HRE Assessment Programme

Highways England - Historical Railways Estate

MKT/461, Baddington Lane Bridge, Cheshire East

BE4 Assessment and Inspection Report

0450660| Form BA

October 2018

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Contents

Executive Summary

Key Facts

Structure Type: Single span propped overbridge

Superstructure Form: Six propped longitudinal cast iron girders with transversely spanning brindle brick jack arches.

Substructure Form: Brindle brick faced common brick abutments with padstones, brindle brick faced common brick wingwalls, and steel propping system supporting all cast iron girders.

Span: Clear skew span: 8.84m; Clear square span: 7.6m

Assessment Code: BE4

Live load capacity: Full C&U loading except for jack arches

Critical members: Internal girders in bending

Capacity factor: 1.76

Restriction: None – subject to repairs being carried out to the jack arches.

Condition: Poor

Local Authority: Cheshire East Council

OS Reference: SJ 646 505

This report presents the load carrying capacity for the bridge and identifies the data used to derive the assessment. It has been prepared by Jacobs for the exclusive use by HRE and should not be relied on by third parties. It has been based on site measurements and investigation by Jacobs or historical information provided by HRE, as appropriate.

The description of condition does not represent a principal inspection; nor should it be relied on for the development of maintenance works. Close inspection of members was limited by the constraints of safe access possible within a single site visit.

Structural soils provided scaffold towers to facilitate tactile inspection of all exposed members. The structural arrangement of the bridge meant that the following elements were not examined as part of the inspection for assessment:

- Jack arch barrel backfill –The trial pits excavated above the 1st internal girder from the north, at mid-span and support, have been logged and revealed concrete and rubble fill. This corroborates a historical drawing within a previous assessment report in Appendix E, and is assumed to be present across the full plan area of the bridge.
- Internal Girders - Due to the jack arch construction, only the underside of the bottom flanges of the internal girders were visible. The webs and top flanges of the internal girders were considered as built-in parts and not amenable for inspection. Unexposed surfaces were assumed to be competent owing to their protection by the deck construction.
- Edge Girders - The parapets supported by the edge girders restricted the inspection of the top flange. The internal face of the web and internal outstands of both flanges were considered as built-in parts and therefore not amenable to inspection. Unexposed surfaces were assumed to be competent owing to their protection by the deck construction or parapets.

1. General Description and Structural Details

Jacobs was appointed by Highways England Historical Railways Estate to undertake a BE4 assessment of overbridge MKT/461.

Structural Soils Ltd excavated two trial pits above the first internal girder from the north; one at the support and one at midspan, to determine top flange dimensions, girder heights, the type and depth of fill over the bridge.

1.1 Location and General Description

Structure MKT/461, Baddington Lane bridge, carries the A530 over the former track bed of Market Drayton - Wellington railway line approximately 7.8km to the south west of Crewe centre.

The bridge carries a bi-directional single lane carriageway, controlled by permanent traffic lights at both ends of the bridge. The carriageway width is restricted by concrete barriers. There is no verge to either side of the carriageway. The road is 5.0m wide at the centre of the span. The overall width between parapets is 6.2m. For carriageway dimensions refer to the plan at road level in Appendix G.

The road is busy with light vehicles with frequent HGV use was observed during the inspection.

The OS grid reference is SJ 646 505

The railway was completed in 1867 and the bridge was probably constructed around this time. The date of propping system installation is unknown.

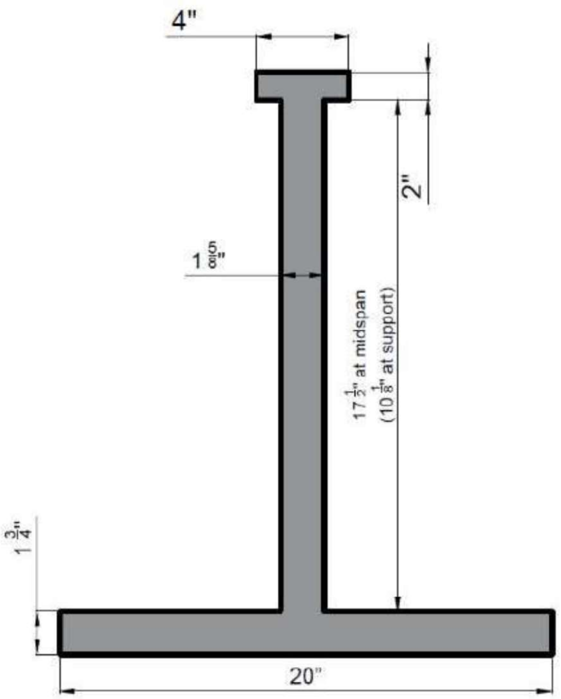
1.2 Construction type

The bridge is a single skew span overbridge with a clear skew span of 8.84m and a clear square span of 7.6m. The skew angle is measured to be 30°. Each main girder is propped to form a continuous three span structure.

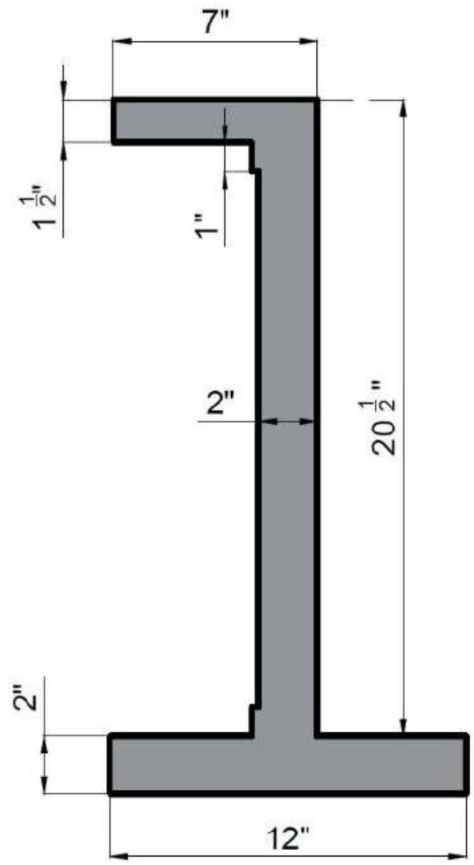
The superstructure comprises six longitudinal cast iron main girders at 1.34m (4' – 4⅝") centres. Common brick jack arches span transversely between the bottom flanges of the longitudinal main girders, with a measured rise at the crown of 260mm, and an assumed barrel thickness of 229mm. The original tie bars are severely corroded, and are considered to be ineffective. Corroded new tie bars of 25mm diameter remaining had been installed to the jack arches at the east end, apart from the centre jack arch. The jack arches are backfilled to the underside of inner girder top flange with concrete.

The internal girders comprise 4" x 2" top flange and 20" x 1 ¾" bottom flange. The web is 17½" x 1⅝" at midspan and 10⅛" x 1⅝" at support.

The edge girders comprise 7" x 1½" top flange and 12" x 2" bottom flange. The web is 20½" x 2" (web thickness is taken from drawings in the previous assessment included in Appendix E).



Section through internal main girder



Section through edge girder

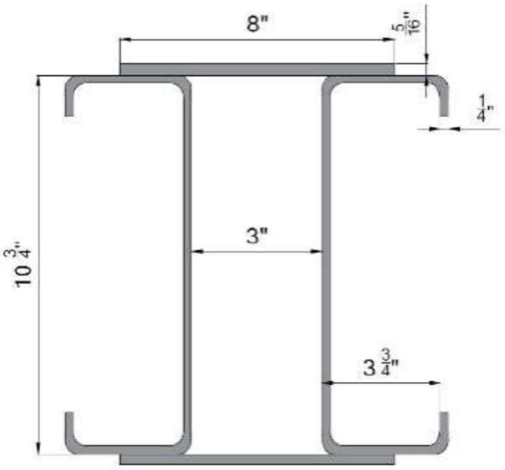
All longitudinal main girders bear directly onto padstones, supported on brindle brick faced common brick abutments, and steel frame propping to the west of the span.

The steel propping system comprises six 254x254x107 UC propping beams one below each cast iron girder. Each propping beam is supported by two columns, located at approximately 1.61m and 3.52m from the west abutment. The columns supporting the internal girders are 254x254x107 UC beams. Columns supporting the edge girders are compound stanchions comprising two 10¾" depth x 3¾" breadth x ¼" thick channels at 3" spacing between webs. The channels are welded together by 8" wide x 9" high x $^{5}/_{16}$" thick plates. There is a screw jack on top of each edge girder column.

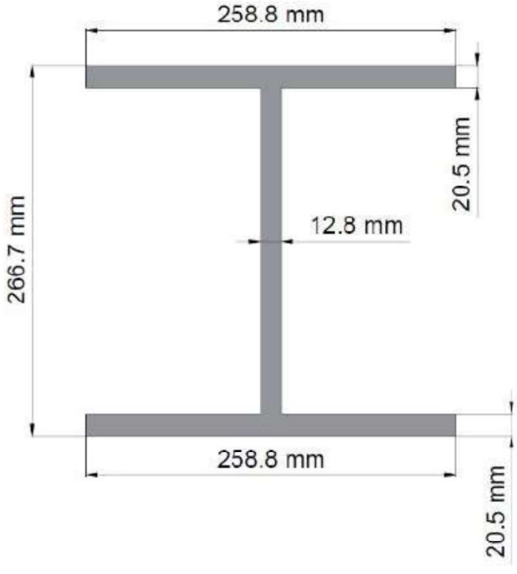
The edge girder columns and the adjacent internal girder columns are diagonally braced by 3" x 3" x $^{5}/_{16}$" angles. All of the columns have been cast into a concrete base.

The parapets and all four wingwalls are of common brick faced with brindle brick.

Sketches of the plan at road level and cross section are included in Appendix G.



Section through external propping column



Section through internal propping column

2. Information Search

2.1 Services search

A services search was carried out by Jacobs. Information is supplied in Appendix F.

2.2 Site Investigation Description Results

No samples were taken from the structure, trials pits and a description of the investigation is included in Appendix D.

2.3 Existing Drawings

Drawing from previous BD21 assessment report by Cheshire Engineering Consultancy, dated September 2000 (enclosed within BE4 assessment report by GIBB, dated May 2001) are included in Appendix E.

3. Structure Condition

3.1 General

The survey and inspection for BE4 assessment were undertaken on Wednesday 20th September 2017. Weather conditions were overcast with a temperature of 14°C.

Full road closure was in place during the site survey for the trial pit excavations.

Parking was available on a private driveway to the west of the bridge, with the permission of the house owner.

Access to the formation was gained via the farm track access located approximately 500m west from the bridge.

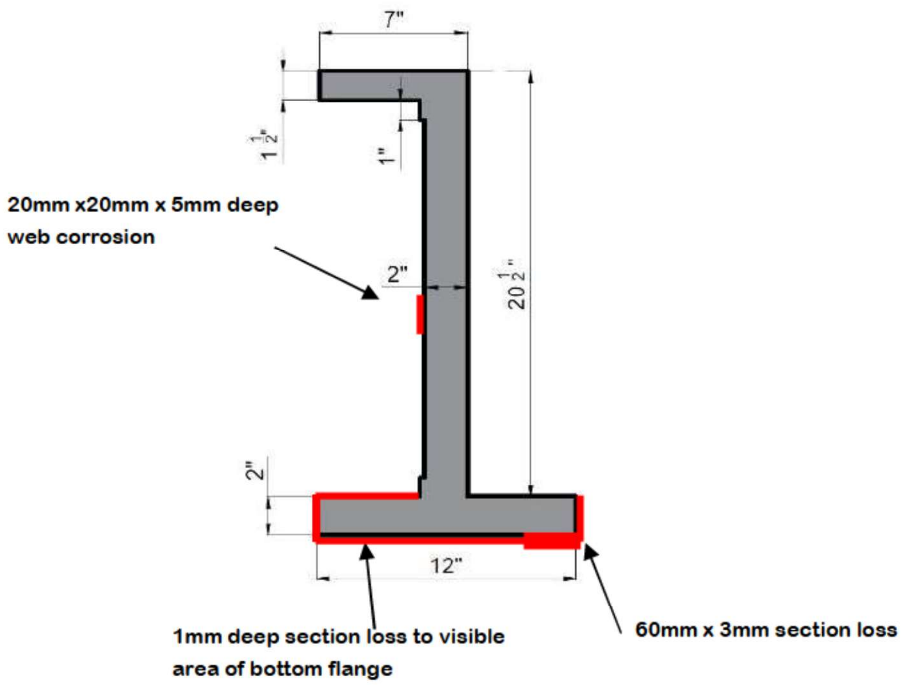
3.2 Structure Condition

3.2.1 Edge girders

Both edge girders are in fair condition.

The south edge girder exhibits delamination to visible areas of the bottom flange up to 1mm deep typically over the historic blast zone (Photograph 21). There is pitting up to 2mm deep to the outer edge of the top flange along the entire length (Photograph 23). The outer face of the girder web shows paint breakdown with surface corrosion (Photograph 22).

The north edge girder exhibits delamination to the bottom flange typically up to 1mm deep over the historic blast zones, with a 60mm wide x 3mm deep section loss adjacent to the new tie bar (Photograph 24). There are corrosion patches up to 20mm x 20mm x 5mm deep to the web between the 1st and 2nd stiffeners from the east abutment (Photograph 25)



Section loss to north edge girder

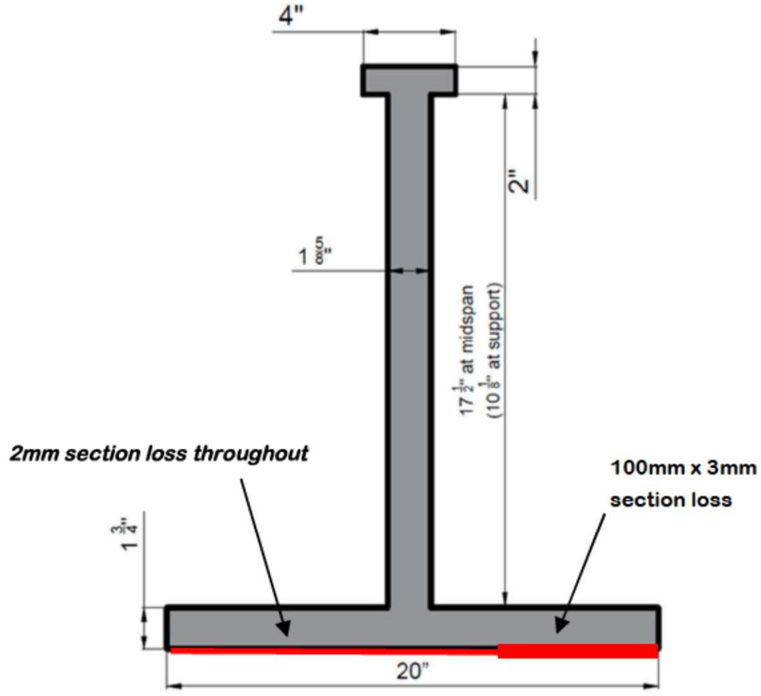
3.2.2 Internal girders

All internal girders are in fair condition.

First internal girder from the south shows delamination to bottom flange over the historic blast zone, typically full width, up to 2mm deep. The bottom flange adjacent to the tie bar also shows an area of corrosion up to 100mm wide x 3mm deep (Photograph 26).

All other internal girders exhibit a similar extent of delamination to the underside of the bottom flange up to 100mm wide x 1mm deep to each side of the flange (Photograph 27).

There are stalactites on all internal girders and also a substantial amount of calcite staining (Photograph 29).



Section loss to 1st internal girder from the south

3.2.3 Jack arches and tie bars

The jack arches and tie bars are in overall poor condition.

Jack arches all shows isolated dropped bricks with deep open joints. The worst case is located at the crown of the central jack arch with up to five bricks dropped by 30mm and open joints up to 100mm deep. (Photograph 28)

There are isolated spalled bricks to all jack arches up to 20mm deep, particularly around the original tie bars (Photograph 29).

Leachate staining and calcite deposit is exhibited on all jack arches, typically concentrated around the interface with the girders and towards the west end of the outer bays.

The original tie bars are typically severely corroded at the intersection with the jack arches, reduced to 10mm diameter (Photograph 30). New tie bars are also corroded; the diameter is typically reduced to 25mm (Photograph 31).

3.2.4 Abutments

Both abutments are in fair condition with minor spalling and isolated open joints up to 100mm deep. (Photograph 10 and Photograph 11)

The east abutment exhibits a full height vertical crack up to 10mm wide. (Photograph 32)

3.2.5 Wingwalls

The wingwalls are all in fair condition with moderate levels of vegetation growth. (Photograph 12 to 15) All wingwalls exhibits a full length diagonal crack of varying severity.

There is a small tree growing through the brickwork and stone coping of the north west wingwall newel, causing dislocation to the surrounding masonry. (Photograph 33)

There is a diagonal fracture through the full length of the south east wingwall typically up to 10mm wide with the east end of the wall separated by up to 50mm at a vertical crack. (Photograph 34)

The north east wingwall is displaced along the diagonal crack by 15mm.

3.2.6 Parapets

Both parapets are in fair condition.

There is a diagonal crack to the west end of the south parapet (Photograph 35)

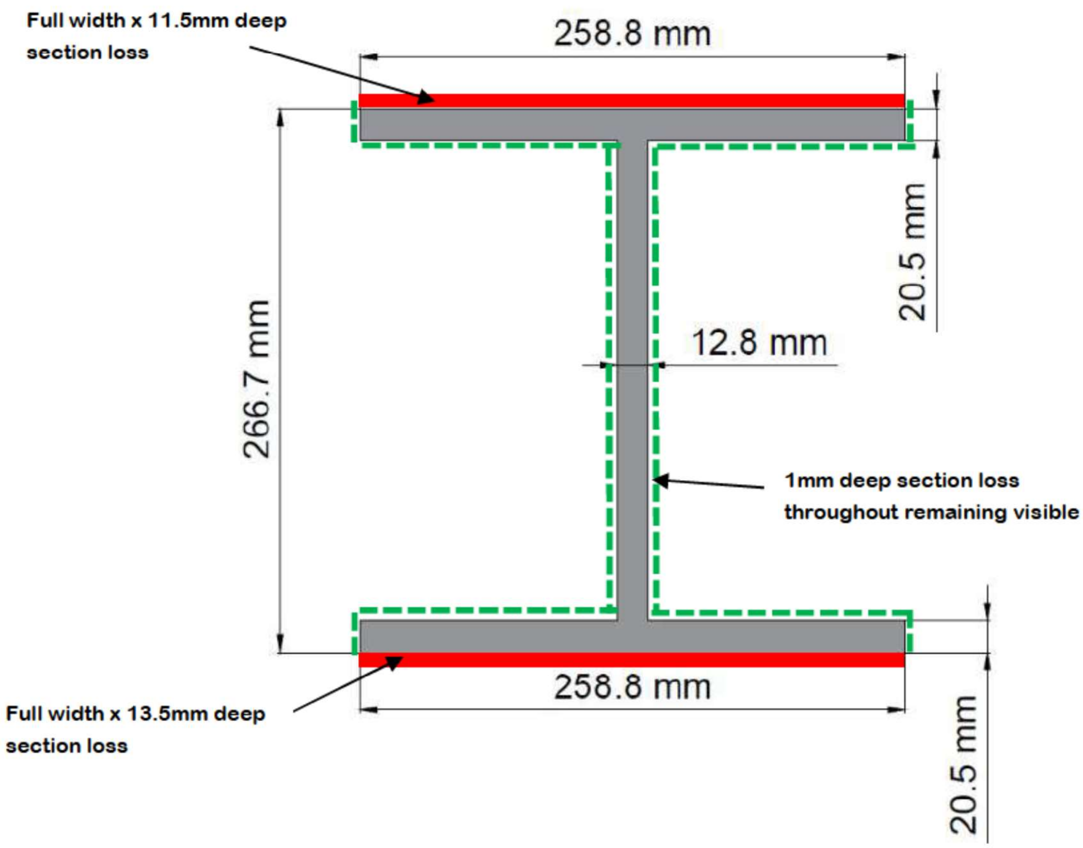
3.2.7 Propping system

The propping system is in overall poor condition.

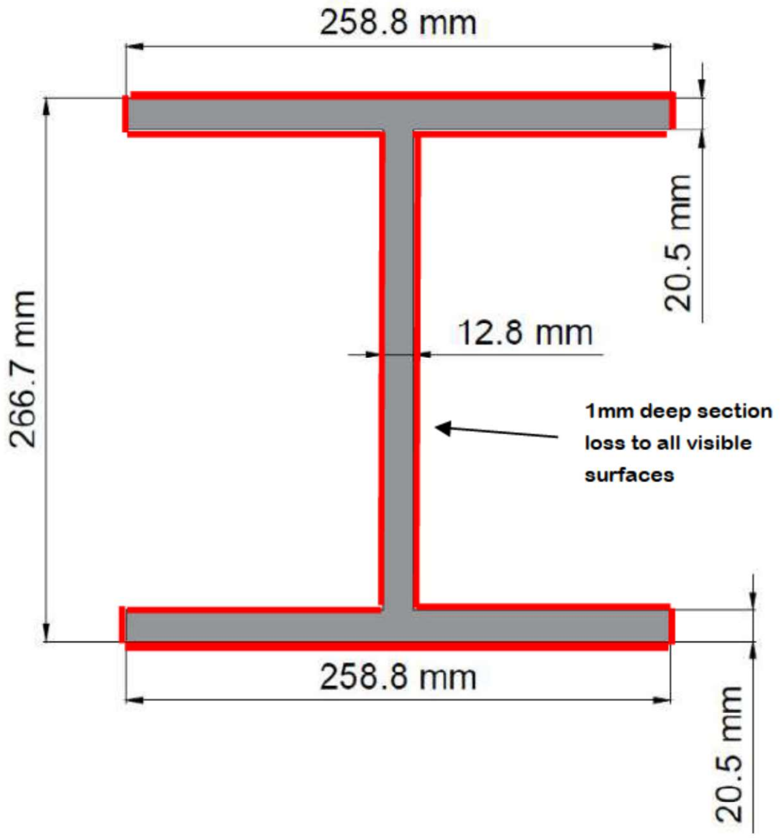
For the propping beams, the underside of the bottom flange is typically delaminated up to 4mm deep. The remaining visible areas exhibit 1mm deep corrosion throughout the entire length. (Photograph 36). The worst case is at the first south internal propping beam, with the top flange reduced to 9mm thick, bottom flange reduced to 7mm thick, and web reduced to a minimum of 3mm thick. (Photograph 37)

The internal columns are typically delaminated by up to 1mm deep along the entire length. (Photograph 38) The edge columns exhibit delamination up to 2mm deep along the entire length of the channels, with delamination to the welded plates up to 1mm deep to the full plate. (Photograph 39)

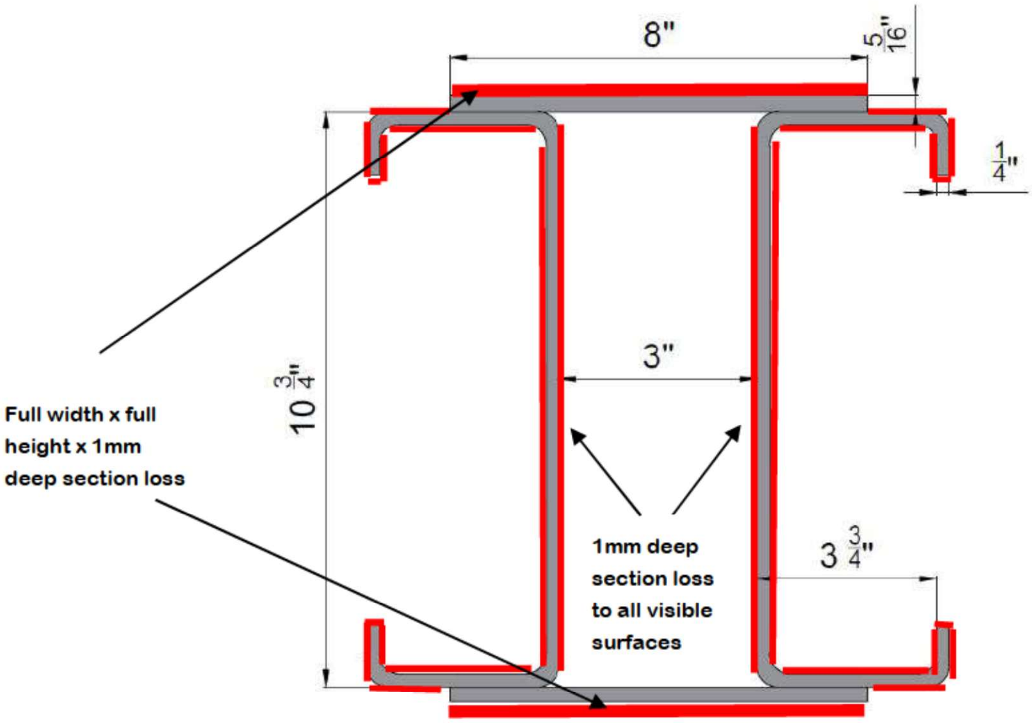
Bracing angles between the edge and first internal columns are typically corroded up to 2mm deep, with the worst case showing 5mm plate thickness remaining to the entire length of member. (Photograph 40) The bracing connection plates to the columns are also delaminated up to 2mm deep. (Photograph 41)



Section loss to propping beam



Section loss to internal propping column



Section loss to external propping column

3.2.8 Formation

The formation beneath the bridge is well kept and used regularly by the farms adjacent for access. (Photograph 18 and Photograph 19).

3.2.9 Road surface

The road surface is in overall poor condition, with a sizeable surface cracking behind the concrete barriers located to the south east of the bridge and crazing to the carriageway surfacing (Photograph 20).

4. Assessment to BE4

4.1 Methodology

Capacities were calculated using estimates of reduced section sizes where corrosion is present; therefore general condition factor will not be applied.

The cast iron main girders and propping beams were assessed using the BE4 Clause 202 Fig 1b vehicle train.

The propping columns were assessed using the maximum reaction forces derived from the applied BE4 loading on the main girders.

The capacities of the cast iron main girders were checked using the permissible stresses prescribed in BE4 Part I, 304 (c).

The capacities of the propping beams and propping columns were checked using the permissible stresses for rolled beams, channels, angles and tees prescribed in BS153 Part 3B Table 3.

Determination of the adequacy of the jack arches was based upon the empirical method described in Bridgeguard 3 Current Information Sheet No 22 (Pro-forma for the empirical assessment of brick, masonry and concrete jack arches and associated ties.).

The abutments were checked qualitatively.

4.2 Results

Element: Cast iron internal girders

Action	Location	Dead Load Effect	Live Load Effect	Total Effect	Assessed Resistance	Load Capacity
Bending	Worst location	41.4 ton.ft	11.4 ton.ft	52.8 ton.ft	61.5 ton.ft	Full C&U vehicle loading

Element: Cast iron edge girders

Action	Location	Dead Load Effect	Live Load Effect	Total Effect	Assessed Resistance	Load Capacity
Bending	Worst location	46.1 ton.ft	15.0 ton.ft	61.1 ton.ft	88.6 ton.ft	Full C&U vehicle loading

Element: Steel propping beams

Action	Location	Dead Load Effect	Live Load Effect	Total Effect	Assessed Resistance	Load Capacity
Bending	Midspan	0.1 ton.ft	4.7 ton.ft	4.8 ton.ft	24.8 ton.ft	Full C&U vehicle loading
Shear	Worst Case Shear	10.0 ton	3.9 ton	13.9 ton	26.0 ton	Full C&U vehicle loading
Buckling	Web	10.0 ton	3.9 ton	13.9 ton	26.5 ton	Full C&U vehicle loading

Element: Propping Columns

Action	Dead Load Effect	Live Load Effect	Total Effect	Assessed Resistance	Load Capacity
Axial Compression	0.2 ton	3.9 ton	4.1 ton	47.0 ton	Full C&U vehicle loading

Element: Jack Arches and Ties

The jack arches and tie bars are deemed inadequate for live loading in accordance with CIS No. 22 empirical assessment owing to the condition of the brickwork.

Element: Abutments

The abutments are considered to be adequate for full C&U vehicle loading to BE4.

5. Conclusions and Recommendations

The assessment demonstrates that the superstructure, the propping beams and columns are adequate for full BE4 loading with excess capacity.

The abutments are adequate for full BE4 loading by qualitative assessment with no significant defects present.

The jack arches are deemed inadequate for full BE4 loading in accordance with CIS 22 “Assessment of jack arches, metal arch plates and associated ties in metal beam bridge decks”, owing to defects in the brickwork. It is recommended that brickwork repair works are to be carried out including repairing the dropped bricks, raking out and repointing open mortar joints. If this action is taken, then the jack arches could be rated for full C&U loading.

It is also recommended that the condition of the propping system is to be monitored by means of routine inspections due to the extensive corrosion present to the columns.

Appendix A. Photographs

Photograph 1 - South elevation

Photograph 2 - North elevation

Photograph 3 - Approach from east of bridge

Photograph 4 - Looking west over bridge

Photograph 5 - Looking east over bridge

Photograph 6 - Approach from west of bridge

Photograph 7 - General view of soffit looking east

Photograph 8 - General view of soffit looking west

Photograph 9 - General view of propping system

Photograph 10 - General view of east abutment

Photograph 11 - General view of west abutment

Photograph 12 - General view of south east wingwall

Photograph 13 - General view of south west wingwall

Photograph 14 - General view of north west wingwall

Photograph 15 - General view of north east wingwall

Photograph 16 - General view of south parapet (carriageway face)

Photograph 17 - General view of north parapet (carriageway face)

Photograph 18 - Formation looking towards south

Photograph 19 - Formation looking towards north

Photograph 20 - Typical carriageway condition – shows heavy crazing

Photograph 21 - South edge girder exhibits delamination to bottom flange.

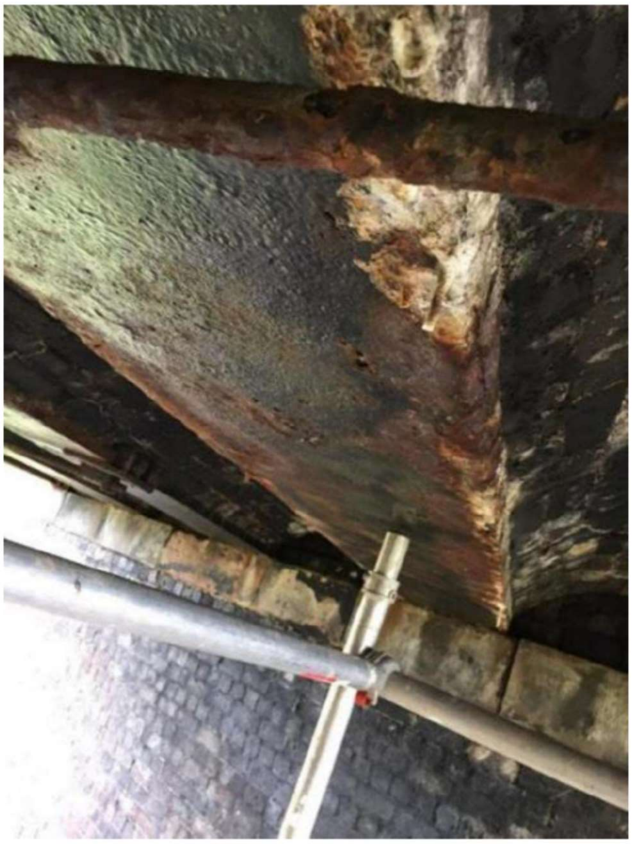
Photograph 22 - South edge girder outer face web print breakdown and surface corrosion

Photograph 23 - South edge girder top flange edge shows pitting along entire length up to 2m deep

Photograph 24 - North edge girder bottom flange delamination typically 1mm deep, up to 3mm deep over historic blast zones.

Photograph 25 - Corrosion patches to north edge girder web up to 5mm deep.

Photograph 26 - Corrosion to bottom flange of first internal girder from the south.



Photograph 27 - Typical 1mm deep corrosion to bottom flange of internal girders.

Photograph 28 - Five bricks dropped by up to 30mm with extensive mortar loss to crown of central jack arch.

Photograph 29 - Isolated spalled bricks to all jack arches up to 30mm deep.

Photograph 30 - Original tie bars severely corroded


Photograph 31 - New tie bars are corroded down to 25mm diameter


Photograph 32 - Full height vertical crack to east abutment up to 10mm wide.

Photograph 33 - Dislocation to masonry of north west wingwall newel due to small tree growth

Photograph 34 - Section of south east wingwall separated away for 50mm at a vertical crack


Photograph 35 - Diagonal crack to west end of south parapet outer face.


Photograph 36 - Delamination and corrosion to propping beams.

Photograph 37 - Worst propping beam beneath 1st internal girder from south web reduced to 5mm at top and 3mm at bottom

Photograph 38 - Internal columns are typically delaminated up to 1mm deep.

Photograph 39 - Edge north column prop channel (south east facing) - section loss of up to 4mm at base.

Photograph 40 - Corrosion to angle bracing with worst case showing 5mm thickness remaining

Photograph 41 - Delamination to bracing connection plates up to 2mm deep

Photograph 42 – Trial pit 1 – Midspan, 1st internal girder from north, exposing crown of 2nd internal jack arch from north

Photograph 43 – Trial pit 2, east support of 1st internal girder from the north

Appendix B. Form AA

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

Appendix: 4

ELR/ Bridge No MKT/461 Issue: 1

Revision: B (Nov 2000)

APPROVAL IN PRINCIPLE FOR ASSESSMENT

Bridge/Line Name: **Baddington Lane / Market Drayton – Wellington**

ELR/Bridge No. MKT/461

Brief Description of Existing Bridge:

(a) Span Arrangement

The bridge is a propped skew overbridge with a clear skew span of 8.84m and a clear square span of 7.6m. The skew angle is measured to be 30°. Each main girder is propped to form a continuous three span structure.

(b) Superstructure Type

The superstructure comprises six longitudinal cast iron main girders at 0.83m (2' – 8-¾") centres. Common brick jack arches span transversely between the bottom flanges of the longitudinal main girders, with a measured rise at the crown of 260mm, and an assumed barrel thickness of 229mm. The original tie bars are severely corroded, and are considered to be ineffective. Corroded new tie bars of 25mm diameter remaining had been installed to the jack arches at the east end, apart from the centre jack arch. The jack arches are backfilled to the underside of inner girder top flange with concrete.

The internal girders comprise 4" x 2" top flange and 20" x 1 ¾" bottom flange. The web is 17½" x 1⅝" at midspan and 10⅛" x 1⅝" at support.

The edge girders comprise 7" x 1½" top flange and 12" x 2" bottom flange. The web is 20½" x 2" (web thickness is taken from drawings in the previous assessment included in Appendix E).

(c) Substructure Type

All longitudinal main girders bear directly onto padstones, supported on common brick abutments, and steel frame propping to the west of the span.

The steel propping system comprises six 254x254x107 UC propping beams below each cast iron girder. Each propping beams are supported by two columns, located at approximately 1.61m and 3.52m from the west abutment. The columns supporting the internal girders are 254x254x107 UC beams. Columns supporting the edge girders are compound stanchions comprises two 10¾" depth x 3¾" breadth x ¼" thick channels at 3" spacing between webs. The channels are welded together by 8" wide x 9" high x $^5/_{16}$" thick plates. There is a screw jack on top of each edge girder column.

The edge girder columns and the adjacent internal girder columns are diagonally braced by 3" x 3" x $^5/_{16}$" angles. All of the columns have been cast into a concrete base.

The parapets and all four wingwalls are of brindle brick.

(d) Planned highway works/modifications at this site

None

(e) Road designation class and whether classed as a heavy load route

The bridge carries a bi-directional single lane carriageway, controlled by permanent traffic lights at both ends of the bridge. The carriageway width is restricted by concrete barriers. There is no verge to either side of the carriageway The road is 5.0m wide at the centre of the span. The overall width between parapets is 6.2m

(f) Any other requirements

None

Assessment Criteria

(a) Loadings and Speed

Section sizes used are obtained from site measurements (See Jacobs report "BE4 Assessment and Inspection repot – Bridge Ref: MKT/461" – December 2017). The bridge is to be assessed with vehicle loading obtained from and applied in accordance with BE4. Standard BE4 loading representative of 24 ton vehicles will be assessed.

(b) Codes to be used

BE4 - "The Assessment of Highway Bridges for Construction and Use Vehicles" Ministry of Transport, 1967 (with amendments to 1969)

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

(c) Proposed Method of Structural Analysis

Capacities of the girders will be calculated using the reduced section sizes of the girders to account for section losses which were identified during the inspection.

APPROVAL IN PRINCIPLE FOR ASSESSMENT

The original cast iron girders will be assessed as simply supported for the maximum dead load stresses; and as having four supports for the live load stresses, with maximum sagging near the east blast zone, and maximum hogging near the east propping column.

The cast iron longitudinal girders will be assessed in bending only accordance with BE4 Part 1-304/c.

The longitudinal cross head beams will be assessed for bending and shear, with the propping columns to be assessed in combined bending and axial compression. Traffic loading for internal girders will be distributed using the distribution curves in BE4 Part 2- Section 3, and distributed using simple static for the edge girders. Bending moments will be derived using BE4 graph no 5.

The D/d enhancement factor will not be considered according to BE4 Part 1-305 – (b, ii) given the presence of a water main and other services in the carriageway.

Determination of the adequacy of the jack arches will be based upon the empirical method described in Bridgeguard 3 Current Information Sheet No 22 (Pro-forma for the empirical assessment of brick, masonry and concrete jack arches and associated ties).

Only the critical component of the bridge such as the worst edge, internal girders and propping system components will be assessed.

Connections and bracing to the propping scheme will not be assessed.

The substructure will be assessed qualitatively.

APPROVAL IN PRINCIPLE FOR ASSESSMENT

Senior Civil Engineer's Comments

Definitions of 'common brick' and 'brindle brick' should be provided with the report so the quality of each type of brick is fully understood.

Proposed Category for Independent Check

Superstructure: 1

Substructure: 1

Name of Checker suggested if Cat 2 or 3: **n/a**

Category 1

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

Signed [redacted]
Title
Date

Category 2 and 3

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

Signed ....
Title ....
Date ....

Signed ....
Title ....
Date ....

Appendix C. Form BA

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

Appendix: 4
Issue: 1
Revision: A (Dec 2005)

ELR/ Bridge No MKT/461

CERTIFICATION FOR ASSESSMENT CHECK

Assessment Group: **Jacobs UK Ltd**

Bridge/Line Name: **Baddington Lane Bridge**

Category of Check: **1**

ELR/ Bridge No: **MKT/461**

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

(1) It has been assessed in accordance with the Approval in Principle as recorded on Form AA approved on 9th October 2018.

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

- BE4 - "The Assessment of Highway Bridges for Construction and Use Vehicles" Ministry of Transport, 1967 (with amendments to 1969)
- BS 153: Parts 3B & 4: 1958 "Steel Girder Bridges" British Standards Institution (with amendments to 12 Sept 1968).

List any departures from the above and additional methods or criteria adopted, with reference and justification for their acceptance.

None

Category 1

Name	Signature	Date	
[redacted]		11/10/2018	Assessor
		11/10/2018	Assessment Checker
		22.10.18	Authorised signatory of the firm of Consulting Engineers to whom Assessor/Checker is responsible.

[redacted]

Highways England Historical Railways Estate	Group Standard

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

Appendix: 4

ELR/ Bridge No MKT/461 Issue: 1

Revision: A (Dec 2005)

CERTIFICATION FOR ASSESSMENT CHECK

[redacted]

Notification of Assessment Check

Assessment Group Jacobs UK Ltd

[redacted]

Bridge Name/Road No. Baddington Lane Bridge

[redacted]

Line Name Market Drayton – Wellington railway line

ELR Code/Structure No. MKT/461

[redacted]

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

Cast iron edge girders	Full C&U vehicle loading
Cast iron internal girders	Full C&U vehicle loading
Steel propping beams	Full C&U vehicle loading
Steel propping columns	Full C&U vehicle loading
Jack arches	Dead load only – owning to defects in the brickwork
Abutments:	Full C&U vehicle loading

Recommended Loading Restrictions

None.

[redacted]

Description of Structural Deficiencies and Recommended Strengthening

The assessment demonstrates that the superstructure, the propping beams and columns are adequate for full BE4 loading with excess capacity. The abutments are deemed adequate for full BE4 loading by qualitative assessment with no significant defects present.

The jack arches are deemed inadequate for full BE4 loading in accordance with CIS 22 "Assessment of jack arches, metal arch plates and associated ties in metal beam bridge decks", owing to defects in the brickwork. It is recommended that brickwork repair works are to be carried out including repairing the dropped bricks, raking out and repointing open mortar joints. If this action is taken, then the jack arches could be rated for full C&U loading.

Name	Signature	Date	
[redacted]		11/10/2018	Assessor
		11/10/2018	Assessment Checker
		22.10.18	Authorised signatory of the firm of Consulting Engineers to whom Assessor/Checker is responsible.

This Certificate is accepted by. [redacted] ....

[redacted]

[redacted]

JACOBS

Appendix D. Site Investigation

HRE Bridges 2017

BRIDGE: MKT/461

Factual Report on Bridge Assessment

Project No: 764283

Client: Jacobs (UK) Ltd

APRIL 2018

STRUCTURAL SOILS LTD

STRUCTURAL SOILS LTD

DOCUMENT ISSUE RECORD

Project No.:	764283
Project Name:	HRE BRIDGES 2017
Document Title	Factual Report on Bridge Assessment
Client:	JACOBS (UK) Ltd
Engineer:	JACOBS (UK) Ltd
Status:	FINAL
Author	[redacted]*Sc (hons) FGS*
Technical Reviewer	[redacted] *BSc (hons)*
Approved by	[redacted]*Sc (hons) FGS*
Report Issue Date	20th April 2018

REVISION RECORD

Revision	Date	Description of revisions	Prepared by
0.0	20/04/2018	1st Submission	[redacted]

STRUCTURAL SOILS LIMITED
The Potteries
Pottery Street
CASTLEFORD
West Yorkshire
WF10 1NJ

Tel: 01977 552255
Email: ask@soils.co.uk
www.soils.co.uk

CONTENTS

1 INTRODUCTION

This investigation was carried out by Structural Soils Ltd (SSL) on the instructions of Jacobs (UK) Ltd (the Client). The work was carried out as part of a term contract to investigate a number of bridges around the United Kingdom

This report relates to bridge MKT/461 which is located in Nantwich, Chesire at British National Grid Reference SJ645504. (see Site Location Map in Appendix A). The bridge consists of cast iron girders and jack arches which is currently propped. This investigation was carried out to provide information for the structural assessment of the bridge.

The investigation has been carried out in accordance with the contract specification, and the general requirements of BS 5930:2015, BS 10175:2011+A1:2013, BS EN 1997-2 (2007), BS EN ISO 22475-1 (2006) and other relevant standards as identified below.

This report presents the factual records of the fieldwork carried out and laboratory testing undertaken. Whilst every attempt is made to record full details of the strata encountered in the exploratory holes, techniques of hole formation and sampling will inevitably lead to disturbance, mixing or loss of material in some soils and rocks. All information given in this report is based on the ground conditions encountered during the site work, and on the results of laboratory and field tests performed during the investigation. However, there may be conditions at the site that have not been taken into account, such as unpredictable soil strata, contaminant concentrations, and water conditions between or below exploratory holes.

This report was prepared by SSL for the sole and exclusive use of Jacobs (UK) Ltd in response to particular instructions. Any other parties using the information contained in this report do so at their own risk and any duty of care to those parties is excluded. No liability will be accepted after a period of 6 years from the date of the report.

STRUCTURAL SOILS LTD

2 FIELDWORK

2.1 General

The fieldwork was commenced and completed on 24th October 2017 and comprised the excavation of two hand dug trial pits (MKT/461 TP01 & TP02) at the locations shown on the Exploratory Hole Location Plan in Appendix A.

The trial pits were excavated in the road adjacent to the northern parapet wall. The pits were excavated to determine the thickness of the fill over the bridge deck and to take a level on the bridge deck. Levelling was carried out by Jacobs (UK) ltd. The trail pits was terminated at 0.21 m and 0.33 m depth respectively on the northern girder.

On completion the trial pits was backfilled with arisings and the road surface reinstated.

The investigation was supervised by an engineer from SSL. The scope of works and positions were selected and set out by Jacobs (UK) Ltd and adjusted where necessary to take account of buried or overhead services, or other restrictions. The exploratory hole and in-situ test locations are shown on the Exploratory Hole Location Plan presented in Appendix A.

The exploratory hole logs are presented in Appendix B. These provide information including the equipment and methods used, samples taken, tests carried out, water observations and descriptions of the strata encountered. Explanation of the terms and abbreviations used on the logs is given in the Key to Exploratory Hole Records in Appendix B, together with other explanatory information.

The holes were logged by an engineer in general accordance with the recommendations of BS 5930:2015 (which incorporates the requirements of BS EN ISO 14688-1, 14688-2 and 14689-1), together with relevant comments, are given on the logs.

3 REFERENCES

3.1 BS 5930:2015 *Code of practice for ground investigations*

3.2 BS EN 1997-1:2004 *Eurocode 7 — Geotechnical Design Part 1 - General Rules* incorporating corrigendum Feb 2009 and Amemdment A1 2013

3.3 BS EN 1997-2:2007 Eurocode 7 — *Geotechnical design Part 2: Ground Investigation and testing*

3.4 BS 10175:2011 *Investigation of potentially contaminated sites: Code of practice*, including amendment A1 2013

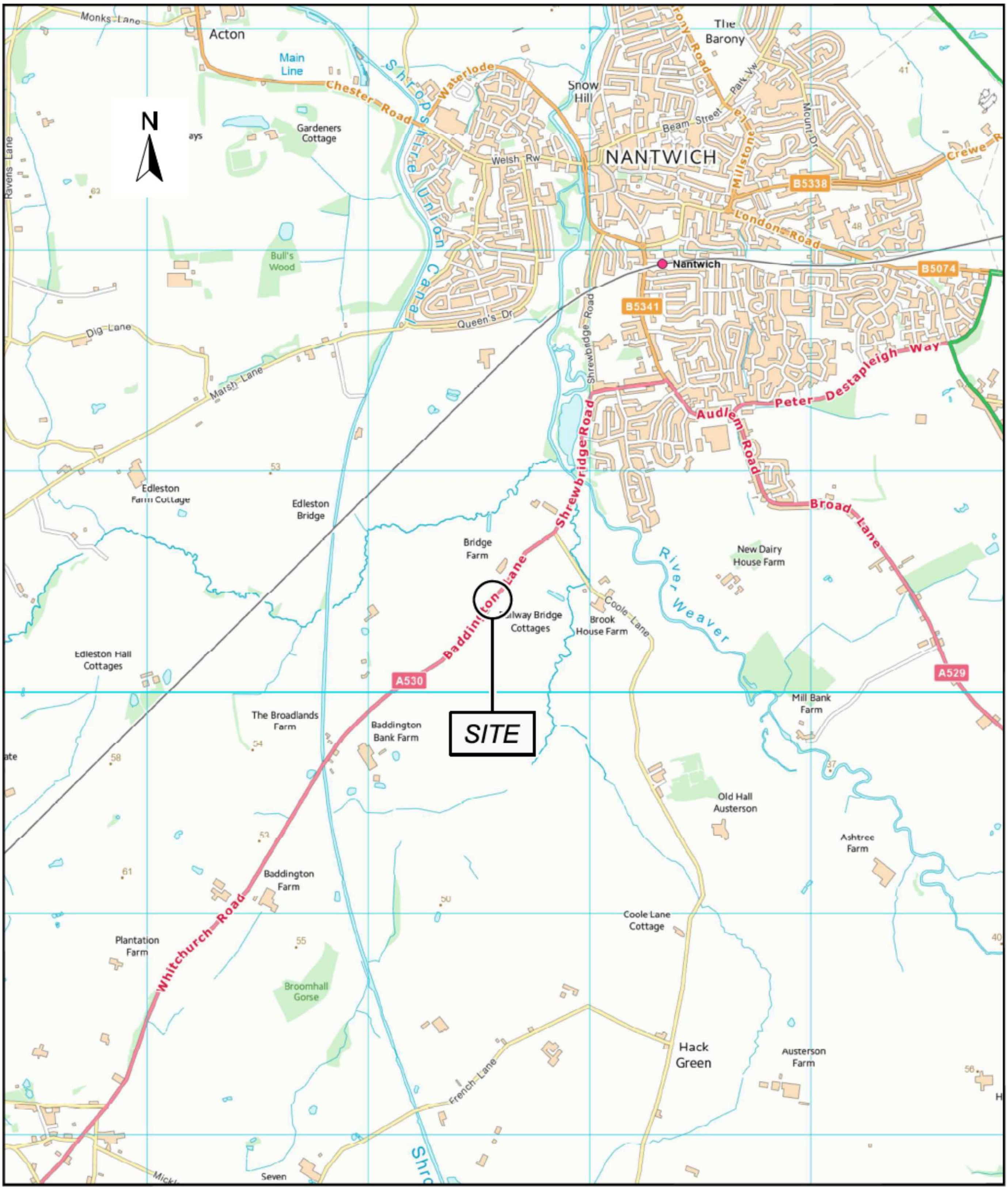
3.5 BS EN ISO 14688-1:2002 *Geotechnical investigation and testing – Identification and classification of soil: Part 1: Identification and description*, including Amendment A1 2013

3.6 BS EN ISO 14688-2:2004 *Geotechnical investigation and testing – Identification and classification of soil: Part 2: Principles for a classification*, including Amendment A1 2013

APPENDIX A -
PLANS AND DRAWINGS

(i) Site Location Plan

(ii) Exploratory Hole Location Plan



STRUCTURAL SOILS
The Potteries
Pottery Street
Castleford
WF10 1NJ
Tel: 01977 552255
ask@soils.co.uk
www.soils.co.uk

REV.	DATE	DESCRIPTION	BY	[redacted]	APR.
00	18.04.2018	-	[redacted]	[redacted]	-

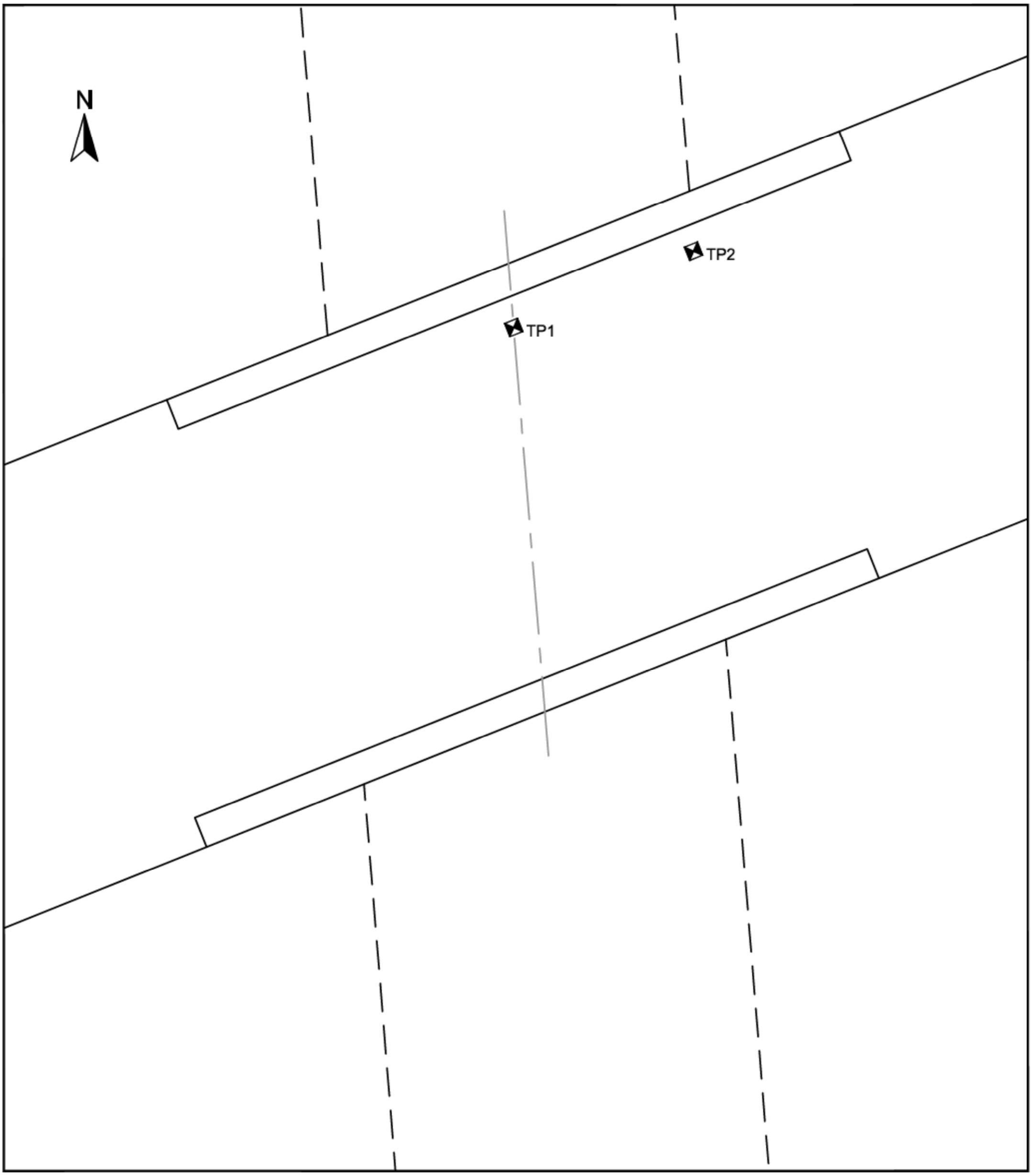
DIMENSION	SCALE	DRAWING STATUS
m	1:25,000	-

CLIENT
Jacobs (UK) Ltd

PROJECT
HRE Bridges - MKT/461

TITLE
SITE LOCATION MAP

JOB NO	GRID REF	SCALE BAR	ORIGIN SIZE	FIGURE
764619	SJ 645 504	0 250 500 750 1,000m	A4	1



LEGEND — Trial Pit Location

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The Potteries
Pottery Street
Castleford
WF10 1NJ

Tel: 01977 552255
ask@soils.co.uk
www.soils.co.uk

REV	DATE	DESCRIPTION	BY	APR
00	18.04.2018	-	[redacted]	-

DIMENSION	SCALE	DRAWING STATUS
m	NTS	-

CLIENT: Jacobs (UK) Ltd

PROJECT: HRE Bridges - MKT/461

TITLE: EXPLORATORY HOLE LOCATION PLAN

JOB NO	SCALE BAR	ORIGIN SIZE	FIGURE
764619		A4	2

APPENDIX B - EXPLORATORY HOLE RECORDS

(i) Key to Exploratory Hole Logs

(ii) Trial Pit Logs

KEY TO EXPLORATORY HOLE LOGS - SUMMARY OF ABBREVIATIONS

ADDITIONAL NOTES

1. All soil and rock descriptions and legends in general accordance with BS EN ISO 14688-1, 14688-2, 14689-1, and BS5930:2015.
2. Material types divided by a broken line (- - -) indicates an unclear boundary.
3. The data on any sheet within the report showing the AGS icon is available in the AGS format.

Structural Soils Ltd, Branch Office - Castleford: The Potteries, Pottery Street, Castleford, West Yorkshire, WF10 1NJ. Tel: 01977-552255, Fax: 01977-552299, Web: www.soils.co.uk, Email:north@soils.co.uk.

KEY TO EXPLORATORY HOLE LOGS - SUMMARY OF GRAPHIC SYMBOLS

MATERIAL GRAPHIC LEGENDS

MADE GROUND

INSTRUMENTATION SYMBOLS

Backfill

Structural Soils Ltd, Branch Office - Castleford: The Potteries, Pottery Street, Castleford, West Yorkshire, WF10 1NJ. Tel: 01977-552255, Fax: 01977-552299, Web: www.soils.co.uk, Email:north@soils.co.uk.

STRUCTURAL SOILS

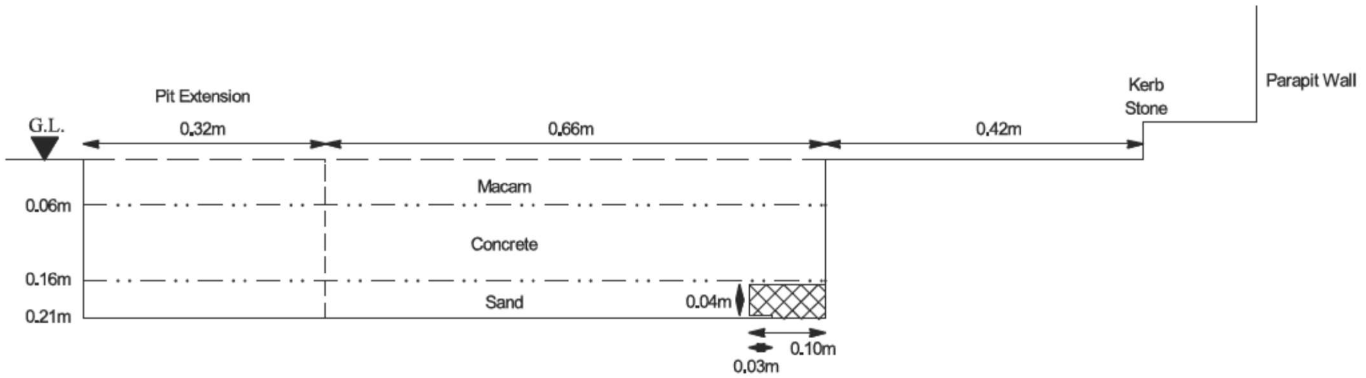
TRIAL PIT LOG

Contract: Jacobs Bridges MKT/461		Client: Jacons UK Ltd		Trial Pit: MKT/461 TP01
Contract Ref: 764619	Start: 15.04.18 End: 15.04.18	Ground Level (m): ---	Co-ordinates: ---	Sheet: 1 of 1

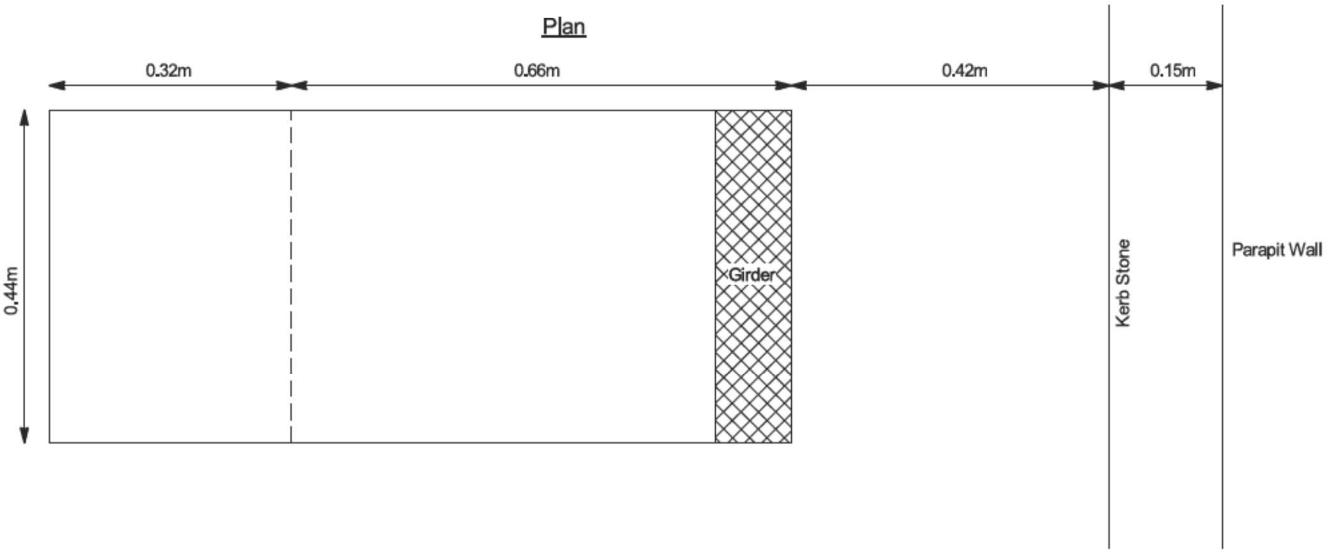
Samples and In-situ Tests: Depth	No	Type	Results	Water	Backfill	Description of Strata	Depth (Thickness)	Material Graphic Legend
						MADE GROUND: MACADAM	0.06	
						MADE GROUND: CONCRETE	0.16	
						MADE GROUND: Orangish brown gravelly SAND. Gravel is angular to subrounded fine to coarse of sandstone and concrete. . . . at 0.17m, cast iron girder	0.21	

Trial pit terminated at 0.21m depth.

Cross Section



Plan



General Remarks

1. Expoloratory hole CAT scanned prior to excavation.
2. No groundwater encountered during excavation
3. Trial pit terminated at 0.21m depth on girder.
4. On completion, borehole backfilled with arisings.
5. Surface reinstated with bituminous macadam upon completion.

All dimensions in metres	Scale: 1:10		
Method Used: Hand dug	Plant Used: Hand tools	Logged By: [redacted]	Checked By:

AGS

STRUCTURAL SOILS

TRIAL PIT LOG

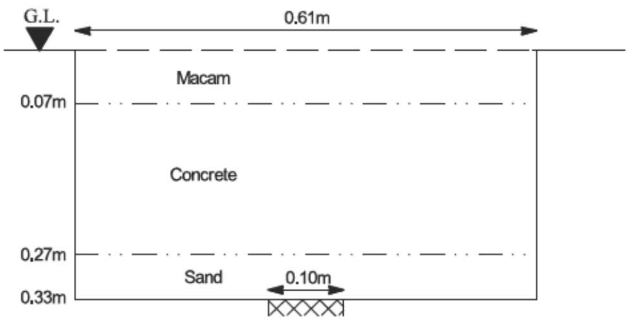
Contract:	Client:	Trial Pit:
Jacobs Bridges MKT/461	**Jacons UK Ltd**	**MKT/461 TP02**

Contract Ref:	Start / End	Ground Level (m):	Co-ordinates:	Sheet:
764619	Start: **15.04.18** End: **15.04.18**	---	---	**1** of **1**

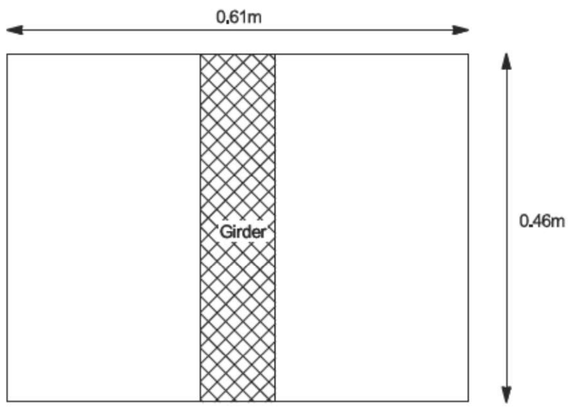
Samples and In-situ Tests: Depth	No	Type	Results	Water	Backfill	Description of Strata	Depth (Thickness)	Material Graphic Legend
						MADE GROUND: MACADAM	0.07	
						MADE GROUND: CONCRETE	(0.20) 0.27	
						MADE GROUND: Orangish brown gravelly SAND. Gravel is angular to subrounded fine to coarse of sandstone, concrete and brick.	0.33	

. . . at 0.33m, cast iron girder
Trial pit terminated at 0.33m depth.

Cross Section



Plan



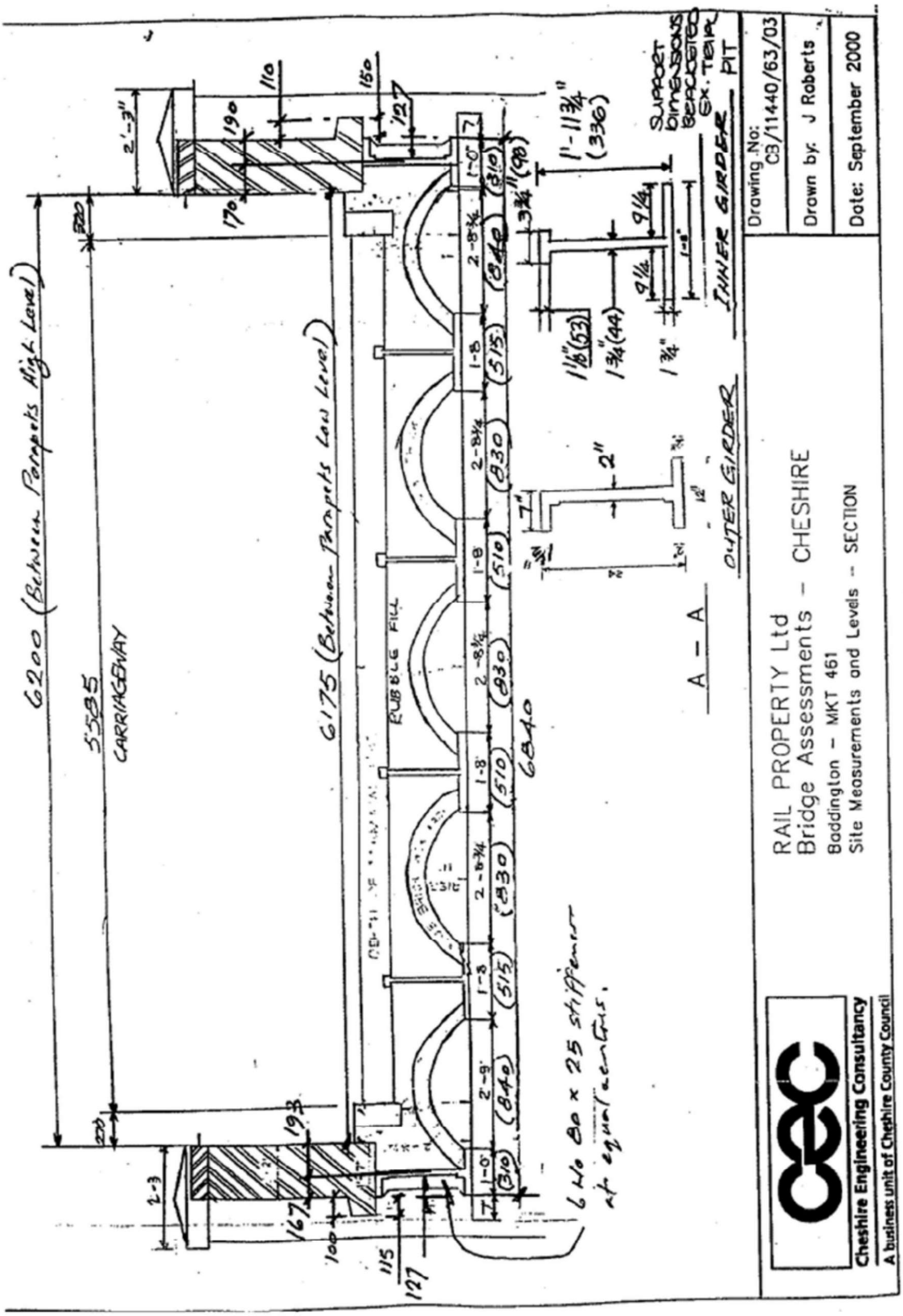
General Remarks

1. Expoloratory hole CAT scanned upon completion.
2. No groundwater encountered during excavation
3. Trial pit terminated at 0.33m depth on girder.
4. On completion, borehole backfilled with arisings.
5. Surface reinstated with bituminous macadam upon completion.

All dimensions in metres	Scale: 1:10		
Method Used: **Hand dug**	Plant Used: **Hand tools**	Logged By: [redacted]	Checked By:

AGS

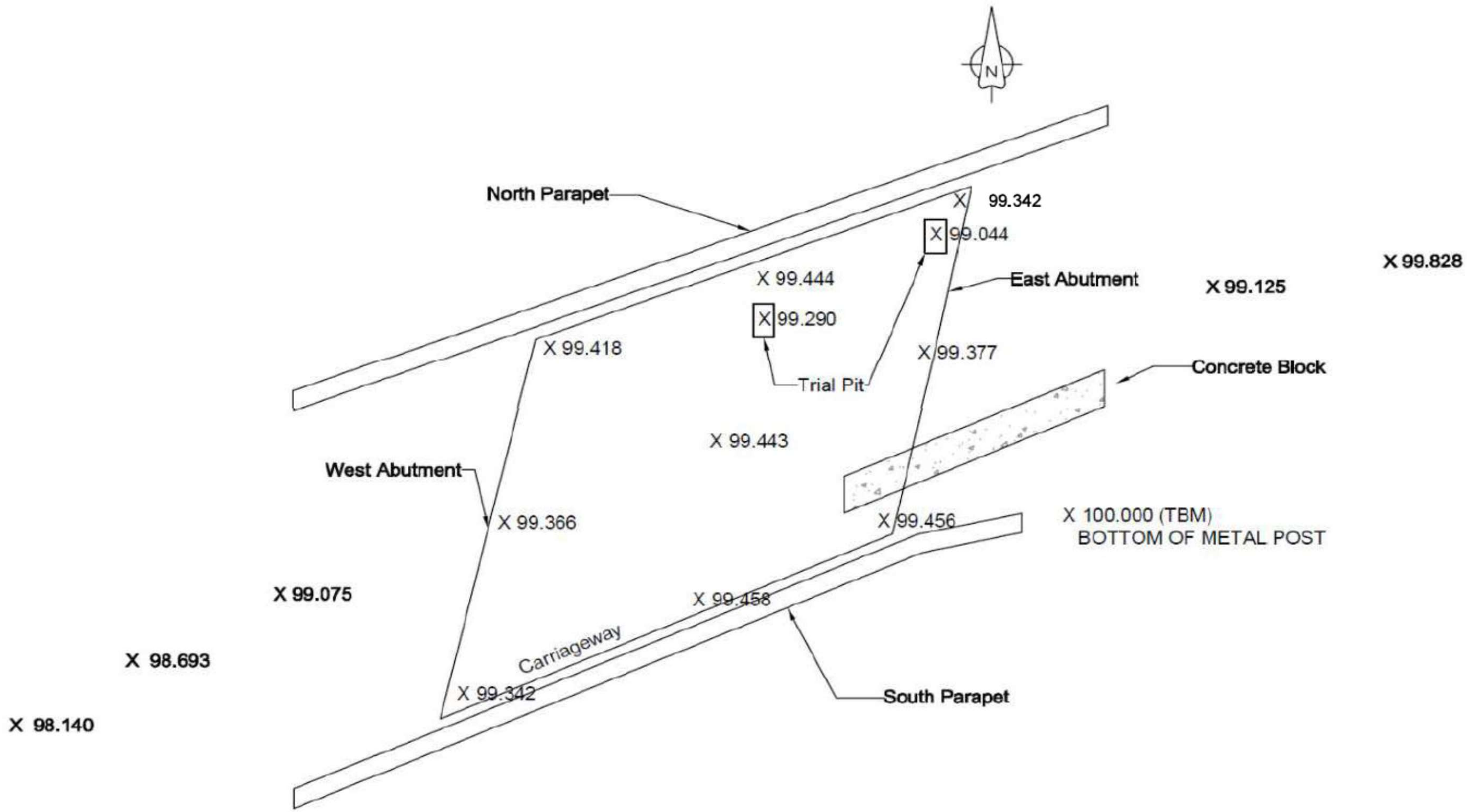
Appendix E. Historical Information

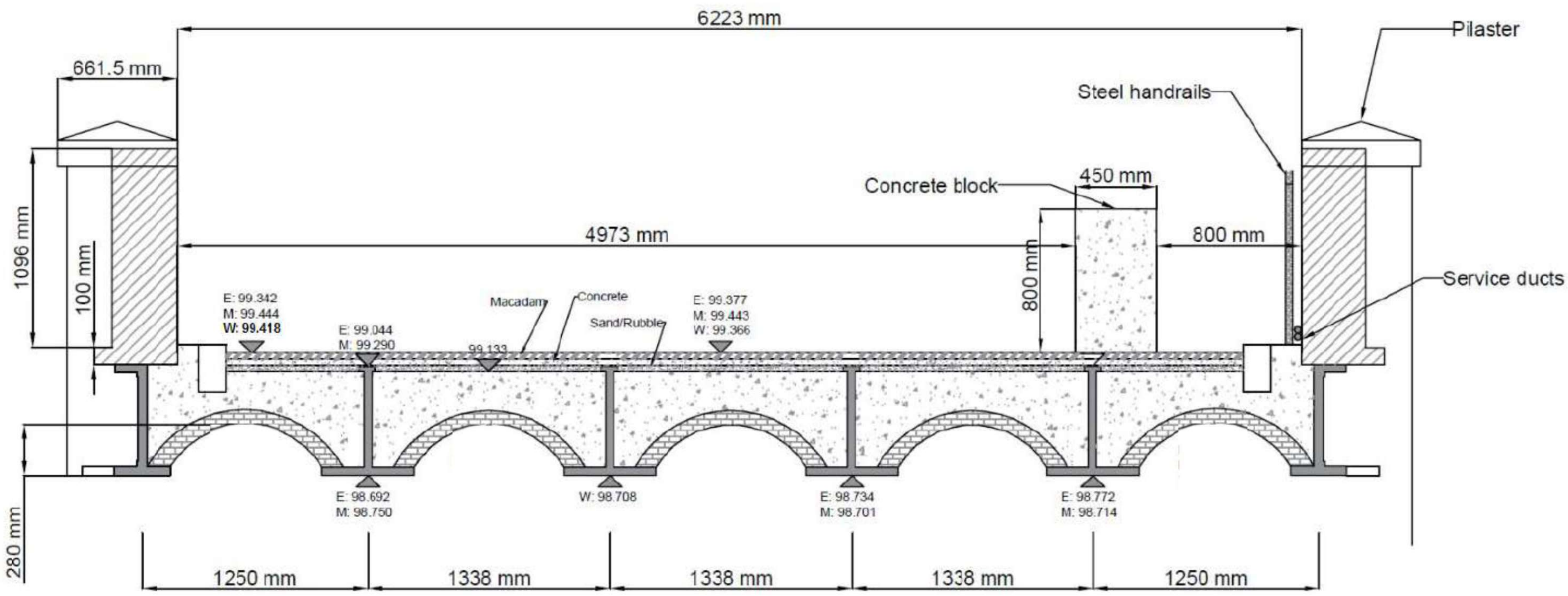


Plan sketch extracted from previous BD21 assessment report by CEC, dated September 2000

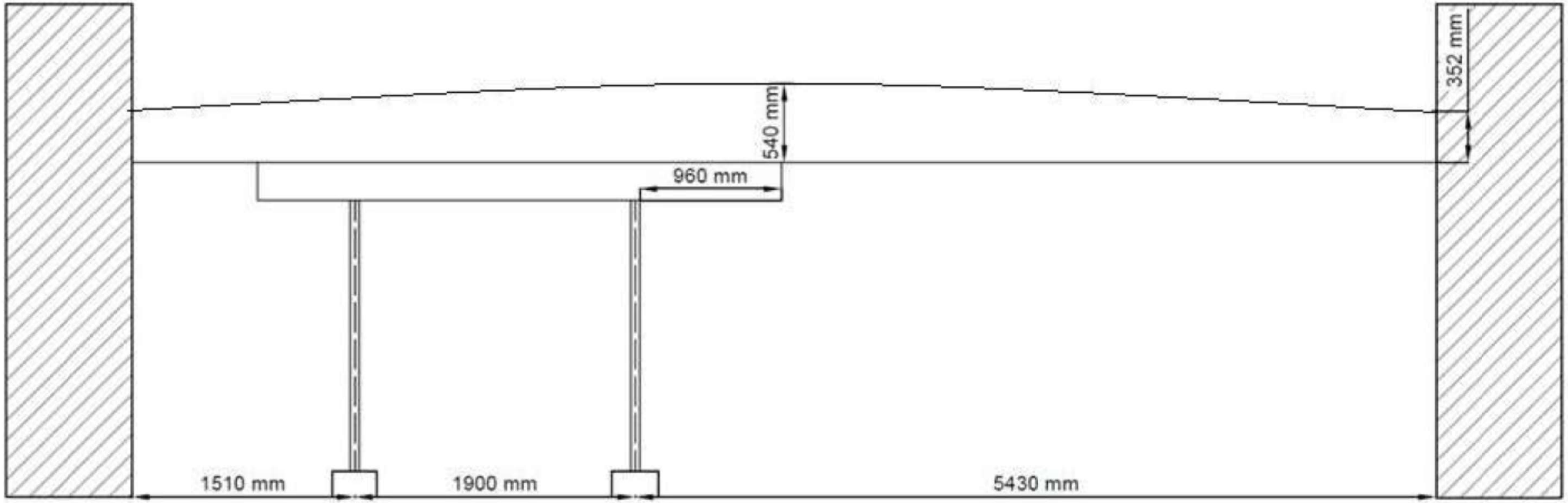
Appendix F. Services Search

Appendix G. Survey Sketches





Section looking east



Long section through the bridge

Appendix H. Calculations

CALCULATION COVER SHEET

Jacobs
Manchester

Project Title: HRE Assessment Programme		Calc. No.: 0450660	
Job No: B28280BT		File: VAR9/5266	
Project Manager	[redacted]	Subject:	**MKT/461**
Assessor			Baddington Lane Bridge
Project Group	31200		BE4 Assessment

	Total Sheets	Made by	Date	Checked by	Date	Reviewed by	Date		
Original	40	[redacted]	Aug-18	[redacted]	Sep-18	[redacted]	Sep-18		
Rev									
Rev									
Rev									
Rev									
Rev									

Superseded by Calculation No. Date

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

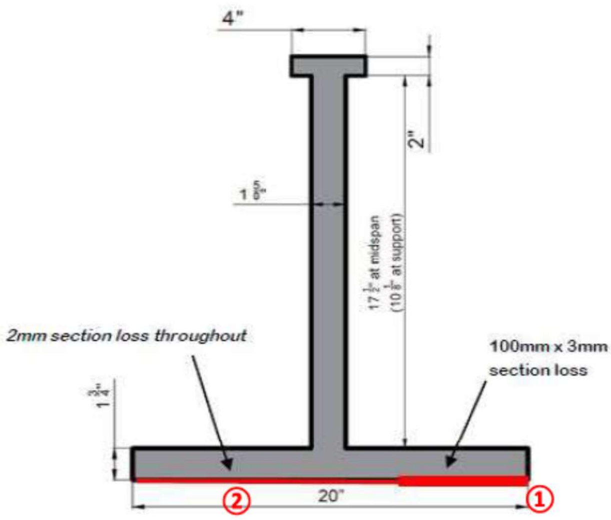
Reference	Calculation	Output

Contents

Page

Reference	Calculation	Output

Internal Main Girder: Midspan



Section loss to 1st internal girder from the south

Corroded section properties about x-x axis:

No.	Section	b (in)	d (in)	A (in^2)	y (in)	A.y (in^3)	A(y-y$_t$)^2 (in^4)
1	Top Flange	4	2	8	1	8	1394.82232
2	Web	1.625	17.5	28.438	10.75	305.7	339.316043
3	Bot. Flange	20	1.75	35	20.375	713.13	1332.72609
1	LOS	3.937	-0.1181	-0.465	21.191	-9.8538	-22.6983383
2	LOS	16.063	-0.0787	-1.2648	21.211	-26.827	-62.0879872
			Total =	**69.708**	in^2		

D	=	Depth of full section	=	21.25	in
d_w	=	Total depth of web panel	=	17.50	in
y_t	=	Distance to neutral axis from top of section	=	14.20	in
y_b	=	Distance to neutral axis from bottom of section	=	7.05	in
A_g	=	Total gross section area	=	70.1727	in^2

No.	Section	I_x (in^4)
1	Top Flange	2.66666667
2	Web	725.748698
3	Bot. Flange	8.93229167
1	LOS	-0.00054056
2	LOS	-0.00065348

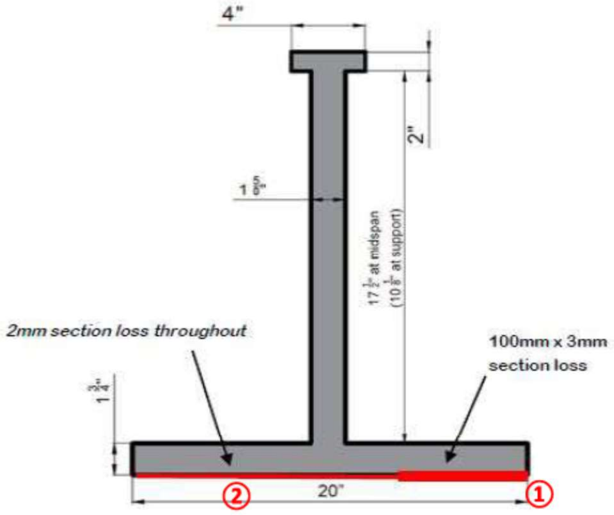
I_{xx}	=	2nd moment of area of beam section	=	3719.42	in^4
Z_{xc}	=	Elastic section modulus (compression flange)	=	261.853	in^3
Z_{xt}	=	Elastic section modulus (tension flange)	=	527.898	in^3

JACOBS CALCULATION SHEET

Office	Manchester	Page No.	B2	of	B12
Job No. & Title	B28280BT HE HRE Assessment Programme	Originator	[redacted]	Date	3/7/18
Section	MKT-461 Bridge Assessment - Section Properties (corroded)	Checker		Date	11/7/18

Reference	Calculation	Output

Internal Main Girder: Support



Section loss to 1st internal girder from the south

Corroded section properties about x-x axis:

No.	Section	b (in)	d (in)	A (in^2)	y (in)	A.y (in^3)	$A(y-y_t)^2$ (in^4)
1	Top Flange	4	2	8	1	8	594.382817
2	Web	1.625	10.125	16.453	7.0625	116.2	107.584741
3	Bot. Flange	20	1.75	35	13	455	399.943869
1	LOS	3.937	-0.1181	-0.465	13.816	-6.4244	-8.18825543
2	LOS	16.063	-0.0787	-1.2648	13.836	-17.499	-22.4815426
			Total =	57.723	in^2		

D = Depth of full section = 13.88 in

y_t = Distance to neutral axis from top of section = 9.62 in

y_b = Distance to neutral axis from bottom of section = 4.26 in

A_g = Total gross section area = 58.1883 in^2

No.	Section	I_x (in^4)
1	Top Flange	2.66666667
2	Web	140.558533
3	Bot. Flange	8.93229167
1	LOS	-0.00054056
2	LOS	-0.00065348

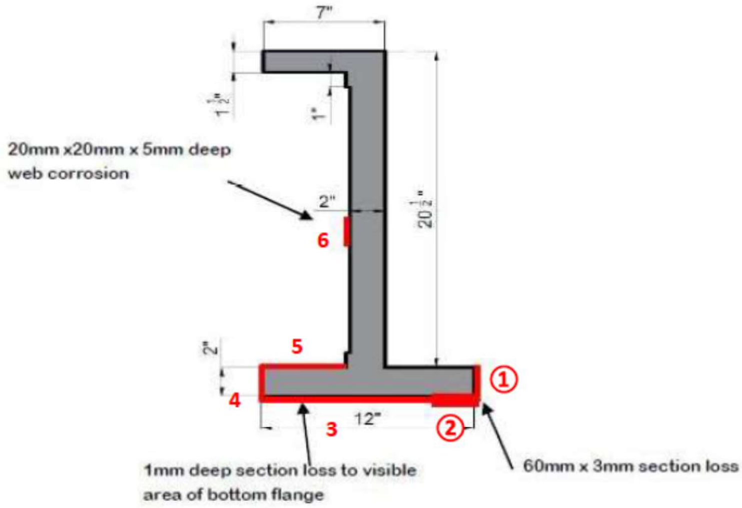
I_{xx} = 2nd moment of area of beam section = 1223.4 in^4

Z_{xc} = Elastic section modulus (compression flange) = 127.177 in^3

Z_{xt} = Elastic section modulus (tension flange) = 287.494 in^3

Reference	Calculation	Output

Edge Girder:



Section loss to north edge girder

Corroded section properties about x-x axis:

No.	Section	b (in)	d (in)	A (in^2)	y (in)	A.y (in^3)	A(y-y$_t$)^2 (in^4)
1	Top Flange	7	1.5	10.5	0.75	7.875	1542.31766
2	Web	2	19	38	11	418	132.841139
3	Bot. Flange	12	2	24	21.5	516	1787.56529
1	LOS	0.0394	-2	-0.0787	21.5	-1.6929	-5.8647038
2	LOS	2.3228	-0.1181	-0.2743	22.441	-6.1567	-25.132747
3	LOS	9.5984	-0.0394	-0.3779	22.48	-8.4951	-34.9033391
4	LOS	0.0394	-2	-0.0787	21.5	-1.6929	-5.8647038
5	LOS	4.9606	-0.0394	-0.1953	20.52	-4.0075	-11.4293666
6	Web corrosion	0.1969	-0.7874	-0.155	11	-1.705	-0.54185231
			Total =	**71.34**	in^2		

D = Depth of full section = 22.50 in

y_t = Distance to neutral axis from top of section = 12.87 in

y_b = Distance to neutral axis from bottom of section = 9.63 in

A_g = Total gross section area = 71.7693 in^2

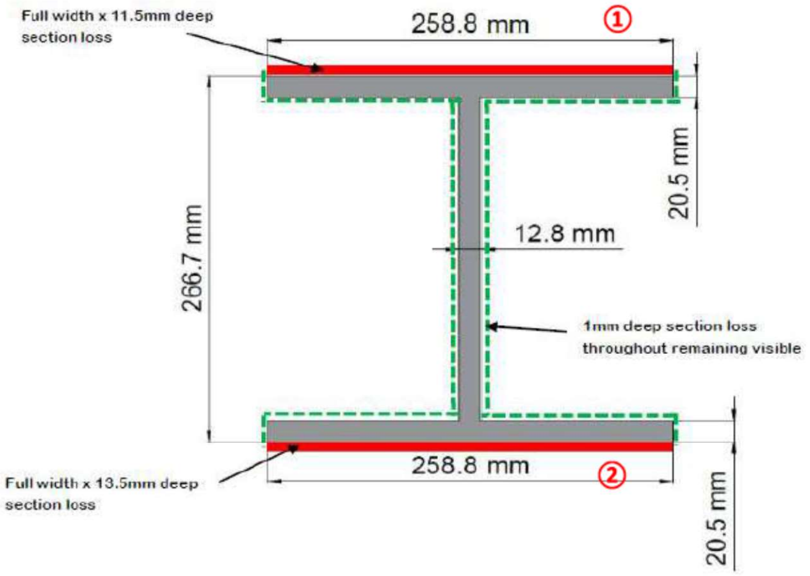
Reference	Calculation	Output

No.	Section	I_x (in^4)
1	Top Flange	1.96875
2	Web	1143.16667
3	Bot. Flange	8
1	LOS	-0.02624667
2	LOS	-0.00031893
3	LOS	-4.8811E-05
4	LOS	-0.02624667
5	LOS	-2.5226E-05
6	Web corrosion	-0.00800836

I_{xx} = 2nd moment of area of beam section = 4532.06 in^4
Z_{xc} = Elastic section modulus (compression flange) = 352.149 in^3
Z_{xt} = Elastic section modulus (tension flange) = 470.605 in^3

Reference	Calculation	Output

Propping Beam: Major Axis



Corroded section properties about x-x axis:

No.	Section	b (in)	d (in)	A (in^2)	y (in)	A.y (in^3)	$A(y-y_t)^2$ (in^4)
1	Top Flange	10.11	0.7677	7.7618	0.3839	2.9794	151.984969
2	Web	0.4252	8.8858	3.7782	5.2106	19.687	0.60969831
3	Bot. Flange	10.11	0.7677	7.7618	10.037	77.908	212.184414
1	LOS TF	10.11	-0.4528	-4.5775	0.2264	-1.0362	-96.125395
2	LOS BF	10.11	-0.5315	-5.3736	10.156	-54.571	-153.608614

Total = **9.3508** in^2

D	=	Depth of full section	=	10.42	in
y_t	=	Distance to neutral axis from top of section	=	4.81	in
y_b	=	Distance to neutral axis from bottom of section	=	5.61	in
A_{web}	=	Net area of web	=	3.78	in^2

No.	Section	I_x (in^4)
1	Top Flange	0.38122613
2	Web	24.8600657
3	Bot. Flange	0.38122613
1	LOS TF	-0.07819381
2	LOS BF	-0.12649702

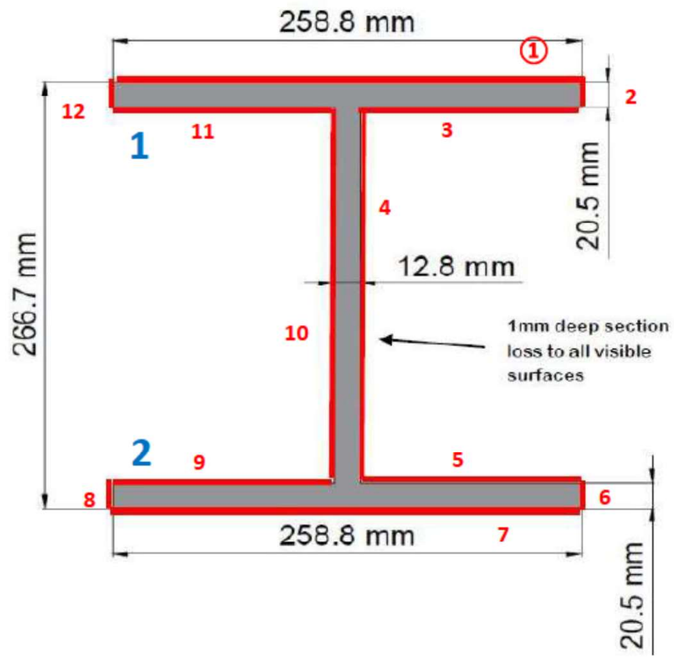
I_{xx}	=	2nd moment of area of beam section	=	140.463	in^4
Z_{xc}	=	Elastic section modulus (compression flange)	=	29.2088	in^3
Z_{xt}	=	Elastic section modulus (tension flange)	=	25.0275	in^3

JACOBS CALCULATION SHEET

Office	Manchester	Page No.	B6	of	B12
Job No. & Title	B28280BT HE HRE Assessment Programme	Originator	[redacted]	Date	3/7/18
Section	MKT-461 Bridge Assessment - Section Properties (corroded)	Checker	[redacted]	Date	11/7/18

Reference	Calculation	Output

Internal Propping Column:



Section loss to internal propping column

Corroded section properties about x-x axis:

No.	Section	b (in)	d (in)	A (in^2)	y (in)	A.y (in^3)	$A(y-y_t)^2$ (in^4)
1	Flange 1	10.189	0.8071	8.2234	0.4035	3.3185	193.152063
2	Web	0.5039	8.8858	4.4779	5.25	23.509	8.8311E-29
3	Flange 2	10.189	0.8071	8.2234	10.096	83.027	193.152063
1	LOS Flange 1	10.189	-0.0394	-0.4011	0.0197	-0.0079	-10.9736913
2	LOS Flange 1	0.0394	-0.7283	-0.0287	0.4035	-0.0116	-0.67352415
3	LOS Flange 1	4.8425	-0.0394	-0.1907	0.7874	-0.1501	-3.79676324
4	LOS Web	0.0394	-8.8858	-0.3498	5.25	-1.8366	-6.8993E-30
5	LOS Flange 2	4.8425	-0.0394	-0.1907	9.7126	-1.8517	-3.79676324
6	LOS Flange 2	0.0394	-0.7283	-0.0287	10.096	-0.2895	-0.67352415
7	LOS Flange 2	10.189	-0.0394	-0.4011	10.48	-4.2041	-10.9736913
8	LOS Flange 2	0.0394	-0.7283	-0.0287	10.096	-0.2895	-0.67352415
9	LOS Flange 2	4.8425	-0.0394	-0.1907	9.7126	-1.8517	-3.79676324
10	LOS Web	0.0394	-8.8858	-0.3498	5.25	-1.8366	-6.8993E-30
11	LOS Flange 1	4.8425	-0.0394	-0.1907	0.7874	-0.1501	-3.79676324
12	LOS Flange 1	0.0394	-0.7283	-0.0287	0.4035	-0.0116	-0.67352415

D = Depth of full section = 10.50 in

y_t = Distance to neutral axis from top of section = 5.25 in

y_b = Distance to neutral axis from bottom of section = 5.25 in

Reference	Calculation	Output

No.	Section	I_x (in^4)
1	Top Flange	0.44638511
2	Web	29.4637847
3	Bot. Flange	0.44638511
1	LOS TF	-5.1814E-05
2	LOS TF	-0.00126765
3	LOS TF	-2.4626E-05
4	LOS Web	-2.30185946
5	LOS BF	-2.4626E-05
6	LOS BF	-0.00126765
7	LOS BF	-5.1814E-05
8	LOS BF	-0.00126765
9	LOS BF	-2.4626E-05
10	LOS Web	-2.30185946
11	LOS TF	-2.4626E-05
12	LOS TF	-0.00126765

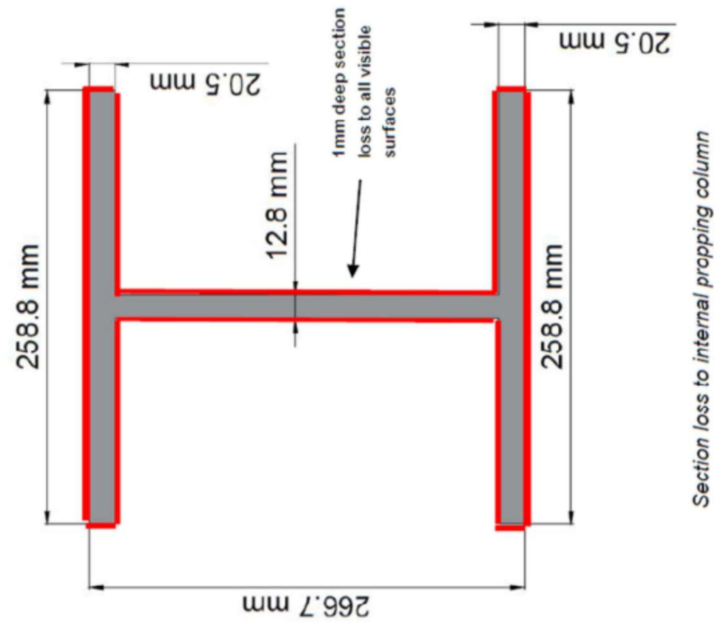
I_{xx} = 2nd moment of area of beam section = 372.22 in^4

Z_{xc} = Elastic section modulus (compression flange) = 70.90 in^3

Z_{xt} = Elastic section modulus (tension flange) = 70.90 in^3

Reference	Calculation	Output

Internal Propping Column: Minor Axis



Gross elastic section properties about x-x axis:

No.	Section	b (in)	d (in)	A (in^2)	y (in)	A.y (in^3)	$A(y-y_t)^2$ (in^4)
1	Top Flange	0.7283	10.11	7.3638	5.0551	37.225	0.000
2	Web	8.8858	0.4252	3.7782	5.0551	19.099	0.000
3	Bot. Flange	0.7283	10.11	7.3638	5.0551	37.225	0.000

D = Depth of full section = 10.11 in

y_t = Distance to neutral axis from top of section = 5.06 in

y_b = Distance to neutral axis from bottom of section = 5.06 in

No.	Section	I_x (in^4)
1	Top Flange	62.7249879
2	Web	0.05692284
3	Bot. Flange	62.7249879

I_{xx} = 2nd moment of area of beam section = 125.507 in^4

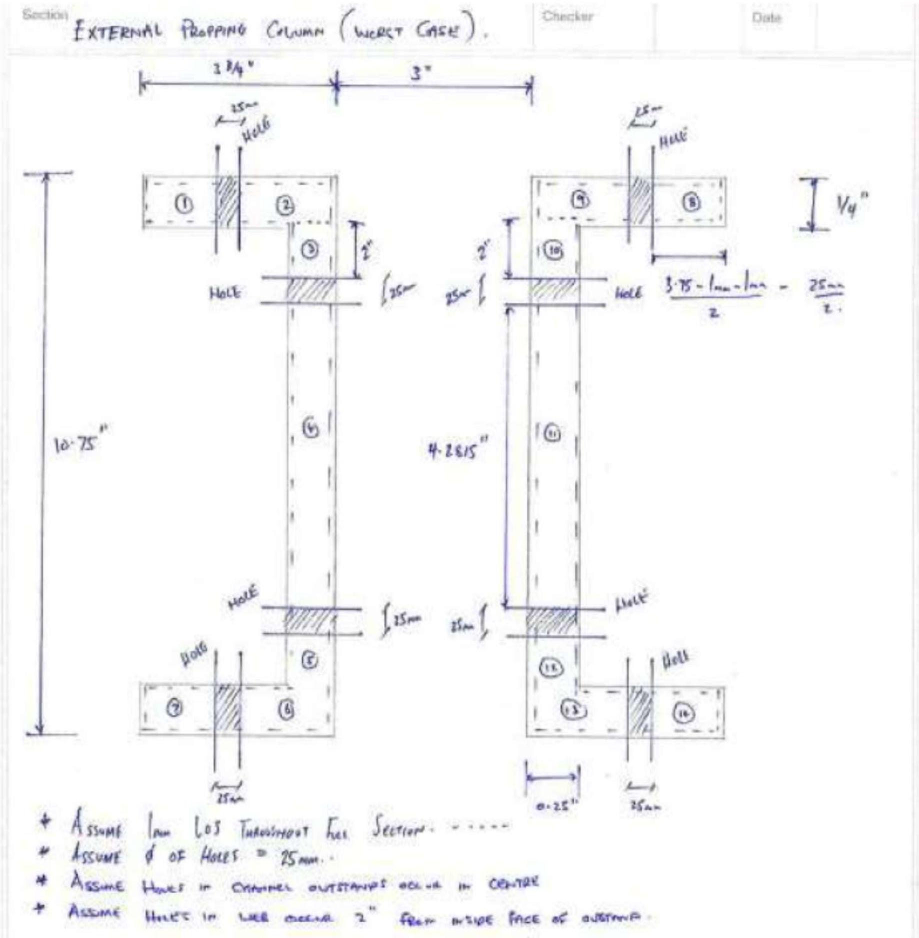
Z_{xc} = Elastic section modulus (compression flange) = 24.8277 in^3

Z_{xt} = Elastic section modulus (tension flange) = 24.8277 in^3

r_y = Radius of gyration of section = 2.60424 in

Reference	Calculation	Output

External Propping Column:



Outstand provides negligible increase in capacity and is ignored for purpose of assessment

Corroded elastic section properties about x-x axis:

No.	Section	b (in)	d (in)	A (in^2)	y (in)	A.y (in^3)	$A(y-y_t)^2$ (in^4)
1	Area 1	1.3435	0.1713	0.2301	0.0856	0.0197	6.34180509
2	Area 2	1.3435	0.1713	0.2301	0.0856	0.0197	6.34180509
3	Area 3	0.1713	2	0.3425	1.1713	0.4012	5.93996748
4	Area 4	0.1713	4.3602	0.7467	5.3356	3.9843	0
5	Area 5	0.1713	2	0.3425	9.5	3.2539	5.93996748
6	Area 6	1.3435	0.1713	0.2301	10.586	2.4356	6.34180509
7	Area 7	1.3435	0.1713	0.2301	10.586	2.4356	6.34180509
8	Area 8	1.3435	0.1713	0.2301	0.0856	0.0197	6.34180509
9	Area 9	1.3435	0.1713	0.2301	0.0856	0.0197	6.34180509
10	Area 10	0.1713	2	0.3425	1.1713	0.4012	5.93996748
11	Area 11	0.1713	4.3602	0.7467	5.3356	3.9843	0
12	Area 12	0.1713	2	0.3425	9.5	3.2539	5.93996748
13	Area 13	1.3435	0.1713	0.2301	10.586	2.4356	6.34180509
14	Area 14	1.3435	0.1713	0.2301	10.586	2.4356	6.34180509

*Assuming holes in channel outstands occur in centre

*Assuming holes in web occur 2" from face of outstand

Reference	Calculation	Output

D = Depth of full section = 10.67 in

y_t = Distance to neutral axis from top of section = 5.34 in

y_b = Distance to neutral axis from bottom of section = 5.34 in

No.	Section	I_x (in^4)
1	Area 1	0.00056237
2	Area 2	0.00056237
3	Area 3	0.1141732
4	Area 4	1.18305228
5	Area 5	0.1141732
6	Area 6	0.00056237
7	Area 7	0.00056237
8	Area 8	0.00056237
9	Area 9	0.00056237
10	Area 10	0.1141732
11	Area 11	1.18305228
12	Area 12	0.1141732
13	Area 13	0.00056237
14	Area 14	0.00056237

I_{xx} = 2nd moment of area of beam section = 77.3216 in^4

Z_{xc} = Elastic section modulus (compression flange) = 14.4916 in^3

Z_{xt} = Elastic section modulus (tension flange) = 14.4916 in^3

Photograph showing location of holes in channel section of external propping column

JACOBS CALCULATION SHEET

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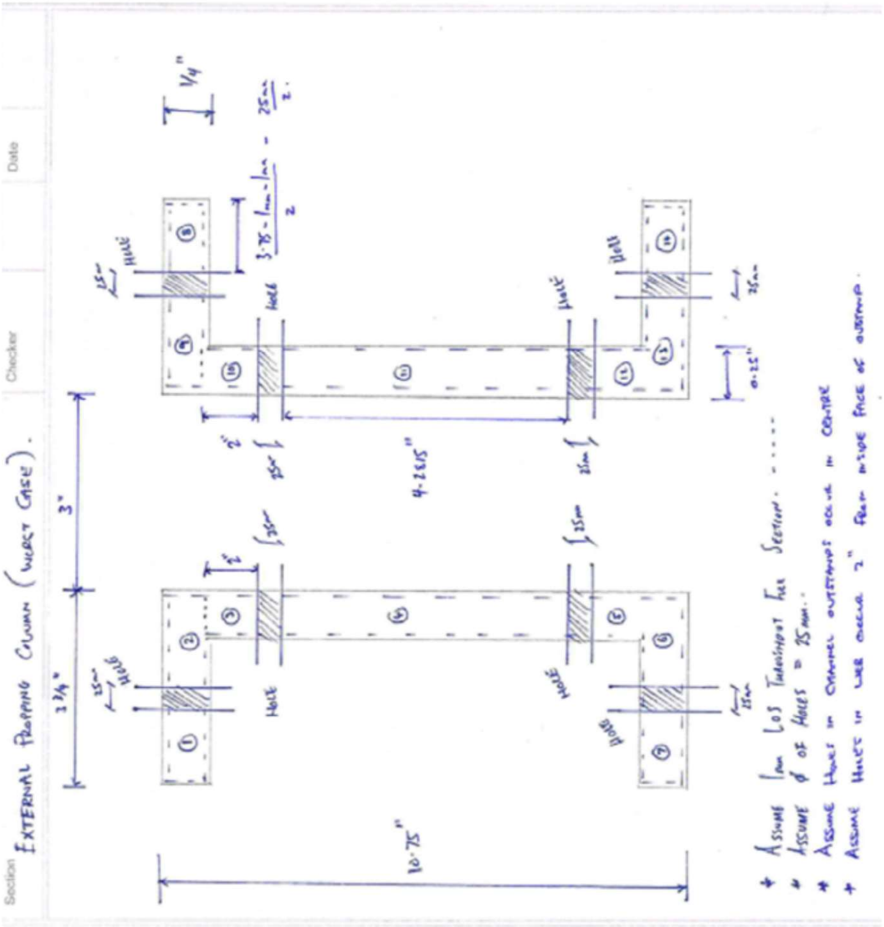
Section	MKT-461 Bridge Assessment - Section Properties (corroded)	Checker		Date	11/7/18

Reference	Calculation	Output

External Propping Column: Minor Axis



Upstand / downstand provides negligible increase in capacity and is ignored for purpose of assessment

Gross elastic section properties about y-y axis:

No.	Section	b (in)	d (in)	A (in^2)	y (in)	A.y (in^3)	$A(y-y_t)^2$ (in^4)
1	Area 1	0.1713	1.3435	0.2301	9.7495	2.2432	4.74014161
2	Area 2	0.1713	1.3435	0.2301	7.4218	1.7077	1.12491489
3	Area 3	2	0.1713	0.3425	6.8356	2.3413	0.90446582
4	Area 4	4.3602	0.1713	0.7467	6.8356	5.1044	1.97184185
5	Area 5	2	0.1713	0.3425	6.8356	2.3413	0.90446582
6	Area 6	0.1713	1.3435	0.2301	7.4218	1.7077	1.12491489
7	Area 7	0.1713	1.3435	0.2301	9.7495	2.2432	4.74014161
8	Area 8	0.1713	1.3435	0.2301	0.6718	0.1546	4.74014161
9	Area 9	0.1713	1.3435	0.2301	2.9995	0.6902	1.12491489
10	Area 10	2	0.1713	0.3425	3.5856	1.2281	0.90446582
11	Area 11	4.3602	0.1713	0.7467	3.5856	2.6775	1.97184185
12	Area 12	2	0.1713	0.3425	3.5856	1.2281	0.90446582
13	Area 13	0.1713	1.3435	0.2301	2.9995	0.6902	1.12491489
14	Area 14	0.1713	1.3435	0.2301	0.6718	0.1546	4.74014161

Total = **4.70425** in^2

Reference	Calculation	Output

Minor Axis

D = Depth of full section = 10.42 in
y_t = Distance to neutral axis from top of section = 5.21 in
y_b = Distance to neutral axis from bottom of section = 5.21 in

No.	Section	I_y (in^4)
1	Area 1	0.03460913
2	Area 2	0.03460913
3	Area 3	0.00083717
4	Area 4	0.00182513
5	Area 5	0.00083717
6	Area 6	0.03460913
7	Area 7	0.03460913
8	Area 8	0.03460913
9	Area 9	0.03460913
10	Area 10	0.00083717
11	Area 11	0.00182513
12	Area 12	0.00083717
13	Area 13	0.03460913
14	Area 14	0.03460913

I_{yy} = 2nd moment of area of beam section = 31.3056 in^4
Z_{yc} = Elastic section modulus (compression flange) = 6.00803 in^3
Z_{yt} = Elastic section modulus (tension flange) = 6.00803 in^3
r_y = Radius of gyration of section = 2.57968 in

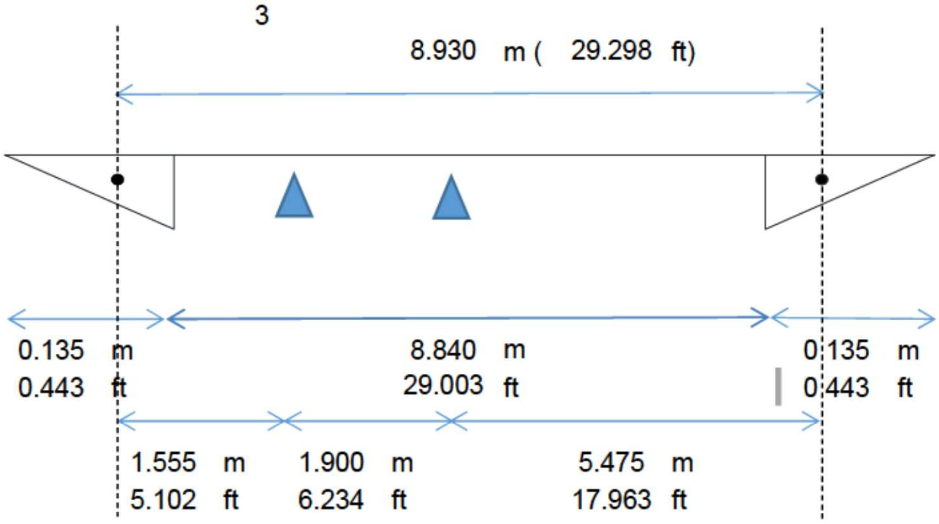
Reference	Calculation				Output
	Effective Span Main Girders				
FormAA	Clear skew span	=	8.840	m	
Pg. B1	MGI depth at midspan, $D_{MGI,m}$	=	21.25	in	
		=	539.75	mm	
Pg. B2	MGI depth at support, $D_{MGI,s}$	=	13.88	in	
		=	352.43	mm	
Pg. B3	MGE depth, D_{MGE}	=	22.50	in	
		=	571.5	mm	

Effective span for main internal girders:

For worst case moment, use larger span length for main girders thus consider maximum depth in bearing area calculation

» Length of bearing area for hard stone = D/4 = 134.94 mm
Assuming linear stress distribution from maximum at front of bearing, to minimum at back. Effective length = distance between bearing pressure diagrams

$$L_{eff} = 8.84 + \left(2 \times \frac{0.135}{3}\right) = 8.930 \text{ m}$$



Internal props shown indicatively as simple supports in above sketch. Prop spacing measured on site and recorded in FormAA

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Section	MKT-461 Bridge Assessment - Effective Span	Checker		Date	11/7/18

Reference	Calculation	Output

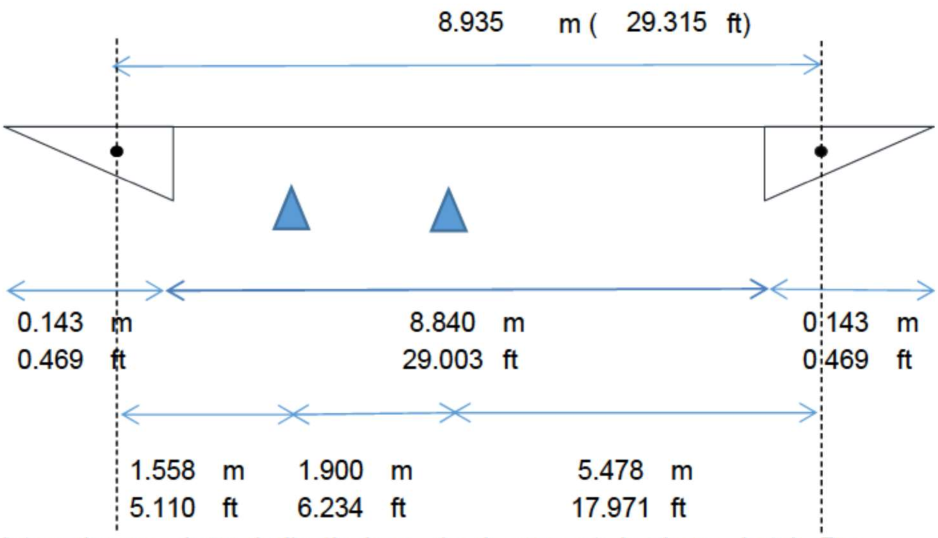
Effective span for main external girders:

For worst case moment, use larger span length for main girders thus consider maximum depth in bearing area calculation

» Length of bearing area for hard stone = D/4 = 142.88 mm

Assuming linear stress distribution from maximum at front of bearing, to minimum at back. Effective length = distance between bearing pressure diagrams

$$L_{eff} = 8.84 + \left(2 \times \frac{0.143}{3}\right) = 8.935 \text{ m}$$

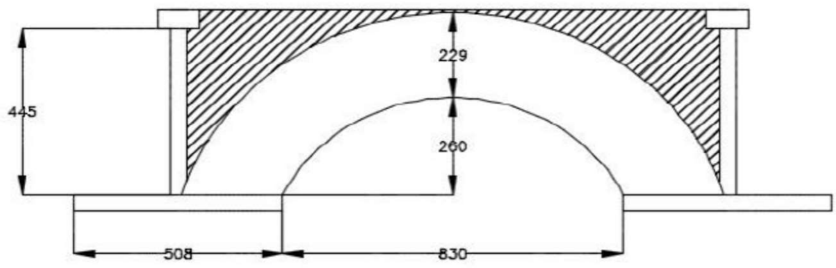


Internal props shown indicatively as simple supports in above sketch. Prop spacing measured on site and recorded in FormAA

Reference	Calculation				Output
	All references to BE4 (The Assessment of Highway Bridges) unless stated otherwise:				
	Weights of materials to be used:				
	Cast Iron	=	450	lb/ft^3	
	Steel	=	490	lb/ft^3	
	Brickwork	=	140	lb/ft^3	
	Concrete	=	150	lb/ft^3	
	Masonry	=	144	lb/ft^3	
	Earth / Sand / Misc	=	135	lb/ft^3	
	Macadam	=	144	lb/ft^3	
	Edge Girder Dead Load:				
Sheet B3	Net area of edge girder	=	72.50	in^2	
	Edge girder self weight	=	226.56	lb/ft	
	Internal Girder Dead Load:				
Sheet B1	Net area of Internal girder	=	71.44	in^2	Midspan
Sheet B2		=	59.45	in^2	Support
	Internal girder self weight	=	223.24	lb/ft	Midspan
		=	185.79	lb/ft	Support
	Parapet Dead Load:				
	Parapet height	=	47.09	in	
Site notes	Parapet width	=	14.17	in	
	Net area of parapet	=	667.37	in^2	
	Parapet self weight	=	667.37	lb/ft	

Reference	Calculation				Output
	Inner Girder Fill Dead Load:				
	Depth of macadam (midspan)	=	2.36	in	
	Depth of concrete fill (midspan)	=	3.94	in	
	Sand fill is assumed to fill full area below the concrete fill.				
Form AA	Girder spacing	=	52.76	in	
	Net area of macadam	=	124.62	in^2	
	Net area of concrete fill	=	207.7	in^2	
AutoCAD	Net area of sand fill	=	244.37	in^2	
	Macadam fill self weight	=	124.62	lb/ft	
	Concrete fill self weight	=	216.35	lb/ft	
	Sand fill self-weight	=	229.1	lb/ft	
	Total UDL from fill	=	570.07	lb/ft	

Reference	Calculation	Output
	Jack Arch Dead Load:	
FormAA	Rise at crown = 10.24 in	
FormAA	Ring thickness = 9.02 in	
FormAA	Span = 51.13 in	

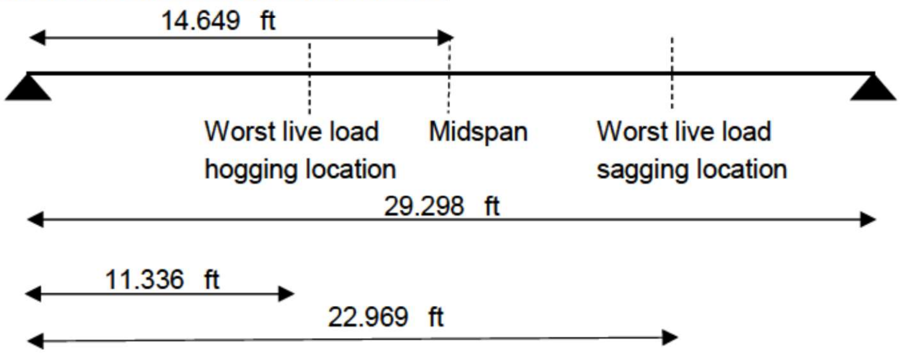


Reference	Calculation	Output
AutoCAD	Net area of Jack Arch = 319508 mm^2	
	= 495.24 in^2	
	Jack Arch self weight = 481.48 lb/ft	
	Propping Beam:	
Sheet B5	Net area of propping beam = 9.3508 in^2	
	Propping beam self weight = 31.819 lb/ft	

Reference	Calculation	Output

Dead load is assumed to be carried by the cast iron girders only.

Bending moment for Internal Girders



Sheet C1 (29.298 ft)

Sheet E4 (11.336 ft; 22.969 ft)

Sheet D1 — Girder selfweight (midspan) $= \dfrac{223.24 \times 29.298^{2}}{8 \times 2240}$

$= 10.693$ ton.ft

Sheet D2 — Fill load (midspan) $= \dfrac{570.07 \times 29.298^{2}}{8 \times 2240}$

$= 27.306$ ton.ft

Sheet D3 — Jack arch $= \dfrac{481.48 \times 29.298^{2}}{8 \times 2240}$

$= 23.063$ ton.ft

Total bending moment at midspan

$= 10.693 + 27.306 + 23.063$

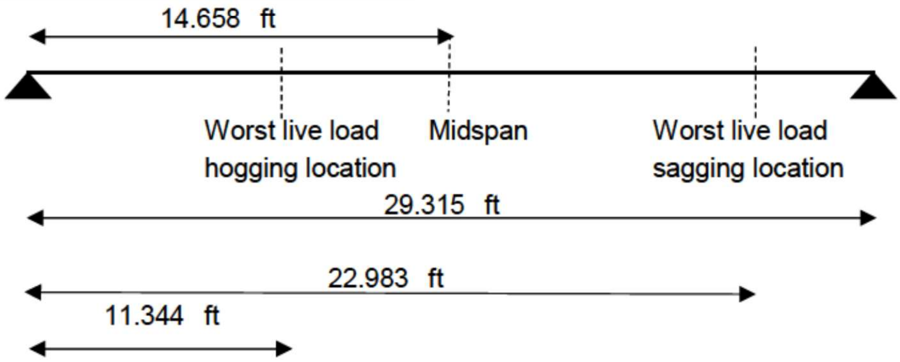
$= 61.062$ ton.ft

Reference	Calculation	Output
	Bending moment at worst sagging live load location for internal girder:	
Sheet D1	Girder selfweight	
	$= \frac{223.24 \times 22.969 \times 29.298 \times (1 - 0.784)}{2 \times 2240}$	
	$= 7.244$ ton.ft	
Sheet D2	Fill load	
	$= \frac{570.07 \times 22.969 \times 29.298 \times (1 - 0.784)}{2 \times 2240}$	
	$= 18.497$ ton.ft	
Sheet D3	Jack arch	
	$= \frac{481.48 \times 22.969 \times 29.298 \times (1 - 0.784)}{2 \times 2240}$	
	$= 15.623$ ton.ft	
	Total DL bending moment at worst live load sagging location	
	$= 7.244 + 18.497 + 15.623$	
	$= 41.364$ ton.ft	
	Bending moment at worst hogging live load location for internal girder:	
	Girder selfweight	
	$= \frac{223.24 \times 11.336 \times 29.298 \times (1 - 0.3869)}{2 \times 2240}$	
	$= 10.146$ ton.ft	
	Fill load	
	$= \frac{570.07 \times 11.336 \times 29.298 \times (1 - 0.3869)}{2 \times 2240}$	
	$= 25.910$ ton.ft	
	Jack arch	
	$= \frac{481.48 \times 11.336 \times 29.298 \times (1 - 0.3869)}{2 \times 2240}$	
	$= 21.883$ ton.ft	
	Total DL bending moment at worst live load hogging location	
	$= 10.146 + 25.910 + 21.883$	
	$= 57.939$ ton.ft	

Reference	Calculation	Output

Dead load is assumed to be carried by the cast iron girders only.

Bending moment for Edge Girders



Sheet C2

Sheet E6

Sheet D1 — Girder selfweight $= \dfrac{226.56 \times 29.315^2}{8 \times 2240}$

$= 10.865$ ton.ft

Sheet D2 — Fill load (midspan) (assumed half of load on internal girders) $= \dfrac{285.04 \times 29.315^2}{8 \times 2240}$

$= 13.669$ ton.ft

Sheet D3 — Jack arch (assumed half of load on internal girders) $= \dfrac{240.74 \times 29.315^2}{8 \times 2240}$

$= 11.545$ ton.ft

Parapet $= \dfrac{667.37 \times 29.315^2}{8 \times 2240}$

32.004 ton.ft

Total bending moment at midspan

$= 10.865 + 13.669 + 11.545 + 32.004$

$= 68.084$ ton.ft

Reference	Calculation	Output

Bending moment at worst sagging live load location for edge girder:

Girder selfweight

$$= \frac{226.56 \times 22.983 \times 29.315 \times (1 - 0.784)}{2 \times 2240}$$

= 7.360 ton.ft

Fill load

$$= \frac{285.04 \times 22.983 \times 29.315 \times (1 - 0.784)}{2 \times 2240}$$

= 9.259 ton.ft

Jack arch

$$= \frac{240.74 \times 22.983 \times 29.315 \times (1 - 0.784)}{2 \times 2240}$$

= 7.820 ton.ft

Parapet

$$= \frac{667.37 \times 22.983 \times 29.315 \times (1 - 0.784)}{2 \times 2240}$$

= 21.679 ton.ft

Total DL bending moment at worst live load sagging location

= 7.360 + 9.259 + 7.820 + 21.679

= 46.119 ton.ft

Reference	Calculation	Output

Bending moment at worst hogging live load location for edge girder:

Girder selfweight

$$= \frac{226.56 \times 11.344 \times 29.315 \times (1 - 0.387)}{2 \times 2240}$$

= 10.310 ton.ft

Fill load

$$= \frac{285.04 \times 11.344 \times 29.315 \times (1 - 0.387)}{2 \times 2240}$$

= 12.971 ton.ft

Jack arch

$$= \frac{240.74 \times 11.344 \times 29.315 \times (1 - 0.387)}{2 \times 2240}$$

= 10.955 ton.ft

Parapet

$$= \frac{667.37 \times 11.344 \times 29.315 \times (1 - 0.387)}{2 \times 2240}$$

= 30.369 ton.ft

Total DL bending moment at worst live load hogging location

= 10.310 + 12.971 + 10.955 + 30.369

= 64.604 ton.ft

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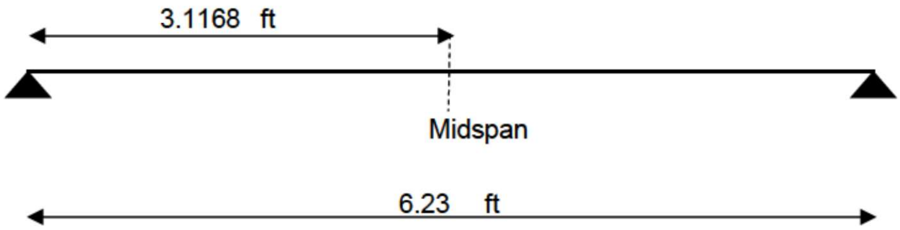
Reference	Calculation	Output

Bending Moment and End Shear for Propping Beam



Sheet C2

Sheet D3

Bending Moment:

Girder selfweight $= \dfrac{31.819 \times 6.234^{2}}{8 \times 2240}$

$= 0.069$ ton.ft

End Shear:

Girder selfweight $= \dfrac{31.819 \times 6.234}{2 \times 9.964}$

$= 10.0$ ton

Reference	Calculation	Output

All references to BE4 (The Assessment of Highway Bridges) unless stated otherwise:

<u>Load train:</u>

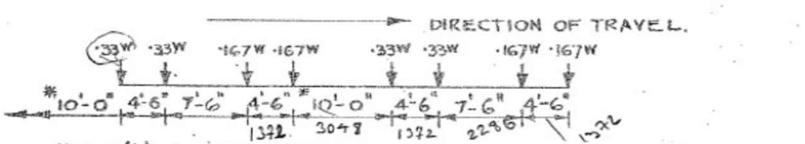
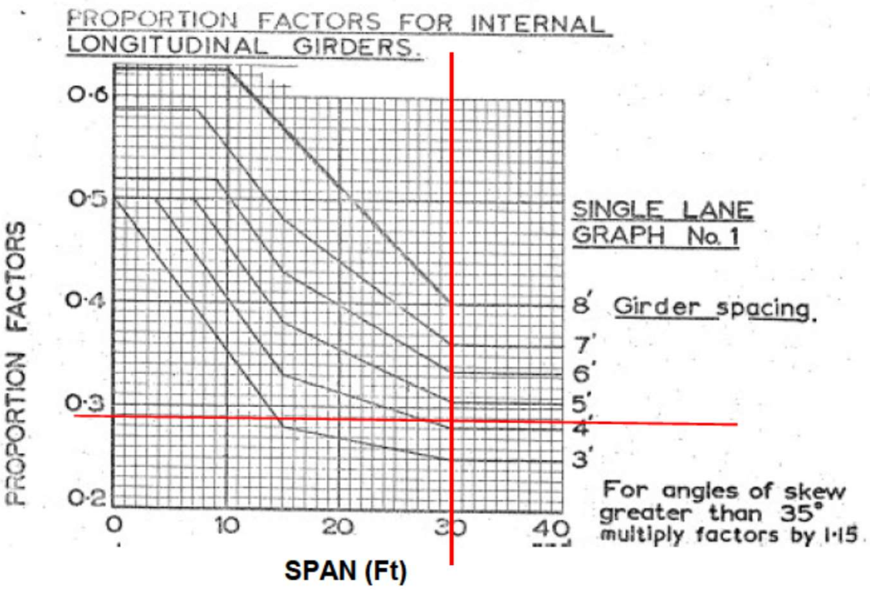


FIG.1.(b) 4 AXLE VEHICLES, W = GROSS WEIGHT OF VEHICLE.

<u>Internal Girders:</u>

Sheet C2	Effective span	=	8.93	m
		=	29.30	ft
Form AA	Girder Spacing	=	1.34	m
		=	4.40	ft
FormAA	Angle of skew	=	30	°

Reference	Calculation	Output



k = 0.29

For 24 tonnes live loading:

w = 24 tonnes

thus 0.33w = 8 ton
0.167w = 4 ton

For 1 MGI girder:

k * 0.33w = 2.32 ton
=
k * 0.167w = 1.16 ton

Reference	Calculation	Output

Live Load for Internal Girders:

	Axle Load (ton)	Axle Spacing (ft)
W1	1.16	
		4.5
W2	1.16	
		7.5
W3	2.32	
		4.5
W4	2.32	
		10
W5	1.16	
		4.5
W6	1.16	
		7.5
W7	2.32	
		4.5
W8	2.32	

'Section' and 'W1 position' are measured from left hand end of Line Beam

	Worst Case Moment(ton.ft)	Section	W1 position
Hog at Support 1	0	0	0
Sag in Span 1	-3.2	4.537	31.037
Hog at Support 2	1.232	5.102	7.53
Sag in Span 2	-3.096	5.104	31.604
Hog at Support 3	11.943	11.336	32.366
Sag in Span 3	-11.429	22.969	61.469
Hog at Support 4	0	29.299	0

	Worst Case Shear(ton)	W1 position
Span 1 left end	-2.8	43.001
Span 1 right end	2.443	17.101
Span 2 left end	2.396	31.601
Span 2 right end	4.014	54.335
Span 3 left end	-4.999	54.337
Span 3 right end	3.943	41.298

Pg. C1

Left Hand End	1	2	3	Right Hand End
Span Length (ft)	5.102	6.234	17.963	
Stiffness EI	1	1	1	
Relative EI/L	0.196	0.160	0.056	

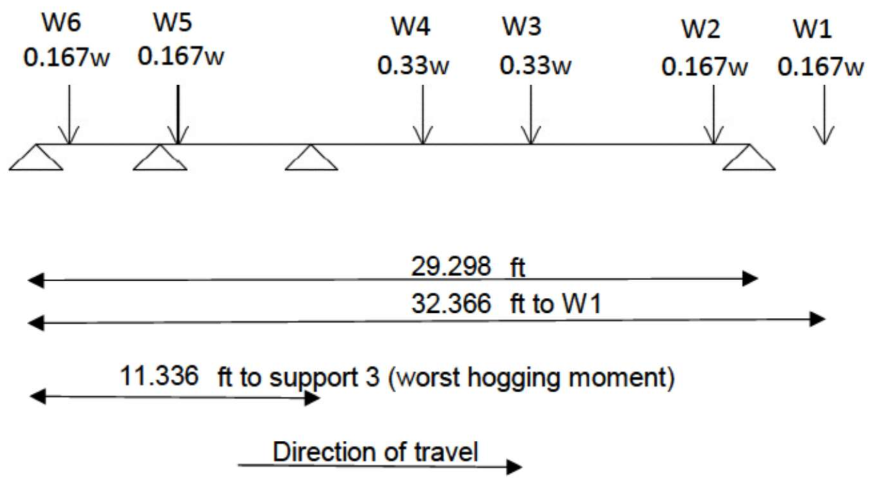
Reference	Calculation	Output

<u>Worst Case Bending Moment for Internal Girders:</u>

Worst Case Hogging Moment

11.943 ton   Hogging at support 3

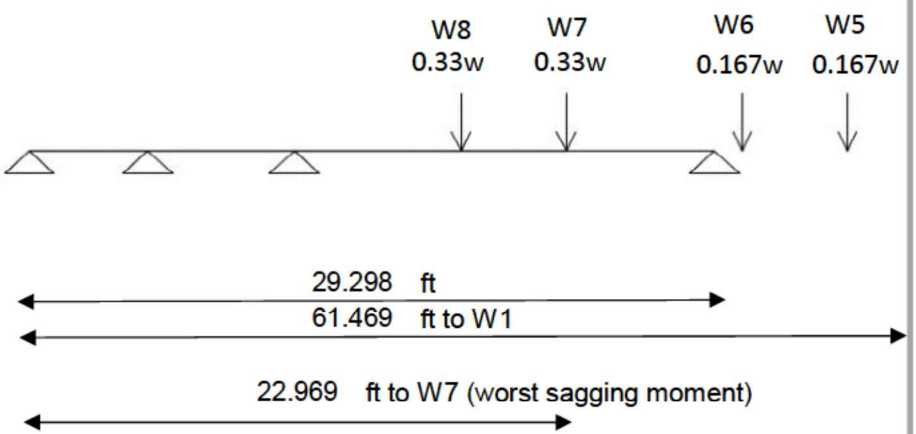
Location of W1 from left hand end of beam = 32.366 ft
Worst bending location from left hand end of beam = 11.336 ft



Worst Case Sagging Moment

-11.429 ton   Sagging in span 3

Location of W1 from left hand end of beam = 61.469 ft
Worst bending location from left hand end of beam = 22.969 ft



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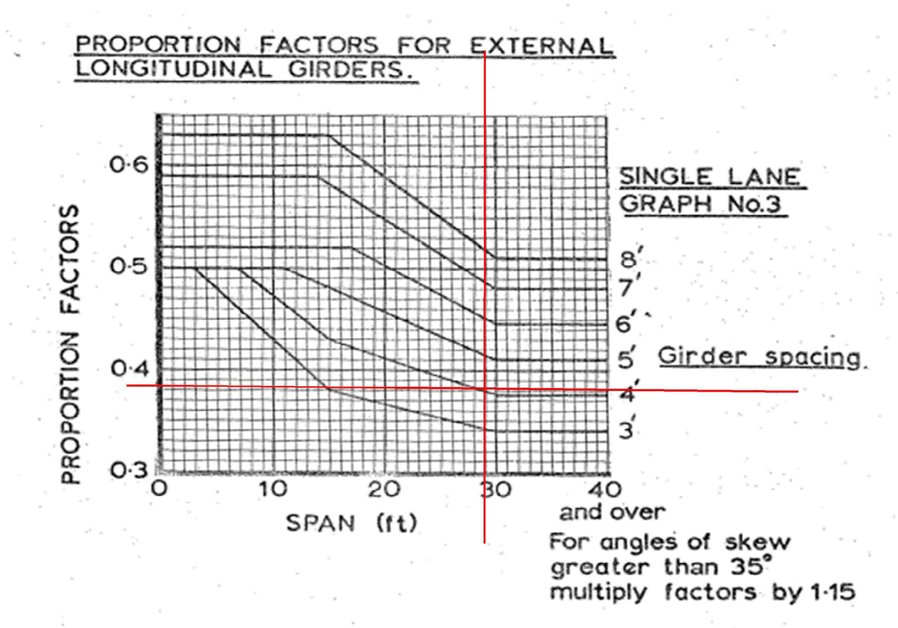
Reference	Calculation	Output

Edge Girders:



Girder Spacing = 1.238 m

Form AA = 4.0617 ft

k = 0.38

For 1 MGE girder:

k * 0.33w = 3.04 ton

k * 0.167w = 1.52 ton

Reference	Calculation	Output

Live Load for Edge Girders:

	Axle Load (ton)	Axle Spacing (ft)
W1	1.52	
		4.5
W2	1.52	
		7.5
W3	3.04	
		4.5
W4	3.04	
		10
W5	1.52	
		4.5
W6	1.52	
		7.5
W7	3.04	
		4.5
W8	3.04	

'Section' and 'W1 position' are measured from left hand end of Line Beam

	Worst Case Moment(ton.ft)	Section	W1 position
Hog at Support 1	0	0	0
Sag in Span 1	-4.192	4.545	31.045
Hog at Support 2	1.616	5.11	7.534
Sag in Span 2	-4.055	5.111	31.611
Hog at Support 3	15.661	11.344	32.383
Sag in Span 3	-14.985	22.983	61.483
Hog at Support 4	0	29.315	0

	Worst Case Shear(ton)	W1 position
Span 1 left end	-3.671	43.001
Span 1 right end	3.207	17.109
Span 2 left end	3.141	31.609
Span 2 right end	5.262	54.343
Span 3 left end	-6.553	54.345
Span 3 right end	5.168	41.314

Pg. C2

Left Hand End	1	2	3	Right Hand End
Span Length (ft)	5.11	6.234	17.971	
Stiffness EI	1	1	1	
Relative EI/L	0.196	0.160	0.056	

Reference	Calculation	Output

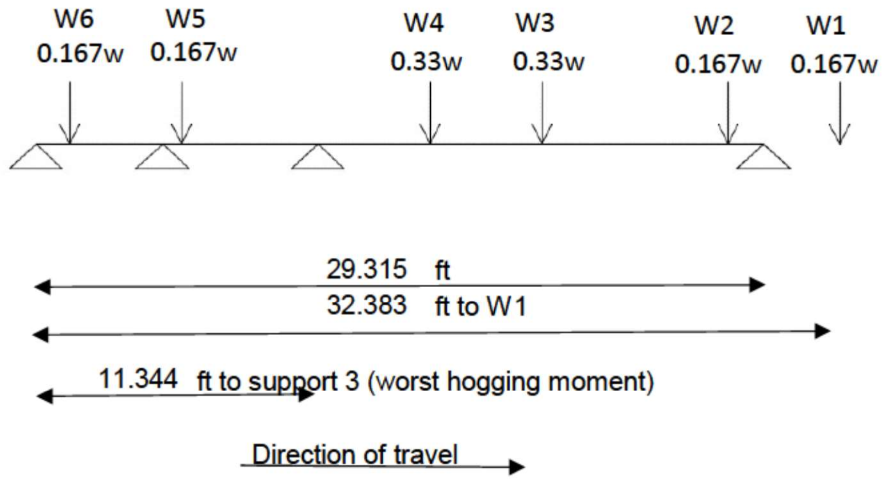
<u>Worst Case Bending Moment for Edge Girders:</u>

Worst Case Hogging Moment

15.661 ton Hogging at support 3

Location of W1 from left hand end of beam = 32.383 ft

Worst bending location from left hand end of beam = 11.344 ft

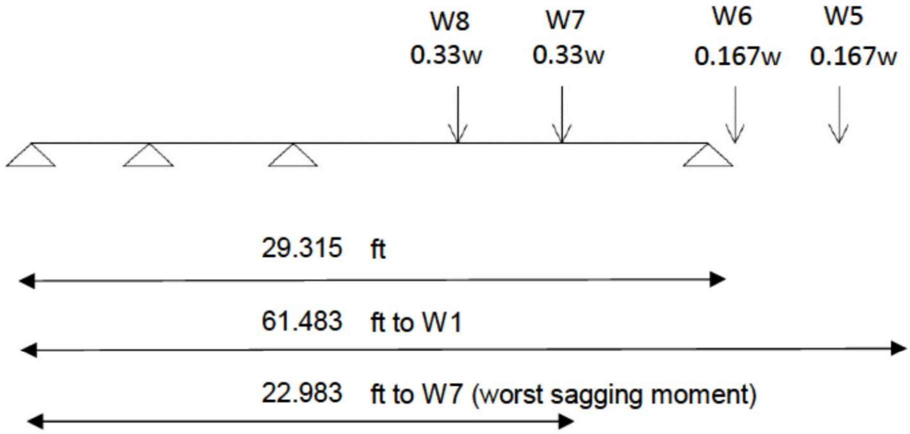


Worst Case Sagging Moment

-14.985 ton Sagging in span 3

Location of W1 from left hand end of beam = 61.483 ft

Worst bending location from left hand end of beam = 22.983 ft



Reference	Calculation	Output

Propping Beams:

Assuming no cantilever

	Axle Load (ton)	Axle Spacing (ft)
W1	1.52	
		4.5
W2	1.52	
		7.5
W3	3.04	
		4.5
W4	3.04	

'Section' and 'W1 position' are measured from left hand end of Line Beam

	Worst Case Moment(ton.ft)	Section	W1 position
Hog at Support 1	0	0	0
Sag in Span 1	-4.738	3.117	15.117
Hog at Support 2	0	6.234	0

	Worst Case Shear(ton)	W1 position
Span 1 left end	-3.885	16.501
Span 1 right end	-3.885	18.233

Left Hand End		Right Hand End
	1	
Span Length (ft)	6.234	
Stiffness EI	1	
Relative EI/L		

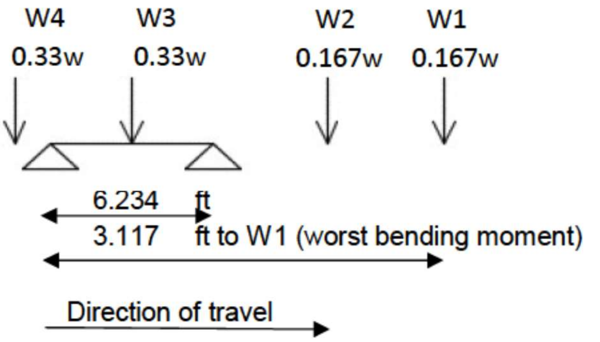
Reference	Calculation	Output

<u>Propping Beam:</u> *External propping beam assessed for the worst case, as this section carries a larger portion of dead load when compared to the internal propping beam*

Reference						
site notes	Column centres	=	6.234	ft		
site notes	End cantilever (east)	=	950	mm =	3.117	ft
site notes	End cantilever (west)	=	463	mm =	1.519	ft
	Total Length	=	10.869	ft		

Worst Case Moment ***(assuming no cantilever)***
-4.738 ton Sagging at midspan

Location of W1 from left hand end of beam = 15.117 ft
Worst bending location from left hand end of beam = 3.117 ft



Worst Case Shear ***(assuming no cantilever)***
-3.885 ton At support

Reference	Calculation	Output
	Internal girder capacity:	
Pg. B2	Z_{xc} = Elastic section modulus (compression flange) = 127.177 in^3	
Pg. B2	Z_{xt} = Elastic section modulus (tension flange) = 287.494 in^3	
	Worst sagging moment load case:	
Pg. D5	Maximum dead load bending moment = 41.36 ton.ft	
Pg. E4	Maximum live load bending moment = 11.43 ton.ft	
	Dead load compressive stress = BM/Z_{xc} = 3.90 ton/in^2	
	Dead load tensile stress = BM/Z_{xt} = 1.73 ton/in^2	
	Live load compressive stress = BM/Z_{xc} = 1.08 ton/in^2	
	Live load tensile stress = BM/Z_{xt} = 0.48 ton/in^2	
	Total compressive stress = 4.98 ton/in^2	Worst compressive and tensile load case
	Total tensile stress = 2.20 ton/in^2	
	Worst hogging moment load case:	
	Maximum dead load bending moment = 57.94 ton.ft	
	Maximum live load bending moment = -11.94 ton.ft	
	Dead load compressive stress = BM/Z_{xc} = 5.47 ton/in^2	
	Dead load tensile stress = BM/Z_{xt} = 2.42 ton/in^2	
	Live load compressive stress = BM/Z_{xc} = -1.13 ton/in^2	
	Live load tensile stress = BM/Z_{xt} = -0.50 ton/in^2	
	Total compressive stress = 4.34 ton/in^2	
	Total tensile stress = 1.92 ton/in^2	
	Capacity check (worst load case - sagging):	
BE4 Part 1, Clause 304c	Permissible compressive stress = 10 ton/in^2 OK	
	Permissible tensile stress = 3 ton/in^2 OK	
	Check < 8 : $5f_L + 2.2f_D$	
	Sagging: 5 x 0.48 + 2.2 x 1.73 = 6.18 ton/in^2 OK	
	Hogging: 5 x -0.50 + 2.2 x 2.42 = 2.83 ton/in^2	
	Tensile Capacity	
	= $(((8 - 2.2f_D)/5) \times Z_{xt}/12)$ + 41.36	
	= ((8 - 2.2 x 1.73) /5) x Z_{xt}/12) + 41.36	
	= 61.50 ton.ft	
	C = (61.50 - 41.36) / 11.43 = 1.76 (sagging)	
	Compressive Capacity = 10 x Z_{xc} /12 = 105.98 ton.ft	
	C = (105.98 - 41.36) / 11.43 = 5.65 (sagging)	

Reference	Calculation	Output
	Edge girder capacity:	
Pg. B4	Z_{xc} = Elastic section modulus (compression flange) = 352.149 in^3	
Pg. B4	Z_{xt} = Elastic section modulus (tension flange) = 470.605 in^3	
	Worst sagging moment load case:	
Pg. D7	Maximum dead load bending moment = 46.12 ton.ft	
Pg. E7	Maximum live load bending moment = 14.99 ton.ft	
	Dead load compressive stress = BM/Z_{xc} = 1.57 ton/in^2	
	Dead load tensile stress = BM/Z_{xt} = 1.18 ton/in^2	
	Live load compressive stress = BM/Z_{xc} = 0.51 ton/in^2	
	Live load tensile stress = BM/Z_{xt} = 0.38 ton/in^2	
	Total compressive stress = 2.08 ton/in^2	Worst compressive and tensile load case
	Total tensile stress = 1.56 ton/in^2	
	Worst hogging moment load case:	
	Maximum dead load bending moment = 64.60 ton.ft	
	Maximum live load bending moment = -15.66 ton.ft	
	Dead load compressive stress = BM/Z_{xc} = 2.20 ton/in^2	
	Dead load tensile stress = BM/Z_{xt} = 1.65 ton/in^2	
	Live load compressive stress = BM/Z_{xc} = -0.53 ton/in^2	
	Live load tensile stress = BM/Z_{xt} = -0.40 ton/in^2	
	Total compressive stress = 1.67 ton/in^2	
	Total tensile stress = 1.25 ton/in^2	
	Capacity check (worst load case - sagging):	
BE4 Part 1, Clause 304c	Permissible compressive stress = 10 ton/in^2 OK	
	Permissible tensile stress = 3 ton/in^2 OK	
	Check < 8 : $5f_L + 2.2f_D$	
	Sagging: 5 x 0.38 + 2.2 x 1.18 = 4.50 ton/in^2 OK	
	Hogging: 5 x -0.40 + 2.2 x 1.65 = 1.63 ton/in^2	
	Tensile Capacity	
	= $(((8 - 2.2f_D)/5) \times Z_{xt}/12)$ + 46.12	
	= ((8 - 2.2 x 1.18) /5) x Z_{xt}/12) + 46.12	
	= 88.57 ton.ft	
	C = (88.57 - 46.12) / 14.99 = 2.83 (sagging)	
	Compressive Capacity = 10 x Z_{xc} /12 = 293.46 ton.ft	
	C = (293.46 - 46.12) / 14.99 = 16.51 (sagging)	

Reference	Calculation	Output
	Propping beam capacity:	
Pg. B5	Z_{xc} = Elastic section modulus (compression flange) = 29.2088 in^3	
Pg. B5	Z_{xt} = Elastic section modulus (tension flange) = 25.0275 in^3	
Pg. D8	Maximum dead load bending moment = 0.07 ton.ft	
Pg. E9	Maximum live load bending moment = 4.74 ton.ft	
	Dead load compressive stress = BM/Z_{xc} = 0.03 ton/in^2	
	Dead load tensile stress = BM/Z_{xt} = 0.03 ton/in^2	
	Live load compressive stress = BM/Z_{xc} = 1.95 ton/in^2	
	Live load tensile stress = BM/Z_{xt} = 2.27 ton/in^2	
	Total compressive stress = 1.97 ton/in^2	
	Total tensile stress = 2.30 ton/in^2	
	Bending:	
BS153, 3B, Tbl 3 (ii)	Permissible compressive stress = 11.88 ton/in^2 **OK** *with 25% enhancement (BE4 Part 1, Clause 304(a))*	
BS153, 3B, Tbl 3	Permissible tensile stress = 11.88 ton/in^2 **OK**	
	Bending capacity of propping beam = ρ_{bt} x Z_{xt} / 12 = 24.77 ton.ft	
	C = (24.77 - 0.07) / 4.74 = 5.21	
	Shear:	
BS153, 3B, Tbl 3	Maximum permissible shear stress = 6.88 ton/in^2 *with 25% enhancement (BE4 Part 1, Clause 304(a))*	
Pg. D8	Maximum dead load shear = 9.95 ton	
Pg. E9	Maximum live load shear = 3.89 ton	
	Total shear = 13.84 ton	
Pg. B5	Net area of web = 3.78 in^2	
	Applied dead load stress = 2.63 ton/in^2	
	Applied live load stress = 1.03 ton/in^2	
	Applied shear stress = 3.66 ton/in^2 **OK**	
	Shear capacity of propping beam = A_{web} x 6.875 = 25.975 tons	
	C = (25.98 - 9.95) / 3.89 = 4.12	

Reference	Calculation	Output

Propping beam web in axial compression

Reference				
Pg. D8	Maximum dead load	=	9.95	ton
Pg. E9	Maximum live load	=	3.89	ton
	Total load	=	13.84	ton

BS153:part 3B, cl. 27; BS153:part 4, cl.27

Net area of web for buckling

Web thickness = 0.4252 in

Effective width = length of bearing plus the additional length given by dispersion at 45 degrees to the level of the neutral axis

$= \sqrt{(10.11^2 + 10.42^2)}$

= 14.52 in

Area = 0.4252 x 14.52 = 6.17 in^2

Dead load stress	=	9.95	/	6.17	=	1.61	ton/in^2
Live load stress	=	3.89	/	6.17	=	0.63	ton/in^2
Total Stress in web	=	13.84	/	6.17	=	2.24	ton/in^2

l_e = Assumed effectively held in position and restrained in direction at one end and the other end partially restrained in direction but not held in position

= 1.5 x 8.8858

= 13.329 in

r = $\sqrt{(bt^3/12) / bt}$ = $t / \sqrt{12}$

= 0.1227 in

l_e/r = 13.329 / 0.12

= 109

Stress in web should not exceed:

p_{ac} = 4.3 ton/in^2 (Table 4) **OK**

Capacity of web buckling:

Area x p_{ac} = 6.17 x 4.3 = 26.5 ton

C = (26.547 - 9.95) / 3.89 = 4.27

Reference	Calculation	Output
	<u>Propping column capacity:</u> Axially loaded compression member	
Pg. B11	Z_{xc} = Elastic section modulus (compression flange) = 14.4916 in^3	
Pg. B11	Z_{xt} = Elastic section modulus (tension flange) = 14.4916 in^3	
Pg. E9	Max. length of propping beam carried by column = 6.234 / 2 + 3.1 = 6.23 ft	
Pg. D3	Dead load of propping beam = 31.82 x 6.23 / 2240 = 0.089 ton	
Site Notes	L = effective length of compression member = 9.747 ft	
Pg. B11	Dead load of column = $\frac{4.704 \times 490 \times 9.747}{12 \times 12 \times 2240}$ = 0.070 ton	
	Total dead load stress = (0.089 + 0.070) / 4.704 = 0.034 ton/in^2	
	Live load axial force = 3.885 ton	
	Live load stress = 3.885 / 4.704 = 0.826 ton/in^2	
Pg. E10	Maximum axial force in column = 0.089 + 0.070 + 3.885 = 4.043 ton	
	Total compressive stress in section = 0.034 + 0.826 = 0.86 ton/in^2	

Reference	Calculation	Output

BS153, 3B, Cl 28 (b)

For sections with $I_y < I_x$

BS153, 3B, Cl 28 (b) i

Critical stress, C_s = $1.2*(170000 / (L/r_y)^2 \sqrt{[1 + 1/20 (LT/r_yD)^2]})$

= 100.53 ton/in^2

r_y = radius of gyration

= 2.5797 in

Pg. B10

D = overall depth of column

= 10.67 in

T = effective thickness of compression flange

= K_1 * mean thickness of horizontal portion of compression flange at point of maximum bending moment

= 0.1713 in

K_1 = 1 for flanges of constant thickness

B.S. 153 : Part 3B : 1958

TABLE 8

Allowable working stress p_{bc} for different values of critical stress C_s (but *see* Clause 25, Table 3).

C_s		p_{bc} for steel to B.S. 15 and B.S. 2762		p_{bc} for steel to B.S. 548 and B.S. 968	
ton/sq. in.	kg/mm²	ton/sq. in.	kg/mm²	ton/sq. in.	kg/mm²
2	3·2	1·0	1·6	1·0	1·6
3	4·7	1·5	2·4	1·5	2·4
4	6·3	2·0	3·2	2·0	3·2
80	126·0	10·0	15·8	13·0	20·5
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
100	157·5	10·0	15·8	13·6	21·4
110	173·2	10·0	15·8	13·8	21·7
120	189·0	10·0	15·8	14·0	22·0
136	214·2	10·0	15·8	14·2	22·4

BS153, 3B, Tbl 8

Allowable working stress, parts in compression, ρ_{bc} = 10 ton/in^2

OK

Capacity of column in compression

= Area x ρ_{bc} = 4.704 x 10 = 47.04 ton

C = (47.04 - 0.16) / 3.89 = 12.07

JACOBS CALCULATION SHEET

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Section	CIS 22 Jack Arch Assessment	**Checker**	[redacted]	**Date**	10/07/2018

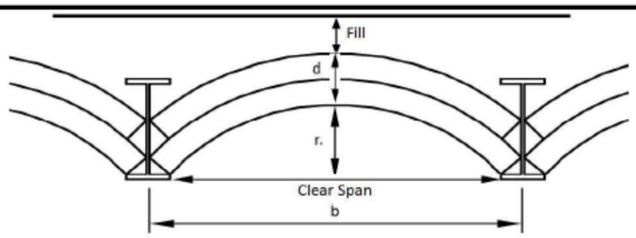
Ref.

NR-GN-CIV-025

Member Reference:

Any Other Reference:

Cl.6.5 CIS22



Fill =	210	mm	Clear Span =	830	mm
d =	229	mm	b =	1338	mm
rc =	260	mm	Depth of concrete Fill =	100	mm

Section 1: Checks for compliance with 40t configuration requirements

Clear Span = Compliant

Gross aspect ratio = Compliant

Do jack arches spring from the bottom flanges? Yes

Arch barrel thickness(including concrete fill) Compliant

Therefore jack arch is is compliant

Section 2: Checks for deficiency

Part 1

Is the backing material structural? Yes Details: concrete

Does the structural backing extend to at least the crown level of the arch extrados? Yes

Effective shear depth of deck D_s = 699 mm Permissible Minimum Effective Shear depth = 303

Therefore effective shear depth is is compliant

Part 2

Do jack arches span longitudinally or transversely between longitudinal girders? Transversely

As jack arches span Transversely Section 3a&3b must be completed

Are ties provided in the edge bays of the jack arches? Yes

Part 3a & 3b

Cross Sectional area of one tie(allow for corrosion loss)? 491 mm^2

What is the number of ties per beam length? 2 No.

What is the clear skew span? 8.84 m

Giving a specific area of: 167 mm^2/m Therefore Fail

260sq.mm/m area required for BD21 loading. However BE4 loading is approx. half of BD21 loading only, therefore the tie bar area could be compliant.

What material are the ties? Steel

What is the maximum tie spacing? 0.37 m

Maximum permissible spacing is 3.00 m therefore Ok

Part 4

Are ties located in crown of arch? No

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Section	CIS 22 Jack Arch Assessment	**Checker**	[redacted]	**Date**	10/07/2018

Ref.

Section 2 Continued

At Support

Does external bay construction provide alternative lateral restraint? Yes

Therefore jack arches pass deficiency checks and meet the requirements of section 2

Section 3

Defect	Present?
Rotation of supporting beam	No
Horizontal displacement of supporting beam	No
Inadequate support to springings (eg. Corrosion to bottom flange of supporting beam over a significant length, missing bedding mortar)	No
Transversely bowed bottom flange of supporting beam	No
Cracking at crown of arch owing to spreading of springings (other than 12 and 13)	No
Distortion and any associated cracking of jack arch barrel	Yes
Arch crack resulting in substructure crack	No
Substructure crack or other distress resulting in crack to jack arch	No

Jack arch deck has significant defects which may be considered to affect capacity.

Dropped bricks with deep open joints to jack arches, especially the central jack arch at the crown is considered to affect capacity.

Following CIS 22 Quantative analysis the jack arches are found to be compliant, are not found to be deficient and have significant defects which may be considered to affect the capacity

Although the BE4 live load is approximately half of the BD21 live load, the capacity of the jack arches is reported as Dead Load only due to the significant defects which may be considered to affect capacity.