder spacing** | **a*** | **b** | **c** | **d** | **e** | **f** | **g** |
| 7.50 | 1.25 | 1.25 | 1.00 | 0.90 | 1.00 | 0.90 | 1.00 | 0.90 |
| 7.50 | 1.65 | 1.60 | 1.25 | 0.90 | 1.20 | 0.90 | 1.15 | 0.90 |
| 7.50 | 2.05 | 1.80 | 1.40 | 0.80 | 1.40 | 0.90 | 1.35 | 0.90 |
| 10.00 | 1.25 | 0.95 | 0.75 | 0.70 | 0.80 | 0.70 | 0.85 | 0.75 |
| 10.00 | 1.65 | 1.20 | 0.95 | 0.70 | 0.95 | 0.70 | 1.00 | 0.70 |
| 10.00 | 2.05 | 1.65 | 1.15 | 0.70 | 1.10 | 0.70 | 1.15 | 0.70 |
| 12.50 | 1.25 | 0.65 | 0.60 | 0.50 | 0.60 | 0.50 | 0.65 | 0.50 |
| 12.50 | 1.65 | 0.75 | 0.75 | 0.50 | 0.75 | 0.50 | 0.75 | 0.50 |
| 12.50 | 2.05 | 1.10 | 0.85 | 0.45 | 0.85 | 0.50 | 0.90 | 0.55 |
| 15.00 | 1.25 | 0.50 | 0.50 | 0.45 | 0.50 | 0.40 | 0.50 | 0.40 |
| 15.00 | 1.65 | 0.70 | 0.60 | 0.40 | 0.65 | 0.40 | 0.65 | 0.40 |
| 15.00 | 2.05 | 1.00 | 0.75 | 0.40 | 0.75 | 0.45 | 0.75 | 0.45 |

*only applicable where there is a masonry parapet of the approximate proportions or larger as shown in Figure 1.

**Table 3.4 Effective Lengths**

For parameters relating to 'a - g' refer to Table 3.5.

| Configuration | Tie area ($mm^2/m$) | Tie height (mm) |
|---|---|---|
| **a** | 0 | 0 |
| **b** | 200 | 0 |
| **c** | 600 | 0 |
| **d** | 200 | 150 |
| **e** | 600 | 150 |
| **f** | 200 | 300 |
| **g** | 600 | 300 |

**Table 3.5 Effective lengths for range of tie configuration**

After determining the appropriate value of effective length $l_e$ of the girder from Table 3.4, the ultimate bending capacity of the edge girder can then be derived in the normal manner from the assessment code.

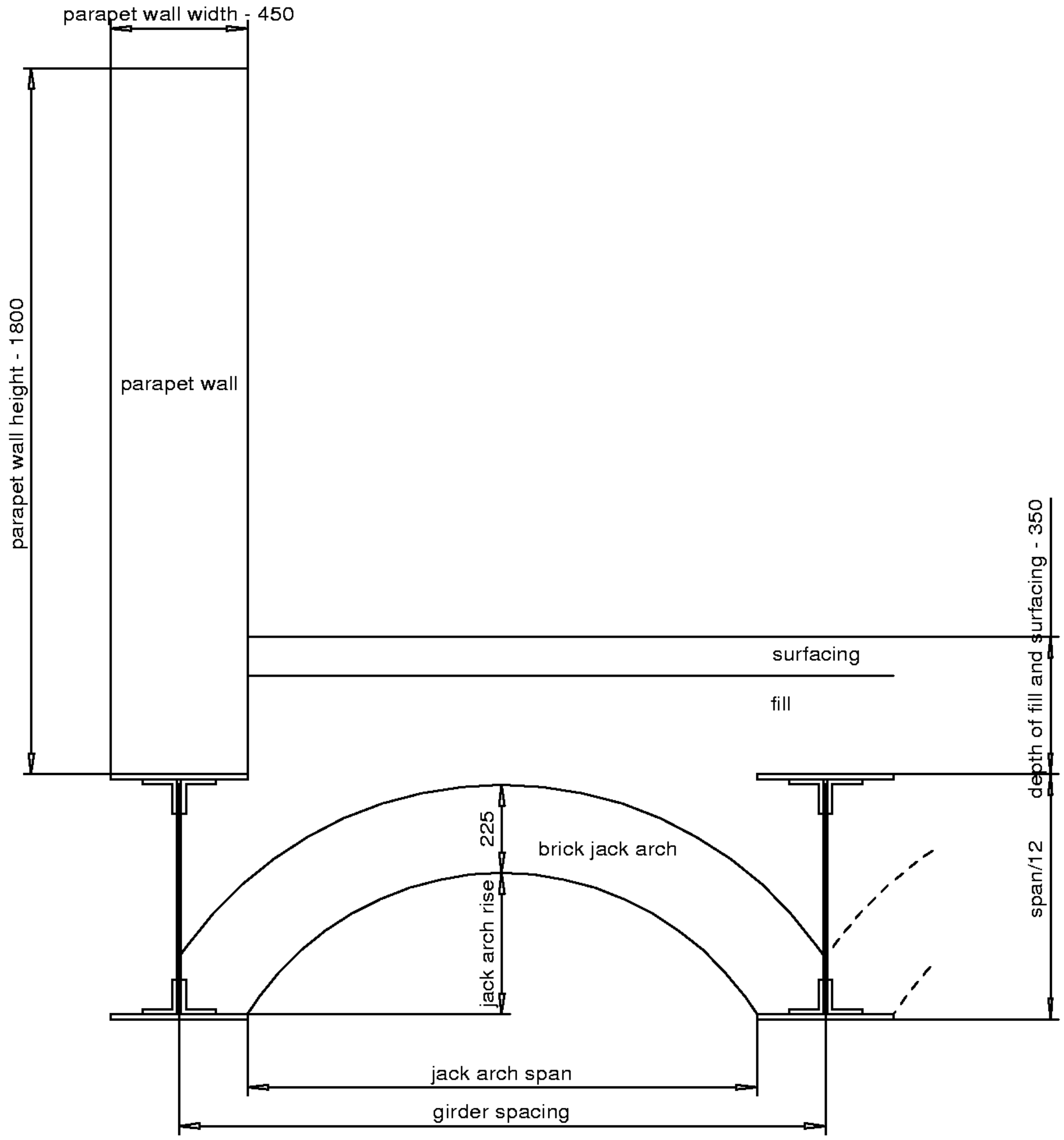



Figure 1 Cross Section of the Typical Structural Arrangement Considered