ESSEX COUNTY COUNCIL
ASSESSMENT CONTRACT 3

APPROVAL IN PRINCIPLE FOR THE ASSESSMENT OF DEBDEN ROAD BRIDGE

ECC BRIDGE NO. 1004
RAIL PROPERTY Ltd BRIDGE NO. AEB/2116

TOPSTOPS

Essex County Council
Transportation & Operational Services

APPROVAL IN PRINCIPLE FOR THE ASSESSMENT OF DEBDEN ROAD BRIDGE

ECC Bridge Number 1004

Rail Property Number AEB/2116

APPROVAL IN PRINCIPLE CONTENTS

- British Railways Board FORM 'AA' (BRIDGES)
- British Railways Board FORM 'AA/1' (BRIDGES)
- Location Plan
- General Arrangement, Cross Section and Idealisation Drawings
- Technical Approval Schedule "TAS" (June 1989)
- Appendix to TAS Schedule dated (June 1989) WS Atkins amended March 1999
- Appendix: Inspection for Assessment

APPROVAL IN PRINCIPLE FOR ASSESSMENT

STRUCTURE / LINE NAME DEBDEN ROAD BRIDGE

ELR / STRUCTURE NO. AEB 2116

BRIEF DESCRIPTION OF EXISTING BRIDGE:

(a) Span Arrangement

The bridge has a clear skew span of 7.43m. The angle of skew is 7°.

(b) Superstructure Type

The deck comprises cast iron beams with brick jack arches.

(c) Substructure Type

Brick abutments.

(d) Details of any Special Features

None

ASSESSMENT CRITERIA

(a) Loadings and Speed

Loadings to be in accordance with BD 21/97. The current permitted traffic speed across the structure is 30mph.

(b) Codes to be used

See attached TAS schedule and March 1999 addendum.

(c) Proposed Method of Structural Analysis

The cast iron beams will be assessed on a worst loaded strip analysis. The jack arches will be assessed as arches springing from the beams using the computer program ARCHIE. For the analysis the following parameters will be adopted.

APPROVAL IN PRINCIPLE FOR ASSESSMENT

STRUCTURE / LINE NAME DEBDEN ROAD BRIDGE

ELR / STRUCTURE NO. AEB 2116

BRIEF DESCRIPTION OF EXISTING BRIDGE:

(a) Span Arrangement

The bridge has a clear skew span of 7.43m. The angle of skew is 17°.

(b) Superstructure Type

The deck comprises cast iron beams with brick jack arches.

(c) Substructure Type

Brick abutments.

(d) Details of any Special Features

None

Superseded.

ASSESSMENT CRITERIA

(a) Loadings and Speed

Loadings to be in accordance with BD 21/97. The current permitted traffic speed across the structure is 30mph.

(b) Codes to be used

See attached TAS schedule and March 1999 addendum.

(c) Proposed Method of Structural Analysis

The cast iron beams will be assessed on a worst loaded strip analysis. The jack arches will be assessed as arches springing from the beams using the computer program ARCHIE. For the analysis the following parameters will be adopted.

ARCHIE

Backing level	None
Masonry self weight	21 kN/m3
Fill self weight	19 kN/m3
Surfacing self weight	23 kN/m3
Ø' for fill	30^{o}
ARCHIE pressure coefficient	0.3
Masonry strength	2.3 N/mm2

Passive pressures generated behind the arch will be limited to 30% of the full passive pressures.

Section sizes and dimensions will be based on drawings AI1877/1004/FIG 01,06

(d) Details of any Special Requirements

None

STRUCTURAL ASSESSMENT ENGINEER'S COMMENTS

CIVIL ENGINEER'S COMMENTS

BRB WORKS GROUP COMMENTS - IF APPLICABLE

APPROVAL IN PRINCIPLE FOR ASSESSMENT

PROPOSED CATEGORY FOR INDEPENDENT CHECK

SUPERSTRUCTURE 1

SUBSTRUCTURE N/A

NAME OF CHECKER SUGGESTED IF CAT 2 OR 3 N/A

THE ABOVE IS SUBMITTED FOR APPROVAL IN PRINCIPLE

SIGNED [redacted]

TITLE ASSESSMENT TEAM LEADER

DATE 22/12/99

FOR AND ON BEHALF OF WS ATKINS CONSULTANTS LTD

CATEGORY 1

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

SIGNED [redacted]

TITLE Senior Civil Engineer

DATE 21/2/00

CATEGORY 2 AND 3

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

SIGNED N/A

TITLE

DATE

APPROVAL IN PRINCIPLE FOR ASSESSMENT

ADDITIONAL INFORMATION REQUIRED FOR BRB PUBLIC ROAD OVERBRIDGES ASSESSED AS PART OF BRIDGEGUARD III

STRUCTURE / LINE NAME DEBDEN ROAD BRIDGE

ELR / STRUCTURE NO. AEB 2116

SCOPE OF ASSESSMENT

An inspection of the structure has been carried out prior to the assessment in order to confirm section sizes and overall dimensions as shown on the drawings. The substructure shows no signs of distress and is deemed satisfactory, therefore no analysis will be carried out.
The beams will be assessed to determine their load carrying capacity using permissible stress analysis and the jack arches will be assessed to determine their capacity at ULS. Should the deck be adequate for 40 Te loading the HB capacity will be determined.
The parapets will not be assessed since they do not meet current standards.

ASSESSMENT CRITERIA

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

See attached TAS schedule and March 1999 addendum.

b) Proposed method of structural analysis

The cast iron beams will be assessed on a worst loaded strip analysis. The jack arches will be assessed as arches springing from the beams using the computer program ARCHIE. For the analysis the following parameters will be adopted.

ARCHIE

Backing level	None
Masonry self weight	21 kN/m3
Fill self weight	19 kN/m3
Surfacing self weight	23 kN/m3
Ø' for fill	30^{o}
ARCHIE pressure coefficient	0.3
Masonry strength	2.3 N/mm2

Passive pressures generated behind the arch will be limited to 30% of the full passive pressures.

Section sizes and dimensions will be based on drawings AI1877/1004/FIG 01,06

c) Planned Highway works / modifications at this site

Intrusive works were carried out and comprised drilling through the arch barrel. There are no other works planned.

d) Road designation / class and whether classed as a heavy load route

Unclassified road. The road is not a heavy load route.

e) Any other requirement

None.

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

SIGNED [redacted]

TITLE County Bridges Manager

DATE 7 February 2000

FOR AND ON BEHALF ESSEX COUNTY COUNCIL TRANSPORTATION AND OPERATIONAL SERVICES DIVISION.

Essex County Council
Transportation & Operational Services

[redacted]
HEAD OF TRANSPORTATION AND
OPERATIONAL SERVICES,
ENVIRONMENTAL SERVICES DIRECTORATE,
COUNTY HALL, CHELMSFORD. CM1 1QH
Telephone [redacted]

WS Atkins

DEBDEN ROAD BRIDGE

Map Ref: TL 553823 237896

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Essex County Council
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A4	DRWG.NO. B1204/1004/LP001–	
	CAD NO. DEB–LP01	
	SCALES 1:2500	
	DATE AUG 99	DRAWN/TRACED TNP
	DATE AUG 99	CHECKED AJS
	DATE	AUTHORISED

SCHEME TITLE

DEBDEN ROAD BRIDGE
SAFFRON WALDEN
LOCATION PLAN

DO NOT SCALE

ECC Bridge No. 1004
Rail Property Board No. EAB/2116

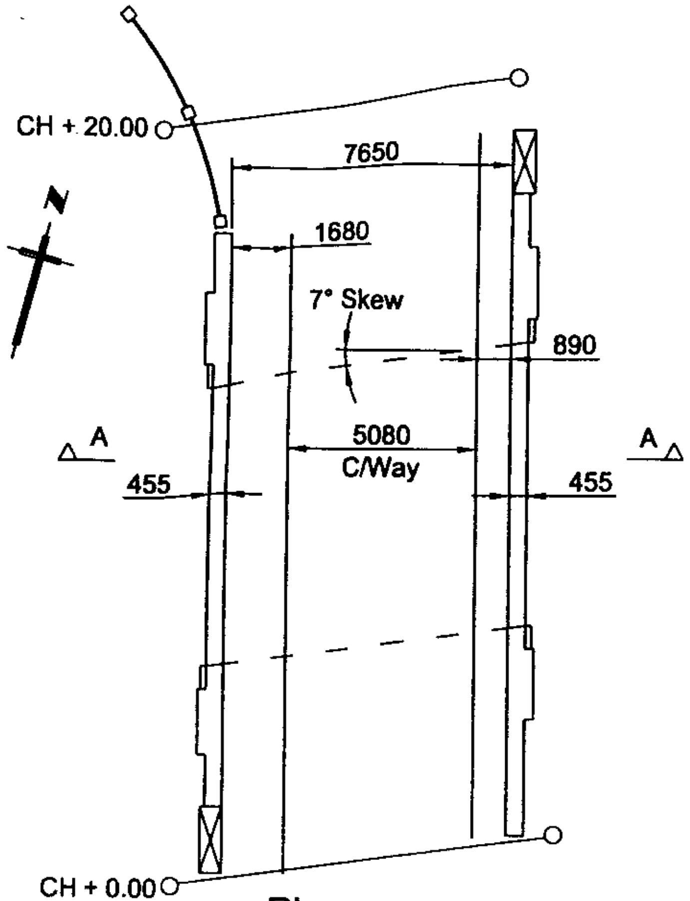
	A	B	C	D	E	F	G
CH 0	10.000	8.998	8.949	9.039	8.980	9.047	9.787
CH 2	9.798	8.995	8.882	8.962	8.906	9.037	9.791
CH 4	9.804	8.924	8.817	8.889	8.829	8.971	9.810
CH 6	9.808	8.854	8.744	8.811	8.749	8.891	9.810
CH 8	9.810	8.780	8.661	8.738	8.676	8.809	9.808
CH 10	9.804	8.669	8.580	8.642	8.579	8.699	9.807
CH 12	9.787	8.598	8.487	8.553	8.494	8.599	9.787
CH 14	9.767	8.509	8.380	8.440	8.368	8.504	9.763
CH 16	9.740	8.384	8.268	8.329	8.244	8.380	9.743
CH 18		8.283	8.139	8.188	8.113	8.238	9.758
CH 20		8.145	7.979	8.042	7.958	8.096	

Table of Levels

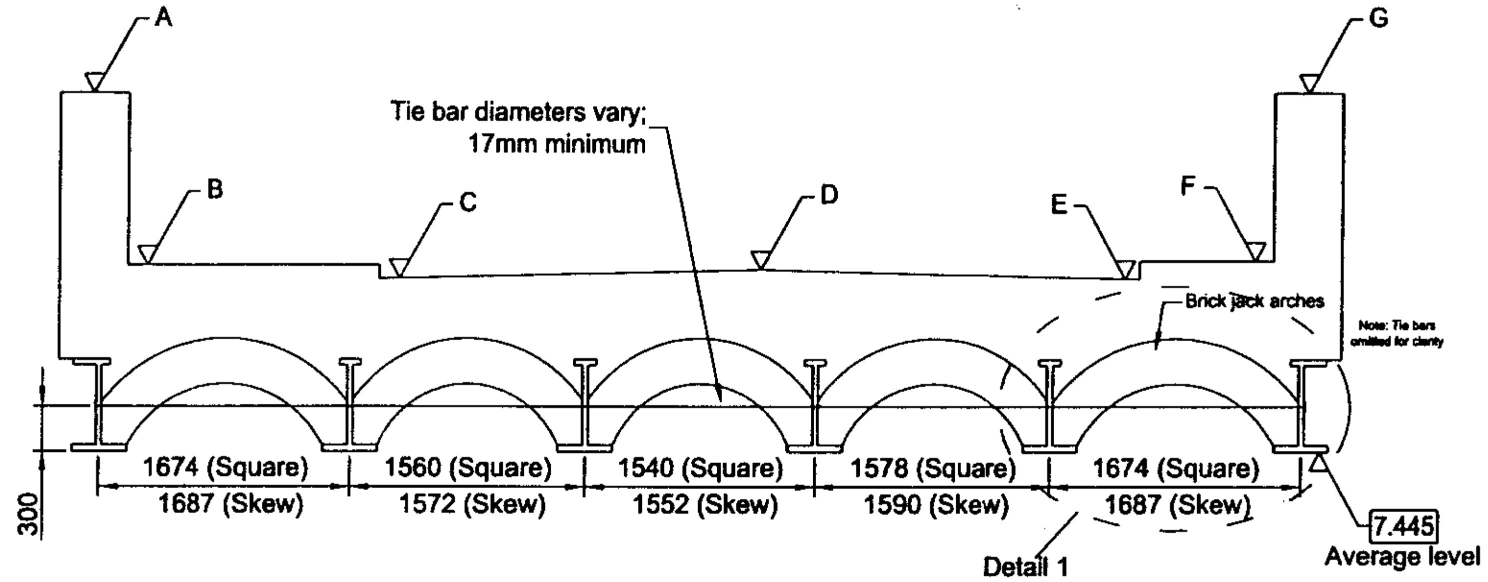
Notes:

All Dimensions in mm

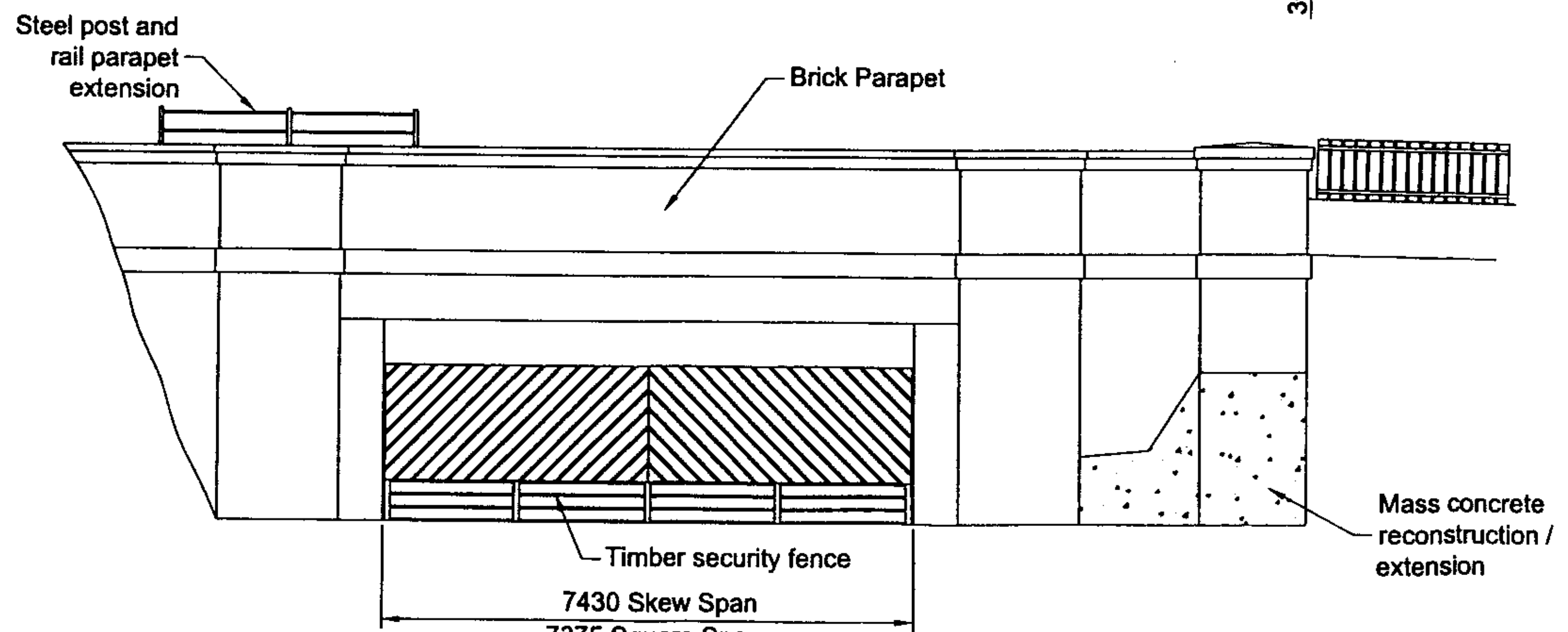
All levels in m above local datum



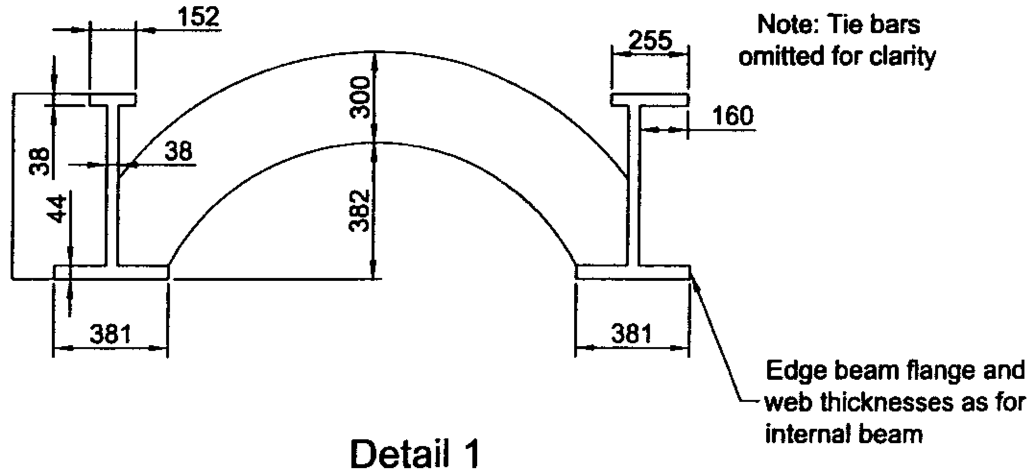
Plan
Scale 1:200



Section A-A
Scale 1:50



East Elevation
Scale 1:100



Detail 1

Essex County Council
Transportation & Operational Services

[redacted]

Atkins

SCALES		SURVEYED	LEVELLED	DESIGNED	DRAWN/TRACED	CHECKED	AUTHORISED	
As Shown	INITIALS	JF	JF		MAD	JF	JF	
	DATE	OCT 99	OCT 99		DEC 99			
CAD FILE N:\AI1877\61\01	REVISION NOTES			REVISION		CHECKED		
	Revision to Skew angle				MAD			

DESCRIPTION OF DRAWING — Sheet 1 of 1

Debden Road Bridge, Saffron Walden
GA Elevation and Cross section

SCHEME TITLE

ECC Assessment Contract 3
Rail Property Ltd Bridges

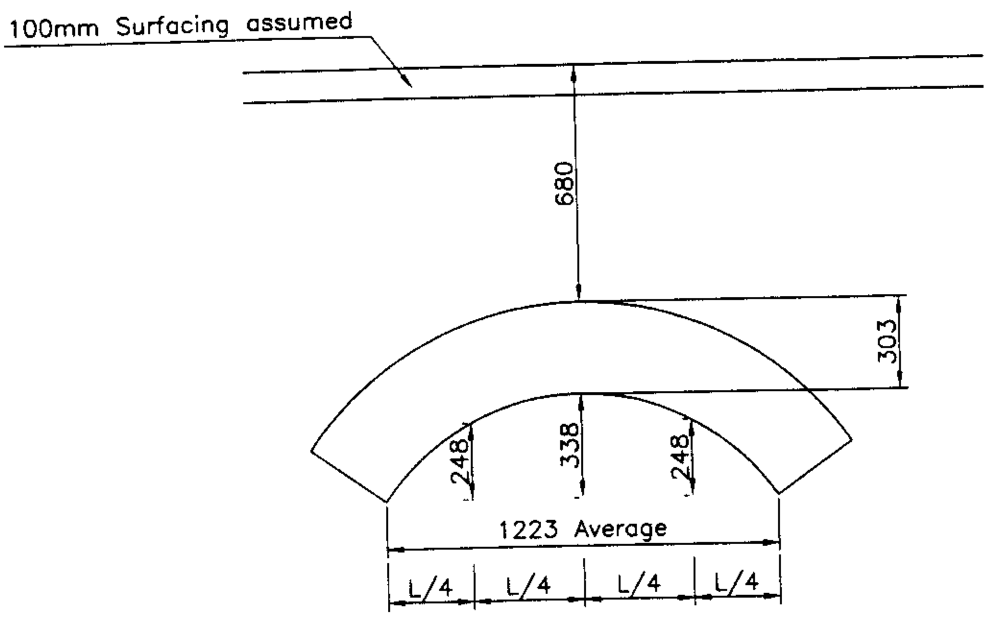
DRAWING NO. AI1877\DWGS\1004\FIG 01

Essex County Council
Transportation & Operational Services

[redacted]

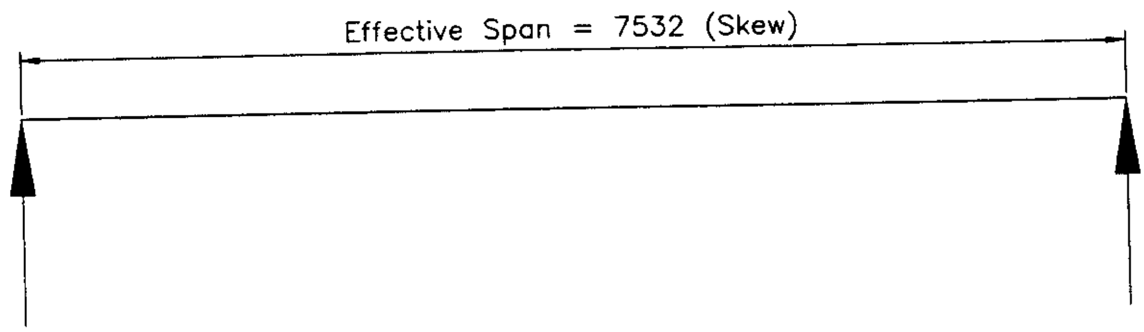
WS Atkins

ECC Bridge No. 1004
Rail Property Board No. AEB/2166



JACK ARCH

Note: All dimensions shown are average for all jack arches



CAST IRON BEAM SECTION

DRWG.NO.	AI1877/dwgs/61/fig06
CAD NO.	N:AI1877/-61-06
SCALES	NTS
DATE NOV 99	DRAWN/TRACED SD
DATE NOV 99	CHECKED JF
DATE	AUTHORISED

SCHEME TITLE ECC ASSESSMENT CONTRACT 3
DEBDEN ROAD BRIDGE, SAFFRON WALDEN
IDEALISATION DIAGRAMS

TECHNICAL APPROVAL SCHEDULE

TECHNICAL APPROVAL SCHEDULE "TAS" (JUNE 1989)

SCHEDULE OF DESIGN DOCUMENTS RELATING TO HIGHWAY BRIDGES & STRUCTURES (All documents are taken to include revisions current at date of this TAS).

1. BRITISH STANDARDS

BS 153 Part 3A, Specification for Steel Girder Bridges (see BE 1/77).

BS 5268 Part 2, Structural Use of Timber

BS 5400 Steel concrete and composite bridges

Part 1: 1978 - General Statement (SEE BD 15/82)

Part 2: 1978 - Specification for loads (see BD 14/82)

Part 3: 1982 - CP for design of steel bridges (see BD 13/82)

Part 4: 1984 - CP for design of concrete bridges (see BD 24/84)

Part 5: 1979 - CP for design of composite bridges (see BD 16/82)

Part 9: 1983 - Bridge bearings (see BD 20/83)

Part 10: 1980 - CP for fatigue (see BD 9/81)

BS 5628: Part 1: 1978 - Unreinforced Masonry

BS 5930: 1981 - Site investigations

BS 6031: 1981 - Earthworks

2. BRITISH STANDARD CODES OF PRACTICE

CP 114 Part 2 Reinforced concrete in buildings (see Tech Memo BE 1/73)

CP 116 Part 2 The structural use of precast concrete (see Tech Memo BE 1/73)

CP 118 The structural use of aluminium

CP 2 Earth retaining structures

CP 2004 Foundations

3. PUBLICATIONS (HMSO)

~~Railway construction and Operation Requirements, Structural and Electrical clearances (1977).~~

~~Railway construction and operation. Requirements for passenger lines and recommendations for goods lines 1950 (reprinted 1970).~~

~~Roads in urban areas and Metric Supplement (as amended by TA 32/82)~~

~~Layout of roads in rural areas and Metric Supplement (as amended by TA 28/82).~~

~~Specification for Highway Works and Notes for Guidance (1986 Edition).~~

~~Highway Construction Details (1987 Edition).~~

~~Simplified Tables of External loads on Buried Pipelines (1970).~~

4. MISCELLANEOUS

~~Circular Roads No 61/72 - Routes for heavy and high abnormal loads.~~

5. TECHNICAL MEMORANDA (BRIDGES)

~~BE 5 The design of Highway bridge parapets (4th revision)~~

~~BE 27 Waterproofing and surfacing of bridge docks.~~

~~BE 3/72 Expansion joints for use La highway bridge docks.~~

~~BE 1/73 Reinforced concrete for highway structures (Relevant ports for the design of buried precast concrete pipes and sign/signal gantries only).~~

~~BE 1/74 The independent checking of erection proposals and temporary works details for major highway structure an trunk roads and motorways.~~

~~BE 8/75 Painting of concrete highway structures~~

~~BE1/77 Standard highway loadings (Relevant parts for the design of buried precast concrete pipes and sign/signal gantries only)~~

~~BE 7/77 Department standard (interim) motorway sign/signal gantries~~

~~BE 1/78 Design criteria for footbridges and sign/signal gantries (Relevant for the design of sign/signal gantries only)~~

~~BE 3/78 Reinforced earth, and anchored earth retaining walls and bridges abutments for embankments~~

6. HIGHWAYS TECHNICAL MEMORANDA

~~H 14/76 Noise barriers - Standard and Materials~~

7. MEMORANDA (BRIDGES)

~~IM 5 Formation of continuity joints in bridge decks~~

8 DEPARTMENTAL STANDARDS

8.1 TRAFFIC ENGINEERING AND CONTROL

~~TD 2/78 Pedestrian Subways - layout and dimensions~~

~~TD 3/79 Combined pedestrian and cycle subways - layout and dimensions~~

~~TD 9/81 Road layout and geometry. Highway link design~~

~~TD 19/83 Safety fences and barriers~~

~~TD) 27/86 Cross Sections and headroom~~

8.2 BRIDGES AND STRUCTURES

BD 2/89 Technical approval of DTp highway structures on motorways and other trunk roads

~~BD 6/81 Approval in principle and calibrating of computer programs for use in DTp highway structures on trunk roads and motorways~~

~~BD 7/81 Weathering steel for highway structures~~

~~BD 9/81 Implementation of BS 5400 Pt 10, CP for fatigue~~

~~BD 10/82 Design of highway structures in areas of mining subsidence~~

~~BD 12/82 Corrugated steel buried structures~~

~~BD 13/82 Design of steel bridges - 'Use of BS 5400 Pt 3: 1982~~

~~BD 14/82 Loads for highway bridges - Use of BS 5400 Pt 2: 1978~~

~~BD 15/82 General principles - Use of BS 5400 Pt 1 : 1978~~

~~BD 16/82 Design of composite bridges - 'Use of B3 5400 Pt 5 , 1979~~

~~BD 19/83 Standard Bridges~~

~~BD 20/83 Bridge Bearings - 'Use of BS S400 Part 9: 1983~~

~~BD 21/84 The assessment of highway bridges and structures~~

~~BD 24/84~~	~~Design of concrete bridges - Use of BS 5400 Pt 4: 1984~~
~~BD 26/86~~	~~Design of lighting columns~~
~~BD 27/86~~	~~Materials for the repair of concrete highway structures~~
~~BD 28/87~~	~~Early thermal cracking of concrete~~
~~BD 29/87~~	~~Design criteria for footbridges~~
~~BD 30/87~~	~~Backfilled retaining walls and bridge abutments~~
~~BD 31/87~~	~~Buried concrete box type structures~~
~~BD 32/88~~	~~Piled foundations~~
~~BD 34/88~~	~~Assessment and Strengthening of Highway Structures on Motorways and other Trunk Roads~~
~~BD 35/88~~	~~Quality Assurance Scheme for paints and similar protective coatings~~
~~BD 36/88~~	~~The Evaluation of Maintenance Costs in Comparing Alternative Designs for Highway Structures~~
BD 37/88	Loads for Highway Bridges

APPENDIX TO TAS SCHEDULE DATED JUNE 1989
(WS Atkins amended March 1999, incorporating relevant technical standards published since June 1989)

1. BRITISH STANDARDS

BS 4360: 1990 - Specification for Weldable Structural Steel.

BS 4466: 1989 - Scheduling, Dimensioning, Bending and Cutting of Steel Reinforcement for Concrete

BS 5400 - Steel, Concrete and Composite Bridges.
- Part 1: 1988 - General Statement (see BD 15/92).
- Part 4: 1990 - CP for Design of Concrete Bridges (see BD 24/92).

BS 5628 - Use of Masonry.
- Part 1: 1992 - Unreinforced Masonry.
- Part 2: 1985 - Reinforced and Prestressed Masonry.

BS 5975: 1996 - CP for Falsework

BS 6651: 1992 - CP for Protection of Structures Against Lightning.

BS 6779 - Highway Parapets for Bridges and Other Structures
- Part 1: 1998 - Specification for Vehicle Containment Parapets of Metal Construction.
- Part 2: 1991 - Specification for Vehicle Containment Parapets of Concrete Construction.
- Part 3: 1994 - Specification for Vehicle Containment Parapets of Combined Metal and Concrete Construction.

BS 7295: 1990: Parts 1 & 2 - Fusion Bonded Epoxy Coated Carbon Steel Bars for the Reinforcement of Concrete

BS 7668: 1984 - Weldable Structural Steels. Hot Finished Structural Hollow Sections in Weather Resistant Steels

BS 8002: 1994 - CP for Earth Retaining Structures.

BS 8004: 1986 - CP for Foundations.

BS 8118 - Structural Use of Aluminium.

BS EN 10025: 1993 - Specification for Hot Rolled Products of Non-alloy Structural Steels - Technical Delivery Conditions.

BS EN 10113: Parts 1-3 - Hot Rolled Products in Weldable Fine Grain Structural Steel.

BS EN 10155: 1993 - Structural Steel with Improved Atmospheric Corrosion Resistance. Technical Delivery Conditions.

3. DoT PUBLICATIONS (HMSO)

Manual of Contract Documents for Highways Works:

- Volume 1: Specification for Highway Works.
- Volume 2: Notes for Guidance on the Specification for Highways Works.
- Volume 3: Highway Construction Details.
- Volume 4: Bills of Quantities for Highways Works.

8. DEPARTMENTAL STANDARDS

8.1 TRAFFIC ENGINEERING AND CONTROL

TD 9/93 - Road Layout and Geometry. Highway Link Design.

TD 27/96 - Road Geometry Links - Cross Sections and Headrooms.

TD 32/93 - Wire Rope Safety Fences.

TD 36/93 - Subways for Pedestrians and Pedal Cyclists - Layout and Dimensions.

8.2 BRIDGES AND STRUCTURES

BD 10/97 - Design of Highway Structures in Areas of Mining Subsidence.

BD 12/95 - Design of Corrugated Steel Buried Structures with Spans not Exceeding 8m, Including Circular Arches.

BD 13/90 - Design of Steel Bridges. Use of BS 5400 Pt 3: 1982.

BD 15/92 - General Principles for the Design and Construction of Bridges - Use of BS 5400 Pt 1: 1988.

BD 20/92 - Bridge Bearings. Use of BS 5400 Pt 9: 1983.

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

BD 24/92 - Design of Concrete Bridges - Use of BS 5400 Pt 4: 1990.

BD 26/94 - Design of Lighting Columns.

BD 33/94 - Expansion Joints for Use in Highway Bridge Decks.

BD 34/90 -	Technical Requirements for the Assessment and Strengthening Programme for Highway Structures on Motorways and Other Trunk Roads. Stage 1 - Older Short Span Bridges and Retaining Structures.
~~BD 35/93 -~~	~~Quality Assurance Schemes for Paints and Similar Protective Coatings.~~
~~BD 36/92 -~~	~~Evaluation of Maintenance Costs in Comparing Alternative Designs for Highway Structures.~~
~~BD 41/97 -~~	~~Reinforced Clay Brickwork Retaining Walls of Pocket Type and Grouted Cavity Type Construction~~.
~~BD 42/94 -~~	~~Design of Embedded Retaining Walls and Bridge Abutments (Unpropped or Propped at the Top).~~
~~BD 43/90 -~~	~~Criteria and Materials for the Impregnation of Concrete Structures.~~
~~BD 44/95 -~~	~~The Assessment of Concrete Highway Bridges and Structures.~~
~~BD 45/93 -~~	~~Identification Marking of Highway Structures~~.
~~BD 46/92 -~~	~~Technical Requirements for the Assessment and Strengthening Programme for Highway Structures.~~ ~~Stage 2 - Modern Short Span Bridges~~.
~~BD 47/94 -~~	~~Waterproofing and Surfacing of Concrete Bridge Decks~~.
~~BD 48/93 -~~	~~The Assessment and Strengthening of Highway Bridge Supports.~~
~~BD 49/93 -~~	~~Design Rules for Aerodynamic Effects on Bridges~~.
~~BD 50/92 -~~	~~Technical Requirements for the Assessment and Strengthening Programme for Highways Structures~~. ~~Stage 3 - Long Span Bridges~~.
~~BD 51/98 -~~	~~Design Criteria for Portal and Cantilever Sign/Signal Gantries~~.
BD 52/93 -	The Design of Highway Bridge Parapets.
~~BD 53/95 -~~	~~Inspections and Records for Road Tunnels~~.
~~BD 54/93 -~~	~~Post Tensioned Concrete Bridges. Prioritisation of Special Inspections~~.
~~BD 56/96 -~~	~~The Assessment of Steel Highway Bridges and Structures~~.

~~BD 57/95 - Design for Durability~~.

~~BD 58/94 - The Design of Concrete Highway Bridges and Structures with External and Unbonded Prestressing.~~

~~BD 60/94 - The Design of Highway Bridges for Vehicle Collision Loads.~~

~~BD 61/96 - The Assessment of Composite Highway Bridges.~~

~~BD 62/94 - As Built, Operational and Maintenance Records for Highway Structures~~.

~~BD 63/94 - Inspection of Highway Structures.~~

~~BD 65/97 - Design Criteria for Collision Protector Beams.~~

~~BD 67/96 - Enclosures of Bridges~~.

~~BD 68/97 - Crib Retaining Walls~~.

~~BD 70/97 - Strengthened / Reinforced Soils and Other Fills for Retaining Walls and Bridge Abutments (Use of BS 8006: 1995).~~

~~SD 4/92 - Procedure for Adoption of Proprietary Manufactured Structures~~.

Appendix

Inspection for Assessment

BRIDGE INSPECTION DETAILS AND CONDITION RATING

ECC Bridge No.: 1004

Rail Property Ltd Bridge No.: AEB/2116

Bridge Name : Debden Road Bridge

Location : Debden Road Bridge, Saffron Walden, Essex
Grid reference TL 53823 37896

Date of Inspection : 20 October 1999
Weather : Overcast & cold

Description : Single span structure comprising cast iron beams with brick jack arches. The structure has brickwork abutments and parapets. The west elevation has been backfilled to carriageway level.

Inspection Method : Hands on

CONSULTING ENGINEERS CONDITION RATING		
	****	Satisfactory Condition
	***	Repairs Required
✓	**	Urgent Repairs Required
	*	Bridge In Dangerous Condition

To be filled in by Essex County Council

		Date
Inspected by	[redacted]	20 Oct 1999
Prepared by		Nov 1999
Checked by		Dec 1999

BRIDGE CLIENT		BRIDGE NO 1004	
File	Initial	Date	Suggested Condition Rating
Read by			
Read by			
Comments			

Index

Section	Description	Page No.
1	**Introduction**	1
2	**Reference Drawings**	2
3	**Inspection Procedure**	3
4	**Condition Report**	4
5	**Intrusive Investigation Report**	8
6	**Conclusions**	9
7	**Recommendations for Assessment**	11
	Appendix – A : Photographs	
	Appendix – B : Defect Diagrams	
	Appendix - C : Statutory Undertakers	

1.0 INTRODUCTION

1.1 Essex County Council (ECC) entered into an agreement with Rail Property Ltd to assess Rail Property Ltd owned bridges carrying publicly maintainable highways. WS Atkins Consultants Ltd – Essex (WSAE) have been appointed by ECC to carry out the visual inspections and assessments of the bridges.

1.2 Debden Road Bridge carries an unclassified road over a dismantled railway line in Saffron Walden, Essex OS Ref. TL 53823 37896.

1.3 An inspection of the structure was carried out on 20 October 1999. The client supplied previous assessment data giving the cast iron beam details and some dimensions of the structure. The inspection included a visual inspection and dimension survey to confirm structural details. The weather was overcast and cold during the inspection.

1.4 The results of the inspection are presented within the text of this report.

1.5 The bridge is a single span structure comprising cast iron beams with brick jack arches and is supported on brickwork abutments. The clear square span is 7.375m and the bridge has a skew of 7°. The parapets are of brickwork construction with a steel post and rail height extension at the south end. The west elevation has been backfilled to carriageway level and now retains a car park for a new residential development. A wooden fence prevents access under the structure.

1.6 The carriageway is 5.08m wide with a 0.89m and 1.68m wide footway on the east and west side of carriageway respectively. The vertical alignment of the carriageway is a gentle hog curve and the horizontal alignment is straight.

1.7 There is a 22 tonnes weight restriction on the structure.

2.0 REFERENCE DRAWINGS

2.1 Rail Property Ltd provided drawings prior to the inspection. The drawing reference is:

Extracts from Examination Report	Dated 30 October 1963

2.2 Following the inspection, survey drawings are produced as below and enclosed in the Approval in Principle for Assessment.

AI1877/DWGS/1004/FIG 01	GA Elevation and Cross Section

2.3 Following the inspection, defect diagrams are produced as below and enclosed in appendix B.

AI1877/DWGS/1004/FIG 02	Defect Diagrams : Abutments
AI1877/DWGS/1004/FIG 03	Defect Diagrams : Parapets
AI1877/DWGS/1004/FIG 04	Defect Diagrams : Elevations
AI1877/DWGS/1004/FIG 05	Defect Diagram : Soffit

3.0 INSPECTION PROCEDURE

3.1 The inspection was undertaken on 20 October 1999. Reference was made to the Bridge Inspection Guide (HMSO 1983) and the Department of Transport standard BD21/97 and advice note BA16/97.

3.2 The visual inspection of the structure was carried out to determine the condition of the bridge. The inspection was carried out within touching distance. Where required, access to the higher level elements of the structure was gained using a ladder.

3.3 A full level and dimensional survey was undertaken. Details of the levels and dimensions taken during the inspection are indicated on Drawings No. AI1877/DWGS/1004/FIG 01 which are included in the Approval in Principle.

3.4 The extent and severity of all defects were recorded. The photographs in Appendix A and the defect diagrams (Drawing Nos. AI1877/DWGS/1004/FIG 02, FIG 03, FIG 04 and FIG 05) in Appendix B illustrate the defects.

3.5 The intrusive investigation works comprised drilling a hole through the crown of a single jack arch barrel. The results of this investigation are given in Section 5.0 of this report.

4.0 CONDITION REPORT

4.1 Foundations

4.1.1 The foundations were not accessible during the inspection. No evidence of any movement or distress was detected.

4.2 Abutments

4.2.1 The substructure of the bridge consists of brickwork abutments. Large areas of the abutments (particularly to the west where backfill for the car park is placed) are buried and could therefore not be inspected (Photo 14).

4.2.2 A service pipe suspended from a cast iron beam has been passed under the structure through both abutments. As such, small areas of brickwork around this have been reconstructed (Photo 3).

4.2.3 The abutments appear to be in fair condition with the following defects identified:

- Large areas of graffiti cover both abutments. Some of the graffiti is offensive, however since access under the bridge is restricted it will not be viewed by the public (Photo 3).
- There are several individual bricks missing from the south abutment with the voids up to depths of 130mm (Photo 15).
- Minor spalling has occurred in small areas on both abutments.
- There is leach staining on the south abutment under the east padstone as well as on the east side face of both abutments (Photo 12).

4.3 Wing walls

4.3.1 The west elevation has been buried up to carriageway level due to backfilling for a car park to a nearby residential development. Inspection was therefore not possible. An area of the car park is vertically displaced by up to 200mm indicating settlement or movement of the fill below (Photo 8). From beneath the bridge, the backfill material used to raise the ground level to above the deck soffit does not appear to have been well placed or compacted.

4.3.2 An area of the east wing wall (possibly where the embankment used to cover the wall) has been repaired or extended with mass concrete as part of the nearby residential development.

4.3.3 The east wing walls are in fair condition with the following defects identified:

- Dense ivy and vegetation growth cover large areas of the wing wall.
- Spalling brickwork to depths of 50mm and mortar loss can be found throughout, particularly along the parapet plinth course (Photo 6).
- The mass concrete used in the section to the north is poor quality with loss of cement paste and subsequent loss of aggregate throughout.
- A 2mm wide diagonal crack runs up the upper half of this mass concrete section.

4.4 Deck beams and jack arches

4.4.1 The beams are cast iron and the jack arches are constructed from London Stock type brickwork with lime mortar.

4.4.2 The cast iron beams are in fair condition with areas of the bottom flange and edges displaying surface corrosion (Photos 4 and 10). The west edge beam could not be fully inspected due to back fill for the car park discussed in section 4.3.

4.4.3 The tie rods are severely corroded with up to 70% section loss (Photo 16).

4.4.4 The brick jack arches are in poor condition with areas of spalling to a depth of 75mm and loss of joint mortar to 60mm (Photo 5). There is graffiti and black staining throughout as well as a small area of leaching. Several individual bricks rang hollow when tapped with a hammer suggesting localised separation of the arch rings.

4.5 Embankments

4.5.1 The embankments adjacent to the south east wing wall are heavily overgrown and show no signs of any significant erosion or slippage.

4.5.2 There are no embankments to the west as the ground level has been raised as part of a nearby residential development. The embankment adjacent to the north east wing wall has been mostly removed with the remaining embankment in good condition and forming part of the gardens for nearby flats.

4.6 Parapets

4.6.1 The parapets comprise 455mm thick brickwork with a capping of a course of blue brickwork. There are lengths of steel post and railing fencing on top of the parapet at the south east and south west ends of the parapets. No vertical movement joints were found along the parapets.

4.6.2 The west parapet is now redundant as the level behind has been raised to road level to accommodate a car park.

4.6.3 The parapets are in poor condition with the following defects identified:

- Brickwork is missing to a depth of 230mm from the carriageway face of the west abutment (Photo 7). There are several smaller areas of spalling and brickwork loss to depth up to 130mm on both parapets.
- Mortar loss up to 50mm deep has occurred along the full length of the lower half of the east parapet carriageway face. Further isolated areas of mortar loss have occurred throughout both parapets.
- Dense ivy growth covered the south east and the north west pilasters. Smaller areas of vegetation growth are also found on both parapets.
- One of the steel posts on top of the east parapet is bent and the top rail has displaced and is also bent. There is surface corrosion and paint loss to both the east and west post and rail sections (Photo 11).
- It is thought that the steel rails raise the parapet height to acceptable level as they coincide with the raised vertical alignment of the carriageway and footway.
- There are several vertical cracks up to 2mm wide, some of which run the full height of the parapets. In all cases the cracks only appear on the carriageway faces.
- The coping unit to the south west pilaster has been displaced, possibly by vehicular impact. There is mortar loss and spalling along the joint between the coping unit and the pilaster (Photo 13).
- There is a hairline crack running through the north west coping unit.
- The south end of the east parapet has been recently reconstructed. There are several loose and missing bricks where the new construction joins the old (Photo 9).

4.7 Road Surface

4.7.1 The road surface over the bridge deck is in good condition with the exception of very minor polishing in the wheel tracks.

4.7.2 The observed traffic flow was less than 7 heavy goods vehicles per hour. This is concluded to be representative.

4.8 Footway Surface

4.8.1 The footway surfacing is in good condition with the exception of a minor depression in the west footway due to a poor reinstatement.

4.9 Waterproofing

4.9.1 The areas of leach staining on the abutments indicate the lack of an effective waterproofing system (Photo 12). Since no intrusive testing was carried out on the upper surface of the deck this could not be confirmed. However, extracts from an Examination Report supplied by the client indicate the presence of a bituminous waterproofing layer. If this still remains it has failed in some locations.

4.10 Signing

4.10.1 The 22 ton weight restriction signs are in good condition.

5.0 INTRUSIVE INVESTIGATION REPORT

5.1 A jack arch barrel thickness of 300mm was obtained from the intrusive investigation.

6.0 CONCLUSIONS

6.1 The structure is in fair condition overall. As well as element specific remedial work there are several areas of spalling, loose and missing brickwork and mortar loss through out the structure that require repair.

6.2 The abutments are in fair condition. However much of the abutments are buried by backfill to support housing and parking facilities adjacent to the bridge. Minor spalling has occurred and several individual bricks are missing. These will require repair. Large areas are covered with graffiti but these are not considered serious since public access under the bridge is restricted.

6.3 The west elevation is buried and could not be inspected. However, settlement of the backfill has occurred with a depression in the adjacent car park surfacing. This will require monitoring for further deterioration.

6.4 The east wing walls are in fair condition. To the north east wing wall some brickwork has been replaced or repaired with mass concrete. This concrete is of poor quality with loss of cement paste and aggregate, and a diagonal crack is present to its upper half. Concrete repairs will be required to prevent further deterioration.

6.5 The cast iron beams are in fair condition. The bottom flanges show signs of minor pitting and surface corrosion. A reduction in section will be required for assessment and the beams cleaned coated with an effective paint system

6.6 The tie bars joining adjacent beams are severely corroded and it is estimated that they have 70% section loss. The badly corroded areas will require repair or replacement and remaining areas cleaned and repainted.

6.7 The brick jack arches are in poor condition. A hammer survey indicated localized separation of the arch rings. Repair work to loose brickwork will be required.

6.8 The parapets are in poor condition. Impact damage is evident to the south pilaster which will require re-setting. Repairs should be carried out to these defects. In addition one of the steel posts and railings on the east parapet have been damaged and it is likely that, if required, the entire metal section of the parapet will need replacing. It should be noted that the west parapet is redundant and that this should be taken into account when considering the cost implications of any repairs.

6.9 The lack of an effective waterproofing system is indicated by the water rundown and staining to the jack arches. The installation of a waterproofing membrane to the deck should prevent further deterioration of brickwork.

6.10 In accordance with BD 21/97 clause 5.24, for HA UDL and KEL a category of Lp is appropriate, corresponding to low traffic flow and poor road surface condition. In determining this category it has been assumed that, whatever the road surface condition at the time of inspection, at some stage in the life of the structure there is likely to be a deterioration to a level corresponding to the 'poor' category.

6.11 Based on the level and dimensional survey the structure has the following geometric features :-

Cast Iron Beams:

Skew span (L)	= 7.532m
Skew angle (α)	= 7°

Jack Arches

Square span(L)	= 1.223m

Dimensions were obtained from levels, site measurements and photographs. See drawings included in the Approval in Principle.

6.12 Based on the inspection and intrusive investigation works the arch barrel has the following properties:

Barrel thickness	=300mm
Masonry strength	=2.3N/mm^2

(BD 21/97 fig. 4.2, London Stocks & lime mortar)
No structurally significant longitudinal cracking.
Some localised arch separation.

6.13 Based on this inspection and the recommendations of BA16/97 Annex D and BD 21/97 6.21, a condition factor F_c of 0.45 will be used for an ARCHIE analysis.

6.14 Based on the inspection, a condition factor F_{cm} of 1.0 would be appropriate for the assessment of the cast iron beams provided the actual beam section is used. The dimensions shown on the drawings included in the Approval in Principle for Assessment allow for any section loss due to corrosion.

6.15 Statutory Undertaker's plant is present on the structure (refer to appendix C) but no detrimental effects were observed.

7.0 RECOMMENDATIONS FOR ASSESSMENT

7.1 The information collected from the site inspection, with respect to defects affecting the structural integrity of the bridge, should be incorporated into the Approval in Principle. Defects affecting the assessment are described in section 6.0. It is recommended that, for the ARCHIE analysis, the condition factor in section 6.13 should be adopted. No allowance need be made for the effects of arch ring separation as the observed defects were localized. For analysis of the cast iron beams the condition factor in 6.14 should be adopted. The tie rods should be analysed based on the corroded section properties stated in section 6.6. No other allowance need be made for the structural deterioration in the assessment calculations.

7.2 For the assessment, the geometric properties and material strengths in sections 6.11 and 6.12 should be adopted.

7.3 For assessment purposes only, the condition of the road should be taken as poor with low traffic flow (Lp).

7.4 Statutory Undertaker's plant is present on the structure (refer to appendix C). This can have a detrimental effect on the interaction of the fill with the jack arches. However, this effect is difficult to measure or quantify and should not be taken into account.

Note that the following are maintenance recommendations and will not affect the proposed assessment.

7.5 The weathered, eroded and missing areas of brickwork should be replaced or repaired. All cracking to structure should be monitored and repaired as necessary.

7.6 A more extensive hammer survey should be carried out to the jack arch barrels and, if necessary, repairs carried out.

7.7 The corroded tie rods should be repaired and painted.

APPENDIX A

Photographs

Photograph 1 – East elevation of Debden Road Bridge

Photograph 2 – View over Debden Road Bridge looking north

Photograph 3 – Typical graffiti on abutment and view of service ducts running under structure

Photograph 4 – Corrosion to bottom flange of cast iron beam

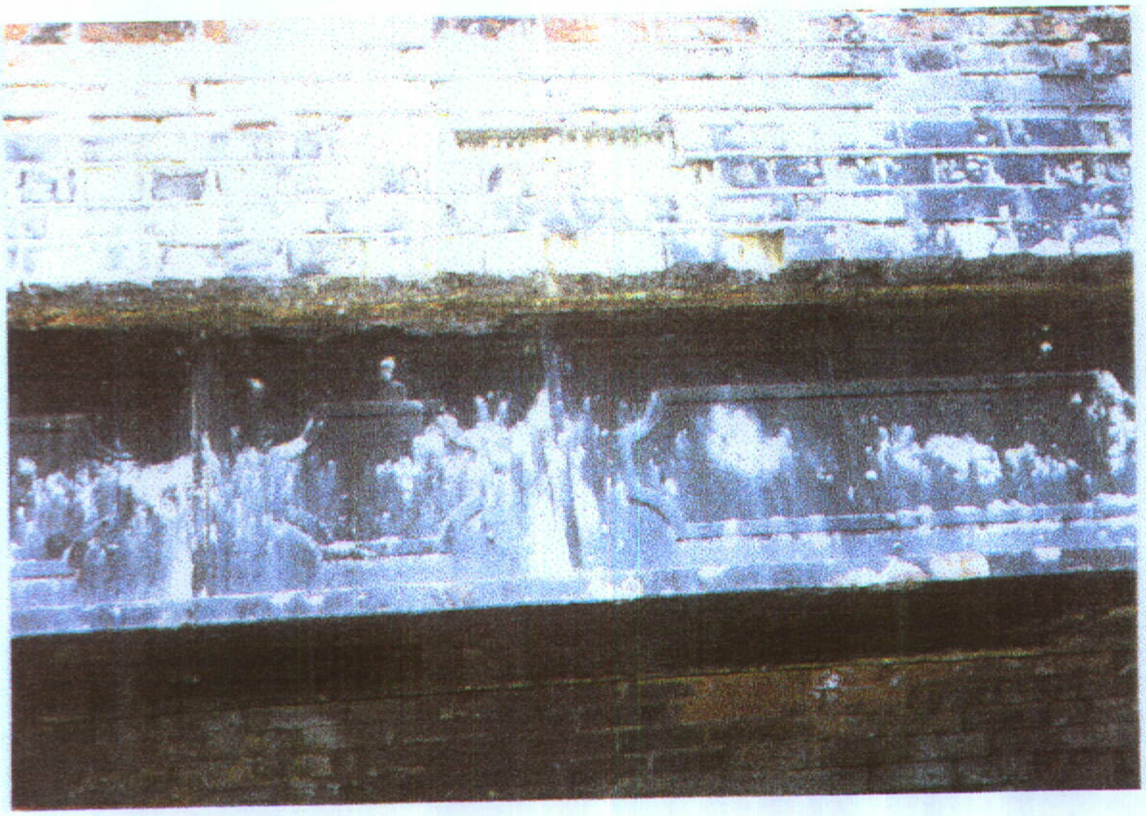
Photograph 5 – Typical spalling brickwork on jack arch soffit

Photograph 6 - North east wing wall showing spalling and vegetation growth

Photograph 7 – Missing brickwork from west parapet

Photograph 8 – Depression and cracking in car park surfacing adjacent to west elevation

Photograph 9 – Loss of brickwork from joint between old and new brickwork at the south end of the east parapet



Photograph 10 – View of east beam. Note spalling brickwork on parapet plinth string course

Photograph 11 – Damage to steel post and rail at south end of east parapet

Photograph 12 – Leach staining on south abutment

Photograph 13 – Displaced coping stone to south pilaster

Photograph 14 – View of east abutment showing backfill covering west elevation.

Photograph 15 – Missing brick, typical to the east abutment

Photograph 16 – Typical corrosion to tie rods

APPENDIX B

Defect Diagrams

ECC ASSESMENT CONTRACT 3 – RAIL PROPERTY Ltd BRIDGES

DETAIL OF STANDARD KEY

KEY

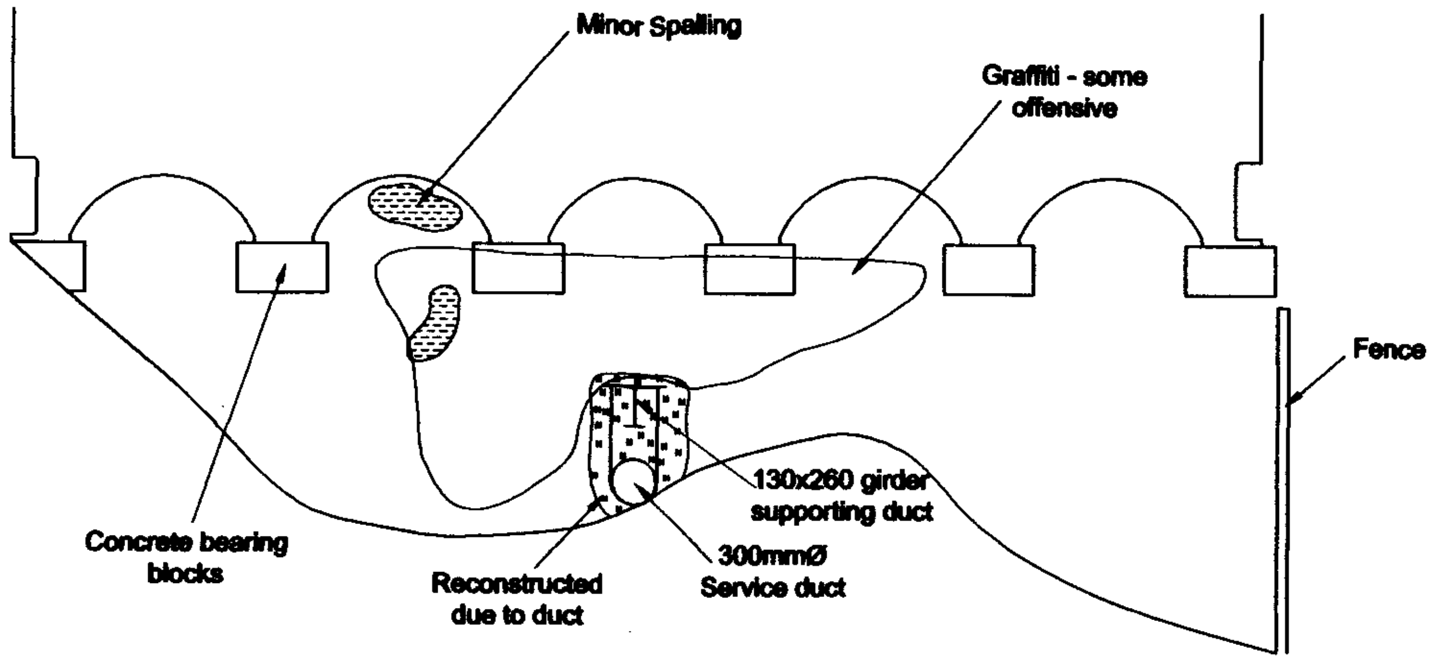
- Damp concrete/brickwork/stonework
- Leaching
- Dry water staining
- Hollow areas (tapping survey)
- Corrosion
- Algae
- Lichen
- Calcareous deposits
- Spalling
- Pointing loss
- Vegetation growth
- Honeycoming
- C=0.3 Crack width in mm
- R Area of repair
- N Area of new brick/stonework
- Efflorescence
- Frost damage

DO NOT SCALE

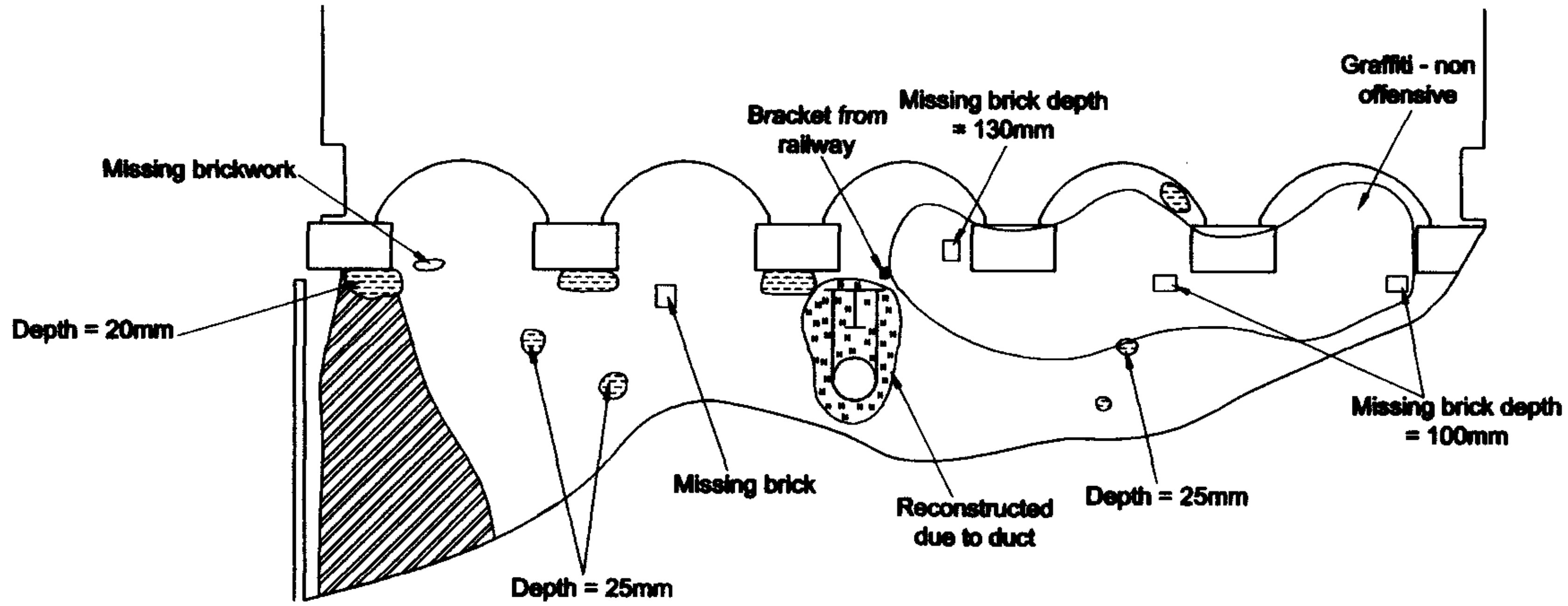
ECC Bridge No. 1004
Rail Property Board No. AEB/2116

Notes:

All Dimensions in mm



North Abutment



South Abutment

Essex County Council Transportation & Operational Services	[redacted]	Atkins	SCALES NTS		SURVEYED	LEVELLED	DESIGNED	DRAWN/TRACED	CHECKED	AUTHORISED	DESCRIPTION OF DRAWING Debden Road Bridge, Saffron Walden Defect diagrams : abutments (Sheet 1 of 1)
				INITIALS	JF	JF		MAD			
				DATE	OCT 98	OCT 98		DEC 98			
			CAD FILE N:\AI1877\61\02	REVISION NOTES			REVISION		CHECKED		

SCHEME TITLE
ECC Assessment Contract 3
Rail Property Ltd Bridges

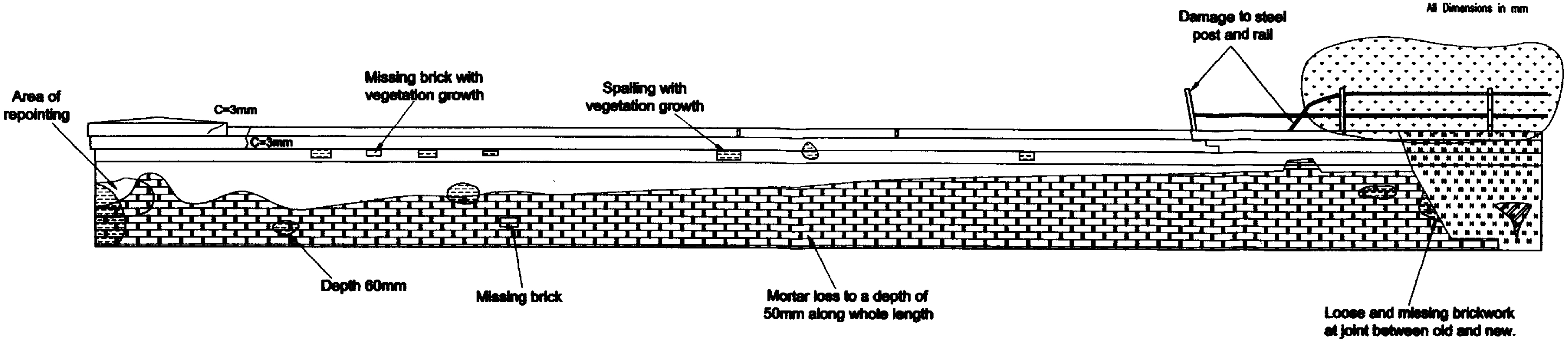
DRAWING NO. AI1877\DWGS\1004\FIG02

DO NOT SCALE

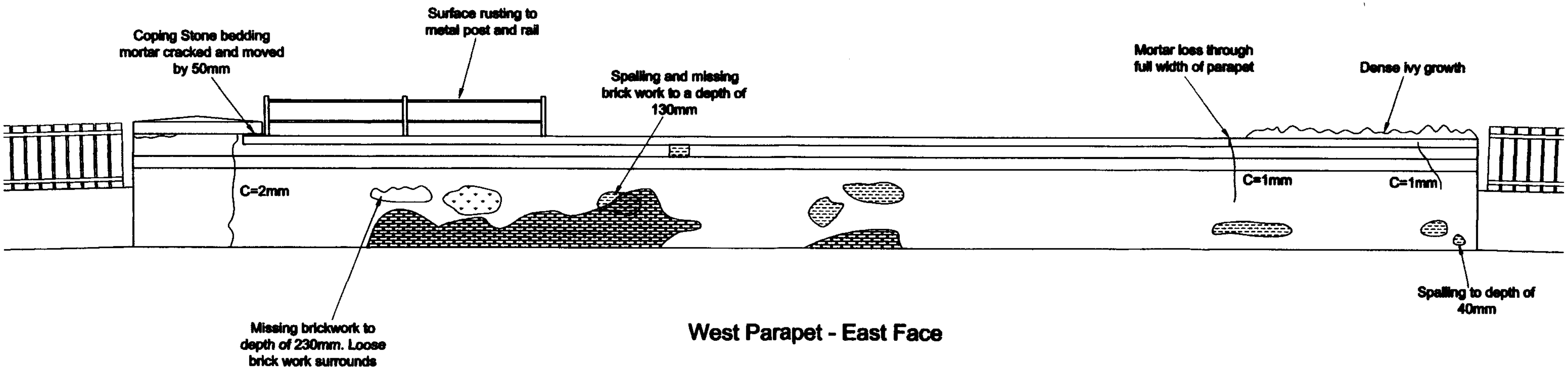
ECC Bridge No. 1004
Rail Property Baord No. AEB/2116

Notes:

All Dimensions in mm



East Parapet - West Face



West Parapet - East Face

Essex County Council Transportation & Operational Services	[redacted]	WS Atkins	SCALES NTS		SURVEYED	LEVELLED	DESIGNED	DRAWN/TRACED	CHECKED	AUTHORISED
				INITIALS	JF	JF		MAD		
				DATE	OCT 98	OCT 98		DEC 98		
			CAD FILE \AI1877\61\FIG03	REVISION NOTES			REVISION		CHECKED	

DESCRIPTION OF DRAWING — Sheet 1 of 1
Debden Road Bridge, Saffron Walden
Defect diagrams : Parapets

SCHEME TITLE
ECC Assessment Contract 3
Rail Property LTD Bridges

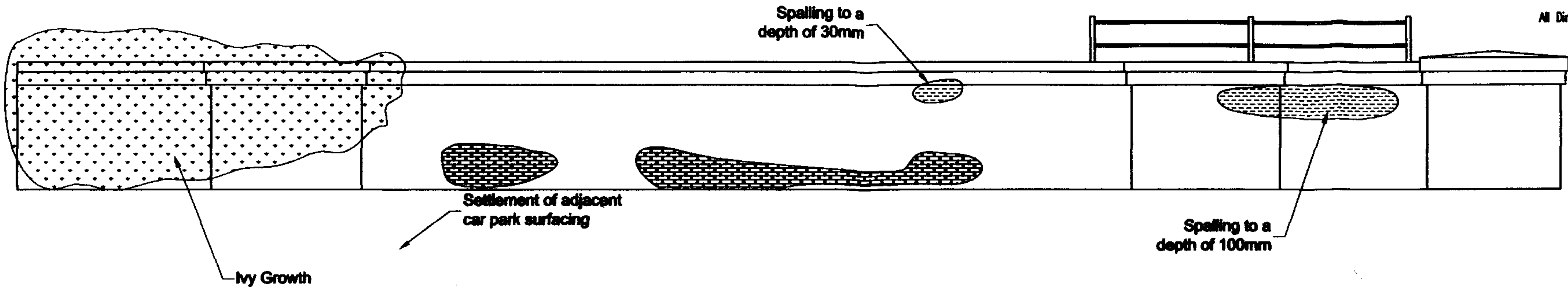
DRAWING NO. AI1877\dwgs\1004\FIG03

DO NOT SCALE

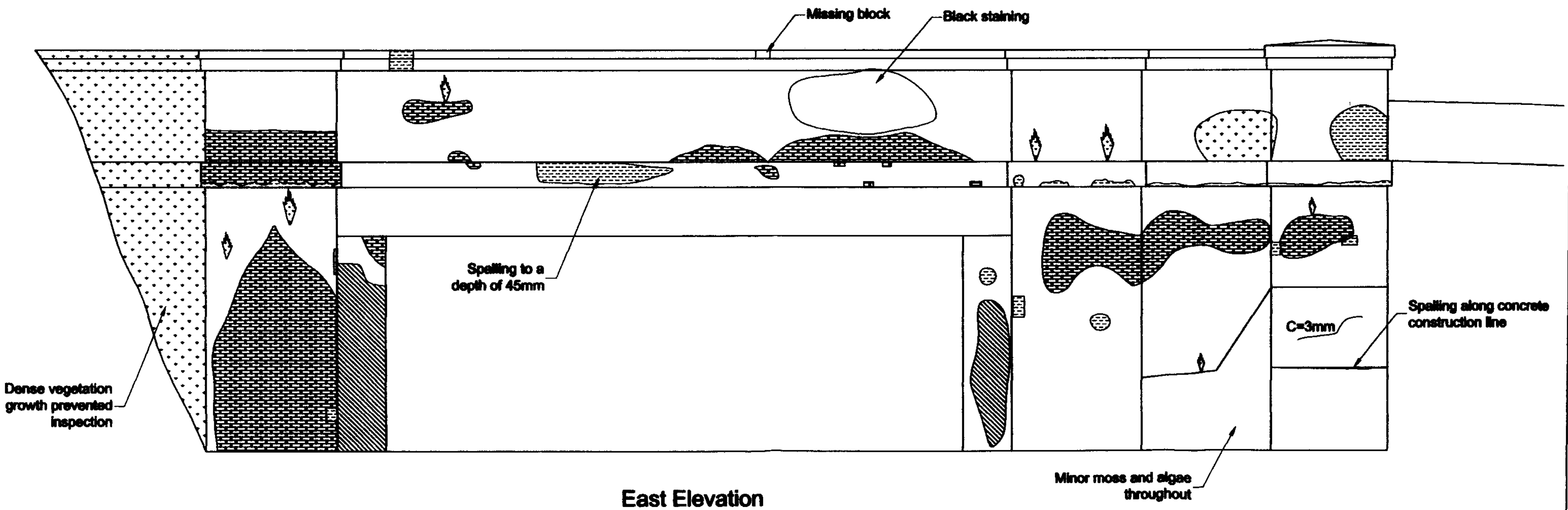
ECC Bridge No.	1004
Rail Property Baord No.	AEB/2116

Notes:

All Dimensions in mm



West Elevation



East Elevation

Essex County Council
Transportation & Operational Services

[redacted]

WS Atkins

SCALES		SURVEYED	LEVELLED	DESIGNED	DRAWN/TRACED	CHECKED	AUTHORISED
NTS	INITIALS	JF	JF		MAD		
	DATE	OCT 99	OCT 99		DEC 99		
CAD FILE N:\AI1877\61\04	REVISION NOTES			REVISION		CHECKED	

DESCRIPTION OF DRAWING — Sheet 1 of 1

Debden Road Bridge, Saffron Walden
Defect diagrams : Elevations

SCHEME TITLE

ECC Assessment Contract 3
Rail Property Ltd Bridges

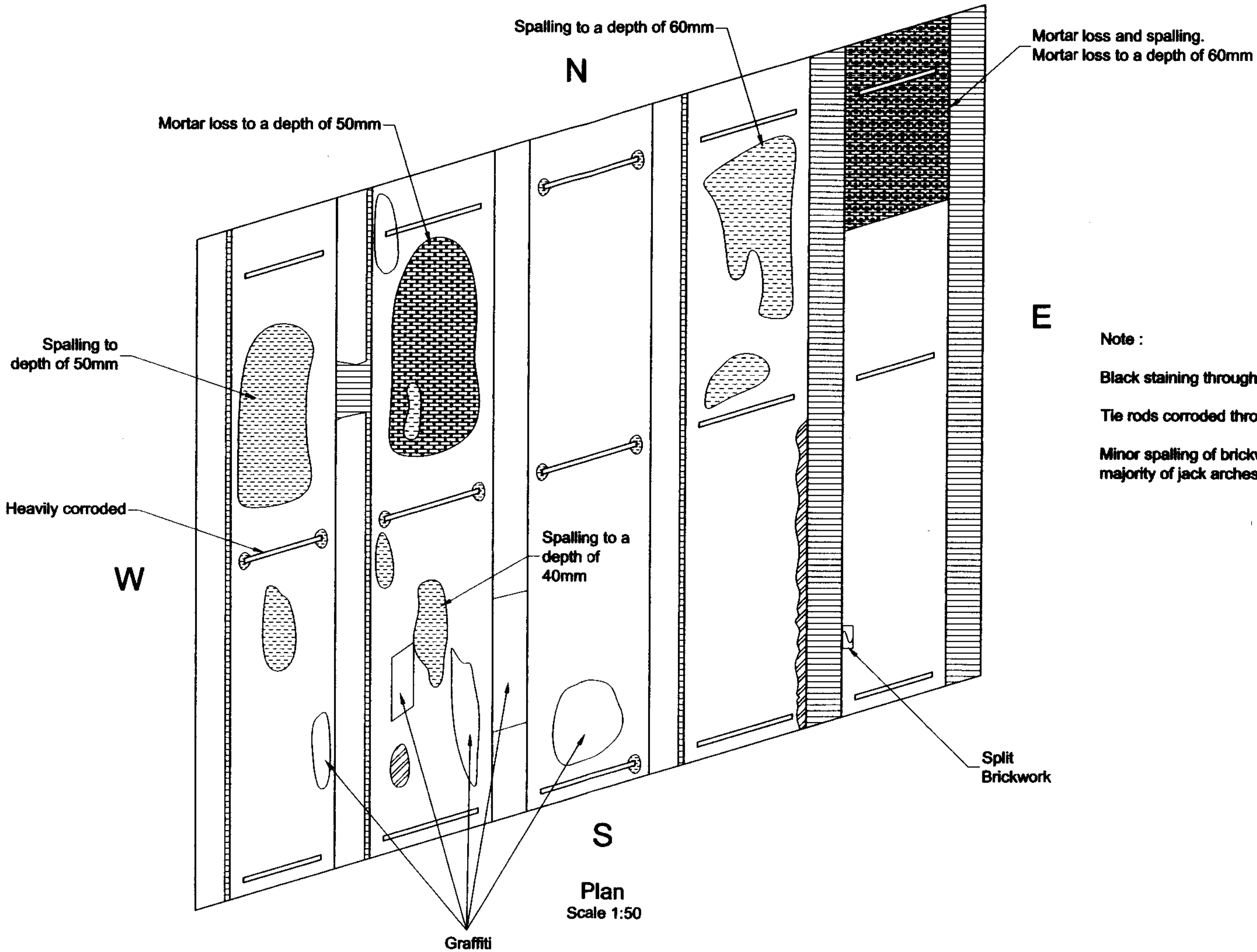
DRAWING NO. AI1877\DWGS\1004\FIG04

DO NOT SCALE

ECC Bridge No.	1004
Rail Property Board No.	AEB/2166

Notes:

All Dimensions in mm



Plan
Scale 1:50

Note :

Black staining throughout soffit

Tie rods corroded throughout

Minor spalling of brickwork to majority of jack arches

Essex County Council
Transportation & Operational Services

[redacted]

Atkins

SCALES		SURVEYED	LEVELLED	DESIGNED	DRAWN/TRACED	CHECKED	AUTHORISED
NTS	INITIALS	JF	JF		MAD		
	DATE	OCT 99	OCT 99		DEC 99		
CAD FILE N:\AI1877\61\05	REVISION NOTES			REVISION		CHECKED	

DESCRIPTION OF DRAWING — Sheet 1 of 1

Debden Road Bridge, Saffron Walden
Defect Diagram : Soffit

SCHEME TITLE

ECC Assessment Contract
Rail Property Ltd Bridges

DRAWING NO. AI1877\DWGS\1004\FIG05

APPENDIX C

Statutory Undertakers

New Roads and Street Works Act (NRSWA) notices have been issued to the following companies. The responses are summarised below:

Company	Service
Anglian Water	Sewers along carriageway centreline.
British Telecom	Cable in the east footway.
Cambridge Cable	Cable in the west footway.
Mercury Communications	No existing plant within the vicinity of the bridge.
Telewest Communications	No existing plant within the vicinity of the bridge.
Transco	No existing plant within the vicinity of the bridge.
British Gas	Pipe in northbound lane of the carriageway.
Eastern Electricity	Cables in the carriageway and east footway.
Essex & Suffolk Water Company	No existing plant within the vicinity of the bridge.
Thames Valley Water	Services in the both footways.
Energis	No existing plant within the vicinity of the bridge
National Grid	No existing plant within the vicinity of the bridge.
Serco Gulf Engineering	No existing plant within the vicinity of the bridge.
Street lighting	No existing plant within the vicinity of the bridge.
English Nature	No comment.
Environment Agency	No comment.

ESSEX COUNTY COUNCIL
ASSESSMENT CONTRACT 3

ASSESSMENT REPORT FOR THE ASSESSMENT OF DEBDEN ROAD BRIDGE

ECC BRIDGE NO. 1004
RAIL PROPERTY Ltd BRIDGE NO. AEB/2116

Read by Bridge Client :
[redacted]
Date : 22 May 2000

Prep[redacted] **Date:** 30/3/00

J Fri[redacted]

Revi[redacted] **Date:** 30/03/00

GR[redacted]

Auth[redacted] **Date:** 15/05/00

Nam[redacted]Eng...MICE.....

Essex County Council
Transportation and Operational Services Division
County Hall
Chelmsford
Essex
CM1 1QH

WS Atkins Consultants - Essex
Threadneedle House
9 - 10 Market Road
Chelmsford
Essex
CM1 1JQ

Rail Property Ltd
Room C5
Hudson House
York
YO1 6HP

Copy No. 1
Version No. 1.0

Assessment Report Index

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	Executive Summary		
	Form BA		
	Form BAA		
1	Introduction		1
2	Conclusions of Inspection Report		2
3	Assessment Methods and Findings		3
4	Conclusions		5
	Appendix A	Summary Results Table	
	Appendix B	Assessment Calculations	
	Appendix C	Approval in Principle and Inspection for Assessment	

EXECUTIVE SUMMARY

Debden Road Bridge, Saffron Walden has been assessed in accordance with the Approval in Principle dated 21 February 2000. This is situated in appendix C of this report.

The bridge is a single span structure comprising cast iron beams with brick jack arches and is supported on brickwork abutments. The clear square span is 7.375m and the bridge has a skew of 7°. The parapets are of brickwork construction with a steel post and rail height extension at the south end. The west elevation has been backfilled to carriageway level and now retains a car park for a new residential development. A wooden fence prevents access under the structure. There is presently a 22 tonne weight restriction on the structure.

Overall the structure is in fair condition.

The results for the main beams are based on a worst loaded strip analysis. The jack arches have been assessed using the ARCHIE computer programme, and the transverse tie rods assessed using the horizontal thrusts obtained from this ARCHIE analysis. The substructure and foundations have been assessed qualitatively.

OVERALL STRUCTURAL CAPACITY	**DEAD LOAD ONLY**

The load carrying capacity of the main structural elements is listed below.

Internal beams:	7.5 tonnes Assessment Live Loading
Edge beams:	7.5 tonnes Assessment Live Loading
Jack arches:	40 tonnes Assessment Live Loading
Tie Rods:	Dead Load Only

Sub-structures, foundations and wingwalls:

A qualitative assessment of the sub-structures, foundations and wingwalls indicate that they are adequate to carry the present traffic loading. According to Clause 8.5 of BD21/97, they may be assumed to be adequate for 40 Tonne assessment loading without further assessment.

Strengthening Requirements

Replacement of the tie rods alone would result in an increase of capacity rating to 7.5 tonnes. Strengthening of the cast iron beams by bonding a steel or carbon fibre plate to the bottom flange would increase the overall capacity of the structure.

Consideration should be given to dismantling the structure and filling the resultant void. This option may be considered advantageous since the west elevation has already been backfilled

FORM 'BA' (BRIDGES)

CERTIFICATION FOR ASSESSMENT CHECK

STRUCTURE / LINE NAME DEBDEN ROAD BRIDGE CATEGORY OF CHECK 1

ELR / STRUCTURE NO AEB/2116

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

(1) It has been assessed in accordance with the Approval in Principle (where appropriate) as recorded on Form AA approved on 21 February 2000

(2) It has been checked for compliance with the following principle British Standards, Codes of Practice, BR Technical notes and Assessment standards. (SEE TAS SCHEDULE IN AIP)

List any departures from the above and additional methods of criteria adopted, with reference and justification for their acceptance (commenting on the results if appropriate).
NONE

CATEGORY 1

NAME SIGNATURE

[redacted]

(ASSESSOR) 27 March 2000

(ASSESSMENT CHECKER) 27 March 2000

DIRECTOR OF THE FIRM OF CONSULTING ENGINEERS TO WHOM THE ASSESSOR / CHECKER IS RESPONSIBLE
27 March 2000

CATEGORY 2 AND 3 (NOTE: CATEGORY 1 CHECK MUST ALSO BE SIGNED)

ASSESSMENT

NAME SIGNATURE

Not Applicable (ASSESSOR)

DIRECTOR OF THE FIRM OF CONSULTING ENGINEERS TO WHOM THE ASSESSOR IS RESPONSIBLE

(b) CHECK

NAME SIGNATURE

Not Applicable (ASSESSMENT CHECKER)

DIRECTOR OF THE FIRM OF CONSULTING ENGINEERS TO WHOM CHECKER IS RESPONSIBLE

FORM 'BAA' (BRIDGES)

CERTIFICATION FOR ASSESSMENT CHECK

NOTIFICATION OF ASSESSMENT CHECK

STRUCTURE NAME / ROAD NO. DEBDEN ROAD BRIDGE

LINE NAME (DISUSED)

ELR CODE / STRUCTURE NO. AEB/2116 ESSEX COUNTY COUNCIL No. 1004

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

STATEMENT OF CAPACITY

DEAD LOAD ONLY (TIE RODS)
7.5 TONNES (BEAMS)

Critical member/s: TIE RODS

RECOMMENDED LOADING RESTRICTIONS

With the transverse tie rods in their present condition it is recommended that vehicular traffic be prevented from using the structure.

Based on the beam capacities it would be recommended that a 7.5 tonnes weight restriction be imposed on the bridge.

DESCRIPTION OF STRUCTURAL DEFICIENCIES AND RECOMMENDED STRENGTHENING

The structure has inadequate capacity for 40 tonnes live loading due to insufficient tensile strength of the highly corroded tie rods and the cast iron beams.

Replacement of the tie rods alone would result in an increase of capacity rating to 7.5 tonnes. Strengthening of the cast iron beams by bonding a steel or carbon fibre plate to the bottom flange would increase the overall capacity of the structure.

Consideration should be given to dismantling the structure and filling the resultant void. This option may be consid[redacted]n backfilled

Name: [redacted]ructural Assessment Engineer

Name[redacted]vil Engineer
[redacted]6/2001

1.0 INTRODUCTION

1.1 Essex County Council (ECC) entered into an agreement with Rail Property Ltd to assess Rail Property Ltd owned bridges carrying publicly maintainable highways. WS Atkins Consultants Ltd – Essex (WSAE) have been appointed by ECC to carry out the visual inspections and assessments of the bridges.

1.2 An Approval in Principle document was submitted and approved on 21 February 2000. This includes a detailed inspection for assessment report. This assessment report should be read in conjunction with the Approval in Principle and Inspection for Assessment Report.

1.3 An inspection of the structure was carried out on 20 October 1999. The client supplied previous assessment data giving the cast iron beam details and some dimensions of the structure. The inspection included a visual inspection and dimension survey to confirm structural details. The weather was overcast and cold during the inspection. The results of the inspection are presented in the inspection for assessment report which forms part of the Approval in Principle dated 21 February 2000.

1.4 A summary of the inspection report findings are listed in section 2 of this assessment report. This includes details of the defects in the bridge which affect the load carrying assessment of the structure.

1.5 Debden Road Bridge carries an unclassified road over a dismantled railway line in Saffron Walden, Essex OS Ref. TL 53823 37896

1.6 The bridge is a single span structure comprising cast iron beams with brick jack arches and is supported on brickwork abutments. The clear square span is 7.375m and the bridge has a skew of 7°. The parapets are of brickwork construction with a steel post and rail height extension at the south end. The west elevation has been backfilled to carriageway level and now retains a car park for a new residential development. A wooden fence prevents access under the structure.

1.7 The carriageway is 5.08m wide with a 0.89m and 1.68m wide footway on the east and west side of carriageway respectively The vertical alignment of the carriageway is a gentle hog curve and the horizontal alignment is straight.

1.8 There is a 22 tonnes weight restriction on the structure.

1.9 The speed limit across the structure is 30mph.

2.0 CONCLUSIONS OF INSPECTION REPORT

Details of the key dimensions of the structure are shown on drawings AI1877/DWGS/1004/FIG 01. These are included in the Approval in Principle document.

Details of the defects in the structure are shown on drawings AI1877/DWGS/1004/FIG 02 to FIG 05. These are situated in the inspection for assessment report which forms an appendix to the Approval in Principle.

The following is a summary of the defects listed in the inspection for assessment report.

2.1 Overall, the bridge is generally in fair condition, however the tie bars, which are severely corroded, and the brick jack arches, which have areas of ring separation, are in poor condition and require repair.

2.2 Based on this inspection and the recommendations of BA16/97 Annex D and BD 21/97 6.21, a condition factor F_c of 0.45 will be used for an ARCHIE analysis.

2.3 Based on the inspection, a condition factor F_{cm} of 1.0 would be appropriate for the assessment of the cast iron beams provided the actual beam section is used. The dimensions shown on the drawings included in the Approval in Principle for Assessment allow for any section loss due to corrosion.

2.4 No allowance need be made for the effects of jack arch ring separation as the observed defects were localized.

2.5 The tie rods should be analysed based on the corroded section properties stated in section 6.6 of the Inspection for Assessment report.

2.6 No other allowance need be made for the structural deterioration in the assessment calculations.

2.7 The weathered, eroded and missing areas of brickwork should be replaced or repaired. All cracking to structure should be monitored and repaired as necessary.

2.8 A more extensive hammer survey should be carried out to the jack arch barrels and, if necessary, repairs carried out

2.9 The corroded tie rods should be repaired and painted.

2.10 The abutments, wing walls and sub-structure show little signs of distress and are assumed to be in sound condition.

3.0 ASSESSMENT METHODS AND FINDINGS

3.1 The assessment of Debden Road Bridge , Saffron Walden has been carried out in accordance with the Approval in Principle dated 21 February 2000. The following drawings, included in the Approval in Principle document have been used.

AI1877/DWGS/1004/FIG 01 GA Elevation and Cross Section

3.2 The following assumptions have been made regarding material strengths.

Masonry Strengths		2.3N/mm2
Cast Iron Strengths:	Compression	154 N/mm2
	Tension	46 N/mm2

3.3 Detailed results are situated in appendix A of this assessment report. Copies of the assessment calculations are situated in appendix B.

MAIN BEAMS

3.4 The main beams were assessed on a worst loading strip analysis. No transverse distribution was assumed. A condition factor of 1.0 was assumed for the beams and the actual section dimensions as measured on site were used. Increased section properties were adopted for live loading in accordance with BD 21/97 Clause 7.13.

3.5 Where the beams are positioned beneath the carriageway HA loading was used for live loading. For such cases the condition of the carriageway was taken as poor with low traffic flow (Lp). The beams beneath the footway were checked with the BD21/97 appendix D vehicles. The beams have been checked at ULS.

3.6 The main beams were assessed at **7.5 TONNES** Assessment Live Loading.

3.7 In accordance with the AIP, since the beams are not able to withstand 40 tonne load effects no HB capacity has been determined.

EDGE BEAMS

3.8 The edge beams were assessed on a worst loading strip analysis. No transverse distribution was assumed. A condition factor of 1.0 was assumed for the beams and the actual section dimensions as measured on site were used. No increase in section properties were adopted for live loading (BD 21/97 Clause 7.13).

3.9 The edge beams were checked with the BD21/97 appendix D vehicles and have been checked at ULS.

3.10 The edge beams were assessed at **7.5 TONNES** Assessment Live Loading.

JACK ARCHES

3.11 The jack arches have been analysed using the ARCHIE computer program.

3.12 The jack arches were assessed at **40 TONNES** Assessment Live Loading.

TIE RODS

3.13 The transverse tie rods were analysed using the worst horizontal thrust lines obtained from the ARCHIE analysis of the jack arches. A condition factor of 1.0 was assumed for the tie rods and actual corroded section dimensions as measured on site were used

3.14 The tie rods were assessed at **DEAD LOAD ONLY**.

SUB-STRUCTURES, FOUNDATIONS, WINGWALLS

3.15 A qualitative assessment of the sub-structures, foundations and wingwalls indicate that they are adequate to carry the present traffic loading. According to Clause 8.5 of BD21/97, they may be assumed to be adequate for 40 Tonne assessment loading without further assessment.

4.0 CONCLUSIONS

4.1 Debden Road Bridge, Saffron Walden has been assessed in accordance with the Approval in Principle dated 21 February 2000.

4.2 Results for the cast iron beams are based on simple beam-strip analysis. The jack arches were assessed using the ARCHIE computer program and the worst horizontal thrusts obtained from ARCHIE were used to analyse the tie rods. A summary of the results is listed below.

4.3 Super Structure

Internal Beams	7.5 tonnes
Edge Beams	7.5 tonnes
Jack Arches	40 tonnes
Tie Rods	Dead load only
Parapets	The parapets do not conform to current standards and have not been assessed.

4.4 Sub-structures, foundations and wing walls

A qualitative assessment of the sub-structures, foundations and wing walls indicate that they are adequate to carry the present traffic loading. According to Clause 8.5 of BD21/97, they may be assumed to be adequate for 40 Tonne assessment loading without further assessment.

4.5 The inspection for assessment showed that the structure requires general minor maintenance as well as high priority repairs to the tie bars and the jack arches. Details are included in section 7 of the inspection report.

4.6 Strengthening Requirements

Replacement of the tie rods alone would result in an increase of capacity rating to 7.5 tonnes. Strengthening of the cast iron beams by bonding a steel or carbon fibre plate to the bottom flange would increase the overall capacity of the structure.

Consideration should be given to dismantling the structure and filling the resultant void. This option may be considered advantageous since the west elevation has already been backfilled

APPENDIX A

SUMMARY RESULTS TABLES

Analysis Results: Internal Cast Iron Beams – Beam Strip Analysis.

Component Name	**Internal Beams**	**Internal Beams**	**Internal Beams**	
Section Location	At Midspan	At Midspan	At Support	
Type of Effect	**Compressive** Stress due to Bending / Nmm^{-2}	**Tensile** Stress due to Bending / Nmm^{-2}	Shear Stress / Nmm^{-2}	
Allowable Dead + Live Load Stress	154.0	46.0	46.0	
Permanent Load Effects	51.8	32.2	6.5	
Allowable Live Load Stress	102.2	13.8	39.5	
Adjusted Permissible Live Load Stress (In accordance with BD21/97 Cl 4.10)	102.2	10.4	21.7	
Adjusted HA Live Load Effects	35.5	22.1	8.2	
C factor	>1	0.47	>1	
Accidental Wheel Load	N/A	N/A	N/A	
Single Axle/Wheel Loading	HA Critical	HA Critical	HA Critical	
PASS/FAIL	PASS	FAIL	PASS	

Assessment LL Rating	40 tonnes	7.5 tonnes	40 tonnes	

45 Units HB Effect				
Associated LL Effect				
HB Rating				

Comments

- Assumed Low Traffic Flow
- Road surface category Poor
- A condition factor of 1.0 was assumed for the beams and the actual section dimensions as measured on site were used
- **Jack Arches were found to be adequate for 40 tonnes Assessment Live Loading**
- **Tie Rods were found to have a capacity rating of DEAD LOAD ONLY** based on the horizontal thrusts obtained from the Jack Arch ARCHIE analysis

Analysis Results: Cast Iron Edge Beams – Beam Strip Analysis.

Component Name	Edge Beams	Edge Beams	Edge Beams	
Section Location	At Midspan	At Midspan	At Support	
Type of Effect	**Compressive** Stress due to Bending / Nmm^{-2}	**Tensile** Stress due to Bending / Nmm^{-2}	Shear Stress / Nmm^{-2}	
Allowable Dead + Live Load Stress	154.0	46.0	46.0	
Permanent Load Effects	35.3	27.0	5.9	
Allowable Live Load Stress	118.7	19.0	40.1	
Adjusted Permissible Live Load Stress (In accordance with BD21/97 Cl 4.10)	118.7	12.7	22.0	
Adjusted HA Live Load Effects	N/A	N/A	N/A	
C factor	N/A	N/A	N/A	
Accidental Wheel Load	RE 17t = 24.4 RF 7.5t = 13.7	RE 17t = 18.6 RF 7.5t = 10.4	Ec3 40t = 12.6	
Single Axle/Wheel Loading	N/A	N/A	N/A	
PASS/FAIL	RE 17t – PASS RF 7.5t – PASS	RE 17t – FAIL RF 7.5t – PASS	Ec3 40t - PASS	

Assessment LL Rating	40 tonnes	7.5 tonnes	40 tonnes	

45 Units HB Effect				
Associated LL Effect				
HB Rating				

Comments

- Assumed Low Traffic Flow
- Road surface category Poor
- A condition factor of 1.0 was assumed for the beams and the actual section dimensions as measured on site were used
- **Jack Arches were found to be adequate for 40 tonnes Assessment Live Loading**
- **Tie Rods were found to have a capacity rating of DEAD LOAD ONLY** based on the horizontal thrusts obtained from the Jack Arch ARCHIE analysis

APPENDIX B

ASSESSMENT CALCULATIONS

WS Atkins

CALCULATION INDEX

PROJECT: ECC ASSESSMENT CONTRACT 3

FILE NAME: DEBDEN ROAD BRIDGE

FILE NO: A1 1 8 7 7 - 6 1 -

C = Calculation
S = Sketch
D = Data

Page

INDEX NO	CALCULATION/SKETCH/DATA	ORIGINATOR	DATE
1	SUMMARY	JF	2/00
2-3	SECTION PROPERTIES	JF	2/00
4-11	LOADING	JF	2/00
12-20	BEAM ANALYSIS	JF	2/00
21	BEAM CAPACITY	JF	2/00
22-25	INTERNAL BEAM - STRESS CHECK	JF	2/00
26-29	EDGE BEAM - STRESS CHECK	JF	2/00
30-31	TIE BAR ASSESSMENT	JF	2/00
32-35	ARCHIVE ANALYSIS DATA	JF	2/00

Continued on Sheet

Atkins	Project: ECC ASSESEMENT CONTRACT 3			Job ref: A1 1877/61	
	Part of structure: DERDEN ROAD			Calc sheet no: /	rev: /
	Drawing ref	Calc by: JF	Date: 2/00	Check by	Date

Ref	Calculations	Output

SUMMARY

INTERNAL BEAMS

Bending;

	Capacity
HA	7.5 TONNES
SAL	>7.5 TONNES

Shear;

HA	40 TONNES
SAL	40 TONNES

EDGE BEAMS

Bending;

	Pass
17T AWL	No.
7.5T AWL	Yes

Shear;

40T AWL	Yes

JACK ARCHES

40 tonnes

TIE RODS

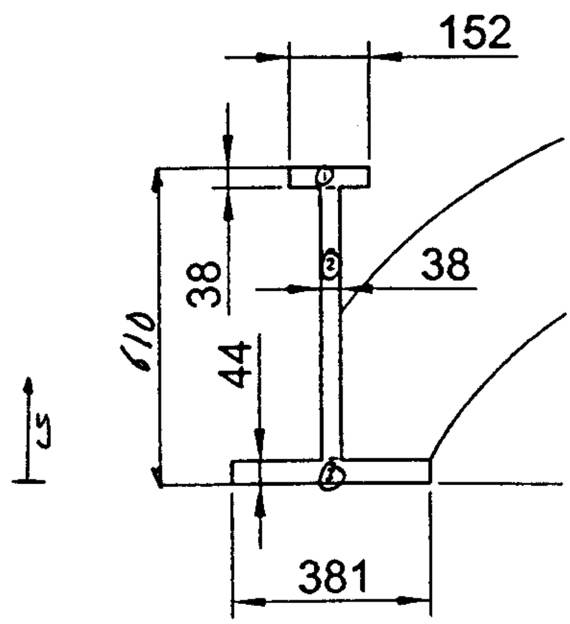
DEAD LOAD ONLY.

Atkins	Project: ECC ASSESSMENT CONTRACT 3			Job ref: AI 1877 /61	
	Part of structure: DERDEN ROAD			Calc sheet no: 2	rev
	Drawing ref	Calc by: VF	Date: 2/00	Check by	Date

Ref	Calculations	Output

SECTION PROPERTIES

INTERNAL BEAM



Section	Area mm^2	y mm	$y-\bar{y}$ mm	$(y-\bar{y})^2$ mm^2	$A(y-\bar{y})^2$ mm^4	I_{self} mm^4
①	5776	591	357	127449	736.1×10^6	0.7×10^6
②	20064	308	74	5476	109.9×10^6 ✓	466.1×10^6 ✓
③	16764	22	-212	44944	753.4×10^6 ✓	2.7×10^6
	Σ 42604				1599.4×10^6	469.5×10^6

$$\bar{y} = \frac{(5776 \times 591) + (20064 \times 308) + (16764 \times 22)}{42604}$$

$= 234$ mm

$I = (1599.4 + 469.5) \times 10^6 = 2.069 \times 10^9$ mm^4

$y_{top} = 376$ mm

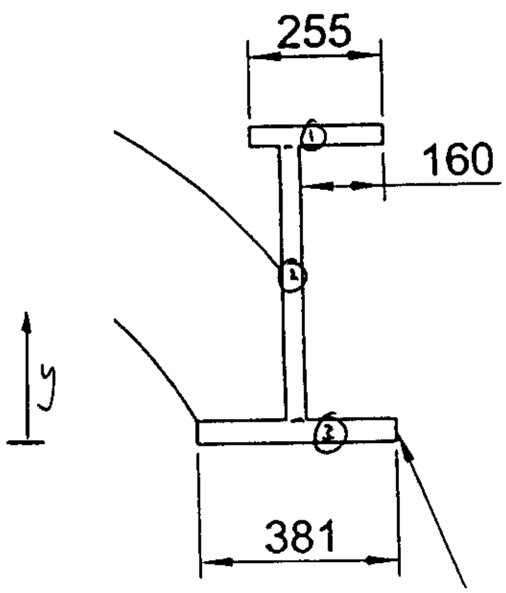
$y_{bot} = 234$ mm

Atkins	Project: ECC ASSESSMENT CONTRACT 3	Job ref: A1 1877/61
	Part of structure: DEBDEN ROAD	Calc sheet no: 13 rev
	Drawing ref: / Calc by: JF / Date: 2/00	Check by / Date

Ref | Calculations | Output

SECTION PROPERTIES

EDGE BEAM



Flange and web thicknesses as for internal beam.

Section	Area mm^2	y mm	$y-\bar{y}$ mm	$(y-\bar{y})^2$ mm^2	$A(y-\bar{y})^2$ mm^4	I_{self} mm^4
①	9690	591	327	106929	1036.1×10^6	1.2×10^6
②	20064	308	44	1936	38.8×10^6	466.1×10^6
③	16764	22	-242	58564	981.8×10^6	2.7×10^6
	Σ=46518				2056.7×10^6	470.0×10^6

$$\bar{y} = \frac{(9690\times591)+(20064\times308)+(16764\times22)}{46518}$$

$= 264$ mm

$I = (2056.7+470.0)\times10^{-6} = 2.527\times10^9\ mm^4$

$y_{top} = 346$ mm

$y_{bot} = 264$ mm

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Atkins	Project: ECC ASSESSMENT CONTRACT 3			Job ref: A1 1877/61	
	Part of structure: DEBDEN ROAD			Calc sheet no: 4	rev
	Drawing ref	Calc by: JF	Date: 2/00	Check by	Date

Ref	Calculations	Output

LOADING

DEAD LOADING

Internal beam self weight; C-S Area = 42604 mm^2

BD21/97 Table 4.1 — Unit weight of cast iron = 7200 Kgm^{-3}

$$\therefore UDL = \frac{42604 \times 7200 \times 9.81}{1 \times 10^9}$$

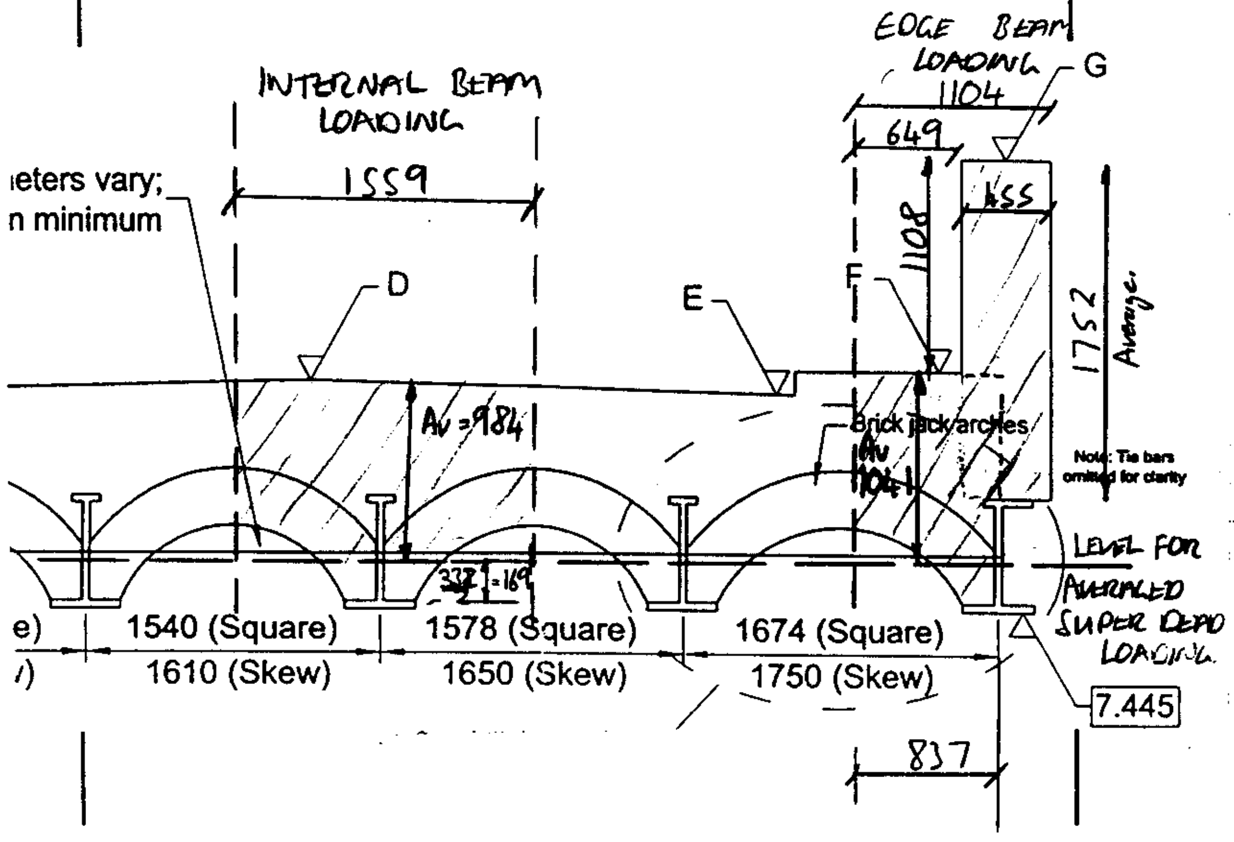
$$= 3.0\ kNm^{-1}\ (Nominal)$$

Edge beam self weight; C-S Area = 46518 mm^2

$$\therefore UDL = \frac{46518 \times 7200 \times 9.81}{1 \times 10^9}$$

$$= 3.3\ kNm^{-1}\ (Nominal)$$

IMPOSED DEAD



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Atkins

Project			Job ref
ECC ASSESSMENT CONTRACT 3			A1 1877/61
Part of structure			Calc sheet no / rev
DEDDEN ROAD			1 5 / 1
Drawing ref	Calc by JF	Date 2/00	Check by / Date

Ref	Calculations	Output

LOADING

IMPOSED DEAD CONT.

For ease of calculating imposed loads, assume the masonry and poor quality lime concrete fill have the same unit density.

INTERNAL BEAMS - Width of deck on beam = 1559mm

Surfacing;

Assume top 100mm to be surfacing.

$$C\text{-}S\ Area = 100 \times 1559 = 155900mm^2$$

$$\therefore UDL = 155900 \times 2400 \times 9.81 \times 10^{-9}$$

$$= \underline{3.7\,kNm^{-1}}\ (Nominal)$$

Fill; Assumed depth of fill = 984mm

$$C\text{-}S\ Area = 984 \times 1559 = 1534056mm^2$$

Adopted unit density = 2100 Kgm^{-3}

$$\therefore UDL = 1534056 \times 2100 \times 9.81 \times 10^{-9}$$

$$= 31.6\,kNm^{-1}\ (Nominal)$$

EDGE BEAMS

Fway surfacing; Assume top 50mm is surfacing

$$C\text{-}S\ Area = 649 \times 50 = 32450mm^2$$

$$\therefore UDL = 32450 \times 2400 \times 9.81 \times 10^{-9}$$

$$= \underline{0.8\,kNm^{-1}}\ (Nominal)$$

Atkins	Project: ECC ASSESSMENT CONTRACT 3	Job ref: A1 1877 /61
	Part of structure: DEBDEN ROAD	Calc sheet no: 1 6 rev 1
	Drawing ref: — Calc by: JF Date: 2/00	Check by: Date:

Ref	Calculations	Output

LOADING

IMPOSED DEAD CONT.

EDGE BEAMS

Fill; C-S Area = 1041 × 837 = 871317 mm²
(Refer to sketch on first loading page)

∴ UDL = 871317 × 2100 × 9.81 × 10^{-9}

= 18.0 kNm⁻¹ (Nominal)

Parapet; C-S Area = (455 × 1108) + (267 × 644)

= 676088 mm²

∴ UDL = 676088 × 2100 × 9.81 × 10^{-9}

= 13.9 kNm⁻¹ (Nominal)

LIVE LOADS

BD 21/97 5.6 — Carriageway width = 5.08 m

∴ No. Notional lanes = 2

HA UDL; Loaded length = 7.532 m

$$W = 336\left(\frac{1}{7.532}\right)^{0.67}$$

= 86.9 kNm⁻¹ / 3.65 m lane

∴ per internal beam = $\frac{86.9}{3.65}$ × 1.559 = 37.1 kNm⁻¹ (Nominal)

HA KEL; = 120 kN / 3.65 m lane

∴ per internal beam = $\frac{120}{3.65}$ × 1.559 = 51.3 kN (Nominal)

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Atkins

Project	Job ref
ECC ASSESSMENT CONTRACT 3	A11877/61
Part of structure	Calc sheet no / rev
DEBDEN ROAD	17 / 1

Drawing ref	Calc by	Date	Check by	Date
	JF	2/00		

Ref	Calculations	Output

LOADING

LIVE LOADING CONT.

Single Axle Load;

Consider a nominal single axle load as an alternative to HA live loads.

From AID

Road category = Lp

∴ SWL = 90 kN

Dispersal of load

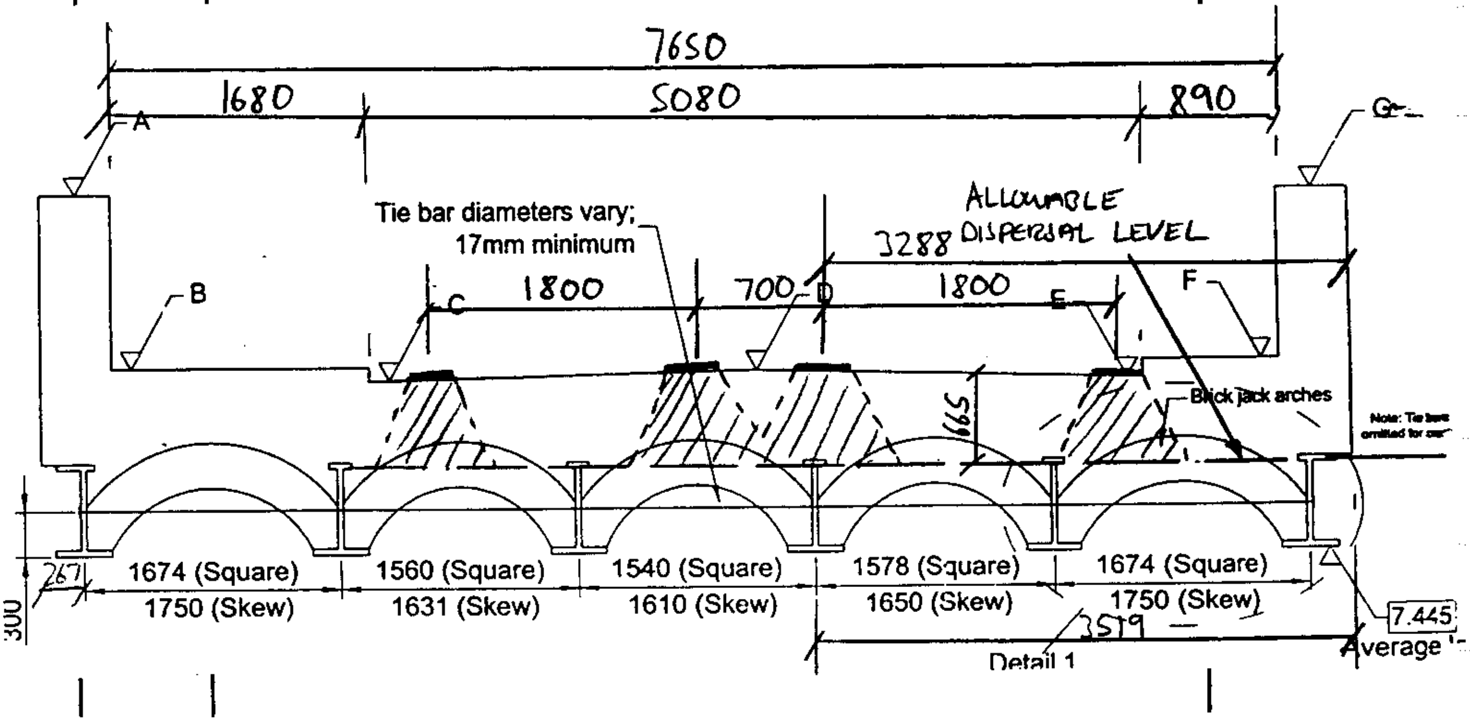
BD 21/97 6.7

For jack arches dispersal is allowed to the level of the mid-depth of the arch ring at the crown.

Effective pressure = $1.1\,\mathrm{Nmm^{-2}}$

Wheel bearing area = $\frac{90000}{1.1} = 81818\,\mathrm{mm}^2$

=> 286 mm square contact area.



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Atkins	Project: ECC ASSESSMENT CONTRACT 3			Job ref: A1 1877/61	
	Part of structure: DEBDEN ROAD			Calc sheet no: / 8	rev: /
	Drawing ref	Calc by: JF	Date: 2/00	Check by	Date

Ref	Calculations	Output

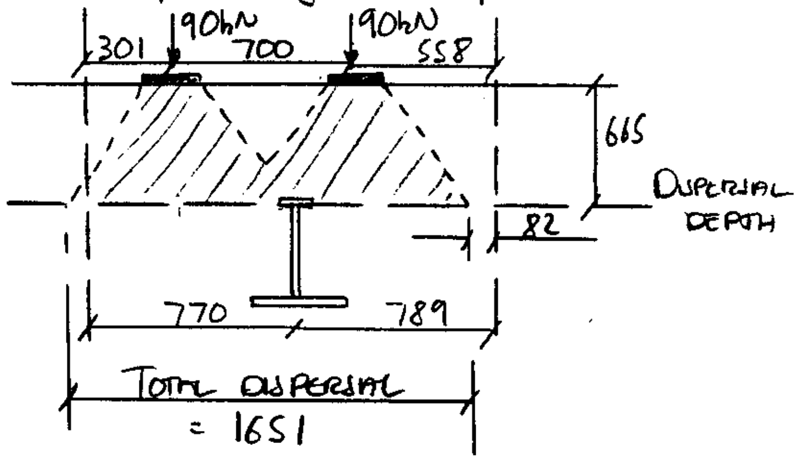
LIVE LOADING CONT

Single Axle Loading cont.

Beam loading:

From sketch on previous page the most onerous loading on an internal beam is shown.

The amount of loading is as follows:



∴ % of wheel loads on beam $= \dfrac{1559-82}{1651}$

$= 89\%$

∴ Effective UDL per beam $= \dfrac{2 \times 90 \times 0.89}{0.665}$

$= 240.9\ kNm^{-1}$ (Nominal)
(over 0.665m length)

Atkins

Project	Job ref
ECC ASSESSMENT CONTRACT 3	A1 1877/61
Part of structure: DEBDEN ROAD	Calc sheet no 19, rev 1
Drawing ref: — ; Calc by: JP ; Date: 2/00	Check by: ; Date:

Ref	Calculations	Output

FACTORED LOADS

Load case	Nominal load	γ_{fL}	γ_{f3}	Factored
Internal beam s.w	$3.0 kNm^{-1}$	1.0	1.0	$3.0 kNm^{-1}$
Edge beam s.w	$3.3 kNm^{-1}$	1.0	1.0	$3.3 kNm^{-1}$
Internal beam surfacing	$3.7 kNm^{-1}$	1.5	1.0	$5.6 kNm^{-1}$
Internal beam fill	$31.6 kNm^{-1}$	1.0	1.0	$31.6 kNm^{-1}$
Edge beam surfacing	$0.8 kNm^{-1}$	1.5	1.0	$1.2 kNm^{-1}$
Edge beam fill	$18.0 kNm^{-1}$	1.0	1.0	$18.0 kNm^{-1}$
Edge beam parapet	$13.9 kNm^{-1}$	1.0	1.0	$13.9 kNm^{-1}$
HA UDL	$37.1 kNm^{-1}$	1.0	1.0	$37.1 kNm^{-1}$
HA KEL	51.3 kN	1.0	1.0	51.3 kN
Single Axle	$240.9 kNm^{-1}$	1.0	1.0	$240.9 kNm^{-1}$ (over 0.66m length)

Total dead on internal beam = $40.2 kNm^{-1}$

" " " edge beam = $36.4 kNm^{-1}$

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Atkins	Project: ECC ASSESSMENT CONTRACT 3			Job ref: AI 1877/61
	Part of structure: DERDEN ROAD			Calc sheet no: / 10 / rev
	Drawing ref	Calc by: JF	Date: 2/00	Check by / Date

Ref	Calculations	Output

LIVE LOADING

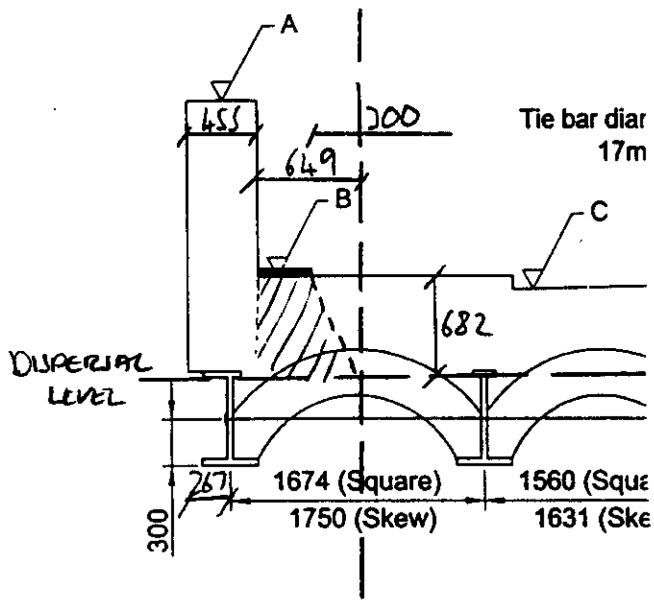
ACCIDENTAL WHEEL LOADS

There are two cases to consider.

i) Wheels adjacent to parapet i.e maximum loading on edge beam.

ii) Wheels adjacent to carriageway ie maximum loading on internal beams.

Case i



100% of wheel load will be taken by edge beam

Longitudinal dispersal = 300 + 682 = 982mm

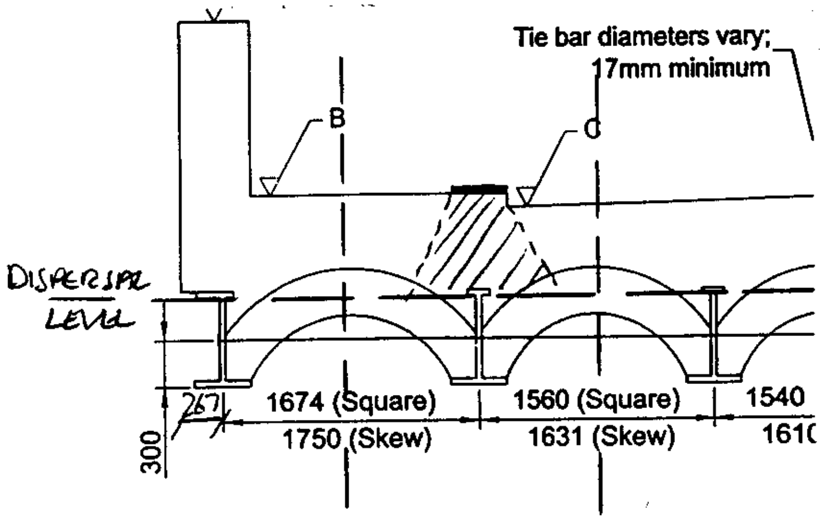
WS Atkins	Project: GCC ASSESSMENT CONTRACT 3			Job ref: AI 1877/61	
	Part of structure: DERBON ROAD			Calc sheet no: 11	rev
	Drawing ref	Calc by: JF	Date: 2/00	Check by	Date

Ref	Calculations	Output

LIVE LOADING

AWL CONT.

(case ii)



By inspection, all of wheel load will be taken by internal beam.

Longitudinal dispersal = 982mm (A for case i)

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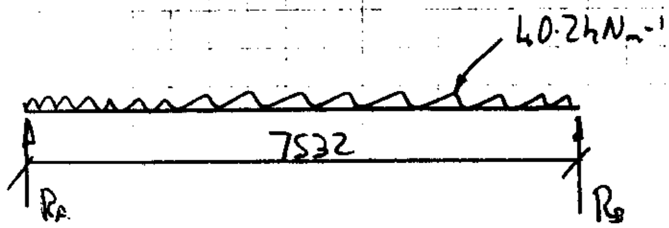
BEAM ANALYSIS

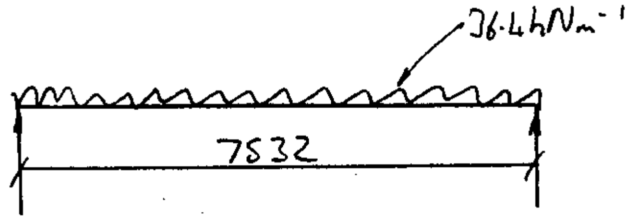
INTERNAL BEAM – PERMANENT LOADS



$$\text{Max shear} = \frac{7.532 \times 40.2}{2} = 151.4\,kN$$

$$\text{Max bending} = \frac{7.532^2 \times 40.2}{8} = 285.1\,kNm$$

EDGE BEAM – PERMANENT LOADS



$$\text{Max shear} = \frac{7.532 \times 36.4}{2} = 137.1\,kN$$

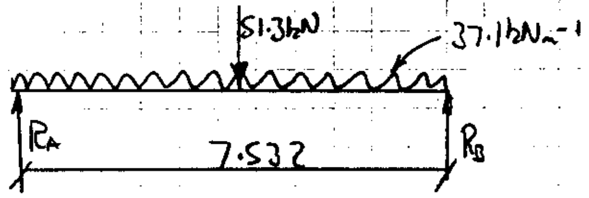
$$\text{Max bending} = \frac{7.532^2 \times 36.4}{8} = 258.1\,kNm$$

Atkins	Project: ECC ASSESSMENT CONTRACT 3		Job ref: A1 1877 /61	
	Part of structure: DEBDEN ROAD		Calc sheet no: / 13	rev: /
	Drawing ref:	Calc by: JP	Date: 2/00	Check by:	Date:

Ref	Calculations	Output

BEAM ANALYSIS - INTERNAL BEAM

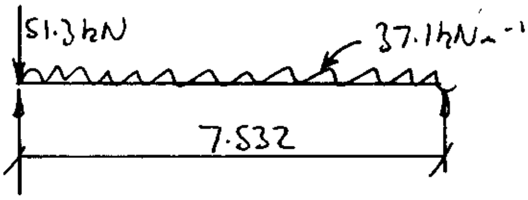
HA LIVE LOAD - MAX BENDING



$$\text{Max bending} = \frac{37.1 \times 7.532^2}{8} + \frac{51.3 \times 7.532}{4}$$

$$= \underline{359.7\,kNm}$$

HA LIVE LOAD - MAX SHEAR



$$\text{Max shear} = 51.3 + \left(\frac{37.1 \times 7.532}{2}\right)$$

$$= \underline{191.0\,kN}$$

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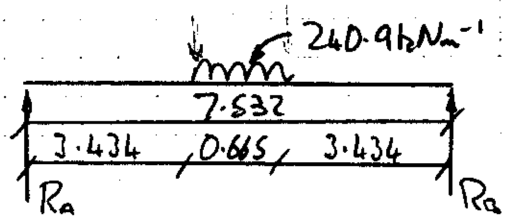
Atkins	Project: ECC ASSESSMENT CONTRACT 3			Job ref: A1 1877/61	
	Part of structure: DEBDEN ROAD			Calc sheet no: 1 14	rev: 1
	Drawing ref:	Calc by: JF	Date: 2/00	Check by:	Date:

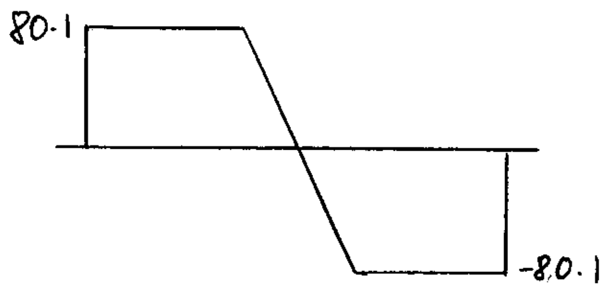
Ref	Calculations	Output

BEAM ANALYSIS - INTERNAL BEAM

SINGLE AXLE LOAD - MAX BENDING



SFD



$$\text{Max bending} = (80.1 \times 3.434) + \left(\frac{80.1 \times 0.665}{4}\right)$$

$$= 288.4\ kNm$$

$\therefore$ HA CRITICAL

Atkins

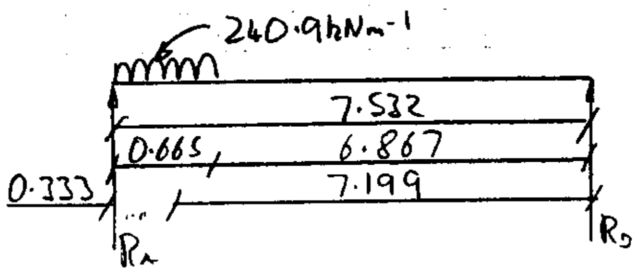
Project	Job ref
GCC ASSESSMENT CONTRACT 3	AI 1877/61
Part of structure	Calc sheet no / rev
DEBDEN ROAD	/ 15 /

Drawing ref	Calc by	Date	Check by	Date
	JF	2/00		

Ref	Calculations	Output

BEAM ANALYSIS - INTERNAL BEAM

SINGLE AXLE LOAD - SHEAR



$$\text{Max shear} = (240.9 \times 0.665) \times \frac{7.199}{7.532}$$

$$= 153.1\text{kN}$$

∴ HA CRITICAL

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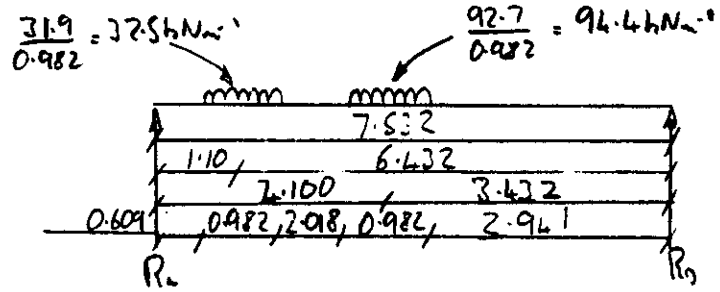
Atkins	Project: ECC ASSESSMENT CONTRACT 2			Job ref: A1 1877/61	
	Part of structure: DERDON ROAD			Calc sheet no: 1 / 11	rev: 1
	Drawing ref	Calc by: JF	Date: 2/00	Check by	Date

Ref	Calculations	Output

BEAM ANALYSIS - EDGE BEAM

VEHICLE RE - 17 TONNES

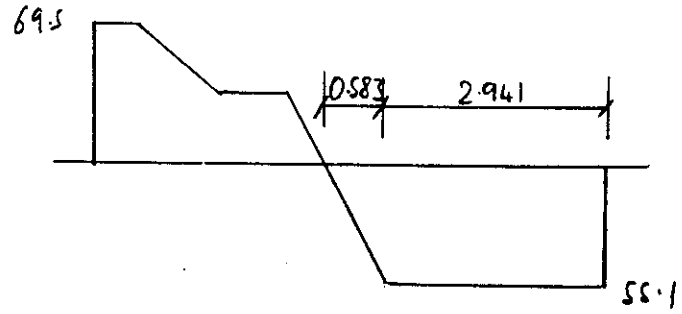
MAXIMUM BENDING



SFD

$$R_A = \frac{31.9 \times 6.432}{7.532} + \frac{92.7 \times 3.432}{7.532} = 69.5\text{kN}$$

$$R_B = \frac{31.9 \times 1.10}{7.532} + \frac{92.7 \times 4.100}{7.532} = 55.1\text{kN}$$



$$\therefore \text{Max Bending} = (55.1 \times 2.941) + (55.1 \times 0.583 \times 0.5)$$

$$= 178.1\text{kNm}$$

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WS Atkins

Project			Job ref
ECC ASSESSMENT CONTRACT 3			A1 1877/61
Part of structure			Calc sheet no / rev
DEBDEN ROAD			/ 17 /
Drawing ref	Calc by	Date	Check by / Date
	JF	2/00	

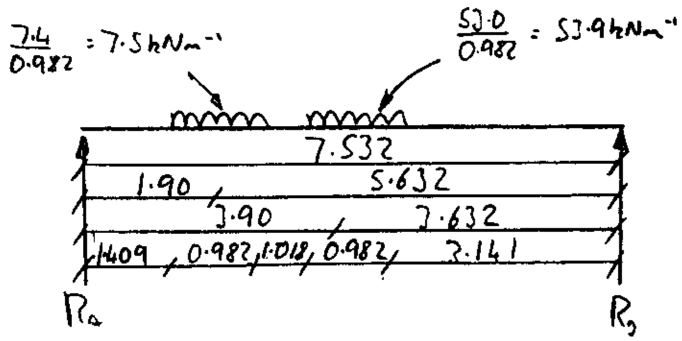
Ref	Calculations	Output

BEAM ANALYSIS - EDGE BEAM

VEHICLE RF - 7.5 TONNES

Refer to spreadsheet for worst vehicle position.

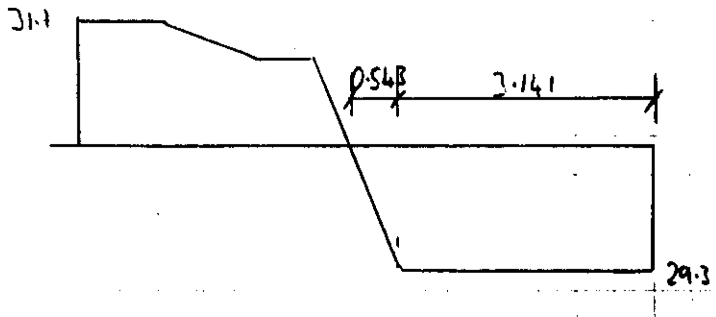
MAXIMUM BENDING;



SFD

$$R_A = \frac{7.4 \times 5.632}{7.532} + \frac{53.0 \times 3.632}{7.532} = 31.1kN$$

$$R_B = \frac{7.4 \times 1.90}{7.532} + \frac{53.0 \times 3.90}{7.532} = 29.3kN$$

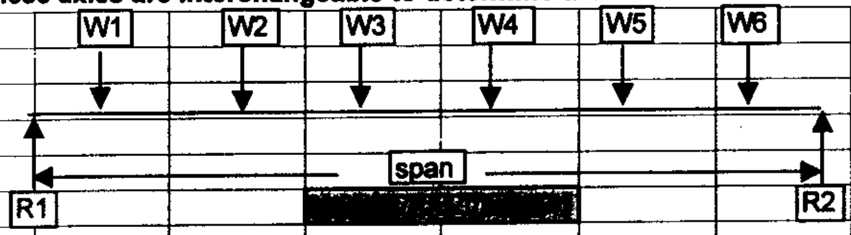


$\therefore$ Max B.M $= (29.3 \times 3.141) + (29.3 \times 0.543 \times 0.5)$

$= 100.0kNm$

Vehicle	W1 (t,kN)	X2 (m)	W2 (t,kN)	X3 (m)	W3 (t,kN)	X4 (m)	W4 (t,kN)	X5 (m)	W5 (t,kN)	X6 (m)	W6 (t,kN)	GVW (t,kN)	Σ X (m)
[redacted]												17	3
impact (γ_{fl})	1		1.18		1		1		1		1		
(inc γ_{fl})	63.765	3	185.409	0	0	0	0	0	0	0	0	249.174	3
*													
*													

*Note: These axles are interchangeable to determine the most adverse effect



[redacted]

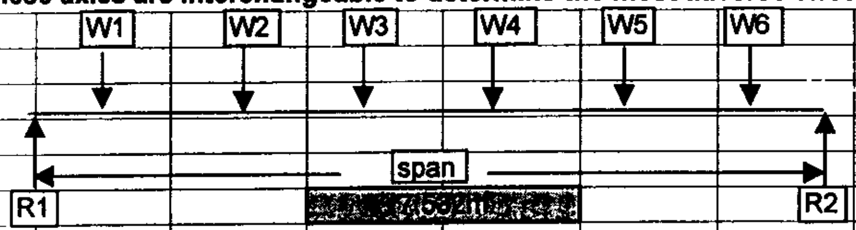
z = distance of first wheel load from support R1 (database handled) in metres

RESULT			+VE BM	ZED	-VE BM	ZED	SHEAR
			[illegible]	1.10	0.00	20.00	222.72

note: look to tables for full picture of shear across deck

Vehicle	W1 (t,kN)	X2 (m)	W2 (t,kN)	X3 (m)	W3 (t,kN)	X4 (m)	W4 (t,kN)	X5 (m)	W5 (t,kN)	X6 (m)	W6 (t,kN)	GVW (t,kN)	Σ X (m)
[redacted]												7.5	2
impact (γ_{fl})	1		1.8		1		1		1		1		
(inc γ_{fl})	14.715	2	105.948	0	0	0	0	0	0	0	0	120.663	2
*													
*													

***Note: These axles are interchangeable to determine the most adverse effect**



7.582m

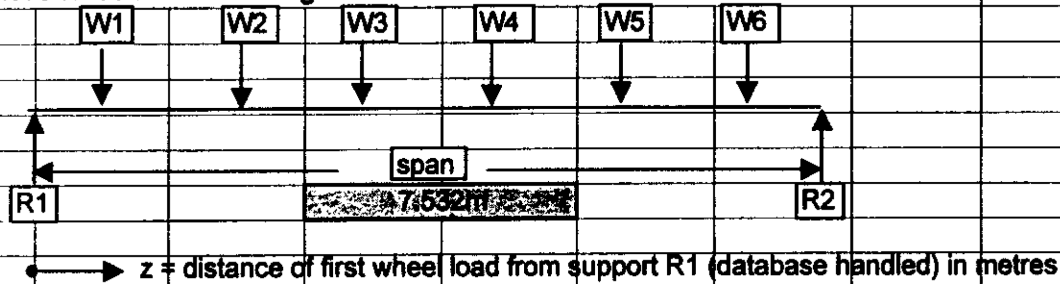
z = distance of first wheel load from support R1 (database handled) in metres

RESULT			+VE BM	ZED	-VE BM	ZED	SHEAR
			212.73	1.90	0.00	-20.00	116.24

note: look to tables for full picture of shear across deck

Vehicle	W1 (t,kN)	X2 (m)	W2 (t,kN)	X3 (m)	W3 (t,kN)	X4 (m)	W4 (t,kN)	X5 (m)	W5 (t,kN)	X6 (m)	W6 (t,kN)	GVW (t,kN)	Σ X (m)
Ec3-a	[illegible]	3.9	11.50	1.3	7.50	3	7	2.8	[illegible]	[illegible]	[illegible]	40	11
impact (γ_{fl})	[illegible]		[illegible]		1		1		[illegible]		[illegible]		
(inc γ_{fl})	68.67	3.9	203.067	1.3	73.575	3	68.67	2.8	68.67	0	0	482.652	11
*													
*													

*Note: These axles are interchangeable to determine the most adverse effect**



RESULT	+VE BM	ZED	-VE BM	ZED	SHEAR
	474.19	0.30	0.00	-20.00	291.85

note: look to tables for full picture of shear across deck

Atkins	Project: CCC ASSESSMENT CONTRACT 3	Job ref: AI 1877/61
	Part of structure: DEBDEN ROAD	Calc sheet no: / 21 / rev: /
	Drawing ref:	Calc by: JF	Date: 2/00	Check by:	Date:

Ref	Calculations	Output

BEAM CAPACITY

BD21/97 7.12

Cast Iron;

Cast iron members are to be assessed on a permissible stress basis.

7.13 The section modulus may be increased for live loading by the factor D/d (Maximum 2.0)

D = overall depth - 75mm
d = Depth of the bare girder at midspan

providing that:

	Internal beam	Edge beam
i) Girders are known to be firmly embedded	✓	X
ii) No services which would decrease the support rendered by the fill. - There is a 6" dia high pressure water main adjacent to one beam, however, it is assumed to be too small to effect support from fill ∴	✓	✓

4.10 **Cast iron - compressive and tensile stresses**

Maximum compressive stress for permanent or combined loads $= 154\ Nmm^{-2}$

Maximum tensile stress for permanent or combined loads $= 46\ Nmm^{-2}$

In addition, for a given permanent load stress the live load stress shall not exceed the value given in table 4.1.

WSAL 042 Rev 1 Dec. 89

Atkins	Project: ECC ASSESSMENT CONTRACT 3			Job ref: A1 1877/61	
	Part of structure: DEBDEN ROAD			Calc sheet no: 1 22	rev: 1
	Drawing ref:	Calc by: JF	Date: 2/00	Check by:	Date:

Ref	Calculations	Output

BENDING STRESS CALCULATIONS

INTERNAL BEAM

Moment due to permanent loads = 285.1kNm

$I = 2.069 \times 10^9 mm^4$

$y_{top} = 376mm$

$y_{bot} = 234mm$

Compressive stress $= \frac{285.1 \times 10^6 \times 376}{2.069 \times 10^9} = \underline{\underline{51.8 Nmm^{-2}}}$

Tensile stress $= \frac{285.1 \times 10^6 \times 234}{2.069 \times 10^9} = \underline{\underline{32.2 Nmm^{-2}}}$

BD21/97 7.13 Increase in section modulus,

$D = 1.197 - 0.075 = 1.122m$

$d = 0.610m$

$\therefore D/d = 1.839 < 2.0$

Factored $Z_{top} = \left(\frac{2.069 \times 10^9}{376}\right) \times 1.839$

$= 10.12 \times 10^6 mm^3$

Factored $Z_{bot} = \left(\frac{2.069 \times 10^9}{234}\right) \times 1.839$

$= 16.26 \times 10^6 mm^3$

Atkins	Project: ECC ASSESSMENT (CONTRACT 2)	Job ref: A1 1877/61
	Part of structure: DEARDEN ROAD	Calc sheet no: 23, rev:
	Drawing ref: — Calc by: JF — Date: 2/00	Check by: — Date:

Ref	Calculations	Output
	BENDING STRESS CALCULATIONS	
	INTERNAL BEAM CONT.	
	Stresses due to live loads;	
	Moment due to live loads = 359.7 kNm	SWL
	∴ Compressive stress = $\frac{359.7 \times 10^6}{10.12 \times 10^6}$ = 35.5 Nmm^{-2}	$\left(\frac{288.4}{10.12}\right.$ = 28.5 Nmm^{-2}
	Tensile stress = $\frac{359.7 \times 10^6}{16.26 \times 10^6}$ = 22.1 Nmm^{-2}	$\left(\frac{288.4}{16.26}\right.$ = 17.7 Nmm^{-2}
	Stresses due to dead + live loads;	
	Compressive stress = 51.8 + 35.5 = 87.3 Nmm^{-2}	< Allowable ∴ C
	Tensile stress = 32.2 + 22.1 = 54.3 Nmm^{-2}	> Allowable ∴ Ten
	Check live load stresses	
BD21/97 4.10	i) Tensile values of f_L, greater value of;	
	f_L = 24.6 − (0.44 × 32.2) = 10.4 Nmm^{-2}	
	OR f_L = 19.6 − (0.76 × 32.2) = −4.9 Nmm^{-2}	
	Allowable f_L = 10.4 Nmm^{-2}	
	Applied tensile stress = 22.1 Nmm^{-2} > 10.41 Nmm^{-2}	
	Reduction factor, k = $\frac{\text{Live load capacity}}{\text{Applied live}}$	
	= $\frac{10.4}{22.1}$ = 0.47	
	∴ **CAPACITY RATING = 7.5 TONNES**	

WSAL-042 Rev 1 Dec. 89

Atkins	Project: ECC ASSESSMENT CONTRACT 3	Job ref: AI 1877/61
	Part of structure: DEBDEN ROAD	Calc sheet no: / 24 ; rev: /
	Drawing ref: ; Calc by: JF ; Date: 2/00	Check by: ; Date:

Ref	Calculations	Output
	BENDING STRESS CALCS.	

INTERNAL BEAM CONT.

ii) Compressive values of f_L, greater value of

$$f_L = -43.9 + (0.79 \times 51.8) = -84.8 Nmm^{-2}$$

$$\text{or } f_L = -81.3 + (3.15 \times 51.8) = 244.5 Nmm^{-2}$$

∴ Allowable $f_L = 154 Nmm^{-2}$

Applied compressive stress $= 35.5 Nmm^{-2}$ < Allowable.

∴ TENSION CRITICAL – ADOPT 7.5 TONNES

SHEAR STRESS CALCULATION

INTERNAL BEAMS

Shear due to permanent loads = 151.4 kN

Web Area = $610 \times 38 = 23180 mm^2$

Shear stress = $\frac{151.4 \times 10^3}{23180} = 6.5 Nmm^{-2}$

Shear due to live loads = 191.0 kN

Shear stress = $\frac{191.0 \times 10^3}{23180} = 8.2 Nmm^{-2}$

Shear stress due to dead + live = $6.5 + 8.2 = 14.7 Nmm^{-2}$

$14.7 Nmm^{-2}$ < Allowable ∴ O.K.

Check live load stresses;

Live load shear acts in the same sense as dead load shear.

∴ $q_L = 24.6 - (0.44 \times 6.5)$

$= 21.7 Nmm^{-2}$

Applied live load shear = 8.2 < 21.7

∴ ADEQUATE FOR 40 TONNES

BENDING STRESS CHECKS.

EDGE BEAM - 7.5T AWL

Moment due to permanent loads = 258.1 kNm

$I = 2.527 \times 10^9 mm^4$

$y_{top} = 346mm$

$y_{bot} = 264mm$

Compressive stress $= \frac{258.1 \times 10^6 \times 346}{2.527 \times 10^9} = 35.3 Nmm^{-2}$

Tensile stress $= \frac{258.1 \times 10^6 \times 264}{2.527 \times 10^9} = 27.0 Nmm^{-2}$

Increase in section modulus;

Not applicable to unbedded edge beam.

Stresses due to live loads.

Moment due to live loads = 100.0 kNm

∴ Compressive stress $= \frac{100.0 \times 10^6 \times 346}{2.527 \times 10^9} = 13.7 Nmm^{-2}$

Tensile stress $= \frac{100.0 \times 10^6 \times 264}{2.527 \times 10^9} = 10.4 Nmm^{-2}$

Stresses due to dead + live loads combined

Compressive stress = $35.3 + 13.7 = 49 Nmm^{-2}$ < Allowable ∴ O.K.

Tensile stress - $27.0 + 10.4 = 37.4 Nmm^{-2}$ < Allowable ∴ O.K

Atkins	Project: ECC ASSESSMENT CONTRACT 3			Job ref: A11877/61	
	Part of structure: DEDDEN ROAD			Calc sheet no: /27	rev: /
	Drawing ref	Calc by: JF	Date: 2/00	Check by	Date

Ref	Calculations	Output

BENDING STRESS CALCS.

EDGE BEAM

Check live load stresses

Tensile stresses; $f_L = 24.6 - (0.44 \times 27.0) = 12.7 Nmm^{-2}$

OR $f_L = 19.6 - (0.76 \times 27.0) = -0.9 Nmm^{-2}$

∴ Allowable $f_L = 12.7 Nmm^{-2}$

Applied live tensile stress $= 10.4 Nmm^{-2} < 12.7 Nmm^{-2}$ ∴ O.K.

Compressive stresses; $f_L = -43.9 + (0.79 \times -35.3) = -71.8 Nmm^{-2}$

OR $f_L = -81.3 + (3.15 \times -35.3) = -192.5 Nmm^{-2}$

∴ Allowable $f_L = 154 Nmm^{-2}$

Applied compressive stress $= 13.7 Nmm^{-2}$ < Allowable

∴ ADEQUATE FOR 7.5 TONNE VEHICLE

Atkins	Project: ECC ASSESSMENT CONTRACT 3			Job ref: A1 1877/61	
	Part of structure: DEBDEN ROAD			Calc sheet no: 1/22	rev: 1
	Drawing ref	Calc by: JF	Date: 2/00	Check by	Date

Ref	Calculations	Output

DENOING STRESS CALCS

EDGE BEAM - 17 TONNES AWL

Stresses due to permanent loads;

As for 7.5t vehicle.

Compressive stress = $35.3\,Nmm^{-2}$

Tensile stress = $27.0\,Nmm^{-2}$

Stresses due to live loads;

Moment due to live loads = 178.1 kNm

Compressive stress = $\frac{178.1 \times 10^6 \times 346}{2.527 \times 10^9} = 24.4\,Nmm^{-2}$

Tensile stress = $\frac{178.1 \times 10^6 \times 264}{2.527 \times 10^9} = 18.6\,Nmm^{-2}$

Stresses due to dead + live loads combined;

Compressive stress = 35.3 + 24.4 = $59.7\,Nmm^{-2}$ < Allowable

Tensile stress = 27.0 + 18.6 = $45.6\,Nmm^{-2}$ < Allowable

Check live load stresses;

As for 7.5t vehicle.

Allowable f_L = $12.7\,Nmm^{-2}$

Applied live tensile stress = $18.6\,Nmm^{-2}$ > Allowable

∴ FAILS 17T VEHICLE

WS Atkins	Project: ECC ASSESSMENT CONTRACT 3	Job ref: A1 1877/61
	Part of structure: DEBDEN ROAD	Calc sheet no: 129 rev: 1
	Drawing ref: — Calc by: JF Date: 2/00	Check by: Date:

Ref	Calculations	Output

SHEAR STRESS CALCS

EDGE BEAM - 40T AWL

Shear due to permanent loads = 137.1kN

Web Area = 23180mm^2

Shear stress : $\frac{137.1 \times 10^3}{23180} = 5.9 Nmm^{-2}$

Shear stress due to live loads;

Shear due to live loads = 291.9kN

Shear stress = $\frac{291.9 \times 10^3}{23180} = 12.6 Nmm^{-2}$

Shear stress due to combined dead + live loads;

$= 5.9 + 12.6 = 18.5 Nmm^{-2} <$ Allowable ∴ O.K

Check live load stresses;

Live load shear acts in the same sense as the dead load shear.

$\therefore q_L = 24.6 - (0.44 \times 5.9)$

$= 22.0 Nmm^{-2}$

Applied live load shear = $12.6 Nmm^{-2} < 22.0 Nmm^{-2}$

∴ ADEQUATE FOR 40 TONNES

Atkins

Project			Job ref
ECC ASSESSMENT CONTRACT 3			A1 1877/61
Part of structure			Calc sheet no / rev
DEBDEN ROAD			1 70 1
Drawing ref	Calc by JF	Date 2/00	Check by / Date

Ref	Calculations	Output

TIE BAR ASSESSMENT

Tie bars to be assessed using the horizontal thrust from the jack arches.

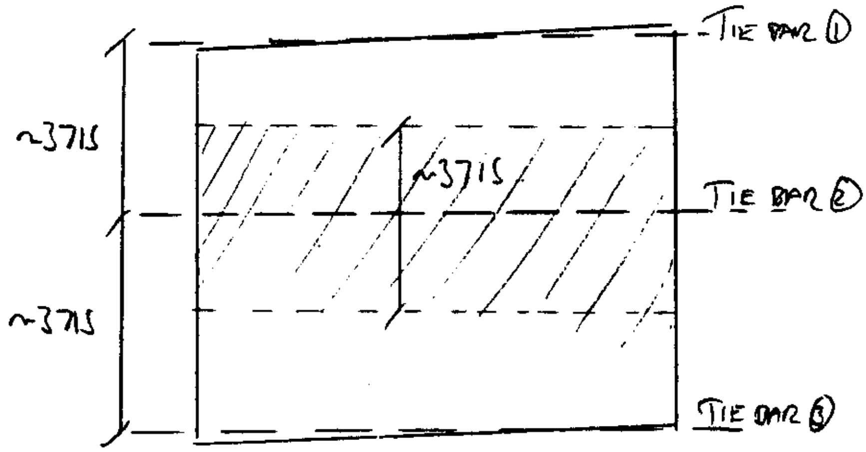
Tie bar capacity

Minimum diameter = 17mm

$$\text{Area} = \left(\frac{17}{2}\right)^2 \times \pi = 227\text{mm}^2$$

Allowable thrust = $227 \times 230 = 52.2$ kN

The length of bridge the tie bar has to restrain is shown below



WS Atkins	Project: ECC ASSESSMENT CONTRACT 3			Job ref: M1 1877 /61	
	Part of structure: DETBDEN ROAD			Calc sheet no: 31	rev
	Drawing ref	Calc by: JF	Date: 2/00	Check by	Date

Ref	Calculations	Output

TIE BAR ASSESSMENT CONT.

Method of assessment;

Using ARCHIE, obtain the horizontal thrust per metre on the following basis.

Adopt a lane width of the tie bar catchment width ie 3.715m

Apply an 11.5t axle to the lane width adopting jack arch dimensions given on recalculation drawing.

Position axle at centre of arch to obtain maximum horizontal thrust.

Refer to ARCHIE output;

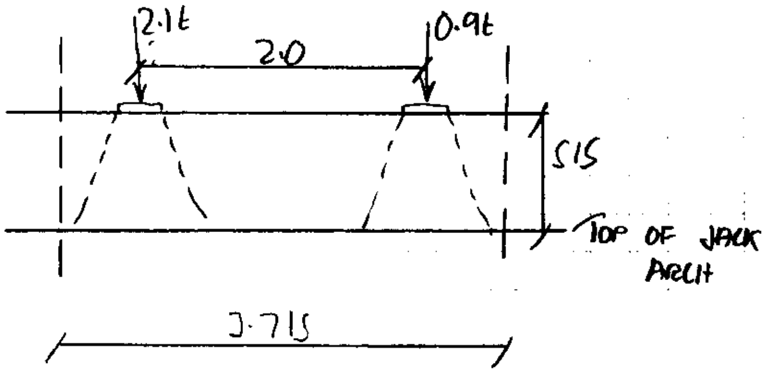
For 11.5t axle, maximum horizontal thrust = 63kN/m

∴ Horizontal thrust per tie = 63.0 × 3.715 = 234 kN

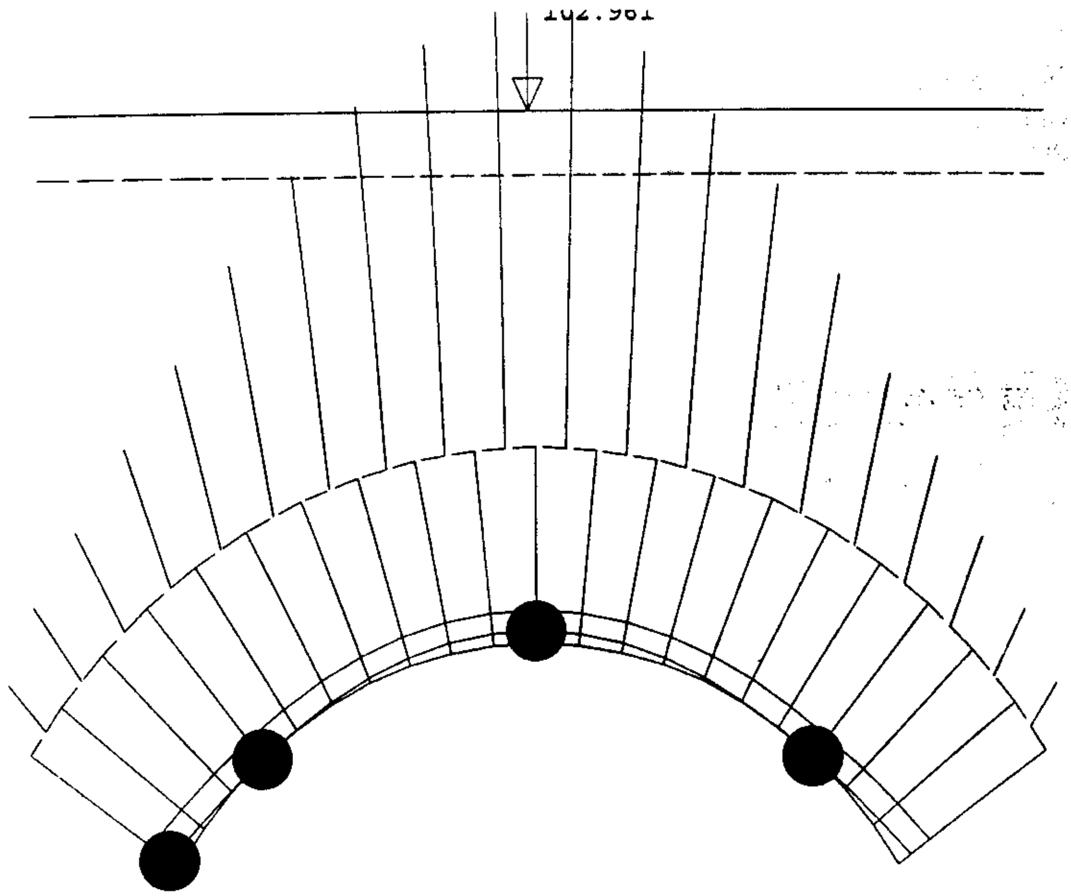
Maximum tonnage tie bar can resist;

$$\frac{52.2}{234} \times 11.5 = 2.6 \text{ tonnes}$$

For 3t axle;



∴ INADEQUATE CAPACITY FOR 3T



Tiebarck

()

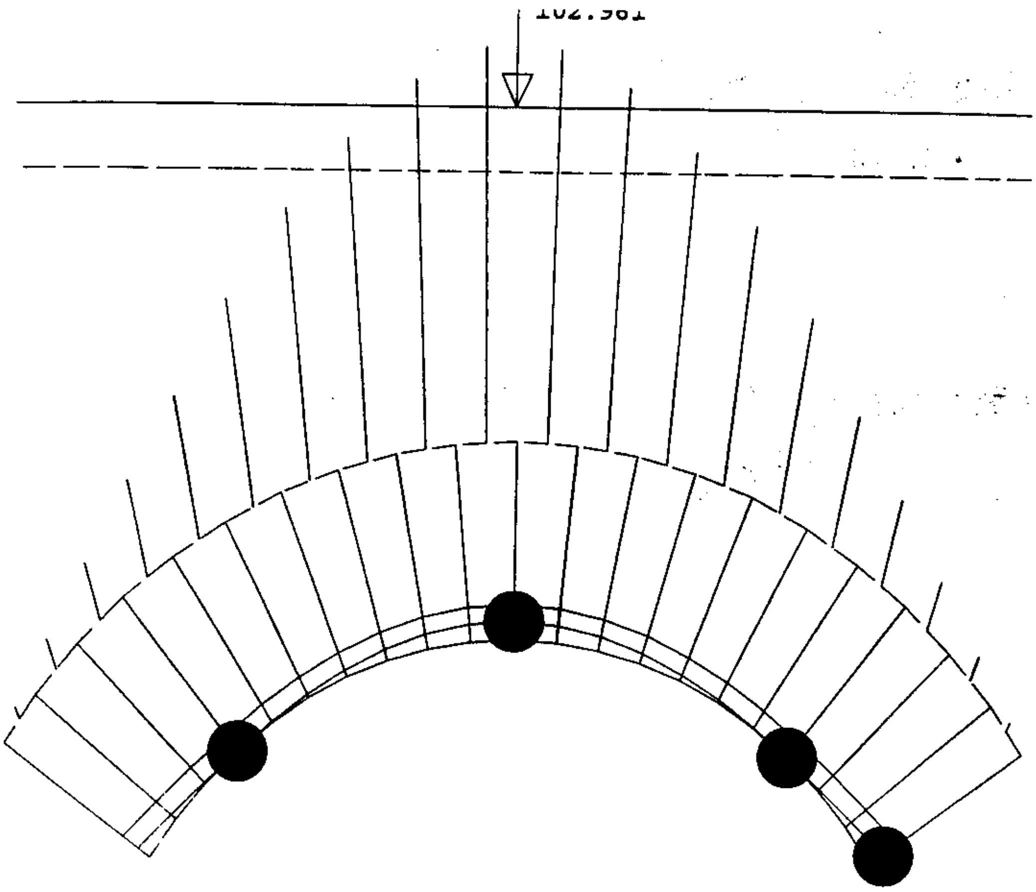
Span	1223 mm	Rise	338 mm
Depth of fill	515 mm	Depth of surfacing	100 mm
Ring depth	303 mm	Ring depth factor	1
Position of backing	0	Depth of mortar loss	0 mm
Fill density	19 kN/m^3	Masonry density	21 kN/m^3
Surfacing density	23 kN/m^3		
Phi for fill	30 deg	Masonry strength	2.3 N/mm^2
Load	Single Axle:11.5t at 600		
Lane width	3715mm		
Required ring depth	66 mm	Geometric F.O.S	4.58
H Left	61 kN/m	H Right	63 kN/m
V Left	69 kN/m	V Right	67 kN/m
Comp. zone at hinge 2	39 mm	Factor on pass. press.	.3

Hinges

1 AT 1   2 AT 4   3 AT 11   4 AT 18

Param(mn).segment

	Stone Weight	Vertical Dead Load	Horizontal Deadload	Vertical Live Load	Horizontal Live Load	Additional Pass Press
1	-.5	-.9	.7	-.3	.2	.3
2	-.6	-1.2	.7	-.8	.3	.5
3	-.6	-1.2	.6	-1.6	.5	.4
4	-.6	-1.2	.4	-2.7	.7	.3
5	-.6	-1.1	.4	-4	.8	.2
6	-.6	-1.1	.3	-5.8	1	.1
7	-.6	-1.1	.2	-7.5	.9	0
8	-.6	-1.1	.1	-8.9	.8	0
9	-.6	-1.1	.1	-10.2	.5	0
10	-.6	-1.1	0	-10.9	.2	0
11	-.6	-1.1	0	-10.8	-.2	0
12	-.6	-1.1	-.1	-10	-.5	0
13	-.6	-1.1	-.1	-8.6	-.8	0
14	-.6	-1.1	-.2	-7.1	-.8	0
15	-.6	-1.1	-.3	-5.4	-.9	0
16	-.6	-1.1	-.4	-3.6	-.8	0
17	-.6	-1.2	-.4	-2.3	-.6	0
18	-.6	-1.2	-.6	-1.3	-.4	0
19	-.6	-1.2	-.7	-.7	-.3	0
20	-.5	-.9	-.7	-.2	-.1	0



(33)

Tiebarck

()

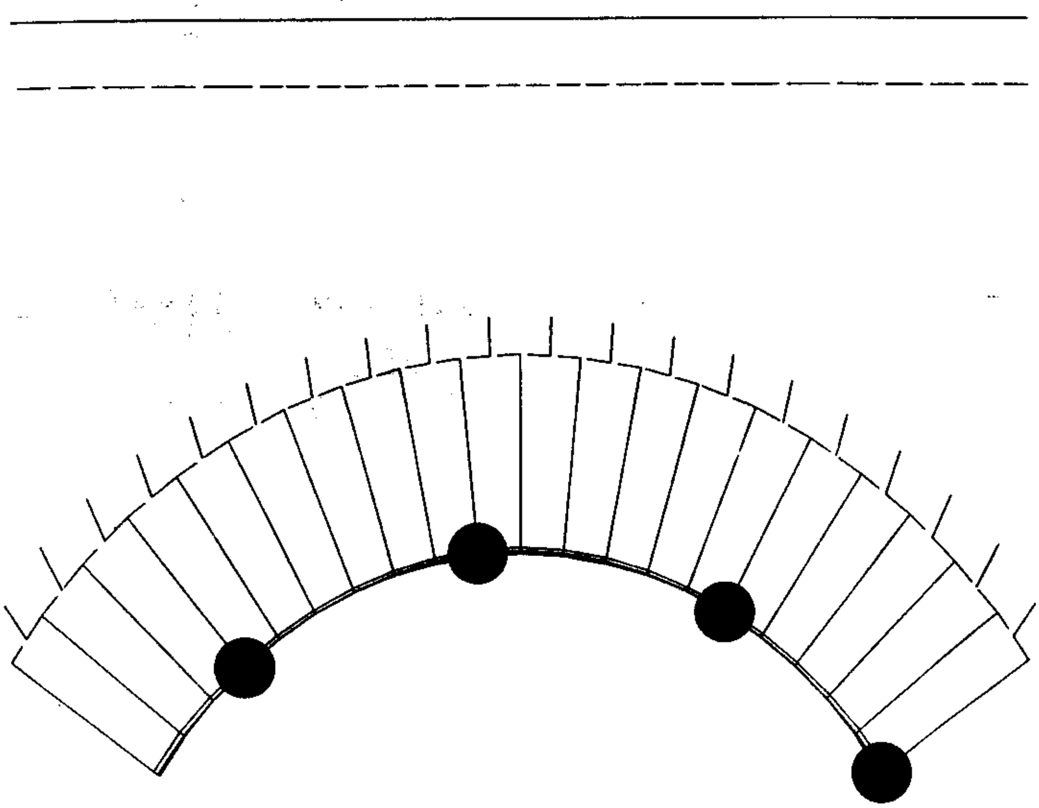
Span	1223 mm	Rise	338 mm
Depth of fill	515 mm	Depth of surfacing	100 mm
Ring depth	303 mm	Ring depth factor	1
Position of backing	0	Depth of mortar loss	0 mm
Fill density	0 kN/m^3	Masonry density	0 kN/m^3
Surfacing density	0 kN/m^3		
Phi for fill	30 deg	Masonry strength	2.3 N/mm^2
Load	Single Axle:11.5t at 600		
Lane width	3715mm		
Required ring depth	59 mm	Geometric F.O.S	5.13
H Left	52 kN/m	H Right	53 kN/m
V Left	52 kN/m	V Right	50 kN/m
Comp. zone at hinge 2	31 mm	Factor on pass. press.	.3

Hinges

1 AT 4 2 AT 11 3 AT 18 4 AT 21

Param(mn).segment

	Stone Weight	Vertical Dead Load	Horizontal Deadload	Vertical Live Load	Horizontal Live Load	Additional Pass Press
1	0	0	0	-.3	.2	0
2	0	0	0	-.8	.3	0
3	0	0	0	-1.6	.5	0
4	0	0	0	-2.7	.7	0
5	0	0	0	-4	.8	0
6	0	0	0	-5.8	1	0
7	0	0	0	-7.5	.9	0
8	0	0	0	-8.9	.8	0
9	0	0	0	-10.2	.5	0
10	0	0	0	-10.9	.2	0
11	0	0	0	-10.8	-.2	0
12	0	0	0	-10	-.5	0
13	0	0	0	-8.6	-.8	0
14	0	0	0	-7.1	-.8	0
15	0	0	0	-5.4	-.9	0
16	0	0	0	-3.6	-.8	0
17	0	0	0	-2.3	-.6	0
18	0	0	0	-1.3	-.4	0
19	0	0	0	-.7	-.3	0
20	0	0	0	-.2	-.1	0



Tiebarck

()

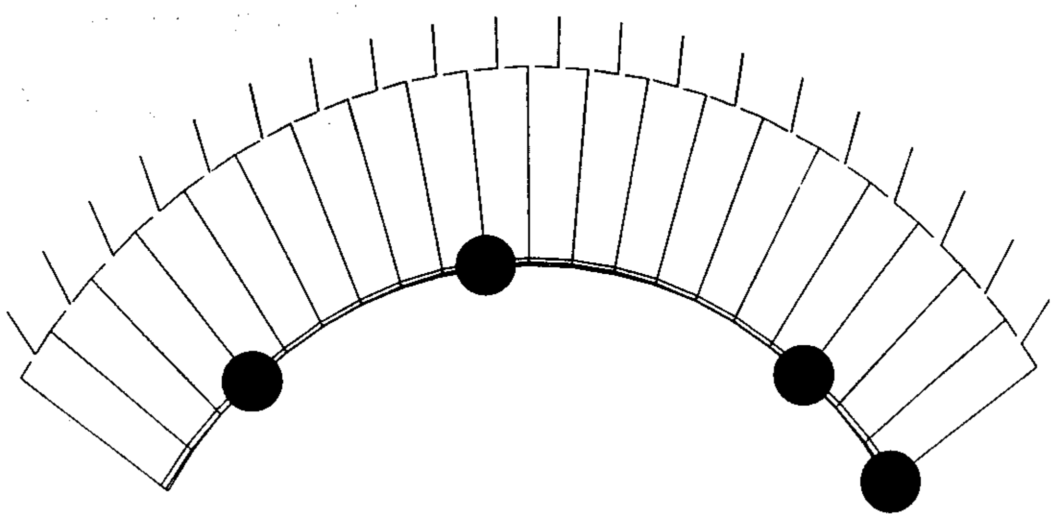
Span	1223 mm	Rise	338 mm
Depth of fill	515 mm	Depth of surfacing	100 mm
Ring depth	303 mm	Ring depth factor	1
Position of backing	0	Depth of mortar loss	0 mm
Fill density	19 kN/m^3	Masonry density	21 kN/m^3
Surfacing density	23 kN/m^3		
Phi for fill	30 deg	Masonry strength	2.3 N/mm^2
Load	Dead Load Only at 500		
Lane width	3715mm		
Required ring depth	13 mm	Geometric F.O.S	24.1
H Left	8 kN/m	H Right	8 kN/m
V Left	17 kN/m	V Right	17 kN/m
Comp. zone at hinge 2	8 mm	Factor on pass. press.	.3

Hinges

1 AT 4 2 AT 10 3 AT 16 4 AT 21

Param(mn).segment

	Stone Weight	Vertical Dead Load	Horizontal Deadload	Vertical Live Load	Horizontal Live Load	Additional Pass Press
1	-.5	-.9	.7	0	0	0
2	-.6	-1.2	.7	0	0	0
3	-.6	-1.2	.6	0	0	0
4	-.6	-1.2	.4	0	0	0
5	-.6	-1.1	.4	0	0	0
6	-.6	-1.1	.3	0	0	0
7	-.6	-1.1	.2	0	0	0
8	-.6	-1.1	.1	0	0	0
9	-.6	-1.1	.1	0	0	0
10	-.6	-1.1	0	0	0	0
11	-.6	-1.1	0	0	0	0
12	-.6	-1.1	-.1	0	0	0
13	-.6	-1.1	-.1	0	0	0
14	-.6	-1.1	-.2	0	0	0
15	-.6	-1.1	-.3	0	0	0
16	-.6	-1.1	-.4	0	0	0
17	-.6	-1.2	-.4	0	0	0
18	-.6	-1.2	-.6	0	0	0
19	-.6	-1.2	-.7	0	0	0
20	-.5	-.9	-.7	0	0	0



Tiebarck

()

Span	1223 mm	Rise	338 mm
Depth of fill	515 mm	Depth of surfacing	100 mm
Ring depth	303 mm	Ring depth factor	1
Position of backing	0	Depth of mortar loss	0 mm
Fill density	22.8 kN/m^3	Masonry density	24 kN/m^3
Surfacing density	40.3 kN/m^3		
Phi for fill	30 deg	Masonry strength	2.3 N/mm^2
Load	Dead Load Only at 500		
Lane width	3715mm		
Required ring depth	14 mm	Geometric F.O.S	21.11
H Left	11 kN/m	H Right	11 kN/m
V Left	21 kN/m	V Right	21 kN/m
Comp. zone at hinge 2	10 mm	Factor on pass. press.	.3

Hinges
1 AT 4   2 AT 10   3 AT 18   4 AT 21

Param(mn).segment

	Stone Weight	Vertical Dead Load	Horizontal Deadload	Vertical Live Load	Horizontal Live Load	Additional Pass Press
1	-.6	-1.2	.9	0	0	0
2	-.6	-1.5	.8	0	0	0
3	-.6	-1.5	.7	0	0	0
4	-.6	-1.5	.6	0	0	0
5	-.6	-1.5	.5	0	0	0
6	-.6	-1.5	.4	0	0	0
7	-.6	-1.5	.3	0	0	0
8	-.6	-1.4	.2	0	0	0
9	-.6	-1.4	.1	0	0	0
10	-.6	-1.4	0	0	0	0
11	-.6	-1.4	0	0	0	0
12	-.6	-1.4	-.1	0	0	0
13	-.6	-1.4	-.2	0	0	0
14	-.6	-1.5	-.3	0	0	0
15	-.6	-1.5	-.4	0	0	0
16	-.6	-1.5	-.5	0	0	0
17	-.6	-1.5	-.6	0	0	0
18	-.6	-1.5	-.7	0	0	0
19	-.6	-1.5	-.8	0	0	0
20	-.6	-1.2	-.9	0	0	0

WS Atkins

CALCULATION INDEX

C = Calculation
S = Sketch
D = Data

PROJECT: ECC ASSESSMENT CONTRACT 3

FILE NAME: DEBDEN ROAD BRIDGE CHECK CALCS

FILE NO: A1877-61

Page

INDEX NO	CALCULATION/SKETCH/DATA		ORIGINATOR	DATE
SUM	SUMMARY		SB	3/00
1	ANALYSIS DATA		SB	3/00
2	ANALYSIS DATA + DEMO LOADING	INTERNAL	SB	3/00
3	SECTION PROPERTIES	INTERNAL	SB	3/00
4-7	LOADING AND ANALYSIS	INTERNAL	SB	3/00
8-10	STRESS CHECK	INTERNAL	SB	3/00
11	SECTION PROPERTIES	EDGE BEAM	SB	3/00
12-14	LOADING AND ANALYSIS	EDGE BEAM	SB	3/00
15-17	STRESS CHECK	EDGE BEAM	SB	3/00
18-19	JACK ARCH		SB	3/00
20-21	TIE RODS		SB	3/00

Continued on Sheet

OF90/WSAEA

WS Atkins

Project			Job ref	
ECC Assessment Contract 3			AI 1877/61	
Part of structure			Calc sheet no	rev
Debden Rd Bridge			/Sum./	
Drawing ref	Calc by SB	Date Feb 00	Check by	Date

Ref	Calculations	Output

Summary

Cast iron beam - internal

Bending = 7.5 tonnes (HA).

Shear = 40 tonnes. (HA).

Cast iron - edge

Bending = 7.5 tonnes (awL)

Shear = 40 tonnes (awl)

Jack Arches

Capacity - 40 tonnes.

Tie bars < 3 tonnes.

WS Atkins	Project: ECC Assessment Contract 3			Job ref: AI1877/61	
	Part of structure: Debden Rd			Calc sheet no / rev: 1 / 1	
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Ref	Calculations	Output
BI204/1004/MSP		

Debden Road Bridge

Single span structure - comprising cast iron beams with brick jack arches.

Brickwork abutments and parapets.

West elevation backfilled to carriageway level.

Clear square span = 7.105m

Skew = 17°.

Parapets - brickwork + steel post and rail height extension at the south end.

Carriageway width = 5.08m wide
East footway = 0.89m wide
West footway = 1.68m wide

Currently has a 22tonne weight restriction

Parapets - 355mm brickwork

Jack arch barrel thickness = 300mm.

Category = Lp.

Cast iron beams - skew span (L) = 7.532m
skew angle (α) = 17°

Jack arches - square span (L) = 1.223m

Arch barrel thickness = 300mm.
Masonry strength = 2.3N/mm².

For archie analysis, F_c = 0.45.

Cast iron beam, F_{cm} = 1.0 (actual beam section used)

Ref	Calculations	Output

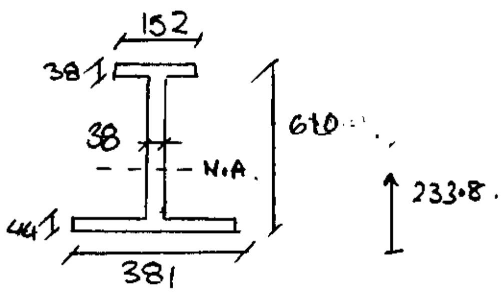
AIP — Traffic speed = 30mph.

Jack arches assessed as arches springing from the beams using computer program ARCHIE.

Information for Archie
Backing level None
Masonry sw = 21 kN/m^3
Fill sw. = 19 kN/m^3
Surfacing sw = 23 kN/m^3
ϕ for fill = 30°
Archie pressure coeff = 0·3
Masonry strength = 2·3 N/mm^2

Cast Iron Beam Assessment

Internal Beam.



BD21/97 — **Dead load and Superdead load.**

Table 4·1. — Unit weight of cast iron = 7200 kg/m^3

= 70·6 kN/m^3

Section Area

= (152 × 38) + (381 × 44) + (38 × 528)

= 42604 mm^2

Atkins	Project: ECC Assessment Contract 3	Job ref: AI1877/61
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	Drawing ref:	Calc by: SB	Date: Jan 00	Check by:	Date:

Ref	Calculations	Output
	Find NA Take moments about bottom edge. $= [(44 \times 381 \times 22) + (38 \times 528 \times 308) + (152 \times 38 \times 591)] / 42604$ $= 233.8$mm from bottom edge. I of section $\left(\frac{381 \times 44^3}{12}\right) + (381 \times 44 \times 211.8^2) = 754.72 \times 10^6$ $+ \left(\frac{38 \times 528^3}{12}\right) + (38 \times 528 \times 74.2^2) = 576.6 \times 10^6$ $+ \left(\frac{152 \times 38^3}{12}\right) + (152 \times \text{~~528~~ } 38 \times 357.2^2) = 737.7 \times 10^6$ $= 2.069 \times 10^9$ mm^4. $Z_t = \frac{2.069 \times 10^9}{233.8} = 48.851 \times 10^6$ mm^3 $Z_b = -\frac{2.069 \times 10^9}{610 - 233.8} = -5.50 \times 10^6$ mm^3	
BD21/97	Permissible stresses.	
4.16.	Compressive stress $\not>$ 154 N/mm^2 Tensile stress $\not>$ 46 N/mm^2.	

WS Atkins	Project: ECC Assessment Contract 3			Job ref: AI 1877/61	
	Part of structure: Debden Rd			Calc sheet no: 4	rev
	Drawing ref	Calc by: SB	Date: Jan 00	Check by	Date

Ref	Calculations	Output

Loading

AI1877\BWGS\1004\FIG1. — Dead loads Internal Beam

Surfacing levels Section AA :- C, D, E

Average carriageway level = 8.558

Surfacing = 100mm
Ring thickness = 300mm
Arch rise = 382mm
Average soffit level = 7.445
Flange thickness = 44mm

∴ Depth of bridge ~~fill above arch~~

= 8.603 − 7.445

= 1.158 m

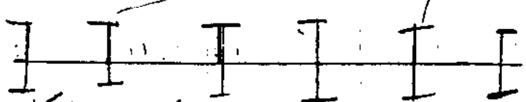
AIP — Assume masonry and poor quality fill have same density.
Average level of jack arch occurs @ $\frac{382}{2} = 191$ from soffit.

AIP — Fill + arch = 1.158 − 0.191 = 0.967m
= 0.967 × 21 = 20.3 kN/m per m.

Table 4.1 — Internal Beam self weight $\frac{7200 \times 9.81}{1000} = 70.6$ kN/m³

= 0.0426 × 70.6 = 3.008 kN/m

Total Dead - outer, internal beam.



1674 1560 1540 1578 1674

Take central internal beams and use width for loading = $\frac{1540 + 1578}{2} = 1559$

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Atkins	Project: ECC Assessment Contract 3			Job ref: AI/877/61	
	Part of structure: Debden Rd			Calc sheet no: 5	rev
	Drawing ref	Calc by: SB	Date: Jan	Check by	Date

Ref	Calculations	Output

Dead load =

(20.3 x 1.559) + (0.1 x 1.559 x 1.5 x 235) + 3.008

fill+barrel   surf.   SW of beam

= 40.15 kN/m

Live Loads

BD21/97 5.25 — HA UDL = $336\left(\frac{1}{L}\right)^{0.67}$

Fig 6. — L = 7.532m $= 336\left(\frac{1}{7.532}\right)^{0.67} = 86.86$ kN/m

Adjustment factor, since L = 7.532

AF = $\bar{a}$/2.5 = 3.65/2.5 = 1.46

Adjusted HA UDL = 86.86/1.46 = 59.5 kN/m

5.27. — Adjusted HA UDL per beam = $\frac{59.5}{2.5}$ x 1.559 = 37.1 kN/m

HA KEL = 120 kN

Adjusted HA KEL per beam = $\frac{120}{1.46 \times 2.5}$ x 1.559 = 51.3 kN/beam

Table 5/3/2 — Single wheel load for Lp = 90kN.

WS/Atkins

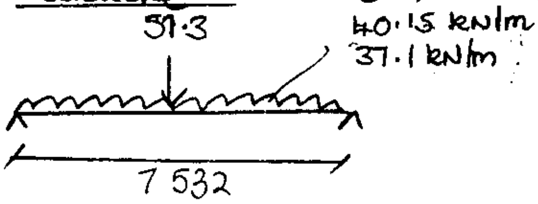
Project	Job ref
ECC Assessment Contract 3	AI 1877/61
Part of structure	Calc sheet no / rev
Debden Rd Bridge	16 / 1

Drawing ref	Calc by	Date	Check by	Date
	SB	Jan 00		

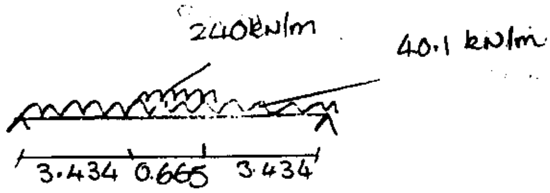
Ref	Calculations	Output

Fig 6.

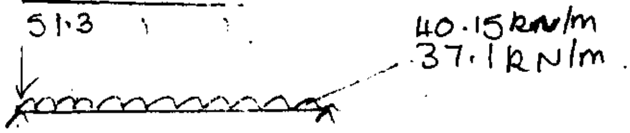
Loadcase 1 - HA UDL, HA KEL, SW udl



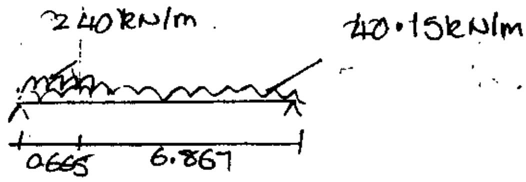
Loadcase 2 - SW UDL, Single Wheel



Loadcase 3 - HA UDL, HA KEL, SW udl



Loadcase 4 - SW UDL, Single Wheel



Loadcase 1

$$M_A = \frac{77.25 \times 7.532^2}{8} + \frac{51.3 \times 7.532}{4} = 644.4\,kNm$$

$$R_A = \frac{77.25 \times 7.532}{2} + \frac{51.3}{2} = 316.6\,kN$$

Loadcase 2

$$M_A = \frac{40.1 \times 7.532^2}{8} + 80.1 \times 3.434 + \frac{80.1 \times 0.665}{4} = 572.7\,kNm$$

~~$R_T = \frac{8.63 \times 7.532}{2} + \frac{90}{2} = 352.4$~~

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WS Atkins	Project: ECC Assessment Contract 3	Job ref: AI1877/63
	Part of structure: South Rd Bridge	Calc sheet no: /6A/ rev
	Drawing ref: — Calc by: SB Date: Mar 00	Check by: Date:

Ref	Calculations	Output

Single Wheel Loading

21/97 table 5/3/2. From LP, single wheel load = 90kN

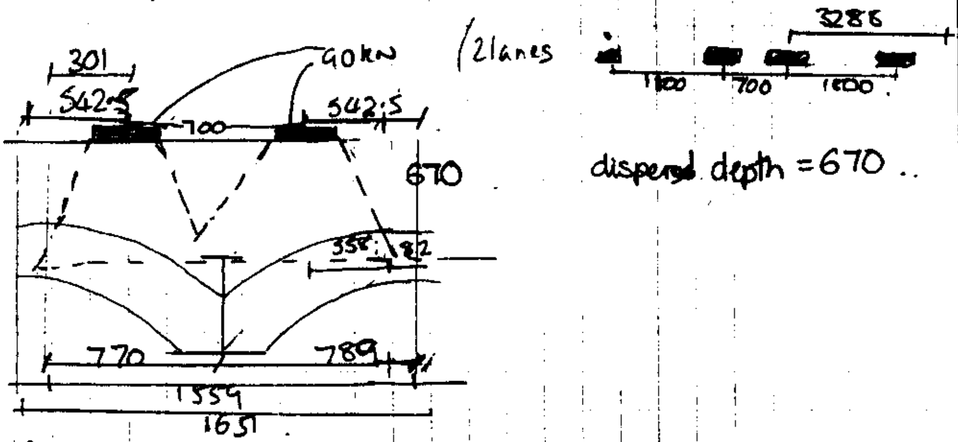
6.7 ,, Dispersal in jack arches to level of mid depth of the arch ring at the crown.

Effective pressure = 1.1N/mm²

Wheel bearing area with square sides $= \sqrt{\frac{90\,000}{1.1}} = 286\text{mm}$.

fig 6.4

pg 4



dispersed depth = 670..

% of wheel loads on beam $= \frac{1559 - 82}{1651} = 89\%$

Effective UDL per beam $= \frac{2 \times 90 \times 0.89}{0.665} = 240.9\text{kN/m}$.

WS Atkins	Project: ECC Assessment Contract 3	Job ref: AI 1877/61
	Part of structure: Debden Rd Bridge	Calc sheet no: 7, rev:
	Drawing ref: — ; Calc by: SB ; Date: Jan 00	Check by: — ; Date: —

Ref	Calculations	Output

Loadcase 3

$M(B) = R_A \times 7.532 = 51.3 \times 7.532 + 77.25 \times \frac{7.532^2}{2}$

$R_A = 342\,kN$

$R_B = 291\,kN$

Loadcase 4

$R_A = (240 \times 0.665)\frac{7.199}{7.532} + 40.15 \times \frac{7.532}{2}$

$R_A = 303\,kN$

$\therefore \; M_A = 644.4\,kNm \quad (LC\,1)$

$V_A = 342\,kN \quad (LC\,3)$

Permissible stresses

4.10 Compressive stress ≯ $154\,N/mm^2$

Tensile stress ≯ $40\,N/mm^2$

$\sigma_c = \frac{748.4 \times 10^3 \times 10^3}{30.76 \times 10^6} = 24.3\,N/mm^2 ≯ 154\,N/mm^2$

$\sigma_t = \frac{748.4 \times 10^3 \times 10^3}{49.49 \times 10^6} = 15.12\,N/mm^2 ≯ 46\,N/mm^2$

i). Tensile values of f_L,

f_L shall not exceed the values given by: the greater of:

WS Atkins

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ECC Assessment Contract 3			A1 1877 / 61
Part of structure			Calc sheet no / rev
Debden Rd Bridge			/ 8 /
Drawing ref	Calc by SB	Date Mar 00	Check by / Date

Ref	Calculations	Output

7
7.12

Strength of members.

- Cast iron assessed on permissible strength
- Section modulus of cast iron girders may be increased for live loading by the factor D/d

D = overall depth − 75mm

d = depth of the bare girder at midspan.

D = 1.158 − 0.075 = 1.083.

d = 0.610.

provided i) girders are firmly embedded in well consolidated filling mat.
ii) no services in carriageway which would decrease the support rendered by the fill.

Both of the above conditions are met.

Factor = D/d = 1.083/0.610 = 1.775.

Increased section modulus

$= 1.775 \times 2.069 \times 10^9 = 3.67 \times 10^9\ mm^4$

Increased $Z_t = 1.775 \times 8.85 \times 10^6 = 15.7 \times 10^6\ mm^3$

Increased $Z_c = 1.775 \times {}^{-}5.5 \times 10^6 = {}^{-}9.76 \times 10^6\ mm^3$

4.10 **Cast Iron - Compressive + Tensile Stress**

Stress due to permanent + live load.

Stress due to permanent load (compressive) $= \dfrac{\frac{40.15 \times 7.532^2}{8} \times (610 - 233.8) \times 10^6}{2.069 \times 10^9}$

$= -51.8 \, N/mm^2$

(tensile) $= \dfrac{284.7 \times 233.8 \times 10^6}{2.069 \times 10^9} = 32.2 \, N/mm^2$

Stress due to live load (compressive) $= \dfrac{359.7 \times 10^6}{-9.76 \times 10^6}$

$= -36.9 \, N/mm^2$

~~Stress~~ (tensile) $= \dfrac{359.7 \times 10^6}{15.7 \times 10^6} = 22.9 \, N/mm^2$

Stress due to permanent + live load

compressive $= -29.2 - 36.9 = -66.1 \, N/mm^2$

tensile $= 18.1 + 22.9 = 41 \, N/mm^2$

∴ ok.

i) for tensile values, the greater of the values given by

$f_L = 24.6 - 0.44 f_a = 24.6 - (0.44 \times 32.2) = 10.43 \, N/mm^2$

$f_L = 19.6 - 0.76 f_d = 19.6 - (0.76 \times 32.2) = -4.87 \, N/mm^2$

∴ Allowable $f_L = 10.43 \, N/mm^2$

Applied f_L > Allowable f_L.

∴ $K = \dfrac{10.43}{22.9} = 0.455$

fig 5/4 Capacity = 7.5te

WS Atkins	Project: CCC Assessment Contract 3	Job ref: AI 1877/61
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	Drawing ref:	Calc by: SB	Date: Mar 00	Check by:	Date:

Ref	Calculations	Output

ii) for compressive values, the greater of the following values.

$f_L = -43.9 + 0.79 f_d = -43.9 + (0.79 \times 51.8) = -84.8 N/mm^2$.

$f_L > -81.3 + 3.15 f_d = -81.3 + (3.15 \times -51.8) = -244.5 N/mm^2$.

Allowable $f_L = 154 N/mm^2$.

Applied $f_L <$ Allowable f_L.

4.11 **Shear Stresses**

Shear stress due to permanent or combined permanent + live $\not> 46 N/mm^2$

Shear due to permanent loads $= \frac{40.15 \times 7.532}{2} = 151.2 kN$.

Shear stress $= \frac{151.2 \times 10^3}{610 \times 38} = 6.5 N/mm^2$

Shear due to live loads $= \left(\frac{37.1 \times 7.532}{2}\right) + 51 = 190.7 kN$.

Shear stress $= \frac{190.7 \times 10^3}{610 \times 38} = 8.2 N/mm^2$

Shear stress due to dead + live $= 6.5 + 8.2 = 14.7 N/mm^2$

Applied shear stress < Allowable shear stress

i). $q_L \not> 24.6 - 0.44 q_d$

$= 24.6 - (0.44 \times 65) = 21.74 N/mm^2$.

∴ ok for 40te.

WS/Atkins

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ECC Assessment contract 3	AI 1877/61
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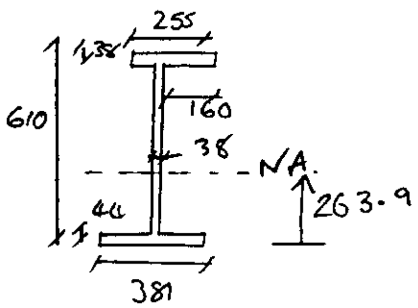
External Beam Edge Beam.



BD21/97 Table 4.1

Dead load and Superimposed load.

Unit weight of cast iron = 7200 kg/m³

Section Area = (381 × 44) + (38 × 528) + (255 × 38)

= 46518 mm²

Find NA

Take moments about bottom edge

= [(381 × 44 × 22) + (38 × 528 × 308) + (255 × 38 × 591)] / 46518

= 263·9 mm from bottom edge.

I of section

$\left(\frac{381 \times 44^3}{12}\right) + (381 \times 44 \times 241.9^2) = 983.66 \times 10^6$

$+ \left(\frac{38 \times 528^3}{12}\right) + (38 \times 528 \times 44.1^2) = 505.15 \times 10^6$

$+ \left(\frac{255 \times 38^3}{12}\right) + (255 \times 38 \times 327.1^2) = 1087.9 \times 10^6$

2.527×10^9

$Z_t = \frac{2.527 \times 10^9}{263.9} = 9.58 \times 10^6\ mm^3$

$Z_c = \frac{-2.527 \times 10^9}{610 - 263.9} = -7.30 \times 10^6\ mm^3$.

WS Atkins	Project: ECC Assessment Contract 3	Job ref: AI 1877/61
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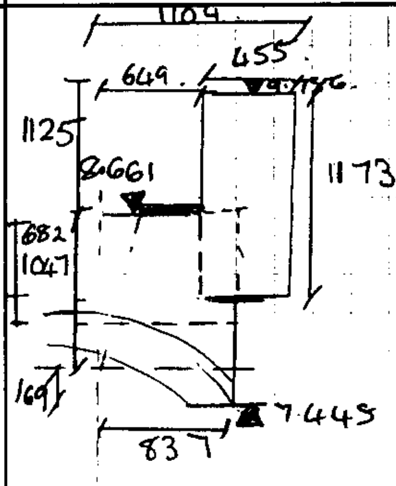
Ref	Calculations	Output



parapet = (.455 x 1.125) + (0.267 x 0.606) = 0.6737 m²

parapet SW = 0.6737 x 20.6 = 13.9 kN/m

Fill = (0.837 x 1.047) = 0.8763 m²
fill SW = 0.8763 x 20.6 = 18.1 kN/m

Surfacing (assume 50mm) = 0.649 x 50 x 10⁻³ x 23.5 x 1.5
= 1.141 kN/m

long. Dispersal = 300 + 682 = 982 mm

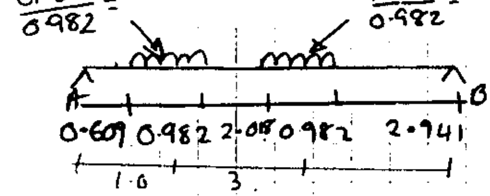
Total SW udl = 13.9 + 18.1 + 1.41 + (0.04518 x 70.6)
= 36.6 kN/m

BD21/97 Table D3 — For RE vehicle = 17 te.

$\frac{6.5}{2} \times 9.81 = 31.88$ kN

$\frac{10.5}{2} \times 9.81 \times 1.8 = 92.7$ kN.

$\frac{31.88}{0.982} = 32.5$ kN/m. $\frac{92.7}{0.982} = 94.4$ kN/m.



$R_A = 31.88 \times \frac{6.432}{7.532} + 92.7 \times \frac{3.432}{7.532} = 69.5$ kN.

$R_B = \frac{31.88 \times 1.10}{7.532} + 92.7 \times \frac{6.100}{7.532} = 55.01$ kN.

WS Atkins

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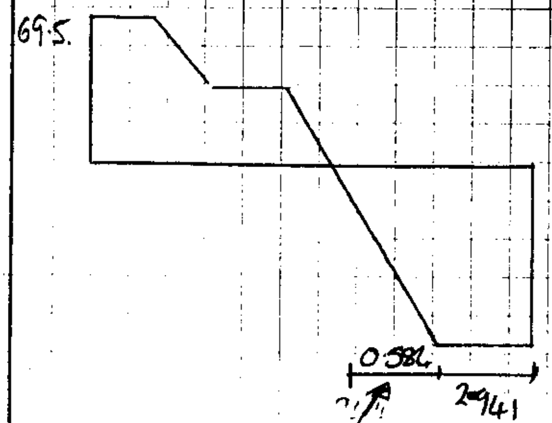
Part of structure				Calc sheet no / rev
Debden Rd Bridge				/ 13 /
Drawing ref	Calc by	Date	Check by	Date
	SB	Mar 00		

Ref	Calculations	Output



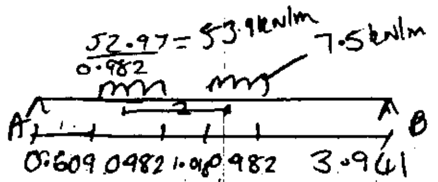
$\frac{55.1}{94.4} = 0.584$

max BM (area under SF diagram

$= (\frac{1}{2} \times 0.584 \times 55.1) + (2.941 \times 55.1)$

$= 178.1$ kNm.

For RF vehicle = 7te

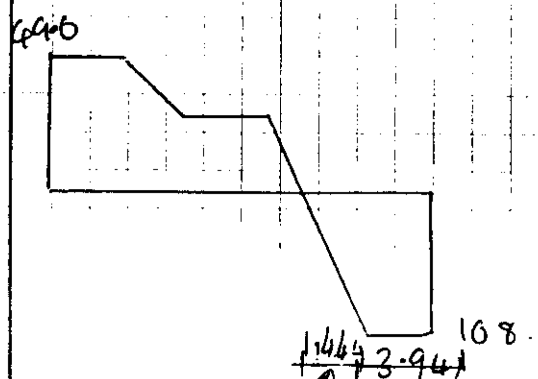


$\frac{6}{2} \times 1.8 \times 9.81 = 52.97$ kN.

$\frac{1.5}{2} \times 9.81 = 7.36$ kN.

$R_A = 52.97 \times \frac{6.432}{7.532} + 7.5 \times \frac{4.432}{7.532} = 49.6$ kN.

$R_B = 52.97 \times \frac{1.1}{7.532} + 7.5 \times \frac{3.1}{7.532} = 10.8$ kN.



$\frac{10.8}{7.5} = 1.44$

BM (area under SF dia)

$= (10.8 \times 3.94) + \frac{1}{2}(1.44 \times 10.8)$

$= 50.3$ kN

4.10 Cast Iron – Compressive + Tensile Stress

Stress due to perm. load

compressive $= - \frac{259 \times 10^6 \times (610 - 263.9)}{2.527 \times 10^9}$

$= -35.4\ N/mm^2$

tensile $= \frac{259 \times 10^6 \times 263.9}{2.527 \times 10^9} = 27\ N/mm^2$

Stress due to live load

compressive $= \frac{50.3 \times 10^6 \times (610 - 263.9)}{2.527 \times 10^9} = 6.9\ N/mm^2$

tensile $= \frac{50.3 \times 10^6 \times 263.9}{2.527 \times 10^9} = 5.3\ N/mm^2$

Stress due to perm + live

comp $= 35.4 + 6.9 = 42.3\ N/mm^2 < 154\ N/mm^2$

tensile $= 27 + 5.3 = 32.3\ N/mm^2 < 46\ N/mm^2$

for tensile values of f_L, the greater of the values given by

$f_L = 24.6 - 0.44 f_d = 24.6 - (0.44 \times 27) = \underline{12.72\ N/mm^2}$

or $f_L = 19.6 - 0.76 f_d = 19.6 - (0.76 \times 27) = -0.92\ N/mm^2$

Applied f_L < Allowable f_L ($12.72\ N/mm^2$)

for compressive stress values of f_L, the greater of the values given by

$f_L = -43.9 + (0.79 f_d) = -43.9 + (0.79 \times -35.4) = -71.9\ N/mm^2$

$f_L = -81.3 + (3.15 \times f_d) = -81.3 + (3.15 \times -35.4) = -192.8\ N/mm^2$

Applied f_L < Allowable f_L

∴ ok for 7.5te

Ref	Calculations	Output

For 17te vehicle - Compressive + Tensile Stress

Stresses due to perm loads

comp. = $-35.4\ N/mm^2$.

tensile = $27\ N/mm^2$.

Stress due to live loads.

Moment due to live load = 178.1 kNm.

Comp. stress = $\frac{178.1 \times 10^6 \times (610-263.9)}{2.527 \times 10^9} = 24.4\ N/mm^2$

tensile stress = $\frac{178.1 \times 10^6 \times 263.9}{2.527 \times 10^9} = 18.6\ N/mm^2$.

Stress due perm + live loads

comp = $-35.4 - 24.4 = 59.8\ N/mm^2$.

tensile = $27 + 18.6 = 45.6\ N/mm^2$

for tensile

$f_L \leq 12.7\ N/mm^2$

∴ Applied f_L (18.6) > Allowable f_L (12.7)

∴ Fails 17te.

WS Atkins	Project: ECC Assessment Contract 3	Job ref: K11877/61
	Part of structure: Debden Rd Bridge	Calc sheet no: 16, rev
	Drawing ref: — · Calc by: SB · Date: Mar 00	Check by: · Date:

Ref	Calculations	Output
4.11.	**Shear Stress**	

Shear stress due to perm = $\frac{137.8\times10^3}{610\times38} = 5.9\ N/mm^2$

Shear stress due to live load = $\frac{291.9\times10^3}{610\times38} = 12.5\ N/mm^2$

Shear stress due to live + perm = $5.9 + 12.5 = 18.4\ N/mm^2 < 46\ N/mm^2$

$q_L \leq 24.6 - 0.44 q_d = 24.6 - 0.44\times5.9 = 22.0\ N/mm^2$

Applied q_L < Allowable q_L

Passes work.

Ref	Calculations	Output

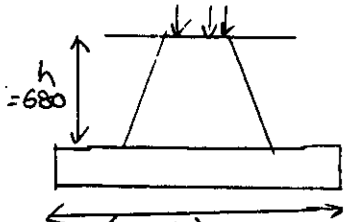
Masonry Arches - Archie.

Backing level - none
Masonry S.W = 21kN/m³
Fill S.W = 19kN/m³
Surfacing SW = 23kN/m³
ϕ for fill = 30°
Pressure coefficient = 0.3
Masonry strength = 2.3N/mm²

$F_c = 0.45$

As the condition factor is 0.45, for Jack Arch we are concerned with single wheel load. Therefore by doubling the factored lane width in Archie.

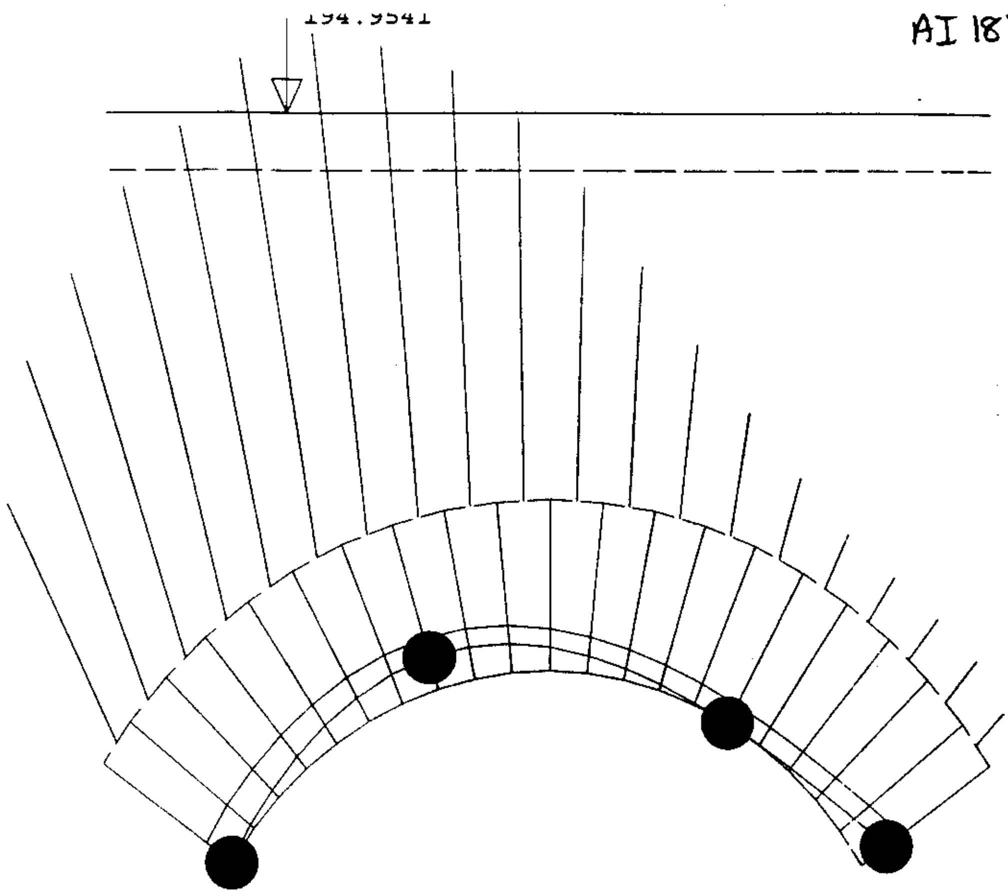
BD21/97



$(1.5+h) = 1.5+0.680 = 2.18m$

∴ Lane width input to Archie = 2.18 × 0.45 × 2
= 1.962m

Pg 29 The Jack Arch passes by inspection
11.5te single axle with FOS = 3.18. - Archie result.



Debden rd

()

Span	1223 mm	Rise	338 mm
Depth of fill	680 mm	Depth of surfacing	100 mm
Ring depth	303 mm	Ring depth factor	1
Position of backing	0	Depth of mortar loss	0 mm
Fill density	19 kN/m^3	Masonry density	21 kN/m^3
Surfacing density	23 kN/m^3		
Phi for fill	30 deg	Masonry strength	2.3 N/mm^2
Load	Single Axle:11.5t at 100		
Lane width	1962mm		
Required ring depth	95 mm	Geometric F.O.S	3.18
H Left	48 kN/m	H Right	68 kN/m
V Left	135 kN/m	V Right	55 kN/m
Comp. zone at hinge 2	36 mm	Factor on pass. press.	.3

Hinges

1 AT 1   2 AT 8   3 AT 16   4 AT 21

Param(mn).segment

	Stone Weight	Vertical Dead Load	Horizontal Deadload	Vertical Live Load	Horizontal Live Load	Additional Pass Press
1	-.5	-1.1	.8	-6.9	3.3	0
2	-.6	-1.4	.8	-10	3.8	0
3	-.6	-1.4	.7	-11.6	3.6	0
4	-.6	-1.4	.5	-13.2	3.3	0
5	-.6	-1.4	.4	-14.1	3	0
6	-.6	-1.4	.4	-15.3	2.6	0
7	-.6	-1.4	.3	-15.4	1.8	0
8	-.6	-1.4	.2	-14.5	1.3	0
9	-.6	-1.4	.1	-13.3	.7	0
10	-.6	-1.4	0	-11.5	.2	0
11	-.6	-1.4	0	-9.2	-.2	0
12	-.6	-1.4	-.1	-6.7	-.3	0
13	-.6	-1.4	-.2	-4.5	-.4	-.1
14	-.6	-1.4	-.3	-2.7	-.3	-.1
15	-.6	-1.4	-.4	-1.3	-.2	-.2
16	-.6	-1.4	-.4	-.5	-.1	-.4
17	-.6	-1.4	-.5	-.1	0	-.4
18	-.6	-1.4	-.7	0	0	-.4
19	-.6	-1.4	-.8	0	0	-.4
20	-.5	-1.1	-.8	0	0	-.2

WS Atkins	Project: ECC Assessment Contract 3		Job ref: AI1877/63	
	Part of structure: Debden Rd Bridge		Calc sheet no: /30/	rev
	Drawing ref	Calc by: SB	Date: Mar 00	Check by	Date

Ref	Calculations	Output

Tie Bars

Tie bar diameter = 17mm

fig 1. Tie bars are 3·7m apart.

Tie bars will assessed using archie and finding horizontal reaction and calculate thrust.

For 11·5te vehicle in archie

31 Horizontal reaction = 55kN/m.

Thrust per tiebar = 60 × 3·7 = 222 kN.

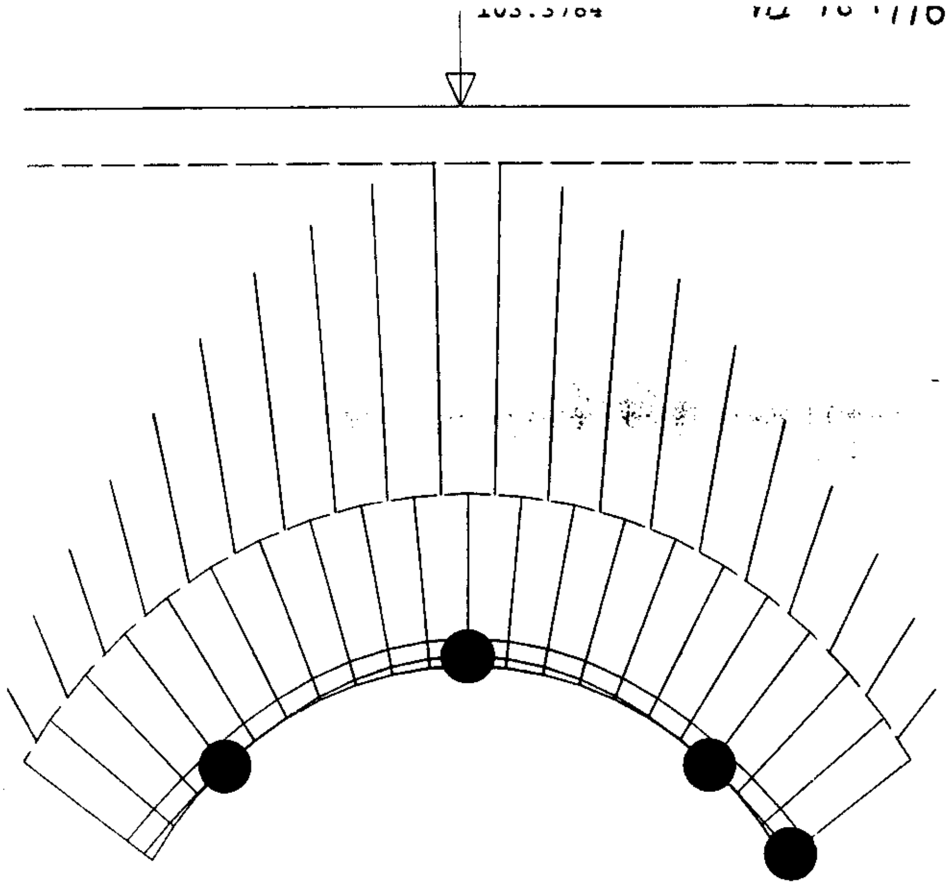
Capacity of tie bar.

$= \frac{17^2}{4} \pi \times 280 = 52 \cdot 2$ kN.

Tie bar fails 11·5te vehicle.

∴ capacity of tie bar

$= \frac{52 \cdot 2}{222} \times 11 \cdot 5 = \underline{2 \cdot 7 te}$.



Tiebar

()

Span	1223 mm	Rise	338 mm
Depth of fill	680 mm	Depth of surfacing	100 mm
Ring depth	303 mm	Ring depth factor	1
Position of backing	0	Depth of mortar loss	0 mm
Fill density	19 kN/m^3	Masonry density	21 kN/m^3
Surfacing density	23 kN/m^3		
Phi for fill	30 deg	Masonry strength	2.3 N/mm^2
Load	Single Axle:11.5t at 600		
Lane width	3700mm		
Required ring depth	60 mm	Geometric F.O.S	5.08
H Left	60 kN/m	H Right	58 kN/m
V Left	71 kN/m	V Right	69 kN/m
Comp. zone at hinge 2	40 mm	Factor on pass. press.	.3

Hinges

1 AT 4   2 AT 11   3 AT 18   4 AT 21

Param(mn).segment

	Stone Weight	Vertical Dead Load	Horizontal Deadload	Vertical Live Load	Horizontal Live Load	Additional Pass Press
1	-.5	-1.1	.8	-.7	.3	0
2	-.6	-1.4	.8	-1.4	.5	0
3	-.6	-1.4	.7	-2.1	.7	0
4	-.6	-1.4	.5	-3.1	.8	0
5	-.6	-1.4	.4	-4.3	.9	0
6	-.6	-1.4	.4	-5.8	1	0
7	-.6	-1.4	.3	-7.2	.9	0
8	-.6	-1.4	.2	-8.3	.7	0
9	-.6	-1.4	.1	-9.3	.5	0
10	-.6	-1.4	0	-9.8	.2	0
11	-.6	-1.4	0	-9.8	-.2	0
12	-.6	-1.4	-.1	-9.1	-.4	0
13	-.6	-1.4	-.2	-8	-.7	0
14	-.6	-1.4	-.3	-6.9	-.8	0
15	-.6	-1.4	-.4	-5.5	-.9	-.1
16	-.6	-1.4	-.4	-4	-.8	-.2
17	-.6	-1.4	-.5	-2.9	-.7	-.3
18	-.6	-1.4	-.7	-1.9	-.6	-.5
19	-.6	-1.4	-.8	-1.2	-.4	-.6
20	-.5	-1.1	-.8	-.6	-.3	-.3

APPENDIX C

APPROVAL IN PRINCIPLE

AND

INSPECTION FOR ASSESSMENT

APPENDIX C

APPROVAL IN PRINCIPLE

AND

INSPECTION FOR ASSESSMENT