Essex Highways

Station Way Bridge
ECC Bridge No. 2100

Assessment Report
May 2025

Ringway Jacobs | Essex County Council

2100 – Station Way Bridge Assessment Report

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Executive Summary

Jacobs has been commissioned by Ringway Jacobs to assess Station Way Bridge in accordance with CS 454 and CS 455. In addition to the assessment live loading specified in CS 454, SV80 loading in accordance with CS 458 has been considered. The assessment has been undertaken using as-built record drawings to determine section dimensions, material properties and general dimensions. The September 2023 inspection for assessment report by Jacobs has been used to obtain the current condition of the structure and to confirm the principal dimensions. The inspection for assessment (IFA) was undertaken in two stages. The topside of the structure was inspected on 5th March 2023, and the underside of the structure was inspected on 19th April 2023.

The following elements have been assessed quantitatively at the ultimate limit state:

- Spans 1 & 5: south-west footway slab (span 1) and original footway slabs (spans 1 & 5)
- Spans 1 & 5: carriageway slabs
- Spans 2 to 4: main beams (parapet beams, kerb beams and carriageway beams)
- Spans 2 to 4: carriageway slab and service bay slabs
- Abutment columns and intermediate-pier columns

Refer to the structure plan in section 2.2 for further details of the span and element referencing.

The various slabs were assessed using either local grillage models or manual methods (including Pucher Charts). The main beams were assessed using global grillage models of spans 2 to 4 (MIDAS software). The abutment columns and intermediate-pier columns were assessed by manual methods, using the worst-case coexistent load effects (reactions) from the global grillage models.

With the exception of the carriageway slab of span 1, all of the superstructure elements supporting the carriageway have been assessed as being adequate for 44 tonnes assessment live loading and SV80 loading. The carriageway slab of span 1 has been assessed as being adequate for permanent loading only, due to the capacity of the dowelled connection with the adjacent run-on slab. It should be noted that if the carriageway slab of span 1 was adequately supported at both ends, it would be adequate for 44 tonnes assessment live loading and SV80 loading.

The superstructure elements supporting the footways have been assessed as being adequate for the following loading:

South-west footway slab (span 1):	Accidental vehicle loading (normal traffic) or pedestrian live loading
Original footway slabs (spans 1 and 5):	Pedestrian live loading only (refer to note below regarding north-east footway slab)
Service bay slabs (spans 2 to 4):	Restricted accidental vehicle loading (3t gross vehicle weight) or pedestrian live loading

Note: propping to the north-east footway slab (span 5) has been installed since the IFA was undertaken. The change in support conditions resulting from the introduction of the propping has not been considered in the assessment of the slab. For the assessment, it has been assumed that the slab is simply supported, and the capacity stated above is dependent on the slab being adequately supported by the main-span abutment. As described in section 3.2, a reduced bearing area at the south-west corner of the slab, where it bears on the main span abutment, was observed during the IFA. Although the slab still appeared to be adequately supported by the main span abutment at the time, it is recommended that suitable remedial works are undertaken to restore full support across the entire width of the slab.

The parapets have been risk-assessed in accordance with section 3 of CS 461. This risk assessment concluded that an N1/N2 upgrade is recommended.

The abutment columns and intermediate-pier columns have been assessed as being adequate for 44 tonnes assessment live loading and SV80 loading.

The following elements have been assessed qualitatively in accordance with CS 459:

- The approach-span abutments (referred to as the 'mass concrete abutments' on the structure plan in section 2.2)
- The curtain walls to the main-span abutments
- The wing walls
- The foundations

During the IFA, it was noted that the western end of the carriageway slab of span 1 was not supported on its intended support (the west approach-span abutment), and a gap of 80 to 100mm between the slab soffit and the abutment bearing shelf was evident, resulting from long-standing settlement of the abutment. The record drawings indicate that this slab is connected to the adjacent run-on slab by steel dowel bars. It is assumed that this dowelled connection is currently providing the support to the western end of the carriageway slab. The dowel bars have been assessed quantitatively and have been found to be adequate for permanent loading only. Due to the settlement observed, the west approach-span abutment is not considered to be adequate for current loading, based on a qualitative assessment.

It was also noted during the IFA that settlement, which also appears to be long-standing, has occurred to the east approach-span abutment. This has caused the north-east footway slab (span 5) to displace horizontally away from the main-span abutment bearing shelf by up to approximately 110mm at the south-west corner of the slab (resulting in a reduced bearing area at this location). Additionally at this location, significant outward movement of the north-east wing wall and parapet was noted (approximately 175mm movement at the top of the parapet). Due to the settlement and movement observed, the east approach-span abutment and north-east wing wall are not considered to be adequate for current loading, based on a qualitative assessment.

In July 2023, after the IFA was undertaken, temporary propping was installed to the north-east footway slab by ECC (refer to section 3.3 for further details).

It is recommended that both of the approach-span abutments, together with the north-east wing wall are subject to a review in accordance with CS 470 (Management of substandard highway structures).

Based on a qualitative assessment, the curtain walls, wing walls (other than the north-east wing wall) and foundations to the main-span abutments and intermediate piers are adequate for current loading.

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Acronyms and Abbreviations

BCI	Bridge Condition Indicator
ECC	Essex County Council
IFA	Inspection for Assessment
AIP	Approval in Principle
ULS	Ultimate Limit State
SLS	Serviceability Limit State
LUL	London Underground Limited
AVL	Accidental Vehicle Loading

1. Introduction

This report details the assessment of Station Way Bridge carried out by Jacobs on behalf of Essex County Council (ECC).

The assessment has been carried out in accordance with CS 454 and CS 455. As-built record drawings have been used to obtain section dimensions, material properties and general dimensions. The September 2023 inspection for assessment report by Jacobs has been used to obtain the current condition of the structure and to confirm the principal dimensions.

As required by ECC, SV80 loading has been considered in accordance with CS 458.

The AIP for the assessment was approved by ECC on 14/05/2024.

2. Structure Details

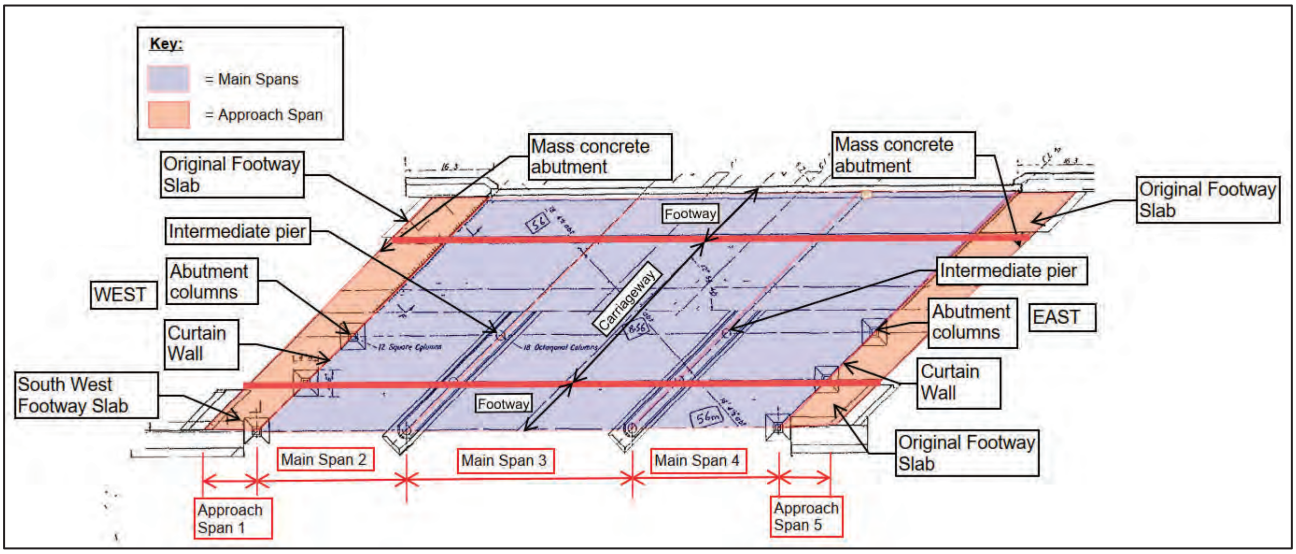
2.1 Description

Station Way Bridge is a five-span structure constructed circa 1939. The structure carries the two-lane carriageway of the unclassified residential road, Station Way, over the London Underground Central Line between Buckhurst Hill and Woodford Stations.

The structure is located in Buckhurst Hill, Epping, Essex at Ordnance Survey Grid Reference TQ 414 929. Refer to Appendix A for the location plan.

The structure has a bituminous carriageway, which has a width of 7.32m. There is a 2.44m wide footway and 0.33m wide parapet upstand on each side of the carriageway.

2.2 Structural Type



Note: ‘mass concrete abutments’ are also referred to as ‘approach-span abutments’

Figure 1: Structure Plan

<u>Main spans (spans 2 to 4)</u>

The main spans (spans 2 to 4) comprise six continuous, reinforced concrete main beams and a reinforced concrete deck slab. For the purposes of this report, the beams are numbered from 1 (north parapet beam) to 6 (south parapet beam), with beams 2 and 5 being referred to as the kerb beams, and beams 3 and 4 being referred to as the carriageway beams. The outermost slab bays (located between beams 1&2 and 5&6) are referred to as the ‘service bay slabs’ in this report and support the service bays and footways. The service bay slabs are constructed at a lower level than the slab supporting the carriageway (the slab located between beams 2 to 5).

The main beams are supported at two intermediate piers, each consisting of six octagonal, reinforced concrete columns, which are integrally connected to the main beams. The columns are supported by a reinforced concrete strip footing and are protected by a reinforced concrete fender.

The main beams are supported at each end by an abutment (referred to as the ‘main-span abutments’ in this report). Each main-span abutment consists of six square, reinforced concrete columns, which are integrally connected to the main beams and are supported by individual reinforced concrete pad foundations. At the upper part of the columns, there is a reinforced concrete curtain wall which spans between the columns.

<u>Approach spans (spans 1 and 5)</u>

There are two simply-supported approach spans, each consisting of three separate reinforced concrete slabs, with one slab supporting the carriageway and two slabs (one on each side of the carriageway slab) supporting the footways. The footway slabs are constructed at a lower level than the carriageway slab. Typically, these slabs are supported at one end by the approach-span abutment, and by the curtain wall of the main-span abutment at the other end (refer to section 2.5 for details of the support conditions for the carriageway slab of span 1 and the north-east footway slab of span 5).

The curtain wall of each main-span abutment creates an enclosed space which, at the west end of the structure, is accessible through an opening in the curtain wall. At the east end of the structure, the former opening has been blocked, and access to the enclosed space has been created via the bomb-shelter tunnels accessible near the north-east corner of the structure. This access was created by ECC in July 2023.

The record drawings indicate that, in 1979, remedial works were undertaken at the south-west corner of the structure. These works involved the replacement of the south-west footway slab (span 1), wing wall, part of the approach-span abutment and part of the main-span abutment. Observations made during the IFA confirmed that these works have been completed as shown on the record drawings. Refer to record drawing B2100/1B for further details of these works.

2.3 Foundation Type

Each abutment column is supported on a 4'6" (1370mm) x 4'6" (1370mm) x 16" (405mm) deep reinforced concrete pad foundation. The columns of each intermediate pier are supported on a 4'6" (1370mm) wide x 3'0" (915mm) deep reinforced concrete strip footing.

As observed during the IFA, there is evidence to suggest that piles were installed as part of the remedial works undertaken in 1979 at the south-west corner of the structure. At this location, the wing wall, part of the approach-span abutment and part of the main-span abutment are supported by a 6675mm long x 3500mm wide x 1000mm deep pile cap and 11 No. 450mm diameter reinforced concrete piles, as shown on record drawing B2100/1B.

2.4 Span Arrangements

Table 1: Details of Structure

Span No. (West to East)	Obstacle Crossed	Construction Type	Square Span (centre to centre of bearings)	Skew Span	Skew (°)
1 (Approach span)	Abutment Cell	Simply Supported RC Slabs	2.13m	3.12m	47
2	Cutting Slope	6No. Continuous RC Beams with integral RC slab	5.60m	8.23m	
3	LUL Central Line	6No. Continuous RC Beams with integral RC slab	8.56m	12.57m	
4	Cutting Slope	6No. Continuous RC Beams with integral RC slab	5.60m	8.23m	
5 (Approach span)	Abutment Cell	Simply Supported RC Slabs	2.13m	3.12m	

2.5 Articulation Arrangements

The main-span beams (spans 2 to 4) are integrally connected to the abutment columns at the end of spans 2 and 4. The abutment columns are hinged at their connection with the supporting pad foundations, with each hinge providing horizontal restraint in the longitudinal and transverse directions.

The intermediate-pier columns are integrally connected to the main-span beams, but the reinforcement arrangement shown on the record drawings indicates that the column-to-beam connection was not designed to be rotationally rigid. This means that whilst these connections provide translational restraint in the longitudinal and transverse directions, there is no significant moment fixity between the columns and the beams. The intermediate-pier columns are hinged at their connection with the supporting footing.

All six slabs of spans 1 and 5 are simply supported, but the north-east footway slab (span 5) is currently propped (refer to section 3.3 for details).

At the time of the IFA, a gap of between 80mm and 100mm was observed between the soffit of the carriageway slab of span 1 and the bearing shelf of the west approach-span abutment. Therefore, the western end of the carriageway slab does not appear to be supported by the abutment. The record drawings indicate that the carriageway slab is connected by steel dowel bars to the run-on-slab behind the abutment, and it is assumed that this dowelled connection is currently providing the support to the western end of the carriageway slab.

3. Inspection for Assessment

3.1 Access

The topside inspection was carried out on 5th March 2023 using the footways over the structure, with no specialist traffic management in place.

The underside inspection was carried out on 19th April 2023 under the protection of a full possession and isolation of the London Underground Central Line. Access to the track was gained from Roding Valley Station.

The IFA report is document number B3553S89-JAC-SBR-2100-IFA-S-001 (dated September 2023).

3.2 Condition

The IFA found the structure to be in fair condition overall, with a $BCI_{(Av)}$ of 78.7% and a $BCI_{(Crit)}$ of 60.4%. The IFA identified the following defects, which have been taken into consideration for the assessment:

- Area of spalled concrete with exposed and corroding longitudinal reinforcement and links to soffit of beam 3 in span 2 (400mm wide x 300mm long x 40mm deep) at approximately 0.40m from west curtain wall. ***Defect considered when determining shear resistance of beam (assumed 2mm loss of diameter to reinforcement over a 1.00m length of beam adjacent to support).***

- Area of spalled concrete with exposed and corroding longitudinal reinforcement and links (530mm long x 140mm high x 35mm deep) at bottom of face of web of beam 3 at midspan of span 3. ***Defect considered when determining bending resistance of beam (assumed 2mm loss of diameter to reinforcement over a 1.00m length at midspan).***

- Area of spalled concrete (560mm wide x 460mm long x 40mm deep) with exposed and corroding longitudinal (secondary) and transverse (primary) reinforcement to soffit of carriageway slab in central bay of span 3. ***Defect considered when determining bending resistance of carriageway slab (assumed 1mm loss of diameter to primary reinforcement over a 1.00m wide section of slab midway between main beams).***

- Movement was noted to the north-east footway slab (span 5), caused by settlement of the east approach-span abutment. The slab has been displaced horizontally away from the main-span abutment bearing shelf by up to approximately 110mm at the south-west corner of the slab (resulting in a reduced bearing area at this location). It should be noted that in July 2023, after the IFA was undertaken, temporary propping was installed to the north-east footway slab (refer to section 3.3 for details). Movement was also noted to the north-east wing wall and parapet (currently being monitored by ECC). ***Defects considered in qualitative assessment of the substructure.***

- There is a gap of between 80mm and 100mm between the soffit of the carriageway slab of span 1 and the bearing shelf of the west approach-span abutment. This is due to settlement of the abutment, and it appears that the western end of the carriageway slab is not supported by the abutment. It is assumed that the dowelled connection between the carriageway slab and the run-on-slab located behind the abutment is currently providing the support to the carriageway slab. ***Defect considered quantitatively by assessing capacity of dowelled connection. Defect also considered in qualitative assessment of substructure.***

Note: the beams are numbered from 1 (north parapet beam) to 6 (south parapet beam)

3.3 Temporary Propping of North-East Footway Slab (Span 5)

In July 2023, ECC implemented a minor-works scheme to prop the north-east footway slab (span 5). The propping system comprises a series of braced, adjustable steel props and is located below the soffit adjacent to the southern edge of the slab, at the centre of the span (refer to figure 2 below). The propping system was installed to provide additional vertical support to the slab in case any further horizontal displacement of the slab away from the main-span abutment bearing shelf (as described in section 3.2) resulted in a localised loss of bearing area at the south-west corner of the slab.

The change in support conditions resulting from the introduction of the propping has not been considered in the assessment of the slab. The slab has been assessed as a simply supported element, assuming that it is adequately supported by the main-span abutment and approach-span abutment (which was the case when the IFA was undertaken).

Figure 2: Interim Propping System (North-East Footway Slab)

3.4 Intrusive Investigations

No intrusive investigations or material testing were carried out in conjunction with the inspection for assessment. The reinforcement details used for the assessment have been taken from the record drawings contained in appendix C.

4. Previous Assessment Summary

The previous assessment of the structure was undertaken by W.S. Atkins in 1997 (assessment report dated June 1997). The structure was assessed at the ultimate limit state in accordance with BD21/93 and BD44/95. The assessed capacity of the structure was 40 tonnes assessment live loading and 30 units of HB loading.

A review of the previous assessment has identified the following differences between the previous assessment and the current assessment:

- The previous assessment was based on a condition factor of 1.0 (i.e. at the time, there were no defects significant to the assessment). The inspection for assessment undertaken for the current assessment identified some significant defects (e.g. exposed and corroding reinforcement), and these defects have been taken into account in the assessment.

- The previous assessment was less comprehensive than the current assessment, and it appears that not all of the elements considered in the current assessment were considered in the previous assessment. The summary table included in the previous assessment report only shows assessment results for the beams and carriageway slab of the main spans (i.e. spans 2 to 4). Whilst some of the other main elements (e.g. the columns) are included in the previous assessment calculations, no clear summary of the assessment results for these other elements is provided.

5. Assessment Methodology

5.1 Scope

The structure has been assessed in accordance with CS 454 and CS 455, with the superstructure and the columns being assessed at the ultimate limit state. In addition to the assessment live loading specified in CS 454, SV80 loading in accordance with CS 458 has been considered.

Details of the current condition of the structure have been taken from the September 2023 inspection for assessment report by Jacobs. The significant defects identified in the inspection for assessment report are summarised in section 3.2 of this report.

5.2 Material Properties

The following material strengths have been used for the assessment:

Concrete (all elements except south-west footway slab of span 1):

Characteristic strength = 15 N/mm^2 (clause 3.1.3 of CS 455)

Concrete (south-west footway slab of span 1 only):

Characteristic strength = 20 N/mm^2 (record drawing B2100/1B)

Reinforcement (all elements except south-west footway slab of span 1):

Characteristic strength = 230 N/mm^2 (clause 3.8.2 of CS 455)

Reinforcement (south-west footway slab of span 1 only):

Characteristic strength = 250 N/mm^2 (record drawing B2100/1B – assuming mild steel in accordance with BS 4449:1969)

Table 2: Unit Weights of Materials

Material	Unit Weight (kg/m^3)
Reinforced concrete	2400
Plain concrete	2300
Surfacing	2400
Miscellaneous fill	2200

5.3 Record Drawing Assumptions

The original (1930s) record drawings are difficult to read, and the concrete cover could only be discerned for the main beams. Therefore, the following assumption has been made:

- The concrete cover for all elements is the same as that stated on the record drawings for the main beams, i.e. 1½" (38mm).

It should be noted that, although the record drawings are difficult to read, it has been possible to obtain all of the information required for the assessment from a combination of the record drawings and the previous assessment report.

5.4 Analysis

The main beams of spans 2 to 4 have been assessed using two grillage models (MIDAS software). The main beams are integrally connected to the abutment columns (at the ends of spans 2 and 4), but the exact degree of moment fixity at the top of the columns is difficult to determine, therefore two separate models/ analyses have been used for spans 2 to 4, as described below:

- Model/ analysis 1: end supports (at abutment column positions) modelled as vertically and rotationally rigid
- Model/ analysis 2: end supports (at abutment column positions) modelled as vertically rigid and rotationally free

For both models/ analyses, the intermediate supports (at the intermediate-pier column positions) have been modelled as vertically rigid and rotationally free. The two analyses have been used to create an envelope of the worst-case load effects for the main beams.

The carriageway slab of spans 2 to 4 has been assessed using Pucher Charts. As the degree of moment fixity provided by the beam-to-slab connection varies and is difficult to determine, the bending capacity of the slab has been assessed for the following two extreme cases. For the hogging capacity, full moment fixity at the beam-to-slab connection has been assumed. For the sagging capacity, zero moment fixity at the beam-to-slab connection has been assumed.

The service bay slabs of spans 2 to 4 have also been assessed using Pucher Charts.

The carriageway slabs of spans 1 and 5 have been assessed using two grillage models (one for each span). Although the two slabs are generally similar, their support conditions differ, and therefore a separate model was required for each.

The south-west footway slab (span 1) has been assessed using a grillage model.

The original footway slabs of spans 1 (north-west footway slab) and 5 (both footway slabs) have been assessed using one grillage model (the three slabs have a similar arrangement).

5.4.1 Foundations

The foundations have been assessed qualitatively in accordance with CS 459.

5.4.2 Intermediate-Pier Columns

The intermediate-pier columns have been assessed quantitatively in accordance with CS 455, using manual methods. The worst-case coexistent load effects (reactions) from the two grillage analyses (for spans 2 to 4) have been applied to the columns.

5.4.3 Intermediate-Pier Column Hinges

The throat of the hinge at the base of each column has been assessed quantitatively in accordance with the basic principles of CS 455 and CS 468. The compressive resistance of the throat was determined using the guidance for rectangular hinges in CS 468.

5.4.4 Abutment Columns

The abutment columns have been assessed quantitatively in accordance with CS 455, using manual methods. The worst-case coexistent load effects (reactions) from the two grillage analyses (for spans 2 to 4) have been applied to the columns.

5.4.5 Abutment Column Hinges

The compressive resistance of the concrete block been assessed in accordance with the basic principles of CS 455. The shear resistance of the steel dowel bar in each hinge has been determined using guidance given in Concrete Society Technical Report No. 34.

5.4.6 Curtain Walls

The curtain walls are effectively deep beams which are subjected to a relatively low level of vertical loading. Therefore, the undertaking of a quantitative assessment is not considered to be necessary for these elements, and instead a qualitative assessment in accordance with CS 459 has been undertaken.

5.4.7 Parapets

A risk assessment of the parapets has been undertaken in accordance with section 3 of CS 461.

6. Assessment Results

With the exception of the carriageway slab of span 1, all of the superstructure elements supporting the carriageway have been assessed as being adequate for 44 tonnes assessment live loading and SV80 loading. The carriageway slab of span 1 has been assessed as being adequate for permanent loading only, due to the capacity of the dowelled connection with the adjacent run-on slab. It should be noted that if the carriageway slab of span 1 was adequately supported at both ends, it would be adequate for 44 tonnes assessment live loading and SV80 loading.

The superstructure elements supporting the footways have been assessed as being adequate for the following loading:

South-west footway slab (span 1):	Accidental vehicle loading (normal traffic) or pedestrian live loading
Original footway slabs (spans 1 and 5):	Pedestrian live loading only (refer to note below regarding north-east footway slab)
Service bay slabs (spans 2 to 4):	Restricted accidental vehicle loading (3t gross vehicle weight) or pedestrian live loading

Note: propping to the north-east footway slab (span 5) has been installed since the IFA was undertaken. The change in support conditions resulting from the introduction of the propping has not been considered in the assessment of the slab. For the assessment, it has been assumed that the slab is simply supported, and the capacity stated above is dependent on the slab being adequately supported by the main-span abutment. As described in section 3.2, a reduced bearing area at the south-west corner of the slab, where it bears on the main span abutment, was observed during the IFA. Although the slab still appeared to be adequately supported by the main span abutment at the time, it is recommended that suitable remedial works are undertaken to restore full support across the entire width of the slab.

The abutment columns and intermediate-pier columns have been assessed as being adequate for 44 tonnes assessment live loading and SV80 loading.

The following tables summarise the assessment results for each superstructure element and the columns. The assessment calculations are contained in appendix B.

The figure below shows the details and locations of the main beam section references used in the summary tables. The vertical dimensions represent the average depth of the main beam in the varying-depth sections.

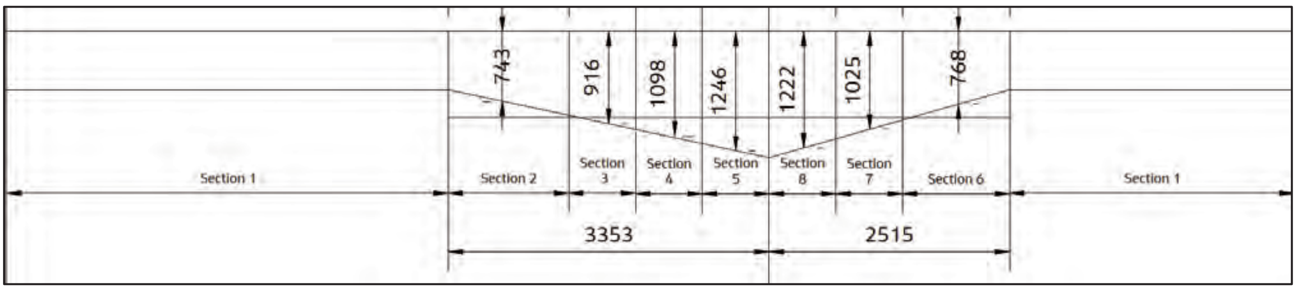


Figure 3: Main Beam Section References

6.1 Spans 1 & 5 – Original Footway Slab - Summary Tables

Table 3: Spans 1 & 5 – Original Footway Slab – Main Slab – Bending

Span 1 - Original Footway Slab – Main Slab – Longitudinal Bending								
ULS Bending (Longitudinal)								
Element	Live Load Case	Section Resistance* (kNm)	DL + SDL Effect* (kNm)	Live Load Effect* (kNm)	Total Load Effect* (kNm)	Utilisation (%)	Result	Section
Main slab	Pedestrian loading	3.6	2.6	0.9	3.5	97	Pass	Midspan (ULS sagging bending)

Table 4: Spans 1 & 5 – Original Footway Slab – Main Slab – Shear

Span 1 - Original Footway Slab – Main Slab - Shear								
ULS Shear								
Element	Live Load Case	Section Resistance* (kN)	DL + SDL Effect* (kN)	Live Load Effect* (kN)	Total Load Effect* (kN)	Utilisation** (%)	Result	Section
Main Slab	Pedestrian loading	23.6	7.6	2.3	9.9	42	Pass	Support (ULS shear)**

Notes (Tables 3 & 4):

The original footway slab (incorporating the main slab and longitudinal downstands) has been assessed using a grillage model (refer to section 5.4 of this report and Appendix D of the AIP for further details).

***Section resistances and load effects relate to width of member in grillage model (0.365m).**

****Utilisation is based on applied shear force at support and shear capacity at 3d from support and is therefore conservative.**

Accidental vehicle loading has not been considered because slab has been found to be only just adequate for permanent loading combined with pedestrian loading (as shown in Table 3). By inspection, slab is not adequate for any level of accidental vehicle loading.

Table 5: Spans 1 & 5 – Original Footway Slab – Downstand – Bending

Span 1 - Original Footway Slab – Downstand – Longitudinal Bending								
ULS Bending (Longitudinal)								
Element	Live Load Case	Section Resistance* (kNm)	DL + SDL Effect* (kNm)	Live Load Effect* (kNm)	Total Load Effect* (kNm)	Utilisation (%)	Result	Section
Downstand	Pedestrian loading	27.0	7.8	2.4	10.2	38	Pass	Midspan (ULS sagging bending)

Table 6: Spans 1 & 5 – Original Footway Slab – Downstand – Shear

Span 1 - Original Footway Slab – Downstand - Shear								
ULS Shear								
Element	Live Load Case	Section Resistance* (kN)	DL + SDL Effect* (kN)	Live Load Effect* (kN)	Total Load Effect* (kN)	Utilisation (%)	Result	Section
Downstand	Pedestrian loading	53.1	25.7	7.6	33.3	63	Pass	d from support (ULS shear)
Downstand	Pedestrian loading	30.8	16.4	5.4	21.8	71	Pass	3d from support (ULS shear)

Notes (Tables 5 & 6):

The original footway slab (incorporating the main slab and longitudinal downstands) has been assessed using a grillage model (refer to section 5.4 of this report and Appendix D of the AIP for further details).

***Section resistances and load effects relate to width of member in grillage model (0.20m wide at bottom of section).**

Accidental vehicle loading has not been considered because slab has been found to be only just adequate for permanent loading combined with pedestrian loading (as shown in Table 3). By inspection, slab is not adequate for any level of accidental vehicle loading.

6.2 Span 1 – Carriageway Slab - Summary Tables

Table 7: Span 1 - Carriageway Slab - Bending

Deck Information: Span 1 Carriageway Slab								
Bending (mid span of carriageway slab)								
Element	Load case	Section Resistance* (kNm)	Dead load + Superimposed Dead load* (kNm)	Live Load Effect* (kNm)	Total Load Effect* (kNm)	Utilisation (%)	Result	Section
Carriageway Slab – Span 1	ALL Model 1	26	2.9	19	22	87	Pass	Bending Longitudinal (Sagging)
	ALL Model 1	14	0.1	6	6	46	Pass	Bending Transverse (Sagging)
	ALL Model 2	26	2.9	20	23	88	Pass	Bending Longitudinal (Sagging)
	ALL Model 2	14	0.1	8	8	60	Pass	Bending Transverse (Sagging)
	ALL Model 1 + SV80	26	2.9	20	24	92	Pass	Bending Longitudinal (Sagging)
	ALL Model 1 + SV80	14	0.1	13	13	99	Pass	Bending Transverse (Sagging)

Note: The slab has been assessed using a grillage model (refer to section 5.1 and Appendix D of the AIP for further details)

***Section resistances and load effects relate to width of member in grillage model (refer to calculations for further details)**

The results above are based on the assumption that the slab is adequately supported at both ends (currently, the dowelled connection at the western end of the slab does not provide adequate support)

Table 8: Span 1 - Carriageway Slab – Shear

Deck Information: Span 1 Carriageway Slab								
Shear @ d from support								
Element	Load case	Section Resistance* (kN)	Dead load + Superimposed Dead load* (kN)	Live Load Effect* (kN)	Total Load Effect* (kN)	Utilisation (%)	Result	Section
Carriageway Slab – Span 1	ALL Model 1	78	5	48	53	68	Pass	d from support
	ALL Model 2	78	8	42	50	64	Pass	d from support
	ALL Model 1 + SV80	78	5	55	61	78	Pass	d from support

Note: The slab has been assessed using a grillage model (refer to section 5.1 and Appendix D of the AIP for further details)

***Section resistances and load effects relate to width of member in grillage model (refer to calculations for further details)**

The results above are based on the assumption that the slab is adequately supported at both ends (currently, the dowelled connection at the western end of the slab does not provide adequate support)

6.3 Span 5 – Carriageway Slab - Summary Tables

Table 9: Span 5 – Carriageway Slab - Bending

Deck Information: Span 5 Carriageway Slab								
Bending (mid span of carriageway slab)								
Element	Load case	Section Resistance* (kNm)	Dead load + Superimposed Dead load* (kNm)	Live Load Effect* (kNm)	Total Load Effect* (kNm)	Utilisation (%)	Result	Section
Carriageway Slab – Span 5	ALL Model 1	26	3	21	24	92	Pass	Bending Longitudinal (Sagging)
	ALL Model 1	14	0.1	6	7	48	Pass	Bending Transverse (Sagging)
	ALL Model 2	26	3	20	27	91	Pass	Bending Longitudinal (Sagging)
	ALL Model 2	14	0.1	8	8	60	Pass	Bending Transverse (Sagging)
	ALL Model 1 + SV80	26	3	20	24	93	Pass	Bending Longitudinal (Sagging)
	ALL Model 1 + SV80	14	0.1	13	23	98	Pass	Bending Transverse (Sagging)

Note: The slab has been assessed using a grillage model (refer to section 5.1 and Appendix D of the AIP for further details)

***Section resistances and load effects relate to width of member in grillage model (refer to calculations for further details)**

Table 10: Span 5 – Carriageway Slab – Shear

Deck Information: Span 5 Carriageway Slab								
Shear @ d from support								
Element	Load case	Section Resistance* (kN)	Dead load + Superimposed Dead load* (kN)	Live Load Effect* (kN)	Total Load Effect* (kN)	Utilisation (%)	Result	Section
Carriageway Slab – Span 5	ALL Model 1	78	5	43	49	62	Pass	d from support
	ALL Model 2	78	8	42	50	64	Pass	d from support
	ALL Model 1 + SV80	78	5	51	57	73	Pass	d from support

Note: The slab has been assessed using a grillage model (refer to section 5.1 and Appendix D of the AIP for further details)

***Section resistances and load effects relate to width of member in grillage model (refer to calculations for further details)**

6.4 Span 1 – South-West Footway Slab - Summary Tables

Table 11: Span 1 – South-West Footway Slab – Bending

Deck Information: Approach Span 1 – South West Footway Slab								
Bending (mid span of carriageway slab)								
Element	Load case	Section Resistance *(kNm)	Dead load + Superimposed Dead load *(kNm)	Live Load Effect* (kNm)	Total Load Effect* (kNm)	Utilisation (%)	Result	Section
South West Footway Slab – Span 1	Pedestrian Load	129	17	7	24	18	Pass	Longitudinal Bending of edge member (Sagging)
	Pedestrian Load	44	5	2	7	16	Pass	Longitudinal Bending (sagging)
	Pedestrian Load	32	4	2	6	18	Pass	Transverse bending (sagging)
	Accidental Vehicle Loading (Normal Traffic)	129	17	42	56	46	Pass	Longitudinal Bending of edge member (Sagging)
	Accidental Vehicle Loading (Normal Traffic)	44	5	7	12	27	Pass	Longitudinal Bending (sagging)
	Accidental Vehicle Loading (Normal Traffic)	32	4	5	9	27	Pass	Transverse bending (sagging)

Note: The slab has been assessed using a grillage model (refer to section 5.1 and Appendix D of the AIP for further details)

***Section resistances and load effects relate to width of member in grillage model (refer to calculations for further details)**

Table 12: Span 1 – South-West Footway Slab - Shear

Deck Information: Approach Span 1 – South West Footway Slab								
Shear @ d from support								
Element	Load case	Section Resistance* (kN)	Dead load + Superimposed Dead load* (kN)	Live Load Effect* (kN)	Total Load Effect* (kN)	Utilisation (%)	Result	Section
South West Footway Slab – Span 1	Pedestrian Load	114	41	12	53	46	Pass	d from support – Longitudinal edge member
	Pedestrian Load	80	10	4	13	17	Pass	d from support – Longitudinal member
	Accidental Vehicle Loading (Normal Traffic)	114	46	55	101	88	Pass	d from support – Longitudinal edge member
	Accidental Vehicle Loading (Normal Traffic)	80	7	17	24	30	Pass	d from support – Longitudinal member

Note: The slab has been assessed using a grillage model (refer to section 5.1 and Appendix D of the AIP for further details)

***Section resistances and load effects relate to width of member in grillage model (refer to calculations for further details)**

6.5 Spans 2 to 4 – Carriageway Beams - Summary Tables

Table 13: Spans 2 to 4 - Carriageway Beams – Bending

Carriageway Beam – Ultimate Limit State Bending									
Bending – Carriageway Beam (effective width = 1730mm) (method of assessment as described in section 5.4)									
Element / Assessment	Load case	Section Reference	Section Resistance (kNm)	Dead load + Superimposed Dead load (kNm)	Live Load Effect (kNm)	Total Load Effect (kNm)	Utilisation (%)	Result	Section
Carriageway Beam - Main Spans 2 -4	ALL Model 1 + SV80	1	630	118	503	620	98	Pass	Max Sagging (Analysis 2) @ Mid Span of Span 3
	ALL Model 1	1	709	118	344	461	65	Pass	Max Sagging (Analysis 2) @ Mid Span of Span 3
	ALL 2	1	709	118	340	445	63	Pass	Max Sagging (Analysis 2) @ Mid Span of Span 3
	ALL 1 + SV80	5 & 8	-1651	-520	-884	-1404	85	Pass	Max Hogging (Analysis 2) @ Intermediate Supports
	ALL 1	5 & 8	-1651	-551	-701	-1252	76	Pass	Max Hogging (Analysis 2) @ Intermediate Supports
	ALL 2	5 & 8	-1651	-520	-637	-1157	70	Pass	Max Hogging (Analysis 2) @ Intermediate Supports

Note: defects summarised in section 3.2 have been taken into account where applicable

Table 14: Spans 2 to 4 - Carriageway Beams – Shear

Carriageway Beam – Ultimate Limit State Shear								
Shear – Carriageway Beam (effective width = 610mm) **(method of assessment as described in section 5.4)**								
Element / Assessment	Load case	Section Resistance (kN)	Dead load + Superimposed Dead load (kN)	Live Load Effect (kN)	Total Load Effect (kN)	Utilisation (%)	Result	Section
Main Spans 2 -4	ALL Model 1	732	65	321	386	53	Pass	Shear @ d (562mm) from end support (analysis 1)
	ALL Model 1	1132	226	355	581	51	Pass	Shear @ d (1204mm) from Intermediate support (analysis 2)
	ALL Model 1	644	26	221	247	38	Pass	Shear @ 3d (1687mm) from end support (analysis 1)
	ALL Model 1	644	87	235	322	50	Pass	Shear @ 3d (3614mm) from intermediate support (analysis 2)
	ALL Model 2	732	65	309	374	51	Pass	Shear @ d (562mm) from end support (analysis 1)
	All Model 2	1132	244	281	525	46	Pass	Shear @ d (1204mm) from Intermediate support (analysis 2)

Carriageway Beam – Ultimate Limit State Shear								
	All Model 2	644	80	226	306	47	Pass	Shear @ 3d (1687mm) from end support (analysis 2)
	All Model 2	644	88	221	310	48	Pass	Shear @ 3d (3614mm) from intermediate support (analysis 2)
	ALL 1 + SV80	732	65	481	547	75	Pass	Shear @ d (562mm) from end support (analysis 1)
	ALL 1 + SV80	1132	244	457	701	62	Pass	Shear @ d (1204mm) from end support (analysis 2)
	ALL 1 + SV80	644	71	219	290	45	Pass	Shear @ 3d (1687mm) from end support (analysis 2)
	ALL 1 + SV80	644	88	360	448	70	Pass	Shear @ 3d (3614mm) from intermediate support (analysis 1)

Note: defects summarised in section 3.2 have been taken into account where applicable

6.6 Spans 2 to 4 – Kerb Beams - Summary Tables

Table 15: Spans 2 to 4 - Kerb Beams - Bending

Kerb Beam – Ultimate Limit State Bending									
Bending – Kerb Beam (effective width = 1730mm) **(method of assessment as described in section 5.4)**									
Element	Load case	Section Reference	Section Resistance (kNm)	Dead load + Superimposed Dead load (kNm)	Live Load Effect (kNm)	Total Load Effect (kNm)	Utilisation (%)	Result	Section
Assessment									
Main Spans 2 - 4	Accidental Vehicle Loading (Normal Traffic)	1	709	215	191	406	57	Pass	Max Sagging (Analysis 1) @ Mid Spans of Span 3
	ALL Model 1 + SV80	1	709	215	332	546	77	Pass	Max Sagging (Analysis 2) @ Mid Spans of Span 3
	ALL Model 1	1	709	215	221	436	62	Pass	Max Sagging (Analysis 2) @ Mid Spans of Span 3
	ALL Model 2	1	709	154	186	387	55	Pass	Max Sagging (Analysis 1) @ Mid Spans of Span 3
	ALL Model 1 + SV80 + Pedestrian	1	709	215	331	630	89	Pass	Max Sagging (Analysis 2) @ Mid Spans of Span 3

Kerb Beam – Ultimate Limit State Bending									
	Accidental Vehicle Loading (Normal Traffic)	5 & 8	-1651	-829	-440	-1269	77	Pass	Max Hogging (Analysis 2) @ Intermediate Supports
	ALL Model 1 + SV80	5 & 8	-1651	-829	-664	-1493	90	Pass	Max Hogging (Analysis 2) @ Intermediate Supports
	ALL Model 1	5 & 8	-1651	-829	-396	-1225	74	Pass	Max Hogging (Analysis 2) @ Intermediate Supports
	ALL Model 2	5 & 8	-1651	-829	-390	-1219	74	Pass	Max Hogging (Analysis 2) @ Intermediate Supports
	ALL Model 1 + SV80 + Pedestrian	5 & 8	-1651	-829	-782	-1611	98	Pass	Max Hogging (Analysis 2) @ Intermediate Supports

Note: defects summarised in section 3.2 have been taken into account where applicable

Table 16: Spans 2 to 4 - Kerb Beams - Shear

Kerb Beam – Ultimate Limit State Shear								
Shear – Kerb Beam (effective width = 610mm) **(method of assessment as described in section 5.4)**								
Element / Assessment	Load case	Section Resistance (kN)	Dead load + Superimposed Dead load (kN)	Live Load Effect (kN)	Total Load Effect (kN)	Utilisation (%)	Result	Section
Main Spans 2 -4	Accidental Vehicle Loading (Normal Traffic)	732	105	189	294	40	Pass	Shear @ d (562mm) from end support (analysis 1)
	Accidental Vehicle Loading (Normal Traffic)	1132	355	233	587	52	Pass	Shear @ d (1204mm) from Intermediate support (analysis 2)
	Accidental Vehicle Loading (Normal Traffic)	644	37	130	167	26	Pass	Shear @ 3d (1687mm) from end support (analysis 1)
	Accidental Vehicle Loading (Normal Traffic)	644	148	141	289	45	Pass	Shear @ 3d (3614mm) from intermediate support (analysis 2)
	ALL Model 1	732	94	175	269	37	Pass	Shear @ d (562mm) from end support (analysis 1)

Kerb Beam – Ultimate Limit State Shear									
	ALL Model 1	1132	346	223	569	50	Pass	Shear @ d (1204mm) from Intermediate support (analysis 2)
	ALL Model 1	644	117	101	218	34	Pass	Shear @ 3d (1687mm) from end support (analysis 2)
	ALL Model 1	644	126	122	249	39	Pass	Shear @ 3d (3614mm) from intermediate support (analysis 2)
	ALL Model 2	732	139	142	280	38	Pass	Shear @ d (562mm) from end support (analysis 1)
	ALL Model 2	1132	350	211	561	50	Pass	Shear @ d (1204mm) from Intermediate support (analysis 2)
	All Model 2	644	55	97	152	24	Pass	Shear @ 3d (1687mm) from end support (analysis 1)
	All Model 2	644	126	98	224	35	Pass	Shear @ 3d (3614mm) from intermediate support (analysis 2)
	ALL Model 1 + SV80	732	139	279	418	57	Pass	Shear @ d (562mm) from end support (analysis 1)

Kerb Beam – Ultimate Limit State Shear								
	ALL Model 1 + SV80	1132	336	450	786	69	Pass	Shear @ d (1204mm) from Intermediate support (analysis 1)
	ALL 1 + SV80	644	17	133	250	39	Pass	Shear @ 3d (1687mm) from end support (analysis 2)
	ALL 1 + SV80	644	126	169	296	46	Pass	Shear @ 3d (3614mm) from intermediate support (analysis 2)
	ALL Model 1 + SV80 + Pedestrian	732	139	302	441	60	Pass	Shear @ d (562mm) from end support (analysis 1)
	ALL Model 1 + SV80 + Pedestrian	1132	342	483	825	73	Pass	Shear @ d (1204mm) from Intermediate support (analysis 2)
	ALL Model 1 + SV80 + Pedestrian	644	117	133	250	39	Pass	Shear @ 3d (1687mm) from end support (analysis 2)
	ALL Model 1 + SV80 + Pedestrian	644	121	191	312	48	Pass	Shear @ 3d (3614mm) from intermediate support (analysis 1)

Note: defects summarised in section 3.2 have been taken into account where applicable

6.7 Spans 2 to 4 – Parapet Beams - Summary Tables

Table 17: Spans 2 to 4 - Parapet Beams – Bending

Parapet Beam – Ultimate Limit State Bending									
Bending – Parapet Beam (effective width = 1730mm) (method of assessment as described in section 5.4)									
Element	Load case	Section Reference	Section Resistance (kNm)	Dead load + Superimposed Dead load (kNm)	Live Load Effect (kNm)	Total Load Effect (kNm)	Utilisation (%)	Result	Section
Assessment									
Main Spans 2 -4	Accidental Vehicle Loading (Normal Traffic)	1	970	323	307	629	65	Pass	Max Sagging (Analysis 2) @ Mid Span of Span 3
	Accidental Vehicle Loading (Normal Traffic)	5 & 8	-1651	-806	-396	-1203	73	Pass	Max Hogging (Analysis 2) @ Intermediate Supports

Note: defects summarised in section 3.2 have been taken into account where applicable

Table 18: Spans 2 to 4 - Parapet Beams – Shear

Parapet Beam – Ultimate Limit State Shear								
Shear – Parapet Beam (effective width = 610mm) **(method of assessment as described in section 5.4)**								
Element	Load case	Section Resistance (kN)	Dead load + Superimposed Dead load (kN)	Live Load Effect (kN)	Total Load Effect (kN)	Utilisation (%)	Result	Section
Assessment								
Main Spans 2 -4	Accidental Vehicle Loading (Normal Traffic)	685	105	184	289	42	Pass	Shear @ d (562mm) from end support (analysis 1)
	Accidental Vehicle Loading (Normal Traffic)	1132	330	190	520	46	Pass	Shear @ d (1204mm) from Intermediate support (analysis 2)

Note: defects summarised in section 3.2 have been taken into account where applicable

6.8 Spans 2 to 4 - Carriageway Slab - Summary Tables

Table 19: Spans 2 to 4 - Carriageway Slab - Bending

Spans 2-4 – Carriageway Slab - Bending								
ULS Bending (for 1.00m width of slab)								
Element	Live Load Case	Section Resistance (kNm/m)	DL + SDL Effect (kNm/m)	Live Load Effect (kNm/m)	Total Load Effect (kNm/m)	Utilisation (%)	Result	Section
Carriageway slab (spans 2-4)	ALL model 1 (44t)	45.1	4.5	34.4	38.9	86	Pass	Midway between main beams (ULS sagging bending)
	ALL model 1 (44t)	-50.1	-3.0	-35.7	-38.7	77	Pass	At connection with main beam (ULS hogging bending)
	SV80	45.1	4.5	34.0	38.5	85	Pass	Midway between main beams (ULS sagging bending)
	SV80	-50.1	-3.0	-27.4	-30.4	61	Pass	At connection with main beam (ULS hogging bending)

Notes:

The bending moments in the slab due to wheel loading have been determined with the aid of Pucher Charts. For the maximum sagging moment, it has been conservatively assumed that the slab is simply supported between the main beams. For the maximum hogging moment, it has been conservatively assumed that there is full moment-fixity at the connection between the slab and each supporting main beam.

The defect identified in Appendix E of the AIP (exposed and corroded reinforcement to the soffit of the slab) has been taken into account when determining the bending (sagging) resistance of the slab.

Table 20: Spans 2 to 4 - Carriageway Slab - Shear

Spans 2-4 – Carriageway Slab - Shear								
ULS Shear (for 1.00m width of slab)								
Element	Live Load Case	Section Resistance (kN/m)	DL + SDL Effect (kN/m)	Live Load Effect (kN/m)	Total Load Effect (kN/m)	Utilisation (%)	Result	Section
Carriageway slab (spans 2-4)	ALL model 1 (44t)	166.3	7.7	144.7	152.4	92	Pass	At d from connection with main beam (ULS shear)
	ALL model 1 (44t)	121.8	5.5	96.4	101.9	84	Pass	At 3d from connection with main beam (ULS shear)
	SV80	166.3	7.7	105.3	113.0	68	Pass	At d from connection with main beam (ULS shear)
	SV80	121.8	5.5	70.0	75.5	62	Pass	At 3d from connection with main beam (ULS shear)

6.9 Spans 2 to 4 - Service Bay Slab - Summary Tables

Table 21: Spans 2 to 4 – Service Bay Slab – Bending

Spans 2-4 - Service Bay Slab - Bending								
ULS Bending (for 1.00m width of slab)								
Element	Live Load Case	Section Resistance (kNm/m)	DL + SDL Effect (kNm/m)	Live Load Effect (kNm/m)	Total Load Effect (kNm/m)	Utilisation (%)	Result	Section
Service bay slab (spans 2-4)	AVL (7.5t)	17.0	12.1	11.0	23.1	136	Fail	Midway between main beams (ULS sagging bending)
	AVL (3t)	17.0	12.1	4.0	16.1	95	Pass	
	Pedestrian loading	17.0	12.1	3.7	15.8	93	Pass	

<u>**Notes:**</u>

AVL = accidental vehicle loading

The bending moments in the slab due to wheel loading have been determined with the aid of Pucher Charts. Based on the reinforcement detailing shown on the record drawings (the top main reinforcement has half of the area of the bottom main reinforcement), it appears that the original design intent was for the slab to be a simply-supported element. The slab has been assessed on the same basis, which is considered to be a reasonable approach.

Table 22: Spans 2 to 4 – Service Bay Slab – Shear (at d from connection with main beam)

Spans 2-4 - Service Bay Slab – Shear (at d from connection with main beam)								
ULS Shear (for 1.00m width of slab)								
Element	Live Load Case	Section Resistance (kN/m)	DL + SDL Effect (kN/m)	Live Load Effect (kN/m)	Total Load Effect (kN/m)	Utilisation (%)	Result	Section
Service bay slab (spans 2-4)	AVL (18t)	95.9	22.5	102.6	125.1	130	Fail	At d from connection with main beam (ULS shear)
	AVL (7.5t)	95.9	22.5	53.7	76.2	79	Pass	
	AVL (3t)	95.9	22.5	19.0	41.5	43	Pass	
	Pedestrian loading	95.9	22.5	7.0	29.5	31	Pass	

Table 23: Spans 2 to 4 – Service Bay Slab – Shear (at 3d from connection with main beam)

Spans 2-4 - Service Bay Slab – Shear (at 3d from connection with main beam)								
ULS Shear (for 1.00m width of slab)								
Element	Live Load Case	Section Resistance (kN/m)	DL + SDL Effect (kN/m)	Live Load Effect (kN/m)	Total Load Effect (kN/m)	Utilisation (%)	Result	Section
Service bay slab (spans 2-4)	AVL (18t)	79.8	16.6	85.0	101.6	127	Fail	At 3d from connection with main beam (ULS shear)
	AVL (7.5t)	79.8	16.6	44.5	61.1	77	Pass	
	AVL (3t)	79.8	16.6	15.8	32.4	41	Pass	
	Pedestrian loading	79.8	16.6	5.1	21.7	27	Pass	

6.10 Abutment Columns - Summary Table

Table 24: Abutment Columns – Summary Table

Abutment Column – Ultimate Limit State Axial, Bending and Shear								
Effective Length = 4572mm								
Element	Load case	Section Resistance	Dead load + Superimposed Dead load	Live Load Effect	Total Load Effect	Utilisation (%)	Result	Section
Assessment								
Main Spans 2 -4	ALL Model 1 + SV80	1216 kN	125 kN	929 kN	1053 kN	87	Pass	Axial
	ALL Model 1 + SV80	98 kNm	5 kNm	69 kNm	74 kNm	75	Pass	Bending
	Braking Force	65 kN	0 kN	61 kN	61 kN	93	Pass	Shear

6.11 Intermediate-Pier Columns - Summary Table

Table 25: Intermediate-Pier Columns – Summary Table

Intermediate Column – Ultimate Limit State Axial and Shear								
Effective Length = 5510								
Element	Load case	Section Resistance (kN)	Dead load + Superimposed Dead load (kN)	Live Load Effect (kN)	Total Load Effect (kN)	Utilisation (%)	Result	Section
Assessment								
Main Spans 2 -4	ALL Model 1 + SV80	2526	525	1190	1715	68	Pass	Axial
	Braking Force	151	0	61	61	40	Pass	Shear

Table 26: Qualitative Assessment - Summary Table

Qualitative Assessment Findings		
Element(s)	Pass/ Fail	Comments
West approach-span abutment (original part)	Fail	Settlement of the main part of the abutment is evident (refer to section 6.12 for details)
West approach-span abutment (reconstructed part supporting south footway)	Pass	Adequate for current loading
East approach-span abutment	Fail	Settlement of the northern part of the abutment is evident (refer to section 6.12 for details)
North-east wing wall and associated parapet	Fail	Significant outward movement of the wing wall and parapet is evident (approximately 175mm movement at the top of the parapet). This movement is currently being monitored by ECC.
Other wing walls and associated parapets	Pass	Adequate for current loading
Curtain walls (main-span abutments)	Pass	Adequate for current loading
Foundations to main-span abutments and intermediate piers	Pass	Adequate for current loading

6.12 Qualitative Assessment of Approach-Span Abutments

The part of the west approach-span abutment supporting the carriageway slab of span 1 has undergone settlement, and the western end of the slab does not appear to be supported by the abutment and instead appears to be supported by the adjacent run-on slab. Based on this, the main part of the abutment is not considered to be adequate for current loading. The reconstructed part of the west approach-span abutment, which supports the south footway, is considered to be adequate for current loading.

The part of the east approach-span abutment supporting the north footway has undergone settlement. This has caused the north-east footway slab (span 5) to displace horizontally away from the main-span abutment bearing shelf. Based on this, this part of the east approach-span abutment is not considered to be adequate for current loading. It should be noted that in July 2023, after the IFA was undertaken, ECC installed temporary propping to the north-east footway slab.

6.13 Parapet Assessment

The parapets were risk-assessed in accordance with section 3 of CS 461. This risk assessment concluded that an N1/N2 upgrade is recommended.

6.14 Intermediate-Pier Column Hinge Assessment

The throat of the hinge at the base of each column has been assessed quantitatively in accordance with the basic principles of CS 455 and CS 468. The compressive resistance of the throat was determined using the guidance for rectangular hinges in CS 468.

Intermediate-Pier Column Hinge Assessment Results				
Element	Resistance (kN)	Applied Axial Force (kN)	Utilisation (%)	Pass / Fail
Intermediate-Pier Column Hinge	774	525	68	Pass

6.15 Abutment Column Hinge Assessment

The compressive resistance of the concrete block in each hinge has been assessed in accordance with the basic principles of CS 455. The shear resistance of the steel dowel bar in each hinge has been determined using guidance given in Concrete Society Technical Report No. 34.

Abutment Column Hinge Assessment Results				
Component/ effect	Resistance	Applied Effect	Utilisation (%)	Pass / Fail
Dowel bar/ shear	243 kN	61 kN	25	Pass
Concrete block/ compression	15.0 N/mm^2	11.3 N/mm^2	75	Pass

6.16 Dowel Bar Assessment (Carriageway Slab of Span 1)

The steel dowel bars which connect the western end of the carriageway slab of span 1 to the adjacent run-on slab have assessed quantitatively and have been found to be adequate for permanent loading only.

7. Category II Check Results

7.1 Spans 1 & 5 – Original Footway Slab – Summary Tables (Cat II Check)

Table 27: Spans 1 & 5 – Original Footway Slab – Main Slab – Bending (Cat II Check)

Span 1 - Original Footway Slab – Main Slab – Transverse Bending								
ULS Bending (Transverse) for 1.00m Width of Slab								
Element	Live Load Case	Section Resistance (kNm/m)	DL + SDL Effect (kNm/m)	Live Load Effect (kNm/m)	Total Load Effect (kNm/m)	Utilisation (%)	Result	Section
Main slab	Pedestrian loading	17.4	13.2	4.1	17.3	99	Pass	Midspan (ULS sagging bending)

Table 28: Spans 1 & 5 – Original Footway Slab – Main Slab – Shear (Cat II Check)

Span 1 - Original Footway Slab – Main Slab - Shear								
ULS Shear for 1.00m Width of Slab								
Element	Live Load Case	Section Resistance (kN/m)	DL + SDL Effect (kN/m)	Live Load Effect (kN/m)	Total Load Effect (kN/m)	Utilisation* (%)	Result	Section
Main Slab	Pedestrian loading	71.7	26.6	8.3	34.9	49	Pass	Support (ULS shear)*

<u>Notes (Tables 27 & 28):</u>

The original footway slab (incorporating the main slab and longitudinal downstands) has been assessed (for the cat II check) as follows:
Main slab assumed to span transversely between longitudinal downstands
Longitudinal downstands assumed to support loading from transversely-spanning main slab

***Utilisation is based on applied shear force at support and shear capacity at 3d from support and is therefore conservative.**

Accidental vehicle loading has not been considered because slab has been found to be only just adequate for permanent loading combined with pedestrian loading (as shown in Table 27). By inspection, slab is not adequate for any level of accidental vehicle loading.

Table 29: Spans 1 & 5 – Original Footway Slab – Downstand – Bending (Cat II Check)

Span 1 - Original Footway Slab – Downstand – Longitudinal Bending								
ULS Bending (Longitudinal)								
Element	Live Load Case	Section Resistance* (kNm)	DL + SDL Effect* (kNm)	Live Load Effect* (kNm)	Total Load Effect* (kNm)	Utilisation (%)	Result	Section
Downstand	Pedestrian loading	32.0	24.6	7.4	32.0	100	Pass	Midspan (ULS sagging bending)

Table 30: Spans 1 & 5 – Original Footway Slab – Downstand – Shear (Cat II Check)

Span 1 - Original Footway Slab – Downstand - Shear								
ULS Shear								
Element	Live Load Case	Section Resistance* (kN)	DL + SDL Effect* (kN)	Live Load Effect* (kN)	Total Load Effect* (kN)	Utilisation (%)	Result	Section
Downstand	Pedestrian loading	31.0	19.5	5.8	25.3	82	Pass	3d from support (ULS shear)

Notes (Tables 29 & 30):

The original footway slab (incorporating the main slab and longitudinal downstands) has been assessed (for the cat II check) as follows:
Main slab assumed to span transversely between longitudinal downstands
Longitudinal downstands assumed to support loading from transversely-spanning main slab

***Section resistances and load effects relate to actual width of downstand section (0.20m wide at bottom of section)**

Accidental vehicle loading has not been considered because slab has been found to be only just adequate for permanent loading combined with pedestrian loading (as shown in Table 29). By inspection, slab is not adequate for any level of accidental vehicle loading.

7.2 Span 1 – Carriageway Slab - Summary Tables (Cat II Check)

Table 31: Span 1 – Carriageway Slab – Bending (Cat II Check)

Deck Information: Span 1 Carriageway Slab								
Bending (mid span of carriageway slab)								
Element	Load case	Section Resistance *(kNm)	Dead load + Superimposed Dead load* (kNm)	Live Load Effect* (kNm)	Total Load Effect* (kNm)	Utilisation (%)	Result	Section
Carriageway Slab – Span 1	ALL Model 1	26	3	22	25	98	Pass	Bending Longitudinal (Sagging)
	ALL Model 1	15	0.1	8	8	57	Pass	Bending Transverse (Sagging)
	ALL Model 2	26	4	21	25	98	Pass	Bending Longitudinal (Sagging)
	ALL Model 2	15	2	7	9	59	Pass	Bending Transverse (Sagging)
	ALL Model 1 + SV80	26	4	19	23	90	Pass	Bending Longitudinal (Sagging)
	ALL Model 1 + SV80	15	0.1	14	14	99	Pass	Bending Transverse (Sagging)

***Section resistances and load effects relate to width of member in grillage model (refer to calculations for further details)**

The results above are based on the assumption that the slab is adequately supported at both ends (currently, the dowelled connection at the western end of the slab does not provide adequate support)

Table 32: Span 1 – Carriageway Slab – Shear (Cat II Check)

Deck Information: Span 1 Carriageway Slab								
Shear @ d from support								
Element	Load case	Section Resistance* (kN)	Dead load + Superimposed Dead load* (kN)	Live Load Effect* (kN)	Total Load Effect* (kN)	Utilisation (%)	Result	Section
Carriageway Slab – Span 1	ALL Model 1	78	5	48	52	67	Pass	d from support
	ALL Model 2	78	9	41	51	64	Pass	d from support
	ALL Model 1 + SV80	78	5	59	63	81	Pass	d from support

***Section resistances and load effects relate to width of member in grillage model (refer to calculations for further details)**

The results above are based on the assumption that the slab is adequately supported at both ends (currently, the dowelled connection at the western end of the slab does not provide adequate support)

7.3 Span 5 – Carriageway Slab - Summary Tables (Cat II Check)

Table 33: Span 5 – Carriageway Slab – Bending (Cat II Check)

Deck Information: Span 5 Carriageway Slab								
Bending (mid span of carriageway slab)								
Element	Load case	Section Resistance* (kNm)	Dead load + Superimposed Dead load* (kNm)	Live Load Effect* (kNm)	Total Load Effect* (kNm)	Utilisation (%)	Result	Section
Carriageway Slab – Span 5	ALL Model 1	26	3	22	25	99	Pass	Bending Longitudinal (Sagging)
	ALL Model 1	15	0.1	8	8	58	Pass	Bending Transverse (Sagging)
	ALL Model 2	26	4	21	25	98	Pass	Bending Longitudinal (Sagging)
	ALL Model 2	15	0.1	9	9	61	Pass	Bending Transverse (Sagging)
	ALL Model 1 + SV80	26	4	19	23	90	Pass	Bending Longitudinal (Sagging)
	ALL Model 1 + SV80	15	0.1	14	14	98	Pass	Bending Transverse (Sagging)

***Section resistances and load effects relate to width of member in grillage model (refer to calculations for further details)**

Table 34: Span 5 – Carriageway Slab – Shear (Cat II Check)

Deck Information: Span 5 Carriageway Slab								
Shear @ d from support								
Element	Load case	Section Resistance* (kN)	Dead load + Superimposed Dead load* (kN)	Live Load Effect* (kN)	Total Load Effect* (kN)	Utilisation (%)	Result	Section
Carriageway Slab – Span 5	ALL Model 1	78	5	48	53	68	Pass	d from support
	ALL Model 2	78	9	41	51	65	Pass	d from support
	ALL Model 1 + SV80	78	5	51	56	71	Pass	d from support

***Section resistances and load effects relate to width of member in grillage model (refer to calculations for further details)**

7.4 Span 1 – South-West Footway Slab - Summary Tables (Cat II Check)

Table 35: Span 1 – South-West Footway Slab – Bending (Cat II Check)

Deck Information: Approach Span 1 – South West Footway Slab								
Bending (mid span of slab)								
Element	Load case	Section Resistance* (kNm)	Dead load + Superimposed Dead load* (kNm)	Live Load Effect* (kNm)	Total Load Effect* (kNm)	Utilisation (%)	Result	Section
South West Footway Slab – Span 1	Pedestrian Load	127	27	6	33	26	Pass	Longitudinal Bending of edge member (Sagging)
	Pedestrian Load	42	4	1	5	12	Pass	Longitudinal Bending (sagging)
	Pedestrian Load	35	3	1	4	12	Pass	Transverse bending (sagging)
	Accidental Vehicle Loading (Normal Traffic)	127	27	42	69	54	Pass	Longitudinal Bending of edge member (Sagging)
	Accidental Vehicle Loading (Normal Traffic)	42	4	6	10	24	Pass	Longitudinal Bending (sagging)
	Accidental Vehicle Loading (Normal Traffic)	35	3	5	7	21	Pass	Transverse bending (sagging)

***Section resistances and load effects relate to width of member in grillage model (refer to calculations for further details)**

Table 36: Span 1 – South-West Footway Slab – Shear (Cat II Check)

Deck Information: Approach Span 1 – South West Footway Slab								
Shear @ d from support								
Element	Load case	Section Resistance* (kN)	Dead load + Superimposed Dead load* (kN)	Live Load Effect* (kN)	Total Load Effect* (kN)	Utilisation (%)	Result	Section
South West Footway Slab – Span 1	Pedestrian Load	119	45	9	54	46	Pass	d from support – Longitudinal edge member
	Pedestrian Load	78	9	2	11	15	Pass	d from support – Longitudinal member
	Accidental Vehicle Loading (Normal Traffic)	119	47	51	98	82	Pass	d from support – Longitudinal edge member
	Accidental Vehicle Loading (Normal Traffic)	78	10	12	22	28	Pass	d from support – Longitudinal member

***Section resistances and load effects relate to width of member in grillage model (refer to calculations for further details)**

7.5 Spans 2 to 4 – Carriageway Beams - Summary Tables (Cat II Check)

Table 37: Spans 2 to 4 – Carriageway Beams – Bending (Cat II Check)

Carriageway Beam – Ultimate Limit State Bending									
Bending – Carriageway Beam (effective width = 1730mm)									
Element / Assessment	Load case	Section Reference	Section Resistance (kNm)	Dead load + Superimposed Dead load (kNm)	Live Load Effect (kNm)	Total Load Effect (kNm)	Utilisation (%)	Result	Section
Carriageway Beam (Span 2-4)	ALL Model 1 + SV80	1	605	102.7	386.9	489.6	80.9	Pass	Sagging @ Mid Span of Span 3 (Analysis 2)
	ALL Model 1	1	605	102.7	314.3	417.0	68.9	Pass	Sagging @ Mid Span of Span 3 (Analysis 2)
	ALL Model 2	1	605	102.7	296.8	399.5	66.0	Pass	Sagging @ Mid Span of Span 3 (Analysis 2)
	ALL Model 1 + SV80	5 & 8	-1655	-521.3	-880	-1401.3	84.7	Pass	Hogging @ Intermediate Supports (Analysis 2)
	ALL Model 1 – 44 Tonnes	5 & 8	-1655	-521.3	-734.7	-1256	75.9	Pass	Hogging @ Intermediate Supports (Analysis 2)
	ALL Model 2	5 & 8	-1655	-521.3	-594.9	-1116.2	67.4	Pass	Hogging @ Intermediate Supports (Analysis 2)

Note: defects summarised in section 3.2 have been taken into account where applicable

Table 38: Spans 2 to 4 – Carriageway Beams – Shear (Cat II Check)

Carriageway Beam – Ultimate Limit State Shear								
Shear – Carriageway Beam (effective width = 610mm)								
Element	Load case	Section Resistance (kN)	Dead load + Superimposed Dead load (kN)	Live Load Effect (kN)	Total Load Effect (kN)	Utilisation (%)	Result	Section
Assessment								
Carriageway Beam (Span 2 -4)	ALL Model 1	776	69.0	219.9	288.9	46.5	Pass	Shear @ d from End Support (analysis 1)
	ALL Model 1	1107	214.7	454.9	672.6	60.5	Pass	Shear @ d from Intermediate Support (analysis 2)
	ALL Model 1	629	31.4	197.4	228.8	36.4	Pass	Shear @ 3d from End Support (analysis 1)
	ALL Model 1	629	82.6	388.2	470.8	74.8	Pass	Shear @ 3d from Intermediate Support (analysis 2)
	ALL Model 2	776	69	242.6	311.6	40.0	Pass	Shear @ d from End Support (analysis 1)
	All Model 2	1107	214.7	270.8	485.5	43.8	Pass	Shear @ d from Intermediate Support (analysis 2)

Carriageway Beam – Ultimate Limit State Shear									
	All Model 2	629	35	123.5	158.5	25.2	Pass	Shear @ 3d from End Support (analysis 2)
	All Model 2	629	82.6	166.4	249.0	39.6	Pass	Shear @ 3d from Intermediate Support (analysis 2)
	ALL Model 1 + SV80	776	69	359.7	428.7	55.2	Pass	Shear @ d from End Support (analysis 1)
	ALL Model 1 + SV80	1107	138.3	441.1	579.4	52.3	Pass	Shear @ d from Intermediate Support (analysis 1)
	ALL Model 1 + SV80	629	35	168.8	203.8	32.4	Pass	Shear @ 3d from End Support (analysis 2)
	ALL Model 1 + SV80	629	96.5	259.8	356.3	56.6	Pass	Shear @ 3d from Intermediate Support (analysis 1)

Note: defects summarised in section 3.2 have been taken into account where applicable

7.6 Spans 2 to 4 – Kerb Beams - Summary Tables (Cat II Check)

Table 39: Spans 2 to 4 – Kerb Beams – Bending (Cat II Check)

Kerb Beam – Ultimate Limit State Bending									
Bending – Kerb Beam (effective width = 1730mm)									
Element / Assessment	Load case	Section Reference	Section Resistance (kNm)	Dead load + Superimposed Dead load (kNm)	Live Load Effect (kNm)	Total Load Effect (kNm)	Utilisation (%)	Result	Section
Kerb Beam (Spans 2 -4)	ALL Model 1 +SV80 + Pedestrian	1	679	207.2	320.9	528.1	77.8	Pass	Sagging @ Mid Span of Span 3 (Analysis 1)
	ALL Model 1	1	679	207.2	221	428.2	59.7	Pass	Sagging @ Mid Spans of Span 3 (Analysis 1)
	Accidental Vehicle Loading (Normal Traffic)	1	679	168.6	207.6	376.2	55.4	Pass	Sagging @ Mid Spans of Span 3 (Analysis 2)
	ALL Model 2	1	679	165.5	173.7	339.2	50	Pass	Sagging @ Mid Spans of Span 3 (Analysis 1)

Kerb Beam – Ultimate Limit State Bending									
	ALL Model 1 +SV80 + Pedestrian	5 & 8	-1655	-809.4	-803.0	-1612.4	97.4	Pass	Hogging @ Intermediate Supports (Analysis 2)
	Accidental Vehicle Loading (Normal Traffic)	5 & 8	-1655	-890	-464.3	-1354.3	81.8	Pass	Hogging @ Intermediate Support (Analysis 2)
	ALL Model 1	5 & 8	-1655	-890	-432.6	-1322.6	80.0	Pass	Hogging @ Intermediate Supports (Analysis 2)
	ALL Model 2	5 & 8	-1655	-809.4	-377.5	-1186.9	71.7	Pass	Hogging @ Intermediate Supports (Analysis 1)

Note: defects summarised in section 3.2 have been taken into account where applicable

Table 40: Spans 2 to 4 – Kerb Beams – Shear (Cat II Check)

Kerb Beam – Ultimate Limit State Shear								
Shear – Kerb Beam (effective width = 610mm)								
Element / Assessment	Load case	Section Resistance (kN)	Dead load + Superimposed Dead load (kN)	Live Load Effect (kN)	Total Load Effect (kN)	Utilisation (%)	Result	Section
Kerb Beam (Spans 2 -4)	ALL Model 1 + SV80 + Pedestrian	776	105.2	295.5	400.7	51.6	Pass	Shear @ d from End Support (analysis 1)
	Accidental Vehicle Loading (Normal Traffic)	776	116	191.2	307.2	39.6	Pass	Shear @ d from End Support (analysis 1)
	ALL Model 1	776	105.2	178.2	283.4	36.5	Pass	Shear @ d from End Support (analysis 1)
	ALL Model 2	776	105.2	154.7	259.9	33.5	Pass	Shear @ d from End Support (analysis 1)
	ALL Model 1 + SV80 + Pedestrian	1107	353.3	404.5	757.8	68.4	Pass	Shear @ d from Intermediate Support (analysis 2)

Kerb Beam – Ultimate Limit State Shear								
	Accidental Vehicle Loading (Normal Traffic)	1107	358.4	224.3	582.7	52.6	Pass	Shear @ d from Intermediate Support (analysis 2)
	ALL Model 1	1107	353.3	197.8	551.1	49.8	Pass	Shear @ d from Intermediate Support (analysis 2)
	ALL Model 2	1107	353.3	177.2	530.5	47.9	Pass	Shear @ d from Intermediate Support (analysis 2)

Note: defects summarised in section 3.2 have been taken into account where applicable

7.7 Spans 2 to 4 – Parapet Beams - Summary Tables (Cat II Check)

Table 41: Spans 2 to 4 – Parapet Beams – Bending (Cat II Check)

Parapet Beam – Ultimate Limit State Bending									
Bending – Parapet Beam (effective width = 1050mm)									
Element / Assessment	Load case	Section Reference	Section Resistance (kNm)	Dead load + Superimposed Dead load (kNm)	Live Load Effect (kNm)	Total Load Effect (kNm)	Utilisation (%)	Result	Section
Parapet Beam (Spans 2 – 4)	ALL Model 1 + SV80 + Pedestrian	1	980	326.7	204.5	531.2	54.2	Pass	Sagging @ Mid Span of Span 3 (Analysis 1)
	Accidental Loading (Normal Traffic)	1	980	306.4	306.7	613.1	62.5	Pass	Sagging @ Mid Span of Span 3 (Analysis 2)
	ALL Model 1	1	980	326.7	42.9	369.6	37.7	Pass	Sagging @ Mid Span of Span 3 (Analysis 1)
	ALL Model 1 + SV80 + Pedestrian	5 & 8	-1655	-837.1	-284.8	-1121.9	67.4	Pass	Hogging @ Intermediate Supports (Analysis 2)
	Accidental Loading (Normal Traffic)	5 & 8	-1655	-837.1	-414.7	-1251.5	75.2	Pass	Hogging @ Intermediate Supports (Analysis 2)
	ALL Model 1	5 & 8	-1655	-837.1	-81.7	-918.8	55.2	Pass	Hogging @ Intermediate Supports (Analysis 2)

Note: defects summarised in section 3.2 have been taken into account where applicable

Table 42: Spans 2 to 4 – Parapet Beams – Shear (Cat II Check)

Parapet Beam – Ultimate Limit State Shear								
Shear – Parapet Beam (effective width = 610mm)								
Element / Assessment	Load case	Section Resistance (kN)	Dead load + Superimposed Dead load (kN)	Live Load Effect (kN)	Total Load Effect (kN)	Utilisation (%)	Result	Section
Parapet Beam (Spans 2 -4)	Accidental Vehicle Loading (Normal Traffic)	652	97.5	183.6	281.1	43.1	Pass	Shear @ d from End Support (analysis 1)
	Accidental Vehicle Loading (Normal Traffic)	1107	322.8	199.3	522.1	47.2	Pass	Shear @ d from Intermediate Support (analysis 2)

Note: defects summarised in section 3.2 have been taken into account where applicable

7.8 Spans 2 to 4 – Carriageway Slab - Summary Tables (Cat II Check)

Table 43: Spans 2 to 4 - Carriageway Slab - Bending (Cat II Check)

Spans 2-4 – Carriageway Slab - Bending								
ULS Bending (for 1.00m width of slab)								
Element	Live Load Case	Section Resistance (kNm/m)	DL + SDL Effect (kNm/m)	Live Load Effect (kNm/m)	Total Load Effect (kNm/m)	Utilisation (%)	Result	Section
Carriageway slab (spans 2-4)	ALL model 1 (44t)	44.8	4.5	37.7	42.2	94	Pass	Midway between main beams (ULS sagging bending)
	ALL model 1 (44t)	-50.4	-3.0	-37.2	-40.2	80	Pass	At connection with main beam (ULS hogging bending)
	SV80	44.8	4.5	36.2	40.7	91	Pass	Midway between main beams (ULS sagging bending)
	SV80	-50.4	-3.0	-28.5	-31.5	63	Pass	At connection with main beam (ULS hogging bending)

Notes:

The bending moments in the slab due to wheel loading have been determined with the aid of Pucher Charts. For the maximum sagging moment, it has been conservatively assumed that the slab is simply supported between the main beams. For the maximum hogging moment, it has been conservatively assumed that there is full moment-fixity at the connection between the slab and each supporting main beam.

The defect identified in Appendix E of the AIP (exposed and corroded reinforcement to the soffit of the slab) has been taken into account when determining the bending (sagging) resistance of the slab.

Table 44: Spans 2 to 4 - Carriageway Slab – Shear (Cat II Check)

Spans 2-4 – Carriageway Slab - Shear								
ULS Shear (for 1.00m width of slab)								
Element	Live Load Case	Section Resistance (kN/m)	DL + SDL Effect (kN/m)	Live Load Effect (kN/m)	Total Load Effect (kN/m)	Utilisation (%)	Result	Section
Carriageway slab (spans 2-4)	ALL model 1 (44t)	121.0	5.0	97.0	102.0	84	Pass	At 3d from connection with main beam (ULS shear)
	SV80	121.0	5.0	78.0	83.0	69	Pass	At 3d from connection with main beam (ULS shear)

7.9 Spans 2 to 4 – Service Bay Slab - Summary Tables (Cat II Check)

Table 45: Spans 2 to 4 – Service Bay Slab – Bending (Cat II Check)

Spans 2-4 - Service Bay Slab - Bending								
ULS Bending (for 1.00m width of slab)								
Element	Live Load Case	Section Resistance (kNm/m)	DL + SDL Effect (kNm/m)	Live Load Effect (kNm/m)	Total Load Effect (kNm/m)	Utilisation (%)	Result	Section
Service bay slab (spans 2-4)	AVL (7.5t)	16.8	12.0	11.0	23.0	137	Fail	Midway between main beams (ULS sagging bending)
	AVL (3t)	16.8	12.0	3.9	15.9	95	Pass	
	Pedestrian loading	16.8	12.0	3.7	15.7	93	Pass	

<u>**Notes:**</u>

AVL = accidental vehicle loading

The bending moments in the slab due to wheel loading have been determined with the aid of Pucher Charts. Based on the reinforcement detailing shown on the record drawings (the top main reinforcement has half of the area of the bottom main reinforcement), it appears that the original design intent was for the slab to be a simply-supported element. The slab has been assessed on the same basis, which is considered to be a reasonable approach.

Table 46: Spans 2 to 4 – Service Bay Slab – Shear (at 3d from connection with main beam) (Cat II Check)

Spans 2-4 - Service Bay Slab – Shear (at 3d from connection with main beam)								
ULS Shear (for 1.00m width of slab)								
Element	Live Load Case	Section Resistance (kN/m)	DL + SDL Effect (kN/m)	Live Load Effect (kN/m)	Total Load Effect (kN/m)	Utilisation (%)	Result	Section
Service bay slab (spans 2-4)	AVL (18t)	79.3	16.5	85.0	101.5	128	Fail	At 3d from connection with main beam (ULS shear)
	AVL (7.5t)	79.3	16.5	44.4	60.9	77	Pass	
	AVL (3t)	79.3	16.5	15.8	32.3	41	Pass	
	Pedestrian loading	79.3	16.5	5.1	21.6	27	Pass	

7.10 Columns - Summary Tables (Cat II Check)

Table 47: Abutment Columns – Summary Table (Cat II Check)

Abutment Column – Ultimate Limit State Axial, Bending and Shear								
Element	Load case	Section Resistance	Dead load + Superimposed Dead load	Live Load Effect	Total Load Effect	Utilisation (%)	Result	Section
Assessment								
Abutment Column	ALL Model 1 + SV80	1151	129.6	838.2	967.8	84.0	Pass	Axial (kN)
	ALL Model 1 + SV80	87	5.4	61.3	66.7	76.7	Pass	Bending (kN.m)
	Braking Force	66	0	61	61	91.9	Pass	Shear (kN)

Table 48: Intermediate-Pier Columns – Summary Table (Cat II Check)

Intermediate Column – Ultimate Limit State Axial and Shear								
Element	Load case	Section Resistance (kN)	Dead load + Superimposed Dead load (kN)	Live Load Effect (kN)	Total Load Effect (kN)	Utilisation (%)	Result	Section
Assessment								
Intermediate Column	ALL Model 1 + SV80	2591	491	1221	1712	66	Pass	Axial
	Braking Force	164	0	60.7	60.7	37	Pass	Shear

8. Conclusions

With the exception of the carriageway slab of span 1, all of the superstructure elements supporting the carriageway have been assessed as being adequate for 44 tonnes assessment live loading and SV80 loading. The carriageway slab of span 1 has been assessed as being adequate for permanent loading only, due to the capacity of the dowelled connection with the adjacent run-on slab. It should be noted that if the carriageway slab of span 1 was adequately supported at both ends, it would be adequate for 44 tonnes assessment live loading and SV80 loading.

The superstructure elements supporting the footways have been assessed as being adequate for the following loading:

South-west footway slab (span 1):	Accidental vehicle loading (normal traffic) or pedestrian live loading
Original footway slabs (spans 1 and 5):	Pedestrian live loading only (refer to note below regarding north-east footway slab)
Service bay slabs (spans 2 to 4):	Restricted accidental vehicle loading (3t gross vehicle weight) or pedestrian live loading

Note: the change in support conditions resulting from the introduction of the propping to the north-east footway slab has not been considered in the assessment. For the assessment, it has been assumed that the slab is simply supported, and the capacity stated above is dependent on the slab being adequately supported by both abutments (which was the case when the IFA was undertaken).

Based on a risk assessment in accordance with section 3 of CS 461, an N1/N2 upgrade of the parapets is recommended.

The abutment columns and intermediate-pier columns have been assessed as being adequate for 44 tonnes assessment live loading and SV80 loading.

Based on a qualitative assessment, the west approach-span abutment, east approach-span abutment and north-east wing wall are not considered to be adequate for current loading, due to the settlement and movement observed.

Based on a qualitative assessment, the curtain walls, wing walls (other than the north-east wing wall) and foundations to the main-span abutments and intermediate piers are adequate for current loading.

9. Recommendations

With regard to the original footway slabs of spans 1 and 5, and the service bay slabs of spans 2 to 4, it is considered that no significant benefit would be gained from undertaking any intrusive investigation works and/ or a reassessment using a more rigorous method of analysis.

It is recommended that a review in accordance with CS 470 is undertaken, in which the following are considered:

- Interim measures to protect the footways from accidental vehicle loading in the short term (e.g. the installation of TVCBs).
- Remedial works to the footway-supporting elements, so that accidental vehicle loading (normal traffic) can be accommodated, or long-term measures to protect the footways from accidental vehicle loading.
- Interim monitoring of the settlement of the west approach-span abutment, followed by remedial works (both structural and geotechnical) to the abutment, to restore the support to the western end of the carriageway slab of span 1 and prevent further settlement of the abutment.
- Interim monitoring of the settlement of the east approach-span abutment, followed by remedial works to the abutment, if deemed necessary following monitoring and further investigation.
- Interim monitoring of the movement of the north-east wing wall, followed by remedial works (both structural and geotechnical) to the wing wall.
- Remedial works to the north-east footway slab, to restore the support to the western end of the slab.
- An upgrade of the parapets to N1/ N2 containment level.

10. Assessment & Check Certificate

Project Details:

Name of Project: Station Way Bridge Assessment

Name of Structure: Station Way Bridge

Structure Ref No. 2100

Section 1

We certify that reasonable professional skill and care have been used in the preparation of the assessment of Station Way Bridge with a view to securing that:

1. It has been assessed in accordance with:

   The Approval in Principle dated 14/05/2024 (date of acceptance) including the following:

   The hinges located at the base of the columns (abutments and intermediate piers) have been assessed quantitatively in accordance with the basic principles of CS 455 and CS 468.

   The steel dowel bars have been assessed using guidance given in Concrete Society Technical Report No. 34.

   The deck slab and service bay slabs of spans 2 to 4 have been assessed using Pucher Charts.

2. The assessed capacity of the structure is as follows:

The structure is adequate for permanent loading only (refer below for further details)

The carriageway slab of span 1 has been assessed as being adequate for **permanent loading only**, due to the capacity of the dowelled connection with the adjacent run-on slab. All other superstructure elements supporting the carriageway have been assessed as being adequate for 44 tonnes assessment live loading and SV80 loading. It should be noted that if the carriageway slab of span 1 was adequately supported at both ends, it would be adequate for 44 tonnes assessment live loading and SV80 loading.

The superstructure elements supporting the footways have been assessed as being adequate for the following loading:

South-west footway slab (span 1):	Accidental vehicle loading (normal traffic) or pedestrian live loading
Original footway slabs (spans 1 and 5):	Pedestrian live loading only (refer to note below regarding north-east footway slab)
Service bay slabs (spans 2 to 4):	Restricted accidental vehicle loading (3t gross vehicle weight) or pedestrian live loading

Note: the change in support conditions resulting from the introduction of the propping to the north-east footway slab has not been considered in the assessment. For the assessment, it has been assumed that the slab is simply supported, and the capacity stated above is dependent on the slab being adequately supported by both abutments (which was the case when the IFA was undertaken).

Based on a risk assessment in accordance with section 3 of CS 461, an N1/N2 upgrade of the parapets is recommended.

The abutment columns and intermediate-pier columns have been assessed as being adequate for 44 tonnes assessment live loading and SV80 loading.

Based on a qualitative assessment, the west approach-span abutment, east approach-span abutment and north-east wing wall are not considered to be adequate for current loading, due to the settlement and movement observed.

Based on a qualitative assessment, the curtain walls, wing walls (other than the north-east wing wall) and foundations to the main-span abutments and intermediate piers are adequate for current loading.

Signed:	[redacted]
Name:	[redacted] (Assessment Team Leader)
Engineering Qualifications:	BEng CEng MICE
Signed:	[redacted]
Name:	[redacted]
Position Held:	Associate Director
Name of Organisation:	Jacobs
Date:	30.05.2025

Section 2

The certificate is agreed and submitted for acceptance

Signed:	[redacted]
Name:	
Engineering Qualifications:	
Name of Organisation:	Ringway Jacobs | Essex County Council
Date:	

The additional criteria given in section 1 are agreed

The certificate is accepted by the TAA

Signed:	[redacted]
Name:	
Position Held:	Structures Manager
TAA:	Essex County Council
Date:	

Project Details:

Name of Project:	Station Way Bridge Assessment
Name of Structure:	Station Way Bridge
Structure Ref No.	2100

Section 1

We certify that reasonable professional skill and care have been used in the preparation of the category 2 check of the assessment of Station Way Bridge with a view to securing that:

1. It has been checked in accordance with:

   The Approval in Principle dated 14/05/2024 (date of acceptance) including the following:

   The hinges located at the base of the columns (abutments and intermediate piers) have been assessed quantitatively in accordance with the basic principles of CS 455 and CS 468.

   The steel dowel bars have been assessed using guidance given in Concrete Society Technical Report No. 34.

   The deck slab and service bay slabs of spans 2 to 4 have been assessed using Pucher Charts.

2. The assessed capacity of the structure is as follows:

The structure is adequate for permanent loading only (refer below for further details)

The carriageway slab of span 1 has been assessed as being adequate for **permanent loading only**, due to the capacity of the dowelled connection with the adjacent run-on slab. All other superstructure elements supporting the carriageway have been assessed as being adequate for 44 tonnes assessment live loading and SV80 loading. It should be noted that if the carriageway slab of span 1 was adequately supported at both ends, it would be adequate for 44 tonnes assessment live loading and SV80 loading.

The superstructure elements supporting the footways have been assessed as being adequate for the following loading:

South-west footway slab (span 1):	Accidental vehicle loading (normal traffic) or pedestrian live loading
Original footway slabs (spans 1 and 5):	Pedestrian live loading only (refer to note below regarding north-east footway slab)
Service bay slabs (spans 2 to 4):	Restricted accidental vehicle loading (3t gross vehicle weight) or pedestrian live loading

Note: the change in support conditions resulting from the introduction of the propping to the north-east footway slab has not been considered in the assessment. For the assessment, it has been assumed that the slab is simply supported, and the capacity stated above is dependent on the slab being adequately supported by both abutments (which was the case when the IFA was undertaken).

Based on a risk assessment in accordance with section 3 of CS 461, an N1/N2 upgrade of the parapets is recommended.

The abutment columns and intermediate-pier columns have been assessed as being adequate for 44 tonnes assessment live loading and SV80 loading.

Based on a qualitative assessment, the west approach-span abutment, east approach-span abutment and north-east wing wall are not considered to be adequate for current loading, due to the settlement and movement observed.

Based on a qualitative assessment, the curtain walls, wing walls (other than the north-east wing wall) and foundations to the main-span abutments and intermediate piers are adequate for current loading.

Signed:	[redacted]
Name:	[redacted] Check Team Leader)
Engineering Qualifications:	MEng CEng MICE
Signed:	[redacted]
Name:	[redacted]
Position Held:	Associate Director
Name of Organisation:	Jacobs
Date:	30.05.2025

Section 2

The certificate is agreed and submitted for acceptance

Signed: [redacted]

Name:

Engineering Qualifications:

Name of Organisation: Ringway Jacobs | Essex County Council

Date:

The additional criteria given in section 1 are agreed

The certificate is accepted by the TAA

Signed: [redacted]

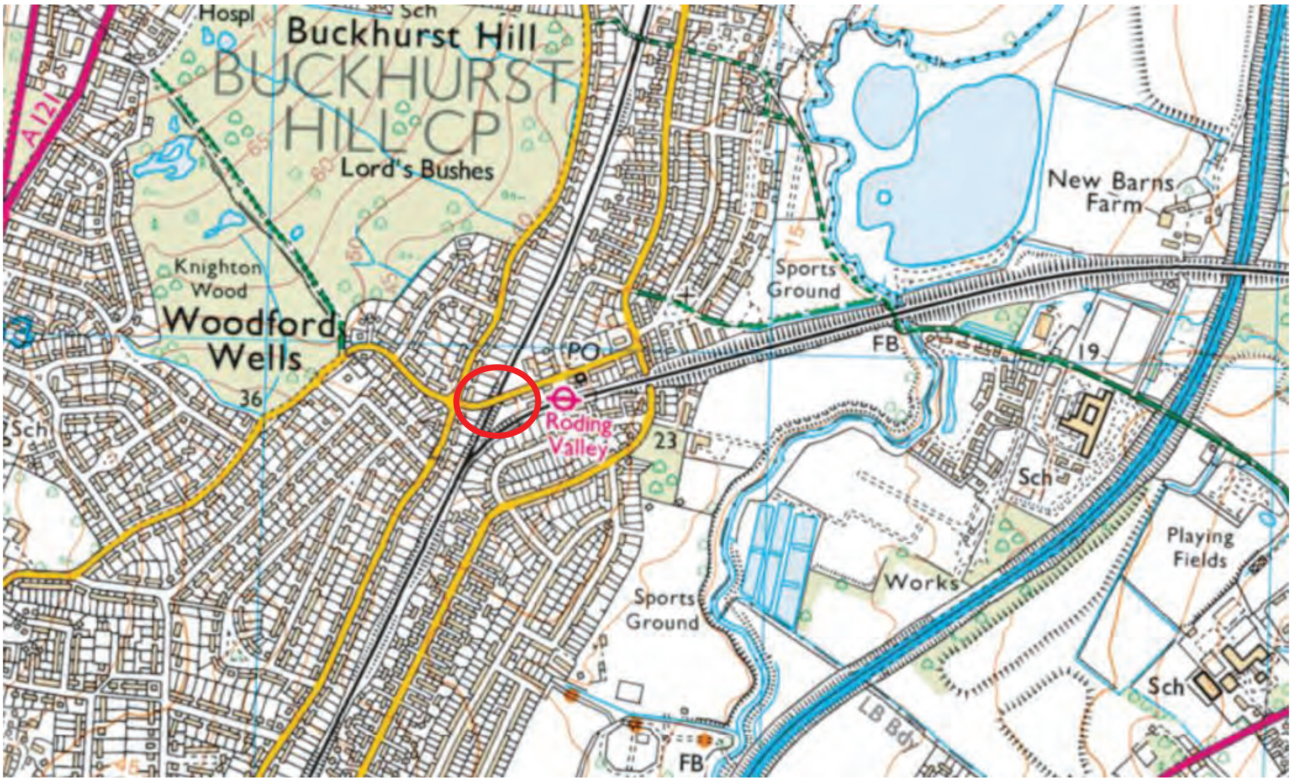
Name:

Position Held: Structures Manager

TAA: Essex County Council

Date:

Appendix A. Location Plan



Location Plan

Station Way Bridge

Grid Reference: TQ 414 929

Easting and Northing: 541460, 192900

Appendix B. Calculations

JACOBS CALCULATION SHEET

Office	Manchester First Street	**Page No.**	1	**Calc No.**	
Job No. & Title	ECC - Assessment of Station Way	**Calcs by**	JF	**Date**	May-24
Section	Contents page	**Checker**	CAT II	**Date**	May-24

Contents

Contents	Page no.	
Pages not used	2 - 17	(Pages removed 04/25)
East Carriageway Slab (Span 5)	18	
West Carriageway Slab (Span 1)	28	
South West Footway Slab (Span 1)	39	
Main Span Beams (Spans 2 to 4)	51	
Pages not used	78 - 91	(Pages removed 04/25)
Intermediate Pier Columns	92	
Abutment Columns	100	
Parapet Assessment	107	
Original Footway Slab (Spans 1 and 5)	A1 - A13	(Pages added 04/25)
Carriageway Slab (Spans 2 to 4)	B1 - B15	(Pages added 04/25)
Service Bay Slab (Spans 2 to 4)	C1 - C10	(Pages added 04/25)

Pages 2 to 17 not used

Material & Section Properties

Reinforced Concrete

CS 455, Cl.3.1.3	Characteristic Strength of Concrete		=	15.00	N/mm^2
CS 454 table 4.1.1a	Density of Reinforced Concrete		=	24	kN/m^3

Steel Reinforcement

CS455, Cl, 3.8.2	Mild steel reinforcement characteristic yield strength	=	230	N/mm^2
	Design Young's Modulus of steel reinforcement	=	210000	N/mm^2

Mass Concrete (Plain Concrete)

CS 454 table 4.1.1a	Unit weight of plain concrete	=	23	kN/m^3

Fill - Miscellaneous

CS 454 table 4.1.1a	Unit weight of miscellaneous fill	=	22	kN/m^3

Carriageway & Footway Surfacing

Reference	Description		Value	Unit
CS 454 table 4.1.1a	Unit weight of Bituminous Macadam (tar)	=	24	kN/m^3

Partial Factors

CS 455 Table 2.13a	Partial Factor for reinforcement	γ_{ms}	=	1.15
CS 455 Table 2.13a	Partial Factor for Concrete	γ_{mc}	=	1.5
CS 455 Table 2.13a	Partial Factor for shear in concrete	γ_{mv}	=	1.15
CS 455 Table 2.13a	Partial Factor for Bond	γ_{mb}	=	1.25
CS 454 Table A.1	Partial Factor for Concrete Deadload		=	1.15
CS 454 Table A.1	Partial Factor for Surfacing		=	1.75
CS 454 Table A.1	Partial Factor for Fill		=	1.2
CS 454 3.9	Inaccurate assessment effects at ULS	$\gamma f3$	=	1.1

Material Thicknesses
These dimensions have be scaled of drawing LC 5/2 in inches and converted to meters and milimeters

Asphalt Cover (depth assumed) = 4 " = 101.6 mm

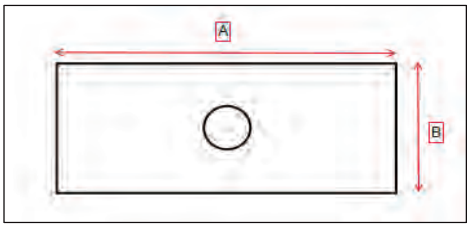
Slab Dimensions used in MIDAS

Skew Span of Slab	=	3.12	m
Square Span of Slab	=	2.13	m
Slab depth	=	0.25	m
Skew of Slab	=	43	°
Width of Slab	=	11.375	m

Autocad Grillage Model

Member dimensions

Longitudinal Edge Slab Member



Section	A (m)	B (m)	Area (m^2)
Longitudinal Slab	0.4564	0.25	0.1141

Self Weight of Section = 3.15 kN/m (applied in MIDAS)

Total Reaction from MIDAS = 167.6 kN

Node	Load	FX (kN)	FY (kN)	FZ (kN)	MX (kN*m)	MY (kN*m)	MZ (kN*m)
17	Self Weig	0.000000	0.000000	3.354087	0.000000	0.000000	0.000000
18	Self Weig	0.000000	0.000000	3.349297	0.000000	0.000000	0.000000
19	Self Weig	0.000000	0.000000	3.359664	0.000000	0.000000	0.000000
20	Self Weig	0.000000	0.000000	3.354701	0.000000	0.000000	0.000000
21	Self Weig	0.000000	0.000000	3.344890	0.000000	0.000000	0.000000
22	Self Weig	0.000000	0.000000	3.350433	0.000000	0.000000	0.000000
23	Self Weig	0.000000	0.000000	3.267884	0.000000	0.000000	0.000000
24	Self Weig	0.000000	0.000000	3.365996	0.000000	0.000000	0.000000
25	Self Weig	0.000000	0.000000	0.480376	0.000000	0.000000	0.000000
26	Self Weig	0.000000	0.000000	10.315259	0.000000	0.000000	0.000000
131	Self Weig	0.000000	0.000000	10.318727	0.000000	0.000000	0.000000
132	Self Weig	0.000000	0.000000	0.476001	0.000000	0.000000	0.000000
133	Self Weig	0.000000	0.000000	3.367337	0.000000	0.000000	0.000000
134	Self Weig	0.000000	0.000000	3.271377	0.000000	0.000000	0.000000
135	Self Weig	0.000000	0.000000	3.339826	0.000000	0.000000	0.000000
136	Self Weig	0.000000	0.000000	3.350215	0.000000	0.000000	0.000000
137	Self Weig	0.000000	0.000000	3.353615	0.000000	0.000000	0.000000
138	Self Weig	0.000000	0.000000	3.354029	0.000000	0.000000	0.000000
139	Self Weig	0.000000	0.000000	3.353827	0.000000	0.000000	0.000000
140	Self Weig	0.000000	0.000000	3.353612	0.000000	0.000000	0.000000
141	Self Weig	0.000000	0.000000	3.353569	0.000000	0.000000	0.000000
142	Self Weig	0.000000	0.000000	3.353753	0.000000	0.000000	0.000000
143	Self Weig	0.000000	0.000000	3.354119	0.000000	0.000000	0.000000
144	Self Weig	0.000000	0.000000	3.354445	0.000000	0.000000	0.000000
145	Self Weig	0.000000	0.000000	3.353954	0.000000	0.000000	0.000000
146	Self Weig	0.000000	0.000000	3.350284	0.000000	0.000000	0.000000
147	Self Weig	0.000000	0.000000	3.337705	0.000000	0.000000	0.000000
148	Self Weig	0.000000	0.000000	3.305978	0.000000	0.000000	0.000000
149	Self Weig	0.000000	0.000000	3.245261	0.000000	0.000000	0.000000
150	Self Weig	0.000000	0.000000	3.160284	0.000000	0.000000	0.000000
151	Self Weig	0.000000	0.000000	3.072836	0.000000	0.000000	0.000000
152	Self Weig	0.000000	0.000000	2.979932	0.000000	0.000000	0.000000
153	Self Weig	0.000000	0.000000	2.799812	0.000000	0.000000	0.000000
154	Self Weig	0.000000	0.000000	2.406079	0.000000	0.000000	0.000000
155	Self Weig	0.000000	0.000000	1.815514	0.000000	0.000000	0.000000
156	Self Weig	0.000000	0.000000	0.021444	0.000000	0.000000	0.000000
160	Self Weig	0.000000	0.000000	2.984040	0.000000	0.000000	0.000000

SUMMATION OF REACTION FORCES PRINTOUT

Load	FX (kN)	FY (kN)	FZ (kN)
Self Weig	0.000000	0.000000	167.603827

MIDAS Comparison

Deadload calculations from the archive drawings

To check the model accuracy compared to the as-built construction of the structure the following is a demonstration of the procedure.

Skew Span	=	3.12	m
Sqaure Span	=	2.13	m
Width of Slab	=	11.62	m
Skew Angle	=	43	°
Depth of slab	=	0.25	m
Area of slab on plan	=	24.725	m^2
Volume of Slab	=	6.18125	m^3
Total Reaction	=	170.6025	kN

Verification

MIDAS calculated = 167.6 kN

As built calculated = 170.6025 kN

Percentage difference = 1.77 % (OK)

<u>Superimposed Deadload</u>

There is no fill over the carriageway slab. The only material which is present ontop of the is asphalt. There is no exact dimensions for the asphalt so the depth has been assumed and is shown below.

Asphalt Depth = 100 mm or 0.1 m (assumed)
Ashphalt Width = 0.4564 m
Asphalt above = 0.04564 m^3

Total weight of = 1.92 kN/m (applied Iin MIDAS)

MIDAS Reactions = 102.16 kN

<u>Superimposed Deadload Calculation using Archive Drawings</u>

Area of slab on plan = 24.725 m^2
Depth of surfacing = 0.1 m^3
Total reactions = 103.85 kN

	Node	Load	FX (kN)	FY (kN)	FZ (kN)	MX (kN*m)	MY (kN*m)	MZ (kN*m)
	17	Superimp	0.000000	0.000000	2.044396	0.000000	0.000000	0.000000
	18	Superimp	0.000000	0.000000	2.041476	0.000000	0.000000	0.000000
	19	Superimp	0.000000	0.000000	2.047795	0.000000	0.000000	0.000000
	20	Superimp	0.000000	0.000000	2.044770	0.000000	0.000000	0.000000
	21	Superimp	0.000000	0.000000	2.038790	0.000000	0.000000	0.000000
	22	Superimp	0.000000	0.000000	2.042169	0.000000	0.000000	0.000000
	23	Superimp	0.000000	0.000000	1.991853	0.000000	0.000000	0.000000
	24	Superimp	0.000000	0.000000	2.051654	0.000000	0.000000	0.000000
	25	Superimp	0.000000	0.000000	0.292800	0.000000	0.000000	0.000000
	26	Superimp	0.000000	0.000000	6.287396	0.000000	0.000000	0.000000
	131	Superimp	0.000000	0.000000	6.289510	0.000000	0.000000	0.000000
	132	Superimp	0.000000	0.000000	0.290134	0.000000	0.000000	0.000000
	133	Superimp	0.000000	0.000000	2.052472	0.000000	0.000000	0.000000
	134	Superimp	0.000000	0.000000	1.993982	0.000000	0.000000	0.000000
	135	Superimp	0.000000	0.000000	2.035703	0.000000	0.000000	0.000000
	136	Superimp	0.000000	0.000000	2.042036	0.000000	0.000000	0.000000
	137	Superimp	0.000000	0.000000	2.044108	0.000000	0.000000	0.000000
	138	Superimp	0.000000	0.000000	2.044361	0.000000	0.000000	0.000000
	139	Superimp	0.000000	0.000000	2.044237	0.000000	0.000000	0.000000
	140	Superimp	0.000000	0.000000	2.044106	0.000000	0.000000	0.000000
	141	Superimp	0.000000	0.000000	2.044080	0.000000	0.000000	0.000000
	142	Superimp	0.000000	0.000000	2.044192	0.000000	0.000000	0.000000
	143	Superimp	0.000000	0.000000	2.044415	0.000000	0.000000	0.000000
	144	Superimp	0.000000	0.000000	2.044614	0.000000	0.000000	0.000000
	145	Superimp	0.000000	0.000000	2.044315	0.000000	0.000000	0.000000
	146	Superimp	0.000000	0.000000	2.042078	0.000000	0.000000	0.000000
	147	Superimp	0.000000	0.000000	2.034411	0.000000	0.000000	0.000000
	148	Superimp	0.000000	0.000000	2.015072	0.000000	0.000000	0.000000
	149	Superimp	0.000000	0.000000	1.978064	0.000000	0.000000	0.000000
	150	Superimp	0.000000	0.000000	1.926268	0.000000	0.000000	0.000000
	151	Superimp	0.000000	0.000000	1.872966	0.000000	0.000000	0.000000
	152	Superimp	0.000000	0.000000	1.816339	0.000000	0.000000	0.000000
	153	Superimp	0.000000	0.000000	1.706552	0.000000	0.000000	0.000000
	154	Superimp	0.000000	0.000000	1.466562	0.000000	0.000000	0.000000
	155	Superimp	0.000000	0.000000	1.106599	0.000000	0.000000	0.000000
	156	Superimp	0.000000	0.000000	0.013071	0.000000	0.000000	0.000000
	160	Superimp	0.000000	0.000000	1.818843	0.000000	0.000000	0.000000
	SUMMATION OF REACTION FORCES PRINTOUT							
		Load	FX (kN)	FY (kN)	FZ (kN)			
		Superimp	0.000000	0.000000	102.158523			

<u>MIDAS Comparison</u>

<u>Verification</u>

MIDAS Calculated = 102.16 kN

As Built Calculated = 103.85 kN

Percentage Difference = 1.64 % (OK)

<u>Capacity</u>

Moment Resistance of Carriageway Slab

Bar Type	Bar Dia (")	Bar Dia (mm)	Area of Bar (mm^2)	Spacing (") Centre to Centre	Spacing (mm) Centre to Centre
Longitudinal Bottom Bar	9/16"	14.3	160.61	4	101.6

JACOBS			CALCULATION SHEET			
Office	Manchester First Street	Page No.	21	Calc No.	
Job No. & Title	ECC - Assessment of Station Way	Calcs by	JF	Date	May-24
Section	Approach Span 5 - East Carriageway Slab	Checker	CAT II	Date	May-24

Grillage Model Dims

Note: Only the 250mm deep slab considered for M_u

Width of section = 456.00 mm

Height of section = 250 mm

Cross Sectional Area = 114000 mm^2

Total number of tension bars per section width = 4.49

Area of tension reinforcement per section width = 720.83 mm^2

Cover to reinforcement = 1 1/2 " = 38 mm (Assumed)

Longitudinal Direction

Effective Depth, d = 204.85 mm

CS 455, Eq

Z =

$$z = \left(1 - \frac{0.84\frac{f_y A_s}{\gamma_{ms}}}{\frac{f_{cu} bd}{\gamma_{mc}}}\right) d$$

= 178.29 mm

Should be less than 0.95d = 194.61 mm (OK)
Take Z = 178.29 mm

Moment resistance equation

CS 455, Eq

$$M_u = \min \left\{ \begin{array}{l} \frac{f_y}{\gamma_{ms}} A_s z \\ \frac{0.225 f_{cu}}{\gamma_{mc}} bd^2 \end{array} \right.$$

M_u = 25.70 kNm

M_u = 43.05 kNm

Therefore moment reistance = 25.70 kNm

Transverse Direction

Bar Type	Bar Dia (")	Bar Dia (mm)	Ara of Bar (mm^2)	Spacing (") Centre to Centre	Spacing (mm) Centre to Centre
Transverse Bottom Bar	1/2	12.7	126.68	5.5	139.7

Grillage Model Dims

Width of section = 425 mm

Height of section = 250 mm

Cross sectional area of section = 106250 mm^2

Total number of tension bars per section = 3.04 or 3 bars

Area of tension reinforcement per section = 385.38 mm^2

Cover to reinforcement = 1 1/2" " = 52.3 mm

Effective depth, d = 185 mm

5 455, Eq 5.2. Z = = 169.76613 mm

$$z = \left(1 - \frac{0.84 \frac{f_y A_s}{\gamma_{ms}}}{\frac{f_{cu} b d}{\gamma_{mc}}}\right) d$$

Should be less than 0.95d = 175.75 mm (OK)

Moment Resistance Equation

CS 455, Eq

$$M_u = \min \left\{ \begin{array}{l} \frac{f_y}{\gamma_{ms}} A_s z \\ \frac{0.225 f_{cu}}{\gamma_{mc}} b d^2 \end{array} \right.$$

M_u = 13.08 kNm

M_u = 32.73 kNm

Therefore Moment Resistance = 13.08 kNm

Shear Resistance - Longitudinal Directions

Maximum shear resistance based on concrete crushing

Equation 5.6a Maximum shear resistance based on concrete crushing

$$V_{\max} = 0.36 \left(0.7 - \frac{f_{cu}}{250}\right) \left(\frac{f_{cu}}{\gamma_{mc}}\right) b_w d$$

where:

b_w is the breadth of the section, taken as the web width for flanged beams

V_{max} = 215.22 kN

Shear resistance more than 3d from the support

Equation 6.5 Shear resistance of concrete slabs more than 3d from a support

$$V_{uc} = \frac{0.27}{\gamma_{mv}} \xi_s \rho_s^{\frac{1}{3}} f_{cu}^{\frac{1}{3}} b_w d$$

Where:

3D = 618 mm

γ_{mv} is the partial factor for shear defined in Section 2.

ξ_s is the depth factor, taken as $\xi_s = \left(\frac{500}{d}\right)^{0.25}$ but not less than 0.7

ρ_s is the ratio of longitudinal reinforcement $\rho_s = \frac{100 A_s}{b_w d}$ but not less than 0.15, nor greater than 3.0

A_s is the area of longitudinal tension reinforcement that continues at least a distance d beyond the section being considered, and, for shear at supports, continues at least to the support.
Where top and bottom reinforcement are provided, the area in tension under the loading which produces the shear force is to be used.

ξ_s = 1.239 < 0.7 (OK) As = 642.4 mm^2

ρs = 0.66 d = 212 mm

V_{uc} = 60.53 kN

Shear resistance within 3d of the support

Equation 5.6c Shear resistance within 3d of a support

$$V_u = \max \left\{ \begin{array}{l} \frac{3d}{a_v} \Gamma V_{uc} \\ \frac{0.24}{\gamma_{mv}} \xi_s (0.15 f_{cu})^{\frac{1}{3}} b_w d \end{array} \right.$$

where:

a_v is the distance of the section measured from the edge of a rigid bearing, the centre-line of a flexible bearing or the face of a support, where $d \leq a_v \leq 3d$

Γ is the factor to account for short anchorage lengths, defined in Equation 5.6d

a_v = 204.85 mm or 3d

T = $\sqrt{\frac{z}{3d}\frac{F_{ub}}{V_{uc}}}$

fub $f_{ub} = \frac{k k_{cov} \beta \sqrt{f_{cu}}}{\gamma_{mb}}$

K	=	1		acon	=	0.40
β	=	0.39		c	=	38 mm
fcu	=	15	N/mm^2	ϕ	=	14.3 mm
γ_{mb}	=	1.4		kcov	=	1.00

fub = 1.08

Fub = $F_{ub} = f_{ub} p L_a$

p = $p = \pi\phi$ for a single bar, = 179.69 mm

L_a = Length taken as d = 204.85 mm

Fub = 39713.86 N or 39.71 kN 144.166079 Limiting value

Γ = $\Gamma = \min\left\{\begin{array}{l}\sqrt{\frac{z}{3d}\frac{F_{ub}}{V_{uc}}}\\ 1.0\end{array}\right.$ = 0.429 > 1 0.47

V_u = $\frac{3d}{a_v}\Gamma V_{uc}$ = 77.88 kN

= $\frac{0.24}{\gamma_{mv}}\xi_s(0.15 f_{cu})^{\frac{1}{3}} b_w d$ = 31656.79 N

31.66 kN

Shear Resistance - Transverse Direction

Maximum shear resistance based on concrete crushing

Equation 5.6a Maximum shear resistance based on concrete crushing

$$V_{max} = 0.36\left(0.7 - \frac{f_{cu}}{250}\right)\left(\frac{f_{cu}}{\gamma_{mc}}\right) b_w d$$

where:

b_w is the breadth of the section, taken as the web width for flanged beams

V_{Max} = 181.15

Shear resistance more than 3d from the support

Equation 6.5 Shear resistance of concrete slabs more than 3d from a support

$$V_{uc} = \frac{0.27}{\gamma_{mv}}\xi_s \rho_s^{\frac{1}{3}} f_{cu}^{\frac{1}{3}} b_w d$$

Where:

γ_{mv} is the partial factor for shear defined in Section 2.

ξ_s is the depth factor, taken as
$\xi_s = \left(\frac{500}{d}\right)^{0.25}$ but not less than 0.7

ρ_s is the ratio of longitudinal reinforcement
$\rho_s = \frac{100 A_s}{b_w d}$ but not less than 0.15, nor greater than 3.0

A_s is the area of longitudinal tension reinforcement that continues at least a distance d beyond the section being considered, and, for shear at supports, continues at least to the support.
Where top and bottom reinforcement are provided, the area in tension under the loading which produces the shear force is to be used.

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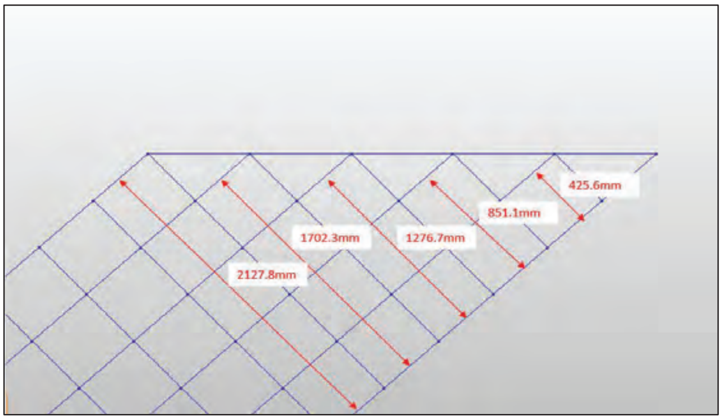
ξ_s = 1.28 < 0.7 OK

ρs = 0.49

Vuc = 17.19 kN

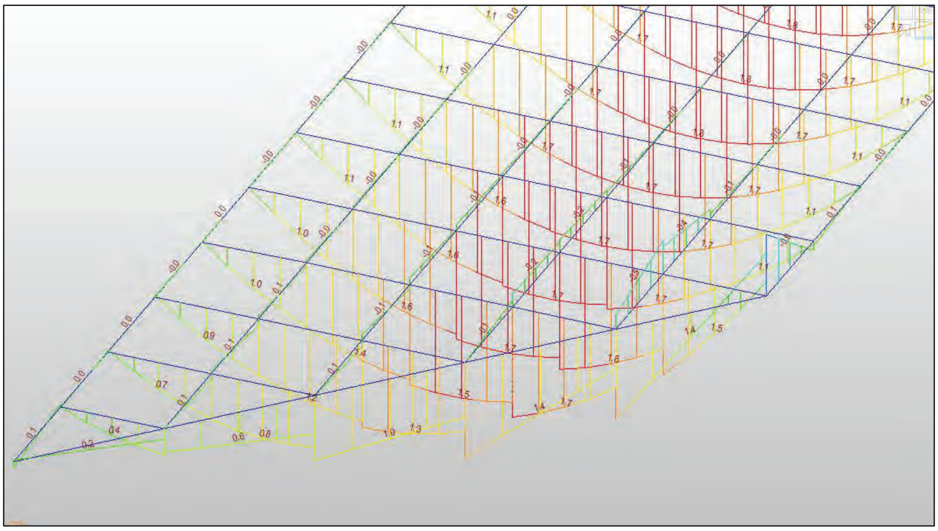
Deadload Capacity Check

Maximun Bending Moment due to deadload M_{ed} depending on length



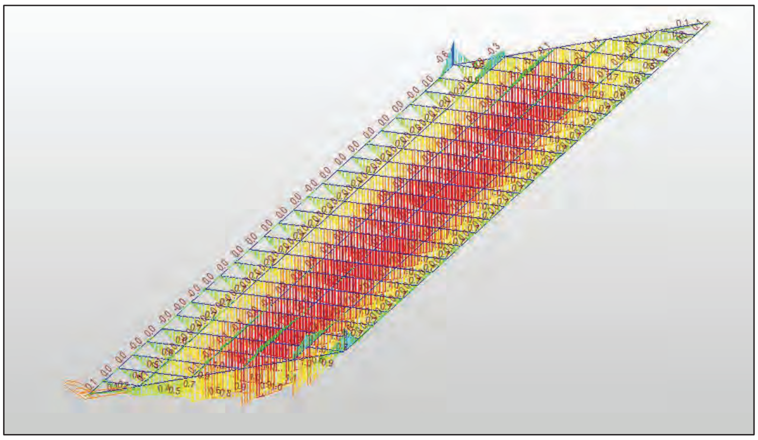
Member Lengths from MIDAS

Length (mm)	Length (m)	W (kNm)	Med (kNm)	MIDAS Results (kNm)	Section Resistance (kNm)	Result
425.6	0.4256	3.15	0.07	0.4	13.08	PASS
851.1	0.8511	3.15	0.29	0.8	13.08	PASS
1276.7	1.2767	3.15	0.64	1.3	13.08	PASS
1702.3	1.7023	3.15	1.14	1.5	13.08	PASS
2127.8	2.1278	3.15	1.78	1.70	13.08	PASS



MIDAS Results Screenshot

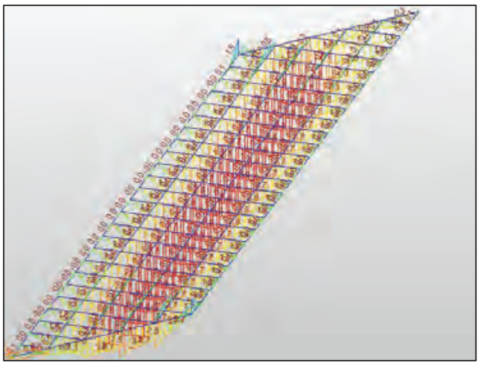
Length (mm)	Length (m)	W (kNm)	Med (kNm)	MIDAS Results (kNm)	Section Resistance (kNm)	Result
425.6	0.4256	1.92	0.04	0.5	13.08	PASS
851.1	0.8511	1.92	0.17	0.8	13.08	PASS
1276.7	1.2767	1.92	0.39	1.00	13.08	PASS
1702.3	1.7023	1.92	0.69	1.00	13.08	PASS
2127.8	2.1278	1.92	1.08	1.00	13.08	PASS



MIDAS Result Screenshot

Load Combination 1 - Deadload and Superimposed Deadload

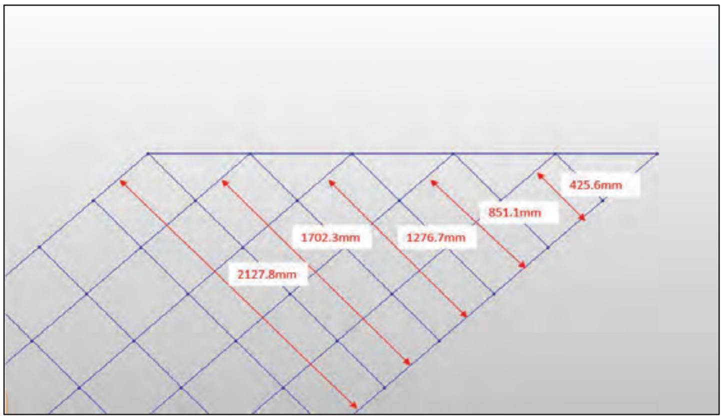
Length (mm)	Length (m)	W (kNm)	Med (kNm)	MIDAS Results (kNm)	Section Resistance (kNm)	Result
425.6	0.4256	5.07	0.11	1.3	13.08	PASS
851.1	0.8511	5.07	0.46	2.1	13.08	PASS
1276.7	1.2767	5.07	1.03	2.50	13.08	PASS
1702.3	1.7023	5.07	1.84	2.70	13.08	PASS
2127.8	2.1278	5.07	2.87	2.70	13.08	PASS



MIDAS Result Screenshot

Deadload Capacity Check - Shear Longitudinal Direction

Maximum Shear Load due to deadload V_{ed} depending on length



Member length from MIDAS

Maximum Shear Due to Deadload Capacity Check

Length (mm)	Length (m)	W (kNm)	Ved (kN)	MIDAS Results (kN)	Section Resistance (kN)	Result
425.6	0.4256	3.15	0.67	-1.6	215.22	PASS
851.1	0.8511	3.15	1.34	-2.3	215.22	PASS
1276.7	1.2767	3.15	2.01	-2.8	215.22	PASS
1702.3	1.7023	3.15	2.68	-3	215.22	PASS
2127.8	2.1278	3.15	3.35	-3.1	215.22	PASS

Maximum Shear Load due to deadload and superimposed deadload

Length (mm)	Length (m)	W (kNm)	Ved +- (kN)	MIDAS Results (kN)	Section Resistance (kN)	Result
425.6	0.4256	5.07	1.08	-2.6	215.22	PASS
851.1	0.8511	5.07	2.16	-3.8	215.22	PASS
1276.7	1.2767	5.07	3.23	-4.4	215.22	PASS
1702.3	1.7023	5.07	4.31	-4.8	215.22	PASS
2127.8	2.1278	5.07	5.39	-5	215.22	PASS

Live Loading - All Model 1

Note: ALL Model 2 will not be applied to the carriageway slab due to the element complying with section 5.6 of CS 454.

Impact Factors and Lane Widths

Load Situation	Impact factor applied to critical axle Poor Road Surface	Notional lane width (m)	Maximum lateral spacing between wheel centres of adjacent vehicles (m)
Single vehicle in each lane	1.8	3	1.2
Convoy of vehicles in each lane	1	2.5	0.7

Traffic Flow Factors

Traffic Flow Category	Traffic Flow Factor
Low	0.9

Lane Factors for ALL Model 1

Lane	Lane Factor
Lane 1	1
Lane 2	1
Lane 3	0.5
Lane 4 and subsequent	0.4

Partial Factor for traffic actions = 1.5

Material & Section Properties

Reinforced Concrete

Reference	Description			Value	Unit
CS 455, Cl.3.1.3	Characteristic Strength of Concrete		=	15.00	N/mm^2
CS 454 table 4.1.1a	Density of Reinforced Concrete		=	24	kN/m^3

Steel Reinforcement

Reference	Description			Value	Unit
CS455, Cl, 3.8.2	Mild steel reinforcement characteristic yield strength		=	230	N/mm^2
	Design Young's Modulus of steel reinforcement		=	210000	N/mm^2

Mass Concrete (Plain Concrete)

Reference	Description			Value	Unit
CS 454 table 4.1.1a	Unit weight of plain concrete		=	23	kN/m^3

Fill - Miscellaneous

Reference	Description			Value	Unit
CS 454 table 4.1.1a	Unit weight of miscellaneous fill		=	22	kN/m^3

Carriageway & Footway Surfacing

Reference	Description			Value	Unit
CS 454 table 4.1.1a	Unit weight of Bituminous Macadam (tar)		=	24	kN/m^3

Partial Factors

Reference	Description	Symbol		Value
CS 455 Table 2.13a	Partial Factor for reinforcement	γ_{ms}	=	1.15
CS 455 Table 2.13a	Partial Factor for Concrete	γ_{mc}	=	1.5
CS 455 Table 2.13a	Partial Factor for shear in concrete	γ_{mv}	=	1.15
CS 455 Table 2.13a	Partial Factor for Bond	γ_{mb}	=	1.25
CS 454 Table A.1	Partial Factor for Concrete Deadload		=	1.15
CS 454 Table A.1	Partial Factor for Surfacing		=	1.75
CS 454 Table A.1	Partial Factor for Fill		=	1.2
CS 454 3.9	Inaccurate assessment effects at ULS	$\gamma f3$	=	1.1

Material Thicknesses
These dimensions have be scaled of drawing LC 5/2 in inches and converted to meters and milimeters

Asphalt Cover (depth assumed) = 4 " = 101.6 mm

Slab Dimensions used in MIDAS

Description		Value	Unit
Skew Span of Slab	=	3.12	m
Square Span of Slab	=	2.13	m
Slab depth	=	0.25	m
Skew of Slab	=	43	°
Width of Slab	=	11.375	m

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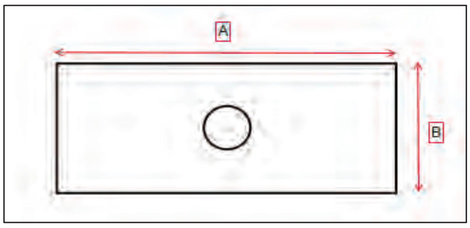
Autocad Grillage Model

Member dimensions

Longitudinal Edge Slab Member



Section	A (m)	B (m)	Area (m^2)
Longitudinal Slab	0.4564	0.25	0.1141

Self Weight of Section = 3.15 kN/m (applied in MIDAS)

Total Reaction from MIDAS = 167.6 kN

Node	Load	FX (kN)	FY (kN)	FZ (kN)	MX (kN*m)	MY (kN*m)	MZ (kN*m)
17	Self Weig	0.000000	0.000000	3.354087	0.000000	0.000000	0.000000
18	Self Weig	0.000000	0.000000	3.349297	0.000000	0.000000	0.000000
19	Self Weig	0.000000	0.000000	3.359664	0.000000	0.000000	0.000000
20	Self Weig	0.000000	0.000000	3.354701	0.000000	0.000000	0.000000
21	Self Weig	0.000000	0.000000	3.344890	0.000000	0.000000	0.000000
22	Self Weig	0.000000	0.000000	3.350433	0.000000	0.000000	0.000000
23	Self Weig	0.000000	0.000000	3.267884	0.000000	0.000000	0.000000
24	Self Weig	0.000000	0.000000	3.365996	0.000000	0.000000	0.000000
25	Self Weig	0.000000	0.000000	0.480376	0.000000	0.000000	0.000000
26	Self Weig	0.000000	0.000000	10.315259	0.000000	0.000000	0.000000
131	Self Weig	0.000000	0.000000	10.318727	0.000000	0.000000	0.000000
132	Self Weig	0.000000	0.000000	0.476001	0.000000	0.000000	0.000000
133	Self Weig	0.000000	0.000000	3.367337	0.000000	0.000000	0.000000
134	Self Weig	0.000000	0.000000	3.271377	0.000000	0.000000	0.000000
135	Self Weig	0.000000	0.000000	3.339826	0.000000	0.000000	0.000000
136	Self Weig	0.000000	0.000000	3.350215	0.000000	0.000000	0.000000
137	Self Weig	0.000000	0.000000	3.353615	0.000000	0.000000	0.000000
138	Self Weig	0.000000	0.000000	3.354029	0.000000	0.000000	0.000000
139	Self Weig	0.000000	0.000000	3.353827	0.000000	0.000000	0.000000
140	Self Weig	0.000000	0.000000	3.353612	0.000000	0.000000	0.000000
141	Self Weig	0.000000	0.000000	3.353569	0.000000	0.000000	0.000000
142	Self Weig	0.000000	0.000000	3.353753	0.000000	0.000000	0.000000
143	Self Weig	0.000000	0.000000	3.354119	0.000000	0.000000	0.000000
144	Self Weig	0.000000	0.000000	3.354445	0.000000	0.000000	0.000000
145	Self Weig	0.000000	0.000000	3.353954	0.000000	0.000000	0.000000
146	Self Weig	0.000000	0.000000	3.350284	0.000000	0.000000	0.000000
147	Self Weig	0.000000	0.000000	3.337705	0.000000	0.000000	0.000000
148	Self Weig	0.000000	0.000000	3.305978	0.000000	0.000000	0.000000
149	Self Weig	0.000000	0.000000	3.245261	0.000000	0.000000	0.000000
150	Self Weig	0.000000	0.000000	3.160284	0.000000	0.000000	0.000000
151	Self Weig	0.000000	0.000000	3.072836	0.000000	0.000000	0.000000
152	Self Weig	0.000000	0.000000	2.979932	0.000000	0.000000	0.000000
153	Self Weig	0.000000	0.000000	2.799812	0.000000	0.000000	0.000000
154	Self Weig	0.000000	0.000000	2.406079	0.000000	0.000000	0.000000
155	Self Weig	0.000000	0.000000	1.815514	0.000000	0.000000	0.000000
156	Self Weig	0.000000	0.000000	0.021444	0.000000	0.000000	0.000000
160	Self Weig	0.000000	0.000000	2.984040	0.000000	0.000000	0.000000
SUMMATION OF REACTION FORCES PRINTOUT							
	Load	FX (kN)	FY (kN)	FZ (kN)			
	Self Weig	0.000000	0.000000	167.603827			

MIDAS Comparison

Deadload calculations from the archive drawings

To check the model accuracy compared to the as-built construction of the structure the following is a demonstration of the procedure.

Skew Span	=	3.12	m
Sqaure Span	=	2.13	m
Width of Slab	=	11.62	m
Skew Angle	=	43	°
Depth of slab	=	0.25	m

Area of slab on plan	=	24.725	m^2
Volume of Slab	=	6.18125	m^3
Total Reaction	=	170.6025	kN

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Verification

MIDAS calculated = 167.6 kN

As built calculated = 170.6025 kN

Percentage difference = 1.77 % (OK)

Superimposed Deadload

There is no fill over the carriageway slab. The only material which is present ontop of the is asphalt. There is no exact dimensions for the asphalt so the depth has been assumed and is shown below.

Asphalt Depth = 100 mm or 0.1 m (assumed)
Ashphalt Width = 0.4564 m
Asphalt above = 0.04564 m^3

Total weight of = 1.92 kN/m (applied lin MIDAS)

MIDAS Reactions = 102.16 kN

Superimposed Deadload Calculation using Archive Drawings

Area of slab on plan = 24.725 m^2
Depth of surfacing = 0.1 m^3
Total reactions = 103.85 kN

Node	Load	FX (kN)	FY (kN)	FZ (kN)	MX (kN*m)	MY (kN*m)	MZ (kN*m)
17	Superimp	0.000000	0.000000	2.044396	0.000000	0.000000	0.000000
18	Superimp	0.000000	0.000000	2.041476	0.000000	0.000000	0.000000
19	Superimp	0.000000	0.000000	2.047795	0.000000	0.000000	0.000000
20	Superimp	0.000000	0.000000	2.044770	0.000000	0.000000	0.000000
21	Superimp	0.000000	0.000000	2.038790	0.000000	0.000000	0.000000
22	Superimp	0.000000	0.000000	2.042169	0.000000	0.000000	0.000000
23	Superimp	0.000000	0.000000	1.991853	0.000000	0.000000	0.000000
24	Superimp	0.000000	0.000000	2.051654	0.000000	0.000000	0.000000
25	Superimp	0.000000	0.000000	0.292800	0.000000	0.000000	0.000000
26	Superimp	0.000000	0.000000	6.287396	0.000000	0.000000	0.000000
131	Superimp	0.000000	0.000000	6.289510	0.000000	0.000000	0.000000
132	Superimp	0.000000	0.000000	0.290134	0.000000	0.000000	0.000000
133	Superimp	0.000000	0.000000	2.052472	0.000000	0.000000	0.000000
134	Superimp	0.000000	0.000000	1.993982	0.000000	0.000000	0.000000
135	Superimp	0.000000	0.000000	2.035703	0.000000	0.000000	0.000000
136	Superimp	0.000000	0.000000	2.042036	0.000000	0.000000	0.000000
137	Superimp	0.000000	0.000000	2.044108	0.000000	0.000000	0.000000
138	Superimp	0.000000	0.000000	2.044361	0.000000	0.000000	0.000000
139	Superimp	0.000000	0.000000	2.044237	0.000000	0.000000	0.000000
140	Superimp	0.000000	0.000000	2.044106	0.000000	0.000000	0.000000
141	Superimp	0.000000	0.000000	2.044080	0.000000	0.000000	0.000000
142	Superimp	0.000000	0.000000	2.044192	0.000000	0.000000	0.000000
143	Superimp	0.000000	0.000000	2.044415	0.000000	0.000000	0.000000
144	Superimp	0.000000	0.000000	2.044614	0.000000	0.000000	0.000000
145	Superimp	0.000000	0.000000	2.044315	0.000000	0.000000	0.000000
146	Superimp	0.000000	0.000000	2.042078	0.000000	0.000000	0.000000
147	Superimp	0.000000	0.000000	2.034411	0.000000	0.000000	0.000000
148	Superimp	0.000000	0.000000	2.015072	0.000000	0.000000	0.000000
149	Superimp	0.000000	0.000000	1.978064	0.000000	0.000000	0.000000
150	Superimp	0.000000	0.000000	1.926268	0.000000	0.000000	0.000000
151	Superimp	0.000000	0.000000	1.872966	0.000000	0.000000	0.000000
152	Superimp	0.000000	0.000000	1.816339	0.000000	0.000000	0.000000
153	Superimp	0.000000	0.000000	1.706552	0.000000	0.000000	0.000000
154	Superimp	0.000000	0.000000	1.466562	0.000000	0.000000	0.000000
155	Superimp	0.000000	0.000000	1.106599	0.000000	0.000000	0.000000
156	Superimp	0.000000	0.000000	0.013071	0.000000	0.000000	0.000000
160	Superimp	0.000000	0.000000	1.818843	0.000000	0.000000	0.000000
SUMMATION OF REACTION FORCES PRINTOUT							
	Load	FX (kN)	FY (kN)	FZ (kN)			
	Superimp	0.000000	0.000000	102.158523			

MIDAS Comparison

Vertification

MIDAS Calculated = 102.16 kN

As Built Calculated = 103.85 kN

Percentage Difference = 1.64 % (OK)

Capacity

Moment Resistance of Carriageway Slab

Bar Type	Bar Dia (")	Bar Dia (mm)	Area of Bar (mm^2)	Spacing (") Centre to Centre	Spacing (mm) Centre to Centre
Longitudinal Bottom Bar	9/16"	14.3	160.61	4	101.6

Grillage Model Dims

Note: Only the 250mm deep slab considered for M_u

Width of section = 456.00 mm

Height of section = 250 mm

Cross Sectional Area = 114000 mm^2

Total number of tension bars per section width = 4.49

Area of tension reinforcement per section width = 721 mm^2

Cover to reinforcement = 1 1/2 " = 38 mm (Assumed)

Longitudinal Direction

Effective Depth, d = 204.85 mm

CS 455, Eq

Z =

$$z = \left(1 - \frac{0.84\frac{f_y A_s}{\gamma_{ms}}}{\frac{f_{cu} bd}{\gamma_{mc}}}\right) d$$

= 178.4 mm

Should be less than 0.95d = 194.61 mm (OK)
Take Z = 178.4 mm

Moment resistance equation

CS 455, Eq

$$M_u = \min \begin{cases} \frac{f_y}{\gamma_{ms}} A_s z \\ \frac{0.225 f_{cu}}{\gamma_{mc}} bd^2 \end{cases}$$

M_u = 26 kNm

M_u = 43.05 kNm

Therefore moment reistance = 26 kNm

Transverse Direction

Bar Type	Bar Dia (")	Bar Dia (mm)	Ara of Bar (mm^2)	Spacing (") Centre to Centre	Spacing (mm) Centre to Centre
Transverse Bottom Bar	1/2	12	113.10	6	152.4

Grillage Model Dims

Width of section	=	425 mm		
Height of section	=	250 mm		
Cross sectional area of section	=	106250 mm^2		
Total number of tension bars per section	=	2.7887139	or	3 bars
Area of tension reinforcement per section	=	339.29201 mm^2		
Cover to reinforcement	=	1 1/2" "	=	52.3 mm
Effective depth, d	=	191.7 mm		

S 455, Eq 5.2. Z = $$z = \left(1 - \frac{0.84\frac{f_y A_s}{\gamma_{ms}}}{\frac{f_{cu} b d}{\gamma_{mc}}}\right) d$$ = 178.28799 mm

Should be less than 0.95d = 182.115 mm (OK)

Moment Resistance Equation

CS 455, Eq
$$M_u = \min \left\{ \begin{array}{l} \frac{f_y}{\gamma_{ms}} A_s z \\ \frac{0.225 f_{cu}}{\gamma_{mc}} b d^2 \end{array} \right.$$

M_u = 12.10 kNm

M_u = 35.14 kNm

Therefore Moment Resistance = 12.10 kNm

Shear Resistance - Longitudinal Directions

Maximum shear resistance based on concrete crushing

Equation 5.6a Maximum shear resistance based on concrete crushing

$$V_{max} = 0.36\left(0.7 - \frac{f_{cu}}{250}\right)\left(\frac{f_{cu}}{\gamma_{mc}}\right) b_w d$$

where:

b_w is the breadth of the section, taken as the web width for flanged beams

V_{max} = 215.22 kN

Shear resistance more than 3d from the support

Equation 5.6b Shear resistance more than 3d from a support

$$V_{uc} = \frac{0.24}{\gamma_{mv}} \xi_s \rho_s^{\frac{1}{3}} f_{cu}^{\frac{1}{3}} b_w d$$

Where:

3D = 618 mm

γ_{mv} is the partial factor for shear defined in Section 2.

ξ_s is the depth factor, taken as $\xi_s = \left(\frac{500}{d}\right)^{0.25}$ but not less than 0.7

ρ_s is the ratio of longitudinal reinforcement $\rho_s = \frac{100A_s}{b_w d}$ but not less than 0.15, nor greater than 3.0

A_s is the area of longitudinal tension reinforcement that continues at least a distance d beyond the section being considered, and, for shear at supports, continues at least to the support.
Where top and bottom reinforcement are provided, the area in tension under the loading which produces the shear force is to be used.

ξ_s = 1.25 < 0.7 (OK)

ρs = 0.69

V_{uc} = 27.93 kN

Shear resistance within 3d of the support

Equation 5.6c Shear resistance within 3d of a support

$$V_u = \max\begin{cases} \frac{3d}{a_v}\Gamma V_{uc} \\ \frac{0.24}{\gamma_{mv}}\xi_s(0.15f_{cu})^{\frac{1}{3}}b_w d \end{cases}$$

where:

a_v is the distance of the section measured from the edge of a rigid bearing, the centre-line of a flexible bearing or the face of a support, where $d \le a_v \le 3d$

Γ is the factor to account for short anchorage lengths, defined in Equation 5.6d

a_v = 614.55 mm or 3d

T = $\sqrt{\frac{z}{3d}\frac{F_{ub}}{V_{uc}}}$

fub $f_{ub} = \frac{k k_{cov}\beta\sqrt{f_{cu}}}{\gamma_{mb}}$

K	=	1		acon	=	0.40	
β	=	0.39		c	=	52.3	mm
fcu	=	15	N/mm^2	ϕ	=	14.3	mm
γ_{mb}	=	1.25		kcov	=	1	

fub = 1.21

Fub = $F_{ub} = f_{ub} p L_a$

p = $p = \pi\phi$ for a single bar, = 44.92 mm

L_a = Length taken as 3d / 2 = 307.275 mm

Fub = 16680.67 N or 16.68 kN

Γ = $\Gamma = \min\begin{cases} \sqrt{\frac{z}{3d}\frac{F_{ub}}{V_{uc}}} \\ 1.0 \end{cases}$ = 7388816583
= 0.086 > 1

V_u	=	$\frac{3d}{a_v}\Gamma V_{uc}$	=	2.40	kN
	=	$\frac{0.24}{\gamma_{mv}}\xi_s(0.15 f_{cu})^{\frac{1}{3}} b_w d$	=	31929.48	N
				31.93	kN

Shear Resistance - Transverse Direction

Maximum shear resistance based on concrete crushing

Equation 5.6a Maximum shear resistance based on concrete crushing

$$V_{\max} = 0.36\left(0.7 - \frac{f_{cu}}{250}\right)\left(\frac{f_{cu}}{\gamma_{mc}}\right) b_w d$$

where:

b_w is the breadth of the section, taken as the web width for flanged beams

V_{Max} = 187.71

Shear resistance more than 3d from the support

Equation 5.6b Shear resistance more than 3d from a support

$$V_{uc} = \frac{0.24}{\gamma_{mv}} \xi_s \rho_s^{\frac{1}{3}} f_{cu}^{\frac{1}{3}} b_w d$$

Where:

γ_{mv} is the partial factor for shear defined in Section 2.

ξ_s is the depth factor, taken as $\xi_s = \left(\frac{500}{d}\right)^{0.25}$ but not less than 0.7

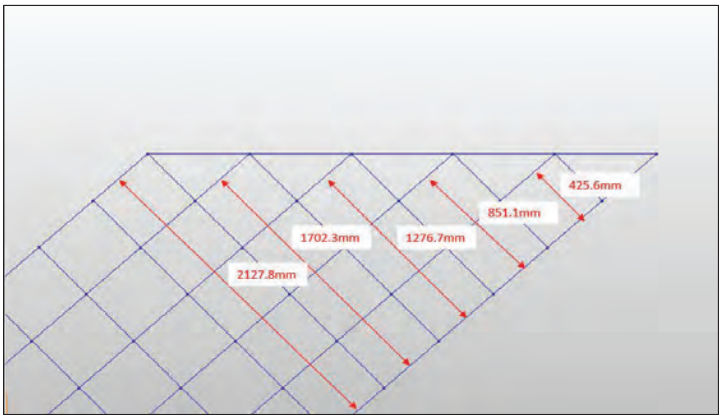
ρ_s is the ratio of longitudinal reinforcement $\rho_s = \frac{100A_s}{b_w d}$ but not less than 0.15, nor greater than 3.0

A_s is the area of longitudinal tension reinforcement that continues at least a distance d beyond the section being considered, and, for shear at supports, continues at least to the support.
Where top and bottom reinforcement are provided, the area in tension under the loading which produces the shear force is to be used.

ξ_s	=	1.27	<	0.7	OK
ρs	=	0.42			
Vuc	=	15.00	kN		

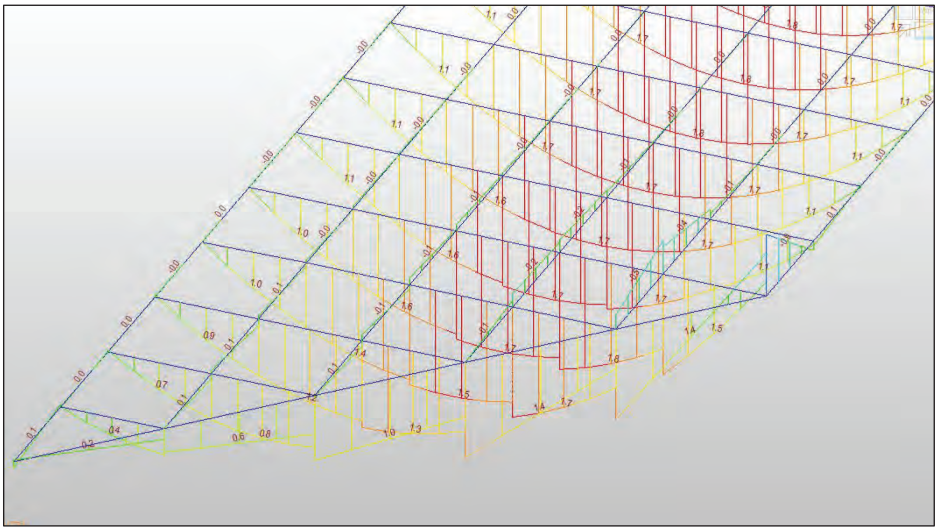
Deadload Capacity Check

Maximun Bending Moment due to deadload M_{ed} depending on length



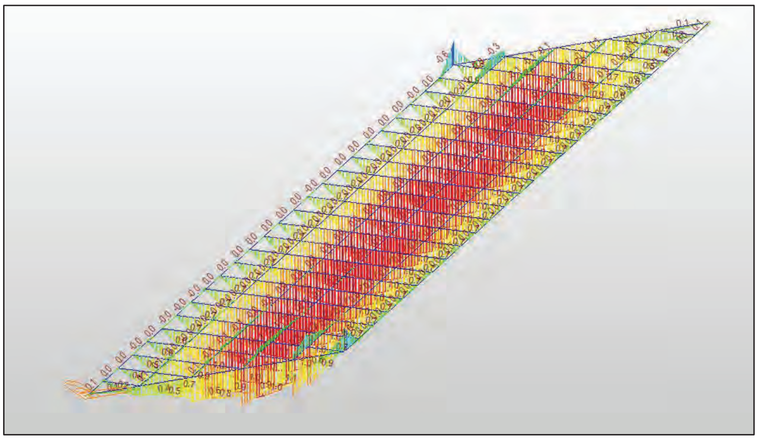
Member Lengths from MIDAS

Length (mm)	Length (m)	W (kNm)	Med (kNm)	MIDAS Results (kNm)	Section Resistance (kNm)	Result
425.6	0.4256	3.15	0.07	0.4	12.10	PASS
851.1	0.8511	3.15	0.29	0.8	12.10	PASS
1276.7	1.2767	3.15	0.64	1.3	12.10	PASS
1702.3	1.7023	3.15	1.14	1.5	12.10	PASS
2127.8	2.1278	3.15	1.78	1.70	12.10	PASS



MIDAS Results Screenshot

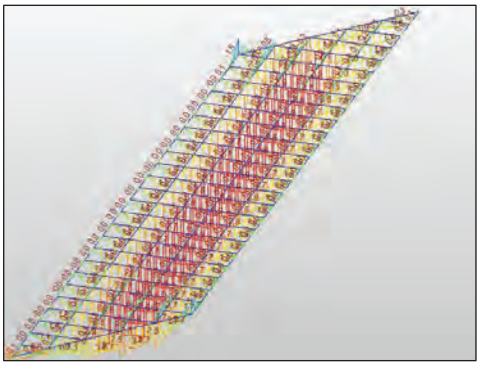
Length (mm)	Length (m)	W (kNm)	Med (kNm)	MIDAS Results (kNm)	Section Resistance (kNm)	Result
425.6	0.4256	1.92	0.04	0.5	12.10	PASS
851.1	0.8511	1.92	0.17	0.8	12.10	PASS
1276.7	1.2767	1.92	0.39	1.00	12.10	PASS
1702.3	1.7023	1.92	0.69	1.00	12.10	PASS
2127.8	2.1278	1.92	1.08	1.00	12.10	PASS



MIDAS Result Screenshot

Load Combination 1 - Deadload and Superimposed Deadload

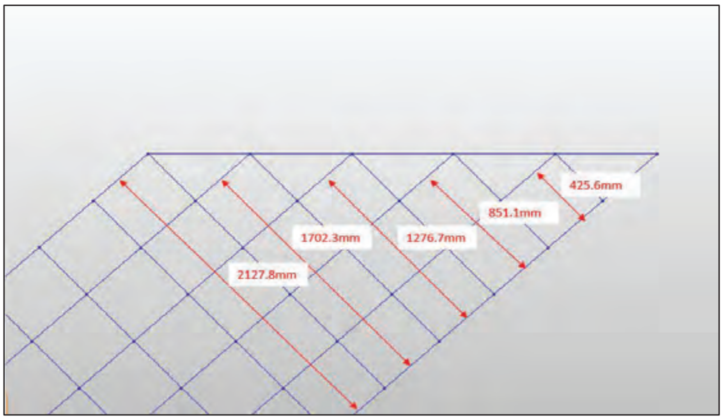
Length (mm)	Length (m)	W (kNm)	Med (kNm)	MIDAS Results (kNm)	Section Resistance (kNm)	Result
425.6	0.4256	5.07	0.11	1.3	12.10	PASS
851.1	0.8511	5.07	0.46	2.1	12.10	PASS
1276.7	1.2767	5.07	1.03	2.50	12.10	PASS
1702.3	1.7023	5.07	1.84	2.70	12.10	PASS
2127.8	2.1278	5.07	2.87	2.70	12.10	PASS



MIDAS Result Screenshot

Deadload Capacity Check - Shear Longitudinal Direction

Maximum Shear Load due to deadload V_{ed} depending on length



Member length from MIDAS

Length (mm)	Length (m)	W (kNm)	Ved (kN)	MIDAS Results (kN)	Section Resistance (kN)	Result
425.6	0.4256	3.15	0.67	-1.6	31.93	PASS
851.1	0.8511	3.15	1.34	-2.3	31.93	PASS
1276.7	1.2767	3.15	2.01	-2.8	31.93	PASS
1702.3	1.7023	3.15	2.68	-3	31.93	PASS
2127.8	2.1278	3.15	3.35	-3.1	31.93	PASS

Maximum Shear Load due to deadload and superimposed deadload

Length (mm)	Length (m)	W (kNm)	Ved +- (kN)	MIDAS Results (kN)	Section Resistance (kN)	Result
425.6	0.4256	5.07	1.08	-2.6	31.93	PASS
851.1	0.8511	5.07	2.16	-3.8	31.93	PASS
1276.7	1.2767	5.07	3.23	-4.4	31.93	PASS
1702.3	1.7023	5.07	4.31	-4.8	31.93	PASS
2127.8	2.1278	5.07	5.39	-5	31.93	PASS

Live Loading - All Model 1

Note: ALL Model 2 will not be applied to the carriageway slab due to the element complying with section 5.6 noted 4 of CS 454.

Impact Factors and Lane Widths

Load Situation	Impact factor applied to critical axle Poor Road Surface	Notional lane width (m)	Maximum lateral spacing between wheel centres of adjacent vehicles (m)
Single vehicle in each lane	1.8	3	1.2
Convoy of vehicles in each lane	1	2.5	0.7

Traffic Flow Factors

Traffic Flow Category	Traffic Flow Factor
Low	0.9

Lane Factors for ALL Model 1

Lane	Lane Factor
Lane 1	1
Lane 2	1
Lane 3	0.5
Lane 4 and subsequent	0.4

Partial Factor for traffic actions = 1.5

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Material & Section Properties

Reinforced Concrete

CS 455, Cl.3.1.3	Characteristic Strength of Concrete		=	20	N/mm^2
CS 454 table 4.1.1a	Density of Reinforced Concrete		=	24	kN/m^3

Steel Reinforcement

CS455, Cl, 3.8.2	Mild steel reinforcement characteristic yield strength		=	250	N/mm^2
	Design Young's Modulus of steel reinforcement		=	210000	N/mm^2

Mass Concrete (Plain Concrete)

CS 454 table	Unit weight of plain concrete		=	23	kN/m^3

Fill - Miscellaneous

CS 454 table 4.1.1a	Unit weight of miscellaneous fill		=	22	kN/m^3

Carriageway & Footway Surfacing

CS 454 table 4.1.1a	Unit weight of Bituminous Macadam (tar)		=	24	kN/m^3

Partial Factors

Table 2.13a	Partial Factor for reinforcement	γ_{ms}	=	1.15
Table 2.13a	Partial Factor for Concrete	γ_{mc}	=	1.5
Table 2.13a	Partial Factor for shear in concrete	γ_{mv}	=	1.15
CS 454 Table A.1	Partial Factor for Concrete Deadload		=	1.15
CS 454 Table A.1	Partial Factor for Surfacing		=	1.75
CS 454 Table A.1	Partial Factor for Fill		=	1.2

Material Thicknesses
These dimensions have be scaled of drawing B 2100/1B

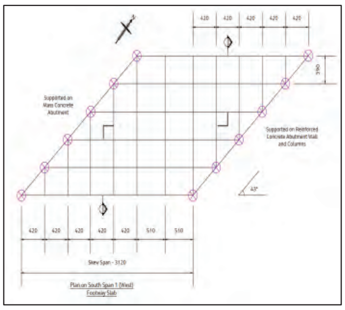
Thickness of Surfacing (assumed) = 100 mm or 0.1 m

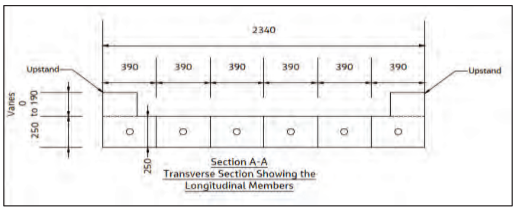
Thickness of Misc Fill =

Height at top fill	=	40.05	m
Height of fill minus surfacing	=	39.95	m
Height of top of upstand	=	39.085	m
Average depth of upstand	=	0.095	m
Thickness of Fill	=	0.960	m

Slab Dimensions

Skew Span	=	3.12	m
Square Span	=	2.13	m
Slab Depth	=	0.25	m
Skew of slab	=	43	°
Width of slab	=	2.34	m

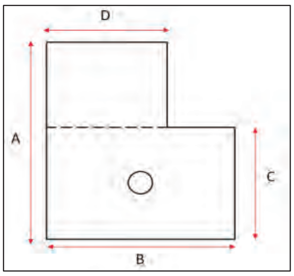




Autocad Grillage Model

Member Dimensions

Longitudinal Edge Memebers & Self Weight



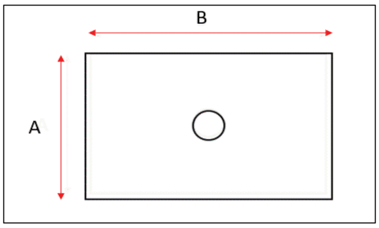
Section	A (m)	B (m)	C (m)	D (m)	Area (m^2)
Edge Member	0.345	0.390	0.250	0.250	0.12125

Self weight of section = 2.91 kN/m (unfactored)

Self weight of section = 3.35 kN/m (factored)

Reaction due to self weight = 10.44 kN (factored) per full length of longitudinal member

Longitudinal Member & Self Weight



Section	A (m)	B (m)	Area (m^2)
Edge Member	0.250	0.390	0.098

Self weight of section = 2.34 kN/m (unfactored)

Self weight of section = 2.69 kN/m (factored)

Total Reaction due to self weight = 8.40 kN (factored) per full length of longitudinal member

Total reactions per member group for self weight

Longitudinal Upstand Member = 20.88 kN

Longitudinal Member = 33.58 kN

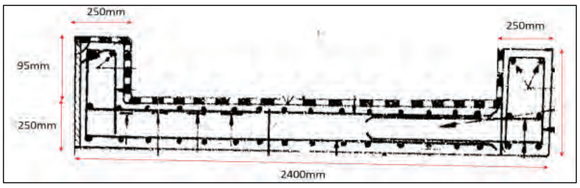
Total Rections = 54.47 kN

Midas Reactions = 54.48 kN

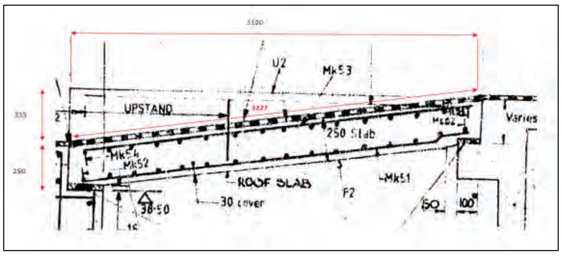
Node	Load	FX (kN)	FY (kN)	FZ (kN)	MX (kN*m)	MY (kN*m)	MZ (kN*m)
1	Deadload	0.000000	0.000000	1.925334	0.000000	0.000000	0.000000
2	Deadload	0.000000	0.000000	3.615398	0.000000	0.000000	0.000000
3	Deadload	0.000000	0.000000	3.602300	0.000000	0.000000	0.000000
4	Deadload	0.000000	0.000000	3.963500	0.000000	0.000000	0.000000
5	Deadload	0.000000	0.000000	-0.182375	0.000000	0.000000	0.000000
6	Deadload	0.000000	0.000000	14.317785	0.000000	0.000000	0.000000
43	Deadload	0.000000	0.000000	14.145091	0.000000	0.000000	0.000000
44	Deadload	0.000000	0.000000	0.057596	0.000000	0.000000	0.000000
45	Deadload	0.000000	0.000000	3.939801	0.000000	0.000000	0.000000
46	Deadload	0.000000	0.000000	3.588135	0.000000	0.000000	0.000000
47	Deadload	0.000000	0.000000	3.592573	0.000000	0.000000	0.000000
48	Deadload	0.000000	0.000000	1.915174	0.000000	0.000000	0.000000
SUMMATION OF REACTION FORCES PRINTOUT							
	Load	FX (kN)	FY (kN)	FZ (kN)			
	Deadload	0.000000	0.000000	54.480311			

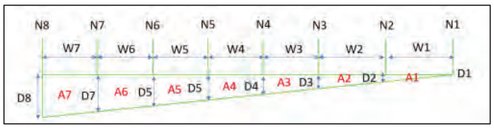
Midas Reaction Output

<u>Deadload Calculations Based on Archive Drawings</u>



Section through Span





Upstand Area - Tapering Section

AutoCAD

Node	Depth (m)	Width (m)	Area (m^2)		Density of Concrete	Unit Weight (kN/m)
1	0	0.250	0.006875		24	0.165
2	0.055	0.250	0.020625		24	0.495
3	0.109	0.250	0.032875		24	0.789
4	0.154	0.250	0.04425		24	1.062
5	0.200	0.250	0.055625		24	1.335
6	0.245	0.250	0.066875		24	1.605
7	0.290	0.250	0.078125		24	1.875
8	0.335	0.250	Σ	0.30525		

Maximum loads on nodes to be applied in MIDAS due to upstand

Node	Area in CS (m^2)	Unit Weight of tapering	Total Unit weight of upstand	Total Unit weight of upstand (Factored)
1	not required	0	2.34	2.69
3	0.02725	0.654	2.994	3.44
5	0.05	1.2	3.54	4.07
8	0.08375	2.01	4.35	5.00

Calculating bending for the upstands using a trapezoidal bending moment equation to verify the bending moment shown in MIDAS

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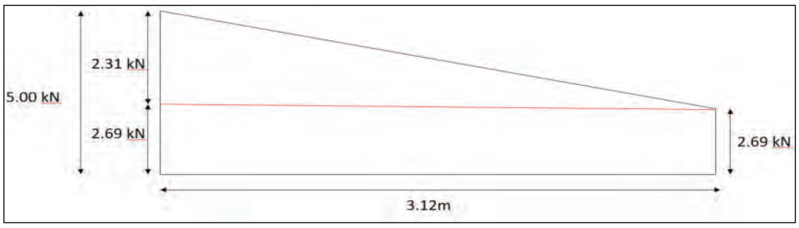
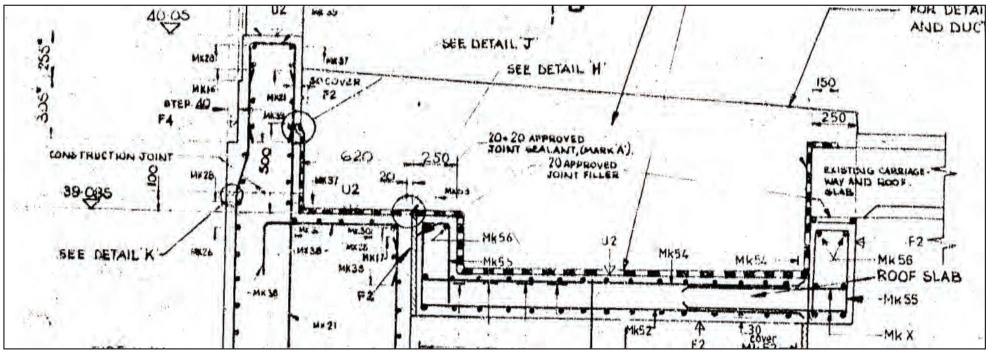


Total Load = 11.9964 = Total Reaction

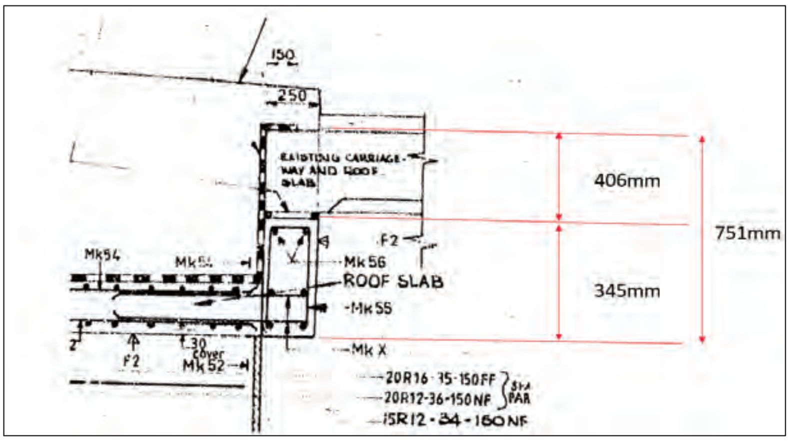
Node	Load	FX (kN)	FY (kN)	FZ (kN)	MX (kN*m)	MY (kN*m)	MZ (kN*m)
1	Deadload	0.000000	0.000000	2.766121	0.000000	0.000000	0.000000
2	Deadload	0.000000	0.000000	2.784458	0.000000	0.000000	0.000000
3	Deadload	0.000000	0.000000	1.144919	0.000000	0.000000	0.000000
4	Deadload	0.000000	0.000000	0.329978	0.000000	0.000000	0.000000
5	Deadload	0.000000	0.000000	0.183905	0.000000	0.000000	0.000000
6	Deadload	0.000000	0.000000	-0.609589	0.000000	0.000000	0.000000
43	Deadload	0.000000	0.000000	8.719179	0.000000	0.000000	0.000000
44	Deadload	0.000000	0.000000	-3.250696	0.000000	0.000000	0.000000
45	Deadload	0.000000	0.000000	0.135300	0.000000	0.000000	0.000000
46	Deadload	0.000000	0.000000	-0.101367	0.000000	0.000000	0.000000
47	Deadload	0.000000	0.000000	-0.047056	0.000000	0.000000	0.000000
48	Deadload	0.000000	0.000000	-0.055520	0.000000	0.000000	0.000000
SUMMATION OF REACTION FORCES PRINTOUT							
	Load	FX (kN)	FY (kN)	FZ (kN)			
	Deadload	0.000000	0.000000	11.999631			

MIDAS total reaction = 11.999 = OK!

<u>Superimposed Deadload</u>



Section of South West Footway Slab



North Upstand - Section Depths

Due to the varyingof the footway slab below is a summary of the levels and weight of fill for superimposed deadload.

Fill Material

Element	Surface Level (m)	Level at Top of Member (m)	Depth of Fill (m)	Unit Weight (kN/m^3)	
South Upstand	39.9225	39.085	0.7375	7.59	Factored
Longitudinal Member	39.9225	38.99	0.8325	8.57	Factored
North Upstand	39.9225	39.741	0.0815	0.84	Factored

Surfacing

Element	Surface Level (m)	Width of Section	Unit Weight (kN/m^3)	
South Upstand	0.1	0.390	1.638	Factored
Longitudinal Member	0.1	0.390	1.638	Factored
North Upstand	0.1	0.390	1.638	Factored

Total Superimposed Deadload to be applied to each member

Element	Total Load (kN/m^3)	
South Upstand	9.23	Factored
Longitudinal Member	10.21	Factored
North Upstand	2.48	Factored

Additional Deadload from Carriageway Slab on the North Upstand

Note: Only the carriageway downstand taken into consideration for the contribution to deadload.

Element	Depth	Width	Unit Weight kN/m	Factored Unit Weight (kN/m)
North Upstand	0.406	1	9.744	11.21

Section Capacities

Moment Resistance of Reinforced Concrete longitudinal members (longitudinal direction)

B2100/1B

Bar type	Bar Dia (mm)	Area of Bar (mm^2)	Spacing Centre to Centre (mm)
Top Compression Bars	16	201.06	150
Bottom Tension Bars	25	490.87	150

Grillage Model Dims

Width of section = 390 mm

Height of Section = 250 mm

Cross Sectional Area = 97500 mm^2

Total Number Compression Bars per section width = 2.6 or 2.5 Bars

Total Number Tension Bars per section width = 2.6 or 2.5 Bars

Area of compression reinforcement per section = 502.65 mm^2

Area of tension reinforcement per section = 1276.27 mm^2

Cover to reinforcement = 30 mm

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<u>Bending equation considering compression reinforcement</u>

Equation 5.2.2c Moment resistance for beams with compression reinforcement

$$M_u = \frac{0.6 f_{cu}}{\gamma_{mc}} bx(d - 0.5x) + f'_s A'_s \left(d - d'\right)$$

where:

A'_s	is the area of compression reinforcement
d'	is the depth to the compression reinforcement from the extreme fibre in compression
f'_s	is equal to $\frac{f_y}{\gamma_{ms} + \frac{f_y}{2000}}$
x	is the neutral axis depth, which may be determined from Equation 5.2.2d

x	=	125	mm	(rectangular section)
d'	=	46	mm	
f'_s	=	196.08	mm	
d	=	220	mm	

M_u	=	78574400	kN/mm	or	78.57	kN/m

Bending Equation Considering Tension Reinforcement Only.

Equation 5.2.2a Moment resistance for beams without compression reinforcement)

$$M_u = \min \left\{ \begin{array}{l} \frac{f_y}{\gamma_{ms}} A_s z \\ \frac{0.225 f_{cu}}{\gamma_{mc}} bd^2 \end{array} \right.$$

where:

M_u	is the moment resistance
f_y	is the characteristic or worst credible strength of the reinforcement
γ_{ms}	is the partial factor for the reinforcement strength
A_s	is the area of tension reinforcement
z	is the lever arm, determined from Equation 5.2.2b, but no greater than $0.95d$
f_{cu}	is the characteristic or worst credible cube strength of the concrete
γ_{mc}	is the partial factor for the concrete strength
b	is the width of the section in compression at the level of the neutral axis
d	is the effective depth of the tension reinforcement from the extreme fibre in compression

Equation 5.2.2b Lever arm for beams without compression reinforcement

$$z = \left(1 - \frac{0.84 \frac{f_y A_s}{\gamma_{ms}}}{\frac{f_{cu} bd}{\gamma_{mc}}}\right) d$$

d = 208 mm

Z = 162.68 mm

Mu = $\frac{250}{1.15}$ x 1276.27 x 162.68 = 45.14 kNm

Mu = $\frac{0.225}{1.5}$ x 20 x 390 x 208^2 = 50.38 kNm

Mu	=	45.14	kNm

<u>Moment Resistance of Reinforced Concrete longitudinal members (Transverse direction)</u>

B2100/1B

Bar type	Bar Dia (mm)	Area of Bar (mm^2)	Spacing Centre to Centre (mm)
Top Compression Bars	16	201.06	150
Bottom Tension Bars	25	490.87	150

Grillage Model Dims

Note: There is two section width is the transverse direction which need to be calculated. The Minimum width of 420mm will be taken into considetation of the moment capacity.

Width of Section	=	420	mm		
Depth of section	=	250	mm		
Cross Sectional Area	=	105000	mm^2		
Total Number of compression bars	=	2.8	or	2	Bars

Total Number of Tension Bars = 2.8 or 2 Bars

Area of compression reinforcment per section = 562.97 mm^2

Area of Tension reinforcement per section = 981.75 mm^2

Cover to reinforcement = 30 mm

Bending Equation using compression reinforcement

Equation 5.2.2c Moment resistance for beams with compression reinforcement

$$M_u = \frac{0.6 f_{cu}}{\gamma_{mc}} bx(d - 0.5x) + f'_s A'_s (d - d')$$

where:

A'_s is the area of compression reinforcement

d' is the depth to the compression reinforcement from the extreme fibre in compression

f'_s is equal to $\frac{f_y}{\gamma_{ms} + \frac{f_y}{2000}}$

x is the neutral axis depth, which may be determined from Equation 5.2.2d

x = 125 mm (rectangular section)
d' = 46 mm
f'_s = 217.5163043 mm
d = 220 mm

Mu = 87457326 kN/mm or 87.46 kN/m

Bending Equation using tension reinforcement only

Equation 5.2.2a Moment resistance for beams without compression reinforcement)

$$M_u = \min \left\{ \begin{array}{l} \frac{f_y}{\gamma_{ms}} A_s z \\ \frac{0.225 f_{cu}}{\gamma_{mc}} bd^2 \end{array} \right.$$

where:

M_u is the moment resistance

f_y is the characteristic or worst credible strength of the reinforcement

γ_{ms} is the partial factor for the reinforcement strength

A_s is the area of tension reinforcement

z is the lever arm, determined from Equation 5.2.2b, but no greater than $0.95d$

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

γ_{mc} is the partial factor for the concrete strength

b is the width of the section in compression at the level of the neutral axis

d is the effective depth of the tension reinforcement from the extreme fibre in compression

Equation 5.2.2b Lever arm for beams without compression reinforcement

$$z = \left(1 - \frac{0.84 \frac{f_y A_s}{\gamma_{ms}}}{\frac{f_{cu} bd}{\gamma_{mc}}}\right) d$$

d = 182.5 mm

Z = 150.49 mm

Mu = $\frac{250}{1.15}$ x 981.75 x 150.49 = 32.12 kNm

Mu = $\frac{0.225}{1.5}$ x 20 x 420.00 x 182.5^2 = 41.97 kNm

Mu = 32.12 kNm

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Moment Capacity of edge members.

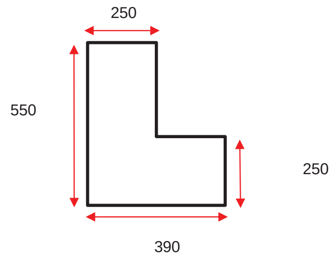
Bar type	Bar Dia (mm)	Area of Bar (mm^2)	Spacing Centre to Centre (mm)
Top Compression Bars	16	201.06	150
Bottom Tension Bars	25	490.87	150

Grillage Model Dims

Overall Width = 390 mm

Height of section (max depth) = 550 mm Assumed height to match CAT II

Cross Sectional Area = 172500 mm^2



Total Number of bars per section = 2.6 or 2.50 bars

Area of tension reinfocement = 1276.27 mm^2

Cover to Reinforcement = 30 mm

Effective depth, d = 507.5 mm

Bending Equation using tension reinforcement only

Equation 5.2.2a Moment resistance for beams without compression reinforcement)

$$M_u = \min \begin{cases} \frac{f_y}{\gamma_{ms}} A_s z \\ \frac{0.225 f_{cu}}{\gamma_{mc}} b d^2 \end{cases}$$

where:

M_u	is the moment resistance
f_y	is the characteristic or worst credible strength of the reinforcement
γ_{ms}	is the partial factor for the reinforcement strength
A_s	is the area of tension reinforcement
z	is the lever arm, determined from Equation 5.2.2b, but no greater than $0.95d$
f_{cu}	is the characteristic or worst credible cube strength of the concrete
γ_{mc}	is the partial factor for the concrete strength
b	is the width of the section in compression at the level of the neutral axis
d	is the effective depth of the tension reinforcement from the extreme fibre in compression

Equation 5.2.2b Lever arm for beams without compression reinforcement

$$z = \left(1 - \frac{0.84 \frac{f_y A_s}{\gamma_{ms}}}{\frac{f_{cu} b d}{\gamma_{mc}}}\right) d$$

Z = 462.68 mm

Mu = $\frac{250}{1.15}$ x 1276.27 x 462.68 = 128.37 kNm

Mu = $\frac{0.225 \times 20}{1.5}$ x 390.00 x 462.68 = 250.47 kNm

Mu = 128.37 kNm

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Shear Reistance without shear reinforcement - Longitudinal Direction

Equation 5.6a Maximum shear resistance based on concrete crushing

$$V_{\max} = 0.36\left(0.7 - \frac{f_{cu}}{250}\right)\left(\frac{f_{cu}}{\gamma_{mc}}\right)b_w d$$

where:

b_w is the breadth of the section, taken as the web width for flanged beams

V_{max} = 259358.4 N or 259.36 kN

Equation 5.6b Shear resistance more than 3d from a support

$$V_{uc} = \frac{0.24}{\gamma_{mv}}\xi_s \rho_s^{\frac{1}{3}} f_{cu}^{\frac{1}{3}} b_w d$$

where:

γ_{mv} is the partial factor for shear defined in Section 2.

ξ_s is the depth factor, taken as $\xi_s = \left(\frac{500}{d}\right)^{0.25}$ but not less than 0.7

ρ_s is the ratio of longitudinal reinforcement $\rho_s = \frac{100A_s}{b_w d}$ but not less than 0.15, nor greater than 3.0

A_s is the area of longitudinal tension reinforcement that continues at least a distance d beyond the section being considered, and, for shear at supports, continues at least to the support.
Where top and bottom reinforcement are provided, the area in tension under the loading which produces the shear force is to be used.

ξs = 1.25
ρs = 1.58

V_{uc} = 66.48 kN

Equation 5.6c Shear resistance within 3d of a support

$$V_u = \max\begin{cases} \frac{3d}{a_v}\Gamma V_{uc} \\ \frac{0.24}{\gamma_{mv}}\xi_s(0.15 f_{cu})^{\frac{1}{3}} b_w d \end{cases}$$

where:

a_v is the distance of the section measured from the edge of a rigid bearing, the centre-line of a flexible bearing or the face of a support, where $d \leq a_v \leq 3d$

Γ is the factor to account for short anchorage lengths, defined in Equation 5.6d

Equation 5.6d Factor to account for the effect of short anchorage lengths

$$\Gamma = \min\begin{cases} \sqrt{\frac{z}{3d}\frac{F_{ub}}{V_{uc}}} \\ 1.0 \end{cases}$$

where:

F_{ub} is the total anchorage force that can be developed in the longitudinal tension reinforcing bars at the front face of the support according to Section 9, but not greater than $\frac{A_s f_y}{\gamma_{ms}}$

z is the flexural lever arm at ULS at a position $3d$ from the support calculated from Equation 5.2.2b

av = 623 mm

Z = 162.68 mm

T = $\sqrt{\frac{z}{3d}\frac{F_{ub}}{V_{uc}}}$

fub = $f_{ub} = \frac{k k_{cov}\beta\sqrt{f_{cu}}}{\gamma_{mb}}$

K	=	1	acon	=	0.40	
β	=	0.39	c	=	30	mm
fcu	=	20	ϕ	=	25	
γ_{mb}	=	1.4	kcov	=	1	

fub = 1.25

Fub = $F_{ub} = f_{ub} p L_a$

P = $p = \pi\phi$ for a single bar, = 157.08 mm^2

La = Length taken as d = 623 mm

Fub = 121.82 kN

Γ $$\Gamma = \min \begin{cases} \sqrt{\frac{z}{3d}\frac{F_{ub}}{V_{uc}}} \\ 1.0 \end{cases}$$ = 0.40 < 1

Γ = 0.40

Equation 5.6c Shear resistance within 3d of a support

$$V_u = \max \begin{cases} \frac{3d}{a_v}\Gamma V_{uc} \\ \frac{0.24}{\gamma_{mv}}\xi_s(0.15 f_{cu})^{\frac{1}{3}} b_w d \end{cases}$$

Vu = $\frac{3d}{a_v}\Gamma V_{uc}$ = 79.68 kN

Vu $\frac{0.24}{\gamma_{mv}}\xi_s(0.15 f_{cu})^{\frac{1}{3}} b_w d$ = 30.35 kN

__Edge Section Shear Resistance__

Equation 5.6a Maximum shear resistance based on concrete crushing

$$V_{\max} = 0.36\left(0.7 - \frac{f_{cu}}{250}\right)\left(\frac{f_{cu}}{\gamma_{mc}}\right) b_w d$$

where:

b_w is the breadth of the section, taken as the web width for flanged beams

V_{MAX} = 589.02 kN

Equation 5.6b Shear resistance more than 3d from a support

$$V_{uc} = \frac{0.24}{\gamma_{mv}}\xi_s \rho_s^{\frac{1}{3}} f_{cu}^{\frac{1}{3}} b_w d$$

where:

γ_{mv} is the partial factor for shear defined in Section 2.

ξ_s is the depth factor, taken as $\xi_s = \left(\frac{500}{d}\right)^{0.25}$ but not less than 0.7

ρ_s is the ratio of longitudinal reinforcement $\rho_s = \frac{100 A_s}{b_w d}$ but not less than 0.15, nor greater than 3.0

A_s is the area of longitudinal tension reinforcement that continues at least a distance d beyond the section being considered, and, for shear at supports, continues at least to the support.
Where top and bottom reinforcement are provided, the area in tension under the loading which produces the shear force is to be used.

ξs = 1.00
ρs = 0.64

Vuc = 96.51 kN

Equation 5.6c Shear resistance within 3d of a support

$$V_u = \max \begin{cases} \frac{3d}{a_v}\Gamma V_{uc} \\ \frac{0.24}{\gamma_{mv}}\xi_s(0.15f_{cu})^{\frac{1}{3}}b_w d \end{cases}$$

where:

a_v is the distance of the section measured from the edge of a rigid bearing, the centre-line of a flexible bearing or the face of a support, where $d \le a_v \le 3d$

Γ is the factor to account for short anchorage lengths, defined in Equation 5.6d

Equation 5.6d Factor to account for the effect of short anchorage lengths

$$\Gamma = \min \begin{cases} \sqrt{\frac{z}{3d}\frac{F_{ub}}{V_{uc}}} \\ 1.0 \end{cases}$$

where:

F_{ub} is the total anchorage force that can be developed in the longitudinal tension reinforcing bars at the front face of the support according to Section 9, but not greater than $\frac{A_s f_y}{\gamma_{ms}}$

z is the flexural lever arm at ULS at a position $3d$ from the support calculated from Equation 5.2.2b

av = 507.5 mm

Z = 462.68 mm

T $\sqrt{\frac{z}{3d}\frac{F_{ub}}{V_{uc}}}$

fub = $f_{ub} = \frac{k k_{cov}\beta\sqrt{f_{cu}}}{\gamma_{mb}}$

K	=	1	acon	=	0.40
β	=	0.39	c	=	30
fcu	=	20	ϕ	=	25
γ_{mb}	=	1.4	kcov	=	1

fub = 1.25

Fub = $F_{ub} = f_{ub} p L_a$

P = $p = \pi\phi$ for a single bar, = 78.540 mm

La = length taken as d = 507.5 mm

Fub = 49.66 kN

r $\Gamma = \min \begin{cases} \sqrt{\frac{z}{3d}\frac{F_{ub}}{V_{uc}}} \\ 1.0 \end{cases}$ = 0.40 < 1.0

r = 0.40

Equation 5.6c Shear resistance within 3d of a support

$$V_u = \max \begin{cases} \frac{3d}{a_v}\Gamma V_{uc} \\ \frac{0.24}{\gamma_{mv}}\xi_s(0.15f_{cu})^{\frac{1}{3}}b_w d \end{cases}$$

Vu = $\frac{3d}{a_v}\Gamma V_{uc}$ = 114.49 kN

Vu = $\frac{0.24}{\gamma_{mv}}\xi_s(0.15f_{cu})^{\frac{1}{3}}b_w d$ = 83.70 kN

Footway Loading - Live Load

Pedestrain Load

Loaded length, L: (0 < L ≤ 36)

Pedestrian live load	=	5.0	kN/m^2
Pedestrian live load factor	=	1.0	
Width factor	=	0.95	
Pedestrian Live Load	=	4.75	kN/m^2

AVL more onerous and cannot be applied at the same time

Accidental vehicle loading (normal traffic)

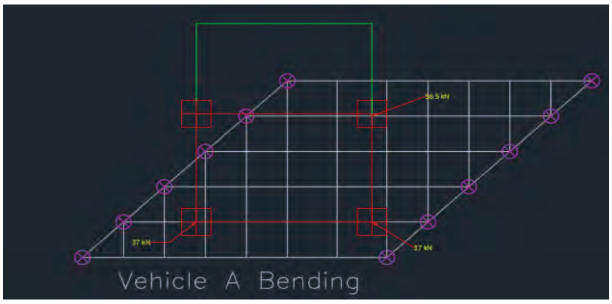
Accidental wheel load is firstly being as applied as a point load according to the most amount of wheels on the member. Due to the skew of the slab only a maximum of three wheel can fit onto the slab at any time

Vehicle Type A

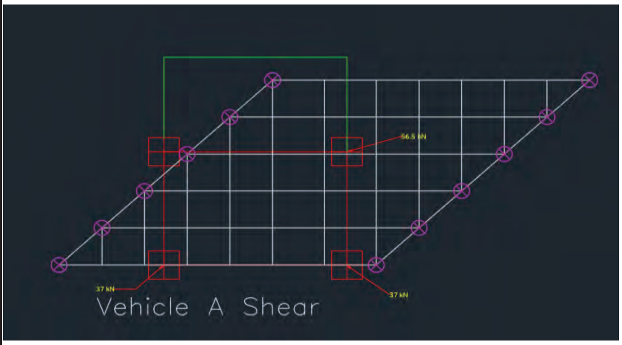
Heaviest Axles

W3 (kN)	Point load (kN)	A1 (m)	W4 (kN)	Point Load (kN)	(factored)
113	84.75	1.3	74	56	

Worse Case Arrangement based on Bending



Worse Case Arrangement based on Shear



Material & Section Properties

Reinforced Concrete

Reference	Description			Value	Unit
CS 455, Cl.3.1.3	Characteristic Strength of Concrete		=	15.00	N/mm^2
CS 454 table 4.1.1a	Density of Reinforced Concrete		=	24	kN/m^3

Steel Reinforcement

Reference	Description			Value	Unit
CS455, Cl, 3.8.2	Mild steel reinforcement characteristic yield strength		=	230	N/mm^2
	Design Young's Modulus of steel reinforcement		=	210000	N/mm^2

Mass Concrete (Plain Concrete)

Reference	Description			Value	Unit
CS 454 table 4.1.1a	Unit weight of plain concrete		=	23	kN/m^3

Fill - Miscellaneous

Reference	Description			Value	Unit
CS 454 table 4.1.1a	Unit weight of miscellaneous fill		=	22	kN/m^3

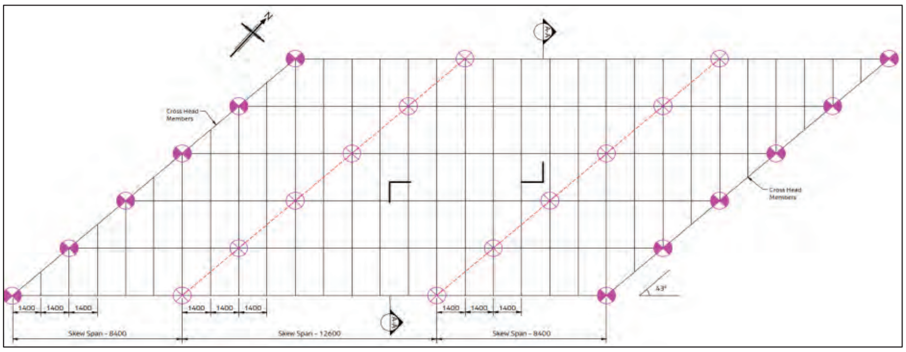
Carriageway & Footway Surfacing

Reference	Description			Value	Unit
CS 454 table 4.1.1a	Unit weight of Bituminous Macadam (tar)		=	24	kN/m^3

Partial Factors

CS 455 Table 2.13a	Partial Factor for reinforcement	γ_{ms}	=	1.15
CS 455 Table 2.13a	Partial Factor for Concrete	γ_{mc}	=	1.5
CS 455 Table 2.13a	Partial Factor for shear in concrete	γ_{mv}	=	1.15
CS 455 Table 2.13a	Partial Factor for Bond	γ_{mb}	=	1.25
CS 454 Table A.1	Partial Factor for Concrete Deadload		=	1.15
CS 454 Table A.1	Partial Factor for Surfacing		=	1.75
CS 454 Table A.1	Partial Factor for Fill		=	1.2
CS 454 3.9	Inaccurate assessment effects at ULS	$\gamma f3$	=	1.1

Grillage Model Span Dimmensions



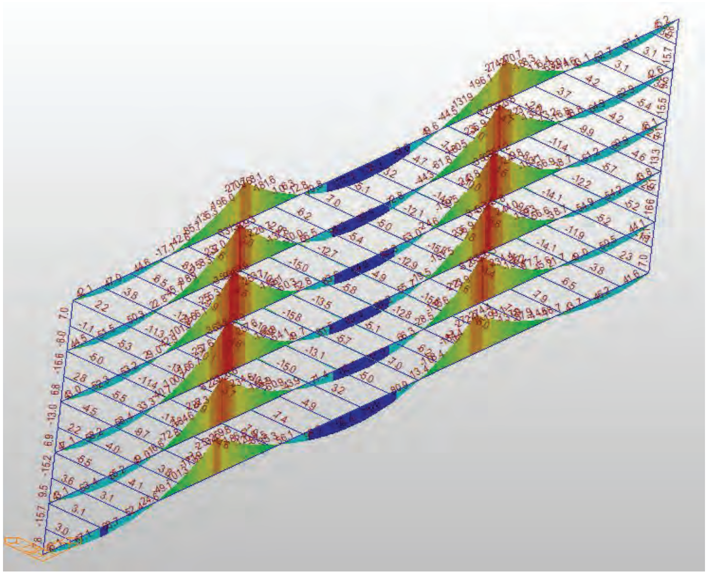
MIDAS Grillage Model

Spans

Main Span 2 Skew Span	=	8.4	m
Main Span 3 Skew Span	=	12.6	m
Main Span 3 Skew Span	=	8.4	m

Deadload Calculations

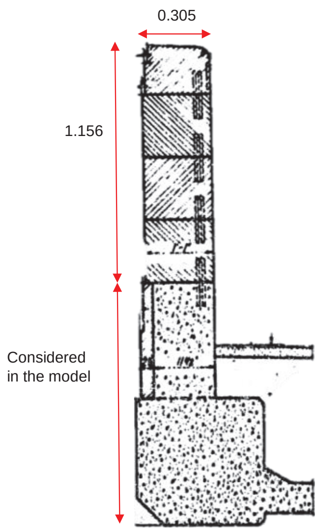
Deadload has been applied using the MIDAS Self Weight Function. Results from the Model shows that bending moment diagrams are working as expected.



Parapet Deadload

The bottom reinforced concrete section below the concrete parapet blocks have been taken into account in the parapet beam.

B2100/8

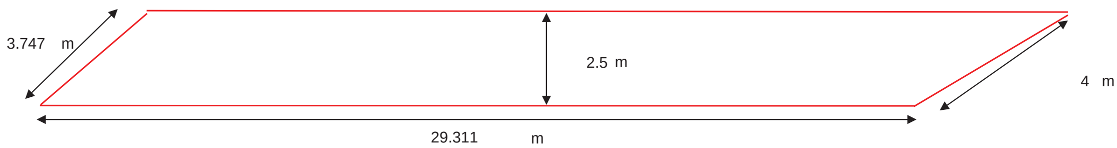


Cross Sectional Area	=	0.35	m^2
Unit weight per m	=	8.46	kN/m
Unit weight per m	=	9.73	kN/m (Factored)

Section through parapet

Superimposed Deadload

North Footway



Typical Plan on Footway

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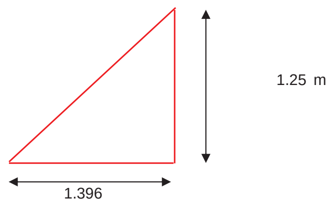
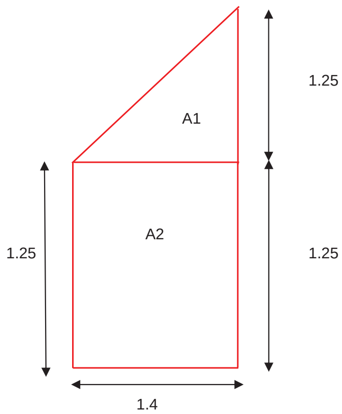
Traingular distribution on span ends



Cross Section Area of fill	=	0.744	x	1.25	=	1	m^2
Unit weight	=	0.93	x	22	=	20	kN/m
Unit weight per m for triangle	=	$\frac{20.46}{2}$			=	10	kN/m

Deadloading at acute corners



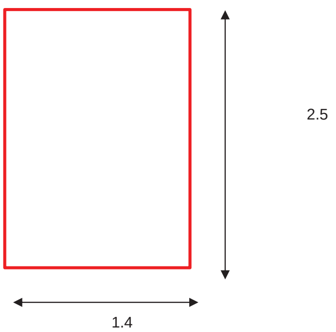
Area 1

Cross section area of fill	=	0.744	x	1.25	=	1	m^2
Unit Weight	=	0.93	x	22	=	20	kN/m
Unit weight per m for triangle	=	$\frac{20.46}{2}$			=	10	kN/m

Area 2

Cross section area of fill	=	0.744	x	1.25	=	1	m^2
Unit Weight	=	0.93	x	22	=	20	kN/m

Normal Sqaure Section



Cross sectional area of fill	=	0.744	x	1.4	=	1	m^2
Unit Weight	=	1.0416	x	22	=	##	kN/m

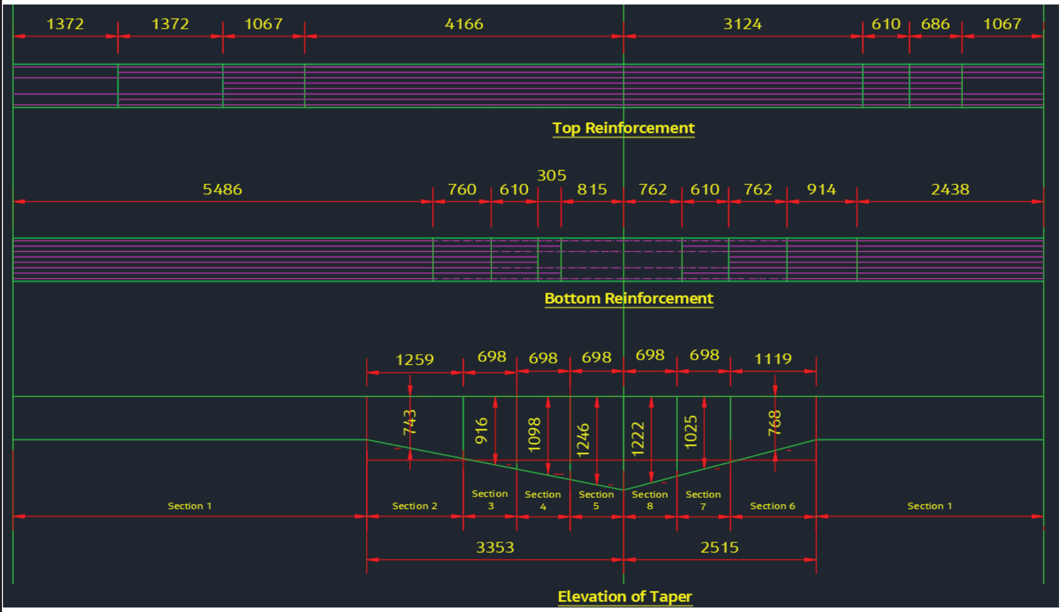
Carriageway Super Imposed Deadload

Using the same principles above, the superimposed deadload will be applied to the transverse memebers.

		Area		**Density**				
Triangular Distribution	=	0.125	x	24	=	$\frac{3}{2}$	=	2 kN/m
		Area 1		**Density**				
Acute Corners	=	0.125	x	24	=	$\frac{3}{2}$	=	2 kN/m
		Area 2		**Density**				
		0.125	x	24	=	3	kN/m	
		Area		**Density**				
Nomral Square Section	=	0.14	x	24	=	3.36	kN/m	

Bending Capacity for Carriageway Beam

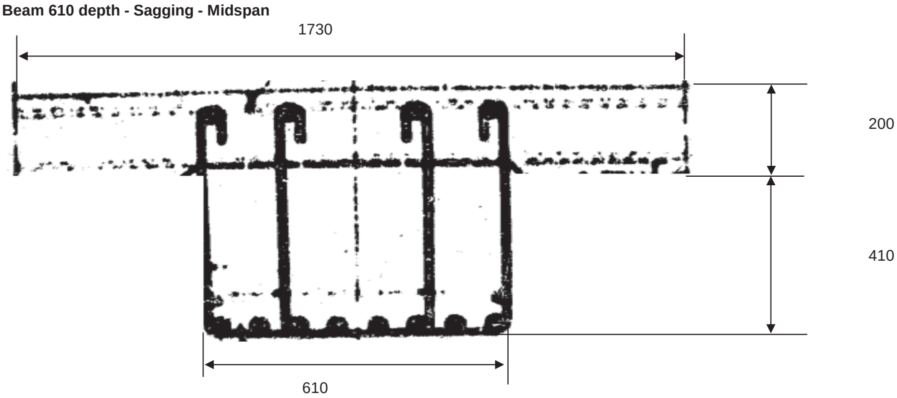
To calculate the bending capacity of the carriageway beam, the point of contraflexure must be defined. However, as this will changed from load case to load case. To overcome this, the beam will be checked at the points below where the reinforcement changes. Both hogging and sagging will be checked.



Reinforcement sketch of cway beam

Reinforced Concrete Beams - Moment Resistance of Beams

Section 1 - Carriageway Beam 610 depth - Sagging - Midspan



Internal Reinforcement for sagging - 610mm depth

Bar Type	Bar Dia (")	Bar Dia (mm)	Area of Bar (mm2)	Numer of Bars	Area of Reinforcement (mm^2)
Longitudinal Bottom Bar	1.375	34.93	957.99	8	7663.95
Longitudinal Top Bar	1.375	34.93	957.99	4	3831.98
Stirrups	0.375	9.525	71.26	N/A	N/A

CS455 Eq 5.2.4

Using Equation 5.2.4, Moment of resistance of flanged beams - Sagging

M_u = Minimum value dervied from the following equations

$$\min\left\{\begin{array}{l}\frac{f_y}{\gamma_{ms}} A_s\left(d-\frac{h_f}{2}\right)\\ \frac{0.6 f_{cu}}{\gamma_{mc}} b h_f\left(d-\frac{h_f}{2}\right)\end{array}\right.$$

Cover to reinforcement = 1 1/2 " = 38 mm

Effective Depth, d = 610 - 38 - 9.525 = 562.38 mm

Depth of Flange, hf = 200 mm

F_{cu} = 15 N/mm^2

$\frac{230}{1.15}$ x 7663.95 x 562.38 - $\frac{200}{2}$ = 708.72 kNm

$\frac{0.6 \quad 15}{1.5}$ x 1730 x 200 x 562.38 - $\frac{200}{2}$ = ## kNm

Mu = 708.72 kNm

Internal Reinforcement for sagging - 610mm depth with 2mm section loss

Bar Type	Bar Dia (")	Bar Dia (mm)	Area of Bar (mm2)	Numer of Bars	Area of Reinforcement (mm^2)
Longitudinal Bottom Bar	1.375	32.93	851.42	8	6811.32
Longitudinal Top Bar	1.375	34.93	957.99	4	3831.98
Stirrups	0.375	9.525	71.26	N/A	N/A

Using Equation 5.2.4, Moment of resistance of flanged beams - Sagging

M_u = Minimum value dervied from the following equations

$$\min\left\{\begin{array}{l}\frac{f_y}{\gamma_{ms}} A_s\left(d-\frac{h_f}{2}\right)\\ \frac{0.6 f_{cu}}{\gamma_{mc}} b h_f\left(d-\frac{h_f}{2}\right)\end{array}\right.$$

Cover to reinforcement = 1 1/2 " = 38 mm

Effective Depth, d = 610 - 38 - 9.525 = 562.38 mm

Depth of Flange, hf = 0 mm

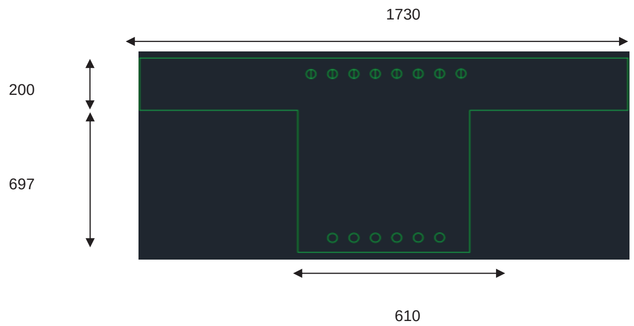
F_{cu} = 15 N/mm^2

$\frac{230}{1.15}$ x 6811.32 x 562.38 - $\frac{200}{2}$ = 629.88 kNm

$\frac{0.6 \quad 15}{1.5}$ x 1730 x 200 x 562.38 - $\frac{200}{2}$ = 959.89 kNm

Mu = 629.88 kNm

Section 2 - Carriageway Beam 897 depth - Sagging and Hogging



Internal Reinforcement

Bar Type	Bar Dia (")	Bar Dia (mm)	Area of Bar (mm2)	Numer of Bars	Area of Reinforcement (mm^2)
Longitudinal Bottom Bar	1.375	34.93	957.99	6	5747.96
Longitudinal Top Bar	1.375	34.93	957.99	8	7663.95
Stirrups	0.375	9.53	71.26	1	71.26

Using Equation 5.2.2, Moment of resistance of flanged beams - Hogging

M_u = Minimum value dervied from the following equations $M_u = \min \left\{ \begin{array}{l} \frac{f_y}{\gamma_{ms}} A_s z \\ \frac{0.225 f_{cu}}{\gamma_{mc}} b d^2 \end{array} \right.$

LC 5/5 Cover to top hogging bars = 1 1/4 " = 31.75 mm

d = 865.25 mm

Z = 654.18 mm

Hogging Bending Capacity Mu

$\frac{230}{1.15}$ x 7663.95 x 654.18 = 1002.72 kNm

$\frac{0.225 \times 15.00}{1.5}$ x 610 x 748657.6 = 1027.53 kNm

Mu = 1002.72 kNm

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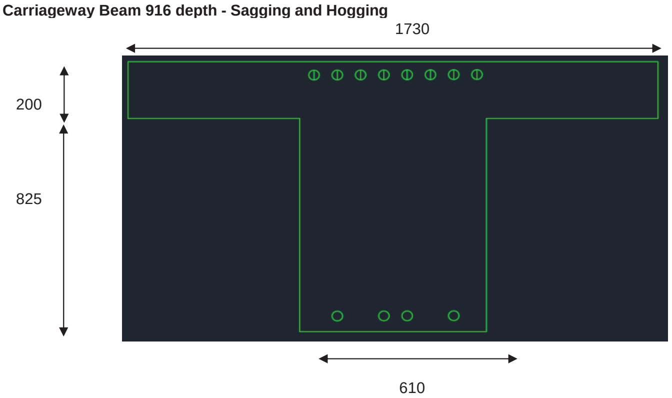
Section 3 - Carriageway Beam 916 depth - Sagging and Hogging



Internal Reinforcement

Bar Type	Bar Dia (")	Bar Dia (mm)	Area of Bar (mm2)	Numer of Bars	Area of Reinforcement (mm^2)
Longitudinal Bottom Bar	1.375	34.93	957.99	4	3831.98
Longitudinal Top Bar	1.375	34.93	957.99	8	7663.95
Stirrups	0.375	9.53	71.26	1	71.26

Using Equation 5.2.2, Moment of resistance of flanged beams - Sagging & Hogging

M_u = Minimum value dervied from the following equations

$$M_u = \min \begin{cases} \frac{f_y}{\gamma_{m}} A_s z \\ \frac{0.225 f_{cu}}{\gamma_{mc}} b d^2 \end{cases}$$

Cover to top hogging bars = 1 1/4" = 31.75 mm

d = 993.25 mm

Z = 782.18 mm

Hogging Bending Capacity

$\frac{230}{1.15}$ x 7663.95 x 782.18 = 1198.91 kNm

0.225 x $\frac{15.00}{1.5}$ x 610 x 986545.6 = 1354.03 kNm

Mu = 1198.91 kNm

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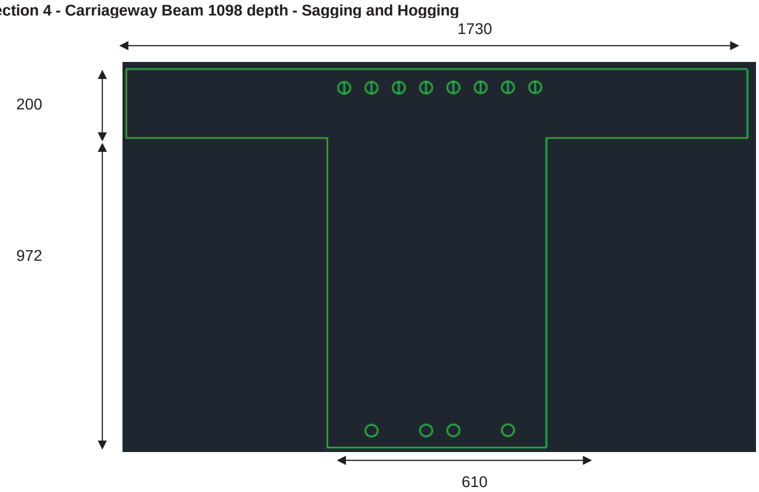
Section 4 - Carriageway Beam 1098 depth - Sagging and Hogging



Internal Reinforcement

Bar Type	Bar Dia (")	Bar Dia (mm)	Area of Bar (mm2)	Numer of Bars	Area of Reinforcement (mm^2)
Longitudinal Bottom Bar	1.375	34.93	957.99	4	3831.98
Longitudinal Top Bar	1.375	34.93	957.99	8	7663.95
Stirrups	0.375	9.53	71.26	1	71.26

Using Equation 5.2.2, Moment of resistance of flanged beams - Sagging & Hogging

M_u = Minimum value dervied from the following equations $M_u = \min \left\{ \begin{array}{l} \frac{f_y}{\gamma_{ms}} A_s z \\ \frac{0.225 f_{cu}}{\gamma_{mc}} b d^2 \end{array} \right.$

Cover to top hogging bars = 1 1/4" = 31.75 mm

d = 1140.25 mm

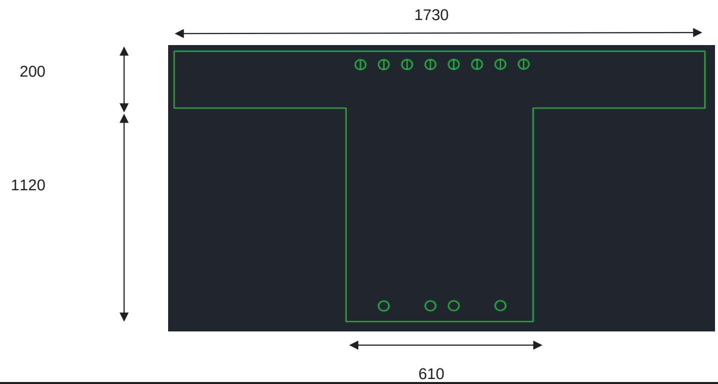
Z = 929.18 mm

Hogging Bending Capacity Mu

$\frac{230}{1.15}$ x 7663.95 x 929.18 = 1424.23 kNm

$\frac{0.225 \times 15.00}{1.5}$ x 610 x 1300170 = 1784.48 kNm
]

Mu = 1424.23 kNm



Bar Type	Bar Dia (")	Bar Dia (mm)	Area of Bar (mm2)	Numer of Bars	Area of Reinforcement (mm^2)
Longitudinal Bottom Bar	1.375	34.93	957.99	4	3831.98
Longitudinal Top Bar	1.375	34.93	957.99	8	7663.95
Stirrups	0.375	9.53	71.26	1	71.26

Using Equation 5.2.2, Moment of resistance beams -Hogging

M_u = Minimum value dervied from the following equations

$$M_u = \min \begin{cases} \frac{f_y}{\gamma_{ms}} A_s z \\ \frac{0.225 f_{cu}}{\gamma_{mc}} bd^2 \end{cases}$$

Cover to top hogging bars = 1 1/4 = 31.75 mm

d = 1288.25 mm

Z = 1077.18 mm

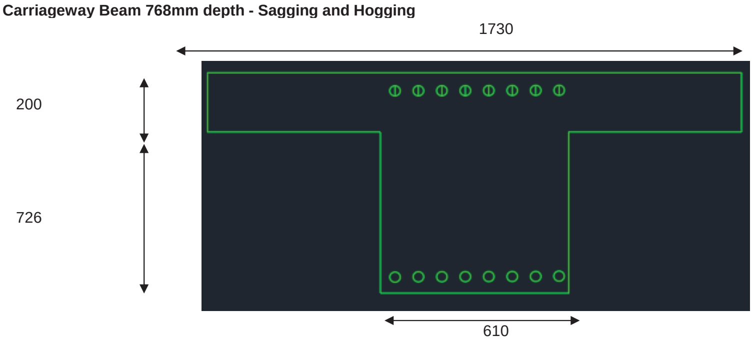
Hogging Bending Capacity Mu

$\frac{230}{1.15}$ x 7663.95 x 1077.18 = 1651.09 kNm

$\frac{0.225 \times 15.00}{1.5}$ x 610 x 1659588 = 2277.78 kNm

Mu = 1651.09 kNm

Section 6 - Carriageway Beam 768mm depth - Sagging and Hogging



Bar Type	Bar Dia (")	Bar Dia (mm)	Area of Bar (mm2)	Numer of Bars	Area of Reinforcement (mm^2)
Longitudinal Bottom Bar	1.375	34.93	957.99	8	7663.95
Longitudinal Top Bar	1.375	34.93	957.99	8	7663.95
Stirrups	0.375	9.53	71.26	1	71.26

Using Equation 5.2.4, Moment of resistance of flanged beams - Sagging & Hogging

M_u = Minimum value dervied from the following equations

$$M_u = \min \left\{ \begin{array}{l} \frac{f_y}{\gamma_{ms}} A_s z \\ \frac{0.225 f_{cu}}{\gamma_{mc}} b d^2 \end{array} \right.$$

Cover to top hogging bars = 1 1/4" = 31.75 mm

d = 894.25 mm

Z = 683.18 mm

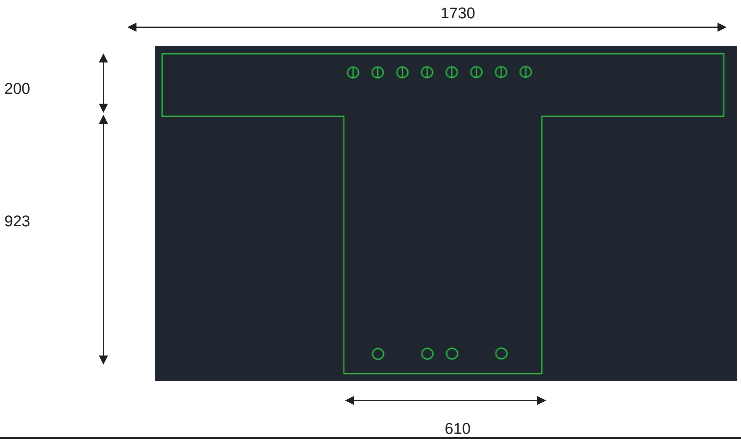
Hogging Bending Capacity Mu

$\frac{230}{1.15}$ x 7663.95 x 683.18 = 1047.17 kNm

$\frac{0.225 \times 15.00}{1.5}$ x 610 x 799683.1 = 1097.57 kNm

Mu = 1047.17 kNm

Section 7 - Carriageway Beam 1025mm depth - Sagging and Hogging



Bar Type	Bar Dia (")	Bar Dia (mm)	Area of Bar (mm2)	Numer of Bars	Area of Reinforcement (mm^2)
Longitudinal Bottom Bar	1.375	34.93	957.99	4	3831.98
Longitudinal Top Bar	1.375	34.93	957.99	8	7663.95
Stirrups	0.375	9.53	71.26	1	71.26

Using Equation 5.2.4, Moment of resistance of flanged beams - Sagging & Hogging

M_u = Minimum value dervied from the following equations $M_u = \min\left\{\begin{array}{l}\frac{f_y}{\gamma_{ms}} A_s z \\ \frac{0.225 f_{cu}}{\gamma_{mc}} b d^2\end{array}\right.$

Cover to top hogging bars = 1 1/4" = 31.75 mm

d = 1091.25 mm

Z = 880.18 mm

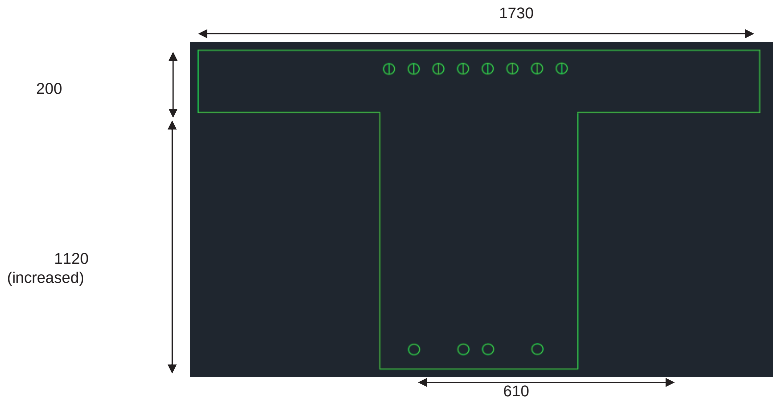
Hogging Bending Capacity Mu

$\frac{230}{1.15}$ x 7663.95 x 880.18 = 1349.127 kNm

$\frac{0.225 \times 15.00}{1.5}$ x 610 x 1190827 = 1634.41 kNm

Mu = 1349.13 kNm

Section 8 - Carriageway Beam 1222mm depth - Sagging and Hogging



Using Equation 5.2.4, Moment of resistance of flanged beams - Sagging & Hogging

M_u = Minimum value dervied from the following equations $\min\left\{\begin{array}{l}\frac{f_y}{\gamma_{ms}}A_s\left(d-\frac{h_f}{2}\right)\\ \frac{0.6f_{cu}}{\gamma_{mc}}bh_f\left(d-\frac{h_f}{2}\right)\end{array}\right.$

Sagging

Cover to reinforcement = 1 1/2 " = 38.1 mm

Effective Depth, d = 1320 - 38.1 - 34.93 = 1246.98 mm

Depth of Flange, hf = 200 mm

F_{cu} = 15 N/mm^2

$\frac{230}{1.15}$ x 7663.95 x 1246.98 - $\frac{200}{2}$ = 1758.07 kNm

$\frac{0.6 \quad 15}{1.5}$ x 1730 x 200 x 1246.98 - $\frac{200}{2}$ = 2381.1 kNm

Mu = 1758.07 kNm

Hogging

Cover to top bars = 1 1/4" = 31.75 mm

Effective Depth,d = 1320 - 31.75 - 34.93 = 1253.33 mm

Depth of Flange, hf = 200 mm

$\frac{230}{1.15}$ x 7663.95 x 1253.33 - $\frac{200}{2}$ = 1767.81 kNm

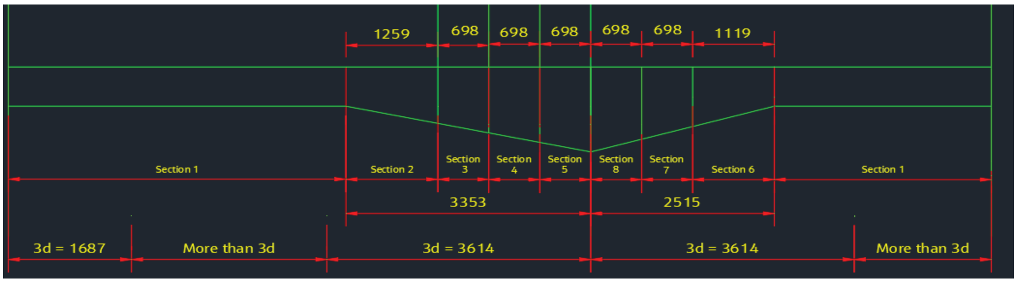
$\frac{0.6 \quad 15}{1.5}$ x 1730 x 200 x 1253.33 - $\frac{200}{2}$ = 2394.3 kNm

Mu = 1767.81 kNm

Bending Section Capacity Summary Table

Section	Sagging Capacity (kNm)	Hogging Capacity (kNm)
Section 1	708.72	N/A
Section 2	N/A	1002.72
Section 3	N/A	1198.91
Section 4	N/A	1424.23
Section 5	N/A	1651.09
Section 6	N/A	1047.17
Section 7	N/A	1349.13
Section 8	N/A	1651.09

Shear Capacity for Carriageway Beam



Shear zones

South Carriageway Beam Shear

The carriageway beam will be checked in 5 locations. As seen above, the locations have been defined by 3d from support and more than 3d from support with the d being defined as the effective depth of the beam. Due to the structure being symmetrical, only one taper will be checked.

Equation 5.7.1 Minimum effective shear reinforcement

Equation 5.7.1 Minimum effective shear reinforcement

$$\frac{A_{sv}}{s_v b_w}(\sin\alpha + \cos\alpha)\left(\frac{f_{yv}}{\gamma_{ms}}\right) \geq 0.2\text{MPa}$$

where

- A_{sv} is the cross-sectional area of shear reinforcement at a particular cross-section
- α is the angle of the shear reinforcement from the longitudinal axis of the beam
- s_v is the spacing of the shear reinforcement along the member
- f_{yv} is the characteristic, or worst credible, strength of the shear reinforcement but not greater than 500 MPa
- b_w is the breadth of the cross-section

$$\frac{957.99}{127.00 \times 610} \times 6.12574E{-}17 + 1 \times \frac{230}{1.15} = 2.4732 > 0.2\text{mpa} =$$

Try without shear reinforcement first

Shear at within 3d from the intermediate support calculations

3d = 3 x 1288.25 = 3865 mm

Bar Type	Bar Dia (")	Bar Dia (mm)	Spacing (mm)	Area of Bar (mm2)	Numer of Bars (3d)	Area of Reinforcement (mm^2)
Main top longitudinal bars	1.375	34.93	127.00	957.99	8.00	7663.95

Shear reinforcement anywhere, Equation 5.6a, V_{MAX}

Equation 5.6a Maximum shear resistance based on concrete crushing

$$V_{\max} = 0.36\left(0.7 - \frac{f_{cu}}{250}\right)\left(\frac{f_{cu}}{\gamma_{mc}}\right)b_w d$$

$$V_{MAX} = 0.36 \times 0.7 - \frac{15.00}{250} \times \frac{15.00}{1.5} \times 610 \times 1288.25 = \text{kN}$$

Shear Resistance more than 3d from a support, Equation 5.6b

$$V_{uc} = \frac{0.24}{\gamma_{mv}}\xi_s \rho_s^{\frac{1}{3}} f_{cu}^{\frac{1}{3}} b_w d$$

where:

- γ_{mv} is the partial factor for shear defined in Section 2.
- ξ_s is the depth factor, taken as $\xi_s = \left(\frac{500}{d}\right)^{0.25}$ but not less than 0.7
- ρ_s is the ratio of longitudinal reinforcement $\rho_s = \frac{100A_s}{b_w d}$ but not less than 0.15, nor greater than 3.0
- A_s is the area of longitudinal tension reinforcement that continues at least a distance d beyond the section being considered, and, for shear at supports, continues at least to the support.
  Where top and bottom reinforcement are provided, the area in tension under the loading which produces the shear force is to be used.

γmv = 1.15

$\xi s = \left(\frac{500}{d}\right)^{0.25} = \left(\frac{500}{1288.25}\right)^{0.25} = 0.79$

$\rho s = \frac{100 \times As}{bw \times d} = \frac{100 \times 7663.95}{610 \times 1288.25} = 0.98$

$Vuc = \frac{0.24}{1.15} \times 0.79 \times 0.98^{1/3} \times 15.00^{1/3} \times 610 \times 1288.25$

Vuc = 316585.13 N/mm

Vuc = 316.59 kN

Shear resistance within 3d of intermeidate support

Equation 5.6c Shear resistance within 3d of a support

$$V_u = \max\begin{cases}\frac{3d}{a_v}\Gamma V_{uc}\\ \frac{0.24}{\gamma_{mv}}\xi_s(0.15f_{cu})^{\frac{1}{3}}b_w d\end{cases}$$

where:

a_v is the distance of the section measured from the edge of a rigid bearing, the centre-line of a flexible bearing or the face of a support, where $d \le a_v \le 3d$

Γ is the factor to account for short anchorage lengths, defined in Equation 5.6d

Equation 5.6d Factor to account for the effect of short anchorage lengths

$$\Gamma = \min\begin{cases}\sqrt{\frac{z}{3d}\frac{F_{ub}}{V_{uc}}}\\ 1.0\end{cases}$$

where:

F_{ub} is the total anchorage force that can be developed in the longitudinal tension reinforcing bars at the front face of the support according to Section 9, but not greater than $\frac{A_s f_y}{\gamma_{ms}}$

z is the flexural lever arm at ULS at a position $3d$ from the support calculated from Equation 5.2.2b

Equation 9.1a Anchorage resistance

$$F_{ub} = f_{ub}pL_a$$

where:

f_{ub} is the average anchorage bond strength over the effective anchorage length, given by Equation 9.1b;

p is the effective perimeter, taken as
$p = \pi\phi$ for a single bar, or
$p = (1.2 - 0.2N)\sum(\pi\phi)$ for a bundled group of N bars, valid up to $N = 4$.

ϕ is the nominal bar diameter

L_a is the effective anchorage length at the position where the resistance is being determined.

Equation 9.1b Average anchorage bond strength

$$f_{ub} = \frac{kk_{cov}\beta\sqrt{f_{cu}}}{\gamma_{mb}}$$

k = 1 γmb = 1.25

B = 0.39 ϕ = 34.93

fcu = 15.00 Kcov = 1.00 (taken as 1)

fub = 1.21

Fub = $F_{ub} = f_{ub}pL_a$

p = 877.76 mm^2

L_a = taken as d from support = 1288.25 mm

Fub = 1366.40 kN $\frac{7663.95 \times 230}{1.15}$ = 1532.79 kN (limiting value)

Fub = 1366.40 kN

Equation 5.6d Factor to account for the effect of short anchorage lengths

$$\Gamma = \min \begin{cases} \sqrt{\frac{z}{3d} \frac{F_{ub}}{V_{uc}}} \\ 1.0 \end{cases}$$

where:

F_{ub} is the total anchorage force that can be developed in the longitudinal tension reinforcing bars at the front face of the support according to Section 9, but not greater than $\frac{A_s f_y}{\gamma_{ms}}$

z is the flexural lever arm at ULS at a position $3d$ from the support calculated from Equation 5.2.2b

Z = 1077.18 mm

Γ = 0.613 < 1 (ok)

Vu = $\frac{3d}{a_v}\Gamma V_{uc}$ = 582.47 kN

$\frac{0.24}{\gamma_{mv}}\xi_s(0.15 f_{cu})^{\frac{1}{3}} b_w d$ = 169.62 kN

Vu = 582.47 kN

Shear at within 3d from the end support calculations

3d = 3 x 562.38 = 1687.125 mm

Bar Type	Bar Dia (")	Bar Dia (mm)	Area of Bar (mm2)	Numer of Bars	Area of Reinforcement (mm^2)
Longitudinal Bottom Bars	1.38	34.93	957.99	8.00	7663.95

Equation 5.6c Shear resistance within 3d of a support

$$V_u = \max \begin{cases} \frac{3d}{a_v}\Gamma V_{uc} \\ \frac{0.24}{\gamma_{mv}}\xi_s(0.15 f_{cu})^{\frac{1}{3}} b_w d \end{cases}$$

where:

a_v is the distance of the section measured from the edge of a rigid bearing, the centre-line of a flexible bearing or the face of a support, where $d \leq a_v \leq 3d$

Γ is the factor to account for short anchorage lengths, defined in Equation 5.6d

Equation 5.6d Factor to account for the effect of short anchorage lengths

$$\Gamma = \min \begin{cases} \sqrt{\frac{z}{3d} \frac{F_{ub}}{V_{uc}}} \\ 1.0 \end{cases}$$

where:

F_{ub} is the total anchorage force that can be developed in the longitudinal tension reinforcing bars at the front face of the support according to Section 9, but not greater than $\frac{A_s f_y}{\gamma_{ms}}$

z is the flexural lever arm at ULS at a position $3d$ from the support calculated from Equation 5.2.2b

Equation 9.1a Anchorage resistance

$$F_{ub} = f_{ub} p L_a$$

where:

f_{ub} is the average anchorage bond strength over the effective anchorage length, given by Equation 9.1b;

p is the effective perimeter, taken as
$p = \pi\phi$ for a single bar, or
$p = (1.2 - 0.2N)\sum(\pi\phi)$ for a bundled group of N bars, valid up to $N = 4$.

ϕ is the nominal bar diameter

L_a is the effective anchorage length at the position where the resistance is being determined.

Equation 9.1b Average anchorage bond strength

$$f_{ub} = \frac{k k_{cov} \beta \sqrt{f_{cu}}}{\gamma_{mb}}$$

k = 1 γmb = 1.25

B = 0.39 ϕ = 34.93

fcu = 15.00 Kcov = 1.00 (taken as 1)

fub = 1.21

Fub = $F_{ub} = f_{ub} p L_a$

p = 877.76 mm^2

L_a = taken as d from support = 562.38 mm

Fub = 596.49 kN $\frac{7663.95 \times 230}{1.15}$ = 1532.79 kN (limiting Value)

Fub = 596.49 kN

Equation 5.6d Factor to account for the effect of short anchorage lengths

$$\Gamma = \min \begin{cases} \sqrt{\frac{z}{3d}\frac{F_{ub}}{V_{uc}}} \\ 1.0 \end{cases}$$

where:

F_{ub} is the total anchorage force that can be developed in the longitudinal tension reinforcing bars at the front face of the support according to Section 9, but not greater than $\frac{A_s f_y}{\gamma_{ms}}$

z is the flexural lever arm at ULS at a position $3d$ from the support calculated from Equation 5.2.2b

Z = 351.30 mm

Γ = 0.286

Vu = $\frac{3d}{a_v}\Gamma V_{uc}$ = 271.46 kN

$\frac{0.24}{\gamma_{mv}}\xi_s(0.15f_{cu})^{\frac{1}{3}}b_w d$ = 74.05 kN

Vu = 271.46 kN

Shear calculations based upon the shear reinforcement

Equation 5.7.1 Minimum effective shear reinforcement

Equation 5.7.1 Minimum effective shear reinforcement

$$\frac{A_{sv}}{s_v b_w}(\sin\alpha + \cos\alpha)\left(\frac{f_{yv}}{\gamma_{ms}}\right) \geq 0.2\text{MPa}$$

where

A_{sv} is the cross-sectional area of shear reinforcement at a particular cross-section

α is the angle of the shear reinforcement from the longitudinal axis of the beam

s_v is the spacing of the shear reinforcement along the member

f_{yv} is the characteristic, or worst credible, strength of the shear reinforcement but not greater than 500 MPa

b_w is the breadth of the cross-section

$\frac{285.02}{127.00 \times 610}$ x 6.12574E-17 + 1 x $\frac{230}{1.15}$ = 0.74 > 0.2mpa = PASS

Shear at within 3d from the intermediate support calculations

3d = 3 x 1288.25 = 3865 mm

Bar Type	Bar Dia (")	Bar Dia (mm)	Spacing (mm)	Area of Bar (mm2)	Numer of Bars (3d)	Area of Reinforcement (mm^2)
Stirrups	3/8	9.53	127.00	71.26	26.40	1881.26

Equation 5.9a Shear resistance more than 3d from a support

$$V_u = V_{uc} + V_{us}$$

where:

V_{uc} is calculated using Equation 5.6b

V_{us} is the component of shear resistance provided by effective shear reinforcement, as defined in Equation 5.9b

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Equation 5.6b Shear resistance more than 3d from a support

$$V_{uc} = \frac{0.24}{\gamma_{mv}} \xi_s \rho_s^{\frac{1}{3}} f_{cu}^{\frac{1}{3}} b_w d$$

where:

- γ_{mv} is the partial factor for shear defined in Section 2.
- ξ_s is the depth factor, taken as $\xi_s = \left(\frac{500}{d}\right)^{0.25}$ but not less than 0.7
- ρ_s is the ratio of longitudinal reinforcement $\rho_s = \frac{100 A_s}{b_w d}$ but not less than 0.15, nor greater than 3.0
- A_s is the area of longitudinal tension reinforcement that continues at least a distance d beyond the section being considered, and, for shear at supports, continues at least to the support. Where top and bottom reinforcement are provided, the area in tension under the loading which produces the shear force is to be used.

γmv = 1.15

ξs = (500 / d) 0.25 = (500 / 1288.25) 0.25 = 0.79

ρs = (100 x As) / (bw x d) = (100 / 610) x (7663.95 / 1288.25) = 0.98

V_{UC} = 245.75 kN

Equation 5.9b Component of shear resistance provided by effective shear reinforcement

$$V_{us} = d \frac{A_{sv}}{s_v} (\sin\alpha + \cos\alpha) \frac{f_{yv}}{\gamma_{ms}}$$

Vus = 1288.25 x (285.02 / 127.00) x 1 + 6.12574E-17 x (230 / 1.15) = 578.24

Vu = 245.75 + 578.24 = 823.99 kN

Shear Resistance withing 3d of support

Equation 5.9c Shear resistance within 3d of a support

$$V_u = \Gamma\left(\frac{3d}{a_v} V_{uc} + V_{us}\right)$$

where:

- Γ is the reduction factor to account for short anchorage lengths, as defined in Equation 5.6 d
- a_v is the distance of the section measured from the edge of a rigid bearing, the centre-line of a flexible bearing or the face of a support, where $d \leq a_v \leq 3d$

Equation 9.1a Anchorage resistance

$$F_{ub} = f_{ub} p L_a$$

where:

- f_{ub} is the average anchorage bond strength over the effective anchorage length, given by Equation 9.1b;
- p is the effective perimeter, taken as $p = \pi\phi$ for a single bar, or $p = (1.2 - 0.2N)\sum(\pi\phi)$ for a bundled group of N bars, valid up to $N = 4$.
- ϕ is the nominal bar diameter
- L_a is the effective anchorage length at the position where the resistance is being determined.

Equation 9.1b Average anchorage bond strength

$$f_{ub} = \frac{k k_{cov} \beta \sqrt{f_{cu}}}{\gamma_{mb}}$$

k	=	1	γmb	=	1.25
B	=	0.39	ϕ	=	34.00
fcu	=	15.00	Kcov	=	0.65

fub = 0.79

Fub = $F_{ub} = f_{ub} p L_a$

p = 854.5132 mm^2

L_a = taken as d from support = 1140.25 mm^2

Fub = 765.30 kN

(7663.95 x 230) / 1.15 = 1532.79 kN (limiting value)

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Equation 5.6d Factor to account for the effect of short anchorage lengths

$$\Gamma = \min \begin{cases} \sqrt{\frac{z}{3d}\frac{F_{ub}}{V_{uc}}} \\ 1.0 \end{cases}$$

where:

F_{ub} is the total anchorage force that can be developed in the longitudinal tension reinforcing bars at the front face of the support according to Section 9, but not greater than $\frac{A_s f_y}{\gamma_{ms}}$

z is the flexural lever arm at ULS at a position $3d$ from the support calculated from Equation 5.2.2b

Z = 929.18 mm Z of section 4

Γ = $\frac{929.177}{3420.75}$ = 0 $\frac{765.30}{245.75}$ = 3.114168312 Γ = 0.92

Equation 5.9c Shear resistance within 3d of a support

$$V_u = \Gamma\left(\frac{3d}{a_v}V_{uc} + V_{us}\right)$$

Vu = 0.920 x $\frac{3421}{1288.25}$ x 245.75 + 578.24 = 1131.99 kN

Shear from the end support calculations

Shear at within 3d from the intermediate support calculations

3d = 3 x 562.38 = 1687 mm

Bar Type	Bar Dia (")	Bar Dia (mm)	Spacing (mm)	Area of Bar (mm2)	Numer of Bars (3d)	Area of Reinforcement (mm^2)
Stirrups	3	9.53	76.20	71.26	22.14	1577.66

Equation 5.9a Shear resistance more than 3d from a support

$$V_u = V_{uc} + V_{us}$$

where:

V_{uc} is calculated using Equation 5.6b

V_{us} is the component of shear resistance provided by effective shear reinforcement, as defined in Equation 5.9b

Equation 5.6b Shear resistance more than 3d from a support

$$V_{uc} = \frac{0.24}{\gamma_{mv}}\xi_s \rho_s^{\frac{1}{3}} f_{cu}^{\frac{1}{3}} b_w d$$

where:

γ_{mv} is the partial factor for shear defined in Section 2.

ξ_s is the depth factor, taken as $\xi_s = \left(\frac{500}{d}\right)^{0.25}$ but not less than 0.7

ρ_s is the ratio of longitudinal reinforcement $\rho_s = \frac{100A_s}{b_w d}$ but not less than 0.15, nor greater than 3.0

A_s is the area of longitudinal tension reinforcement that continues at least a distance d beyond the section being considered, and, for shear at supports, continues at least to the support.
Where top and bottom reinforcement are provided, the area in tension under the loading which produces the shear force is to be used.

γmv = 1.15

ξs = $\frac{500}{d}$ 0.25 = $\frac{500}{562.38}$ 0.25 = 0.97

ρs = $\frac{100 \text{ x } As}{bw \text{ x } d}$ = $\frac{100}{610}$ $\frac{\text{x}}{\text{x}}$ $\frac{7663.95}{562.38}$ = 2.23

Vuc = 224.13 kN

Equation 5.9b Component of shear resistance provided by effective shear reinforcement

$$V_{us} = d\frac{A_{sv}}{s_v}(\sin\alpha + \cos\alpha)\frac{f_{yv}}{\gamma_{ms}}$$

Vus = 562.38 x $\frac{285.02}{76.20}$ x 1 + 6.12574E-17 x $\frac{230}{1.15}$ = 420.71 kN

Vu = 224.13 + 420.71 = 644.84 kN

Shear Resistance withing 3d of support

Equation 5.9c Shear resistance within 3d of a support

$$V_u = \Gamma\left(\frac{3d}{a_v}V_{uc} + V_{us}\right)$$

where:

Γ is the reduction factor to account for short anchorage lengths, as defined in Equation 5.6
d
a_v is the distance of the section measured from the edge of a rigid bearing, the centre-line of a flexible bearing or the face of a support, where $d \le a_v \le 3d$

Equation 9.1a Anchorage resistance

$$F_{ub} = f_{ub} p L_a$$

where:

f_{ub} is the average anchorage bond strength over the effective anchorage length, given by Equation 9.1b;
p is the effective perimeter, taken as
$p = \pi\phi$ for a single bar, or
$p = (1.2 - 0.2N)\sum(\pi\phi)$ for a bundled group of N bars, valid up to $N = 4$.
ϕ is the nominal bar diameter
L_a is the effective anchorage length at the position where the resistance is being determined.

Equation 9.1b Average anchorage bond strength

$$f_{ub} = \frac{k k_{cov} \beta \sqrt{f_{cu}}}{\gamma_{mb}}$$

k = 1 γmb = 1.25

B = 0.39 ϕ = 34.93

fcu = 15.00 Kcov = 0.65

fub = 0.79

Fub = $F_{ub} = f_{ub} p L_a$

p = 877.76 mm^2

L_a = taken as d from support = 562.38 mm m

Fub = 388.13 kN $\frac{7663.95 \quad x \quad 230}{1.15}$ = 1532.79 kN (limiting Value)

Equation 5.6d Factor to account for the effect of short anchorage lengths

$$\Gamma = \min\left\{\begin{array}{l}\sqrt{\frac{z}{3d}\frac{F_{ub}}{V_{uc}}} \\ 1.0\end{array}\right.$$

where:

F_{ub} is the total anchorage force that can be developed in the longitudinal tension reinforcing bars at the front face of the support according to Section 9, but not greater than $\frac{A_s f_y}{\gamma_{ms}}$

z is the flexural lever arm at ULS at a position $3d$ from the support calculated from Equation 5.2.2b

Z = 468.56 mm 504

Γ = 0.671 < 1 (OK)

Equation 5.9c Shear resistance within 3d of a support

$$V_u = \Gamma\left(\frac{3d}{a_v}V_{uc} + V_{us}\right)$$

Vu = 0.671 x $\frac{1687}{562.38}$ x 224.13 + 420.71 = 733.47 kN

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Material & Section Properties

Reinforced Concrete

Reference	Description	Symbol	=	Value	Unit
CS 455, Cl.3.1.3	Characteristic Strength of Concrete		=	15.00	N/mm^2
	Density of Reinforced Concrete		=	24	kN/m^3
CS 454 table 4.1.1a					

Steel Reinforcement

Reference	Description	=	Value	Unit
CS455, Cl, 3.8.2	Mild steel reinforcement characteristic yield strength	=	230	N/mm^2
	Design Young's Modulus of steel reinforcement	=	210000	N/mm^2

Mass Concrete (Plain Concrete)

Reference	Description	=	Value	Unit
CS 454 table 4.1.1a	Unit weight of plain concrete	=	23	kN/m^3

Fill - Miscellaneous

Reference	Description	=	Value	Unit
CS 454 table 4.1.1a	Unit weight of miscellaneous fill	=	22	kN/m^3

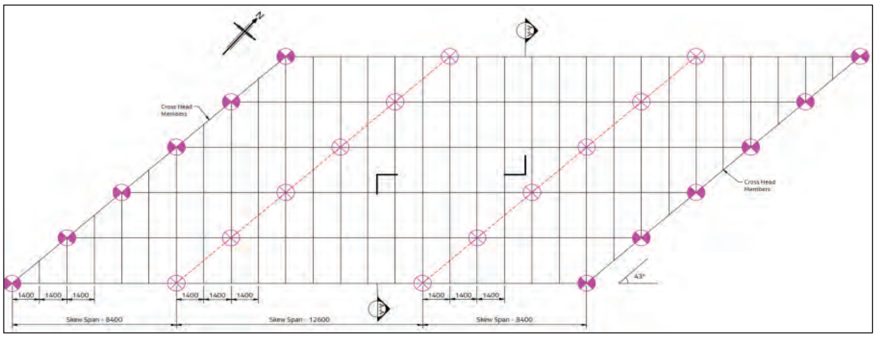
Carriageway & Footway Surfacing

Reference	Description	=	Value	Unit
CS 454 table 4.1.1a	Unit weight of Bituminous Macadam (tar)	=	24	kN/m^3

Partial Factors

Reference	Description	Symbol	=	Value
CS 455 Table 2.13a	Partial Factor for reinforcement	γ_{ms}	=	1.15
CS 455 Table 2.13a	Partial Factor for Concrete	γ_{mc}	=	1.5
CS 455 Table 2.13a	Partial Factor for shear in concrete	γ_{mv}	=	1.15
CS 455 Table 2.13a	Partial Factor for Bond	γ_{mb}	=	1.25
CS 454 Table A.1	Partial Factor for Concrete Deadload		=	1.15
CS 454 Table A.1	Partial Factor for Surfacing		=	1.75
CS 454 Table A.1	Partial Factor for Fill		=	1.2
CS 454 3.9	Inaccurate assessment effects at ULS	$\gamma f3$	=	1.1

Grillage Model Span Dimmensions



MIDAS Grillage Model

Spans

Main Span 2 Skew Span	=	8.4	m
Main Span 3 Skew Span	=	12.6	m
Main Span 3 Skew Span	=	8.4	m

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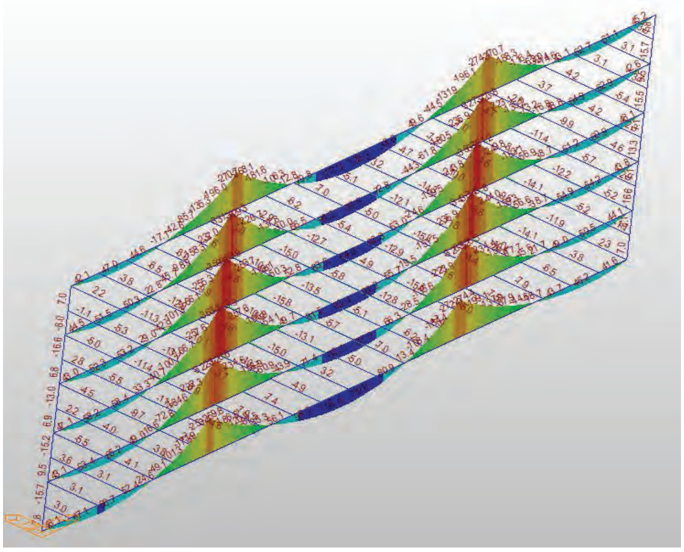
Deadload Calculations

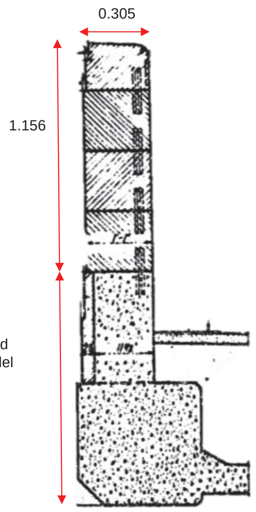
Deadload has been applied using the MIDAS Self Weight Function. Results from the Model shows that bending moment diagrams are working as expected.



Parapet Deadload

The bottom reinforced concrete section below the concrete parapet blocks have been taken into account in the parapet beam.

B2100/8

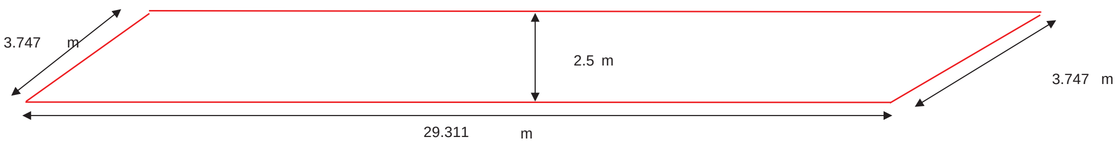


Cross Sectional Area	=	0.35	m^2	
Unit weight per m	=	8.46	kN/m	
Unit weight per m	=	9.73	kN/m	(Factored)

Section through parapet

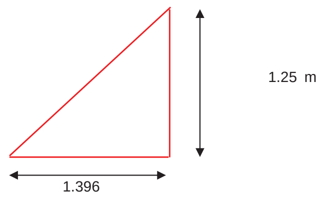
Superimposed Deadload

North Footway



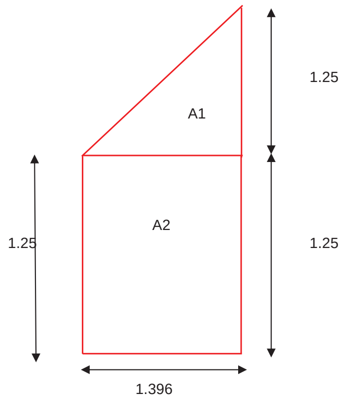
Typical Plan on Footway

Traingular distribution on span ends



Cross Section Area of fill	=	0.744	x	1.25	=	0.93	m^2
Unit weight	=	0.93	x	22	=	20.46	kN/m
Unit weight per m for triangle	=	$\frac{20.46}{2}$			=	10.23	kN/m

Deadloading at acute corners



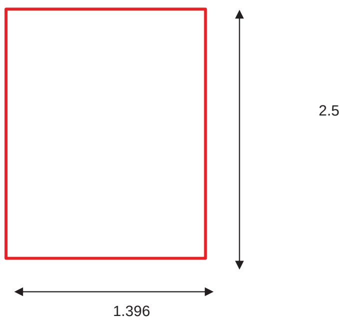
Area 1

Cross section area of fill	=	0.744	x	1.25	=	0.93	m^2
Unit Weight	=	0.93	x	22	=	20.46	kN/m
Unit weight per m for triangle	=	$\frac{20.46}{2}$			=	10.23	kN/m

Area 2

Cross section area of fill	=	0.744	x	1.25	=	0.93	m^2
Unit Weight	=	0.93	x	22	=	20.46	kN/m

Normal Sqaure Section



Cross sectional area of fill	=	0.744	x	1.396	=	1.0386	m^2
Unit Weight	=	1.038624	x	22	=	22.85	kN/m

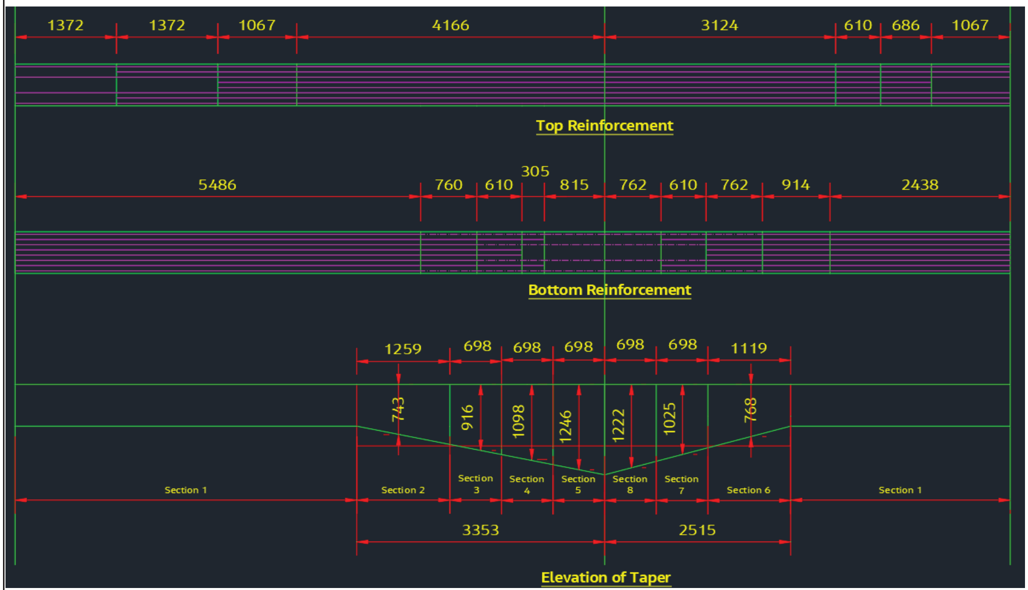
Carriageway Super Imposed Deadload

Using the same principles above, the superimposed deadload will be applied to the transverse memebers.

		Area		**Density**					
Triangular Distribution	=	0.125	x	24	=	$\frac{3}{2}$	=	2	kNm
		Area 1		**Density**					
Acute Corners	=	0.125	x	24	=	$\frac{3}{2}$	=	2	kNm
		Area 2		**Density**					
		0.125	x	24	=	3			kNm
		Area		**Density**					
Nomral Square Section	=	0.1396	x	24	=	3.3504			kNm

Bending Capacity for Carriageway Beam

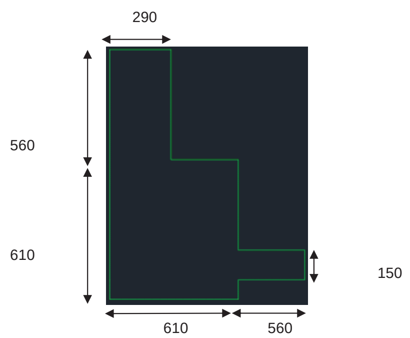
To calculate the bending capacity of the carriageway beam, the point of contraflexure must be defined. However, as this will changed from load case to load case. To overcome this, the beam will be checked at the points below where the reinforcement changes. Both hogging and sagging will be checked.



Reinforcement sketch of cway beam

Reinforced Concrete Beams - Moment Resistance of Beams

Section 1 - Carriageway Beam 610 depth - Sagging - Midspan



Internal Reinforcement for sagging - 610mm depth

Bar Type	Bar Dia (")	Bar Dia (mm)	Area of Bar (mm2)	Numer of Bars	Area of Reinforcement (mm^2)
Longitudinal Bottom Bar	1.375	34.93	957.99	7	6705.96
Longitudinal Top Bar	1.375	34.93	957.99	4	3831.98
Stirrups	0.375	9.525	71.26	N/A	N/A

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CS455 Eq 5.2.2a

Using Equation 5.2.2a, Moment of resistance of beams without compression reinforcement - Sagging

M_u = Minimum value dervied from the following equations $M_u = \min\left\{ \frac{f_y}{\gamma_{ms}} A_s z,\ \frac{0.225 f_{cu}}{\gamma_{mc}} b d^2 \right.$

Cover to reinforcement = 1 1/2 " = 38 mm

Effective Depth, d = 1170 - 38 - 9.525 = 1122.38 mm

Z = 937.69 mm

F_{cu} = 15 N/mm^2

Mu = $\frac{230}{1.15}$ x 6705.96 x 397.67 = 533.35 kNm

Mu = $\frac{0.225 \times 15}{1.15}$ x 610 x 1122.38^2 = 2255.18

Mu = 533.35 kNm

Using compression reinforcement, Equation 5.2.2c Moment Resistance of beam with compression reinforcement

d' = 562.38 mm

x = 270.00 mm (rearranged from eq 5.2.2d from CS455)

f's = $\frac{230}{1.15 + \frac{230}{2000}}$ = 181.82 N/mm

Mu = $\frac{0.6 \times 15}{1.5}$ x 610 x 270.00 x 1122.38 - 386.16 = 727.52 kNm

181.82 x 3831.98 x 1122.38 - 772.33 = 243.88 kNm

Mu = 971.41 kNm

The remainder of the sections have the same capacity as the rest of the beams.

Utilising the full section height to improve the bending resistance

Area 1 (top hat) = 560 x 290 = 162400 mm^2

Area 2 (Beam) = 610 x 610 = 372100 mm^2

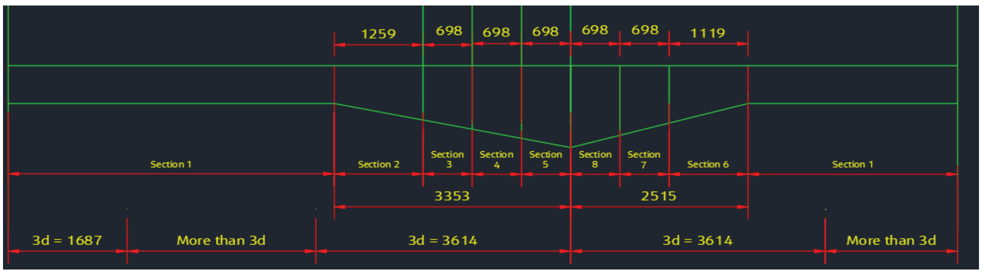
Area Total = 534500 mm^2

Σay = 372100 x $\frac{610}{2}$ + 162400 x 610 + $\frac{560}{2}$ = 2E+08 mm2

ybtm = $\frac{212554780}{534500}$ = 397.67 mm

y top = 610 + 560 - 397.67 = 772.33 mm

<u>Shear Capacity for parapet beam with reduced section</u>



<u>Shear zones</u>

Parapet Beam with 2mm section loss - shear calculations

Equation 5.7.1 Minimum effective shear reinforcement

Equation 5.7.1 Minimum effective shear reinforcement

$$\frac{A_{sv}}{s_v b_w}(\sin\alpha + \cos\alpha)\left(\frac{f_{yv}}{\gamma_{ms}}\right) \geq 0.2\text{MPa}$$

where

A_{sv} is the cross-sectional area of shear reinforcement at a particular cross-section

α is the angle of the shear reinforcement from the longitudinal axis of the beam

s_v is the spacing of the shear reinforcement along the member

f_{yv} is the characteristic, or worst credible, strength of the shear reinforcement but not greater than 500 MPa

b_w is the breadth of the cross-section

$$\frac{851.42}{127.00 \times 610} \times 6.12574E-17 + 1 \times \frac{230}{1.15} = 2.198052 > 0.2\text{mpa}$$ = Try without shear reinforcement first

Shear at within 3d from the end support calculations

3d = 3 x 562.38 = 1687 mm

Bar Type	Bar Dia (")	Bar Dia (mm) 2mm section loss	Spacing (mm)	Area of Bar (mm2)	Numer of Bars (3d)	Area of Reinforcement (mm^2)
Main top longitudinal bars	1.375	32.93	127.00	851.42	8.00	6811.32

Shear reinforcement anywhere, Equation 5.6a, V_{MAX}

Equation 5.6a Maximum shear resistance based on concrete crushing

$$V_{\max} = 0.36\left(0.7 - \frac{f_{cu}}{250}\right)\left(\frac{f_{cu}}{\gamma_{mc}}\right)b_w d$$

$$V_{MAX} = 0.36 \times 0.7 - \frac{15.00}{250} \times \frac{15.00}{1.5} \times 610 \times 562.38 = 790.38 \text{ kN}$$

Shear Resistance more than 3d from a support, Equation 5.6b

$$V_{uc} = \frac{0.24}{\gamma_{mv}}\xi_s \rho_s^{\frac{1}{3}} f_{cu}^{\frac{1}{3}} b_w d$$

where:

γ_{mv} is the partial factor for shear defined in Section 2.

ξ_s is the depth factor, taken as $\xi_s = \left(\frac{500}{d}\right)^{0.25}$ but not less than 0.7

ρ_s is the ratio of longitudinal reinforcement $\rho_s = \frac{100A_s}{b_w d}$ but not less than 0.15, nor greater than 3.0

A_s is the area of longitudinal tension reinforcement that continues at least a distance d beyond the section being considered, and, for shear at supports, continues at least to the support.
Where top and bottom reinforcement are provided, the area in tension under the loading which produces the shear force is to be used.

γmv = 1.15

$$\xi s = \frac{500}{d}^{0.25} = \frac{500}{562.38}^{0.25} = 0.97$$

$$\rho s = \frac{100 \times As}{bw \times d} = \frac{100 \times 6811.32}{610 \times 562.38} = 1.99$$

$$Vuc = \frac{0.24}{1.15} \times 0.97 \times 1.99^{1/3} \times 15.00^{1/3} \times 610 \times 562.38$$

Vuc = 215490.2 N/mm

Vuc = 215.49 kN

Shear reinforcement

Bar Type	Bar Dia (")	Bar Dia (mm) 2mm section loss	Spacing (mm)	Area of Bar (mm2)	Numer of Bars (3d)	Area of Reinforcement (mm^2)
Stirrups	3	7.53	76.20	44.47	22.14	984.68

Equation 5.9b Component of shear resistance provided by effective shear reinforcement

$$V_{us} = d\frac{A_{sv}}{s_v}(\sin\alpha + \cos\alpha)\frac{f_{yv}}{\gamma_{ms}}$$

Vus = 562.38 x $\frac{228.6}{76.20}$ x 1 + 6.12574E-17 x $\frac{230}{1.15}$ = 337.425 kN

Vu = 215.49 + 337.425 = 552.92 kN

Shear Resistance within 3d of support

Equation 5.9c Shear resistance within 3d of a support

$$V_u = \Gamma\left(\frac{3d}{a_v}V_{uc} + V_{us}\right)$$

where:

Γ is the reduction factor to account for short anchorage lengths, as defined in Equation 5.6 d

a_v is the distance of the section measured from the edge of a rigid bearing, the centre-line of a flexible bearing or the face of a support, where $d \le a_v \le 3d$

Equation 9.1a Anchorage resistance

$$F_{ub} = f_{ub}pL_a$$

where:

f_{ub} is the average anchorage bond strength over the effective anchorage length, given by Equation 9.1b;

p is the effective perimeter, taken as
$p = \pi\phi$ for a single bar, or
$p = (1.2 - 0.2N)\sum(\pi\phi)$ for a bundled group of N bars, valid up to $N = 4$.

ϕ is the nominal bar diameter

L_a is the effective anchorage length at the position where the resistance is being determined.

Equation 9.1b Average anchorage bond strength

$$f_{ub} = \frac{kk_{cov}\beta\sqrt{f_{cu}}}{\gamma_{mb}}$$

k = 1 γmb = 1.25

B = 0.39 ϕ = 32.93

fcu = 15 Kcov = 0.67

fub = 0.81

Fub = $F_{ub} = f_{ub}pL_a$

P = 827.50 mm^2

La = taken as d from support = 562.38 mm

Fub = 377.03 kN

Equation 5.6d Factor to account for the effect of short anchorage lengths

$$\Gamma = \min\begin{cases}\sqrt{\frac{z}{3d}\frac{F_{ub}}{V_{uc}}} \\ 1.0\end{cases}$$

where:

F_{ub} is the total anchorage force that can be developed in the longitudinal tension reinforcing bars at the front face of the support according to Section 9, but not greater than $\frac{A_s f_y}{\gamma_{ms}}$

z is the flexural lever arm at ULS at a position $3d$ from the support calculated from Equation 5.2.2b

Z = 468.56 mm

Γ = 0.70 < 1 OK

Equation 5.9c Shear resistance within 3d of a support

$$V_u = \Gamma\left(\frac{3d}{a_v}V_{uc} + V_{us}\right)$$

Vu = 0.70 x $\frac{1687}{562.38}$ x 215.49 + 337.425 = 685.86 kN

Pages 78 to 91 not used

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Material & Section Properties

Reinforced Concrete

Reference	Description	Symbol		Value	Unit
CS 455, Cl.3.1.3	Characteristic Strength of Concrete		=	15.00	N/mm^2
	Density of Reinforced Concrete		=	24	kN/m^3
CS 454 table 4.1.1a					

Steel Reinforcement

Reference	Description		Value	Unit
CS455, Cl, 3.8.2	Mild steel reinforcement characteristic yield strength	=	230	N/mm^2
	Design Young's Modulus of steel reinforcement	=	210000	N/mm^2

Mass Concrete (Plain Concrete)

Reference	Description		Value	Unit
CS 454 table 4.1.1a	Unit weight of plain concrete	=	23	kN/m^3

Fill - Miscellaneous

Reference	Description		Value	Unit
CS 454 table 4.1.1a	Unit weight of miscellaneous fill	=	22	kN/m^3

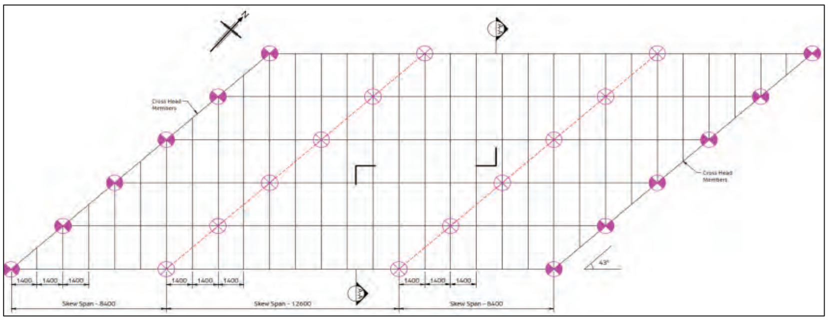
Carriageway & Footway Surfacing

Reference	Description		Value	Unit
CS 454 table 4.1.1a	Unit weight of Bituminous Macadam (tar)	=	24	kN/m^3

Partial Factors

Reference	Description	Symbol		Value
CS 455 Table 2.13a	Partial Factor for reinforcement	γ_{ms}	=	1.15
CS 455 Table 2.13a	Partial Factor for Concrete	γ_{mc}	=	1.5
CS 455 Table 2.13a	Partial Factor for shear in concrete	γ_{mv}	=	1.15
CS 455 Table 2.13a	Partial Factor for Bond	γ_{mb}	=	1.25
CS 454 Table A.1	Partial Factor for Concrete Deadload		=	1.15
CS 454 Table A.1	Partial Factor for Surfacing		=	1.75
CS 454 Table A.1	Partial Factor for Fill		=	1.2
CS 454 3.9	Inaccurate assessment effects at ULS	$\gamma f3$	=	1.1

Grillage Model Span Dimmensions



MIDAS Grillage Model

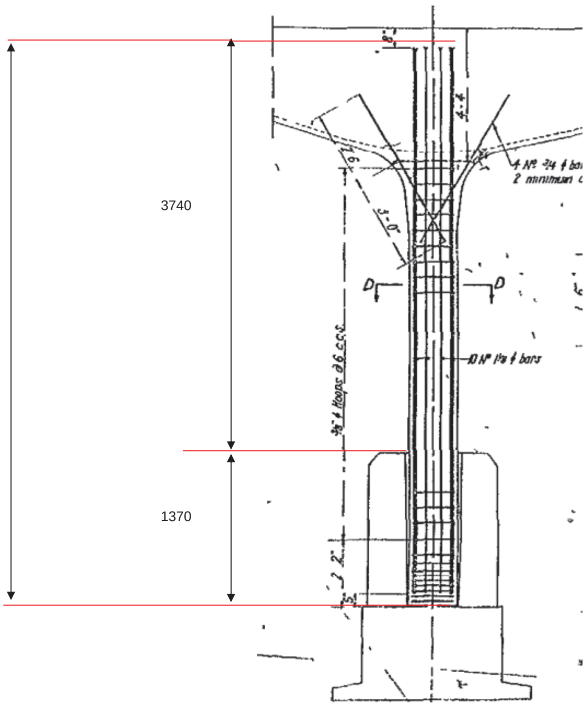
Spans

Main Span 2 Skew Span	=	8.4	m
Main Span 3 Skew Span	=	12.6	m
Main Span 3 Skew Span	=	8.4	m

Tranverse slab effective widths

Longitudinal Direction	=	1400	mm
Transverse Direction	=	2500	mm

Intermediate Column

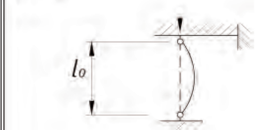


Dimensions above have been determined from the Survey drawing conducted on the 10/10/2023 and the As-Built drawing LC 5/3

Effective Height of the Column

The effective height of the column has been determined using CS 455 Table 7.2. The effective height is determining that the column is pinned at the top and bottom.

CS 455, Table 7.2

Case 3	Top	Full	None	
	Bottom	Full	None	$1.0l_o$

Lo = 5110 mm

Effective Height Le = 5110 mm

Slenderness criterion for short columns

$\frac{Le}{h} < 12 = \frac{5110}{457.2} = 11.18 < 12$ Slender

Forces applied onto the column

In accordance with CS 454 Equation 5.35.1 Longitudinal braking or traction Loads can be determined using the following equation

Equation 5.35.1 Longitudinal braking or traction load

$$Q_L = \min(8L + 250, 750)$$

where:

Q_L is the braking or traction load, in kN

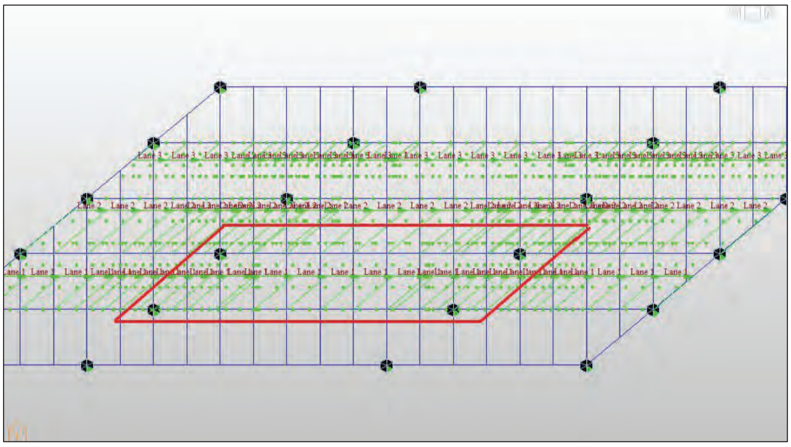
L is the loaded length, in m

L = Loaded Length = 29.4 m

Q_L = Minimim = ## < 750

Q_L = 485.2 kN

The braking load in accordance with CS 454 5.35.1 can only be applied to one lane. Therefore, it has been determined that the worse case loading will be applied to lane 1 and considering that 4 columns are working together to resist the braking force.



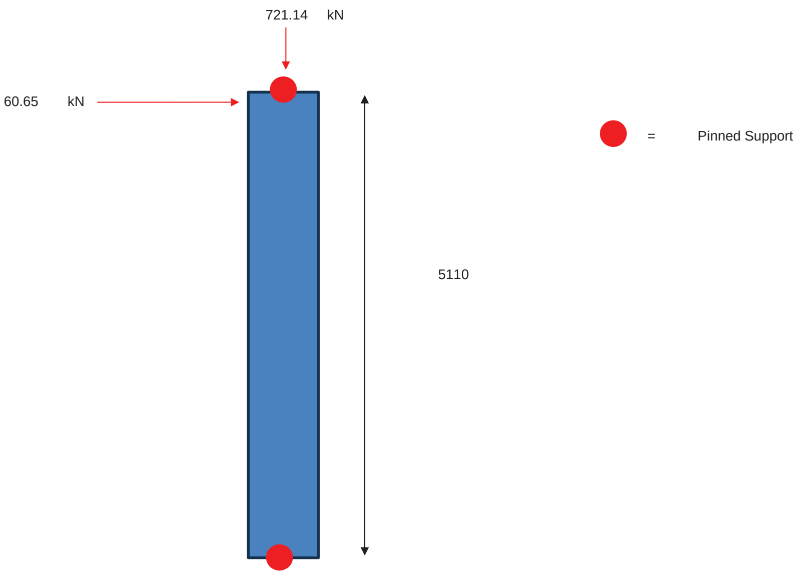
Midas Model showing worse case pier columns from braking

Braking forces distributed over 4 pier columns

Force = $\frac{Q_L}{\text{No. Columns}}$ = $\frac{485.2}{8}$ = 60.65 kN

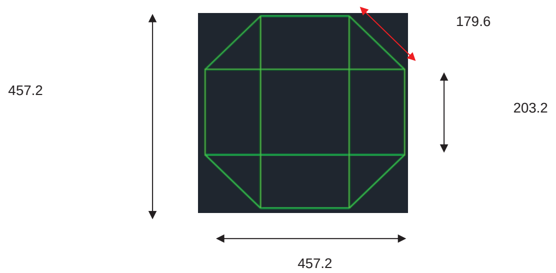
Worse case reaction due to live loading on deck = 721.14 kN (Vehicle D2 + SV80) Node 51

Idealised diagram of worse affected column (live loading)



Shear Resistance of the Intermediate Column

Section Properties of the Octogonal Column



Internal Reinforcement

Bar Type	Bar Dia (")	Bar Dia (mm)	Area of Bar (mm^2)	Spacing (mm)	No. Bars	ea of Reinforcement (mı
Hoop	3/8	9.525	71.26	152.4	0.00	0.00
Vertical Bar	1 1/8	28.575	641.30	N/A	10	6413.02

Cover to reinforcement = 1 1/2 = 38.1 mm

Effective Depth, d = 409.58 mm

Area of the Pier = 155746.52 mm^2

Equation 5.6a Maximum Shear Based on Concrete Crushing

Equation 5.6a Maximum shear resistance based on concrete crushing

$$V_{max} = 0.36\left(0.7 - \frac{f_{cu}}{250}\right)\left(\frac{f_{cu}}{\gamma_{mc}}\right)b_w d$$

b_W = is taken as the column diameter

$$V_{MAX} = 0.36 \times 0.7 - \frac{15.00}{250} \times \frac{15.00}{1.5} \times 457.2 \times 409.58 = 431.44 \text{ kN}$$

Equation 5.6b Shear resistance more than 3d from a support

$$V_{uc} = \frac{0.24}{\gamma_{mv}}\xi_s \rho_s^{\frac{1}{3}} f_{cu}^{\frac{1}{3}} b_w d$$

where:

γ_{mv} is the partial factor for shear defined in Section 2.

ξ_s is the depth factor, taken as $\xi_s = \left(\frac{500}{d}\right)^{0.25}$ but not less than 0.7

ρ_s is the ratio of longitudinal reinforcement $\rho_s = \frac{100A_s}{b_w d}$ but not less than 0.15, nor greater than 3.0

A_s is the area of longitudinal tension reinforcement that continues at least a distance d beyond the section being considered, and, for shear at supports, continues at least to the support.
Where top and bottom reinforcement are provided, the area in tension under the loading which produces the shear force is to be used.

$$\xi s = \left(\frac{500}{409.58}\right)^{0.25} = 1.05$$

$$Ps = \frac{100 \times 3206.51}{457.2 \times 409.58} = 1.71 = \text{limited to } 3.00$$

Shear resistance Vuc is enahanced by carrying out Equation 7.10.1

$$1 + \frac{0.15N}{A_c}$$

where:

N is the ultimate axial load in Newtons, but not greater than $0.11 f_{cu} A_c$

A_c is the area of the entire concrete section, in units of mm^2

Ultimate axial load = 721.14 x 1.65 = 1189.881 kN or = 1189881 N

= 0.11 x 15.00 x 155746.52 = 256981.8 N

Ultimate axial load = min value of = 1189881 or 256981.75 = 256981.8 N

Shear stress enhancement Factor = 1 + $\frac{0.15 \times 256981.7545}{155746.52}$ = 0.2475

Vuc = $\frac{0.24}{1.15}$ x 1.05 x $1.71^{1/3}$ x $15.00^{1/3}$ x 457.2 x 409.58 = 121.20 kN

Enchancement Factor = 121.20 x 0.2475 = 30.00 kN

Vuc = 151.20 kN

Bucking of the slender column

Using the equation 7.9.1 from CS 455, buckling will be exterted onto the column regardless of the support conditions being pinned.

$$M_{ty} = M_{iy} + \frac{N h_x}{K}\left(\frac{l_e}{h_x}\right)^2\left(1 - \frac{0.0035 l_e}{h_x}\right)$$

where:

- M_{iy} is the initial assessment moment but not less than $0.02N$
- h_x is the overall depth of the cross-section in the plane of bending
- K is a coefficient taken as 1750 for normal concrete or 1200 for lightweight aggregate concrete
- l_e is the effective height either in the plane of bending or in the plane at right-angles, whichever is greater

M_{iy} = 0.02 x 256981.7545 = 5139.64 Nmm

h_x = 457.2 mm

K = 1750

le = 5110 mm

M_{ty} = 5147.69 kNm

Ultimate axial of capacity

The axial force shall be calculated as per the same rules as short columns in accordance with CS 455 Cl 7.8

$$N_{uz} = 0.675 \frac{f_{cu}}{\gamma_{mc}} A_c + f_y A_{sc}$$

where:

- A_c is the area of concrete
- A_{sc} is the total area of longitudinal reinforcement

N_{uz} = 0.675 x $\frac{15.00}{1.5}$ x 155746.52 + 230 x 6413.02 = 2526.28 kN

Summary Table

Element	Section Capacity	DI + SI Effects	Live Load Effects	Total Load Effect	Uitilsation	Pass / Fail	Remarks
Intermediate Column	151.20	10.6	60.65	71.25	0.47	PASS	Shear
	5147.69	0.00	5139.64	5139.64	1.00	PASS	Bending
	2526.28	525.15	793.25	1318.404	0.52	PASS	Axial

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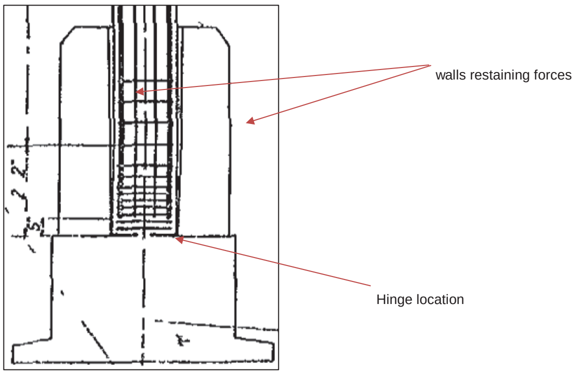
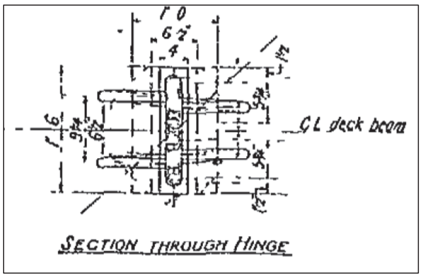
Hinge Capacity

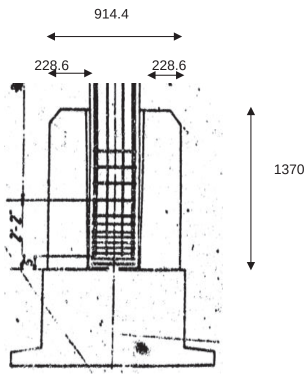
Due to the hinge at the base of the column being restricted transversely and longitudinally, it is assumed that the outer walls of the fender are restraining the base of the column.



Section on Fender



Section on Hinge



Internal Reinforcement in the fender walls

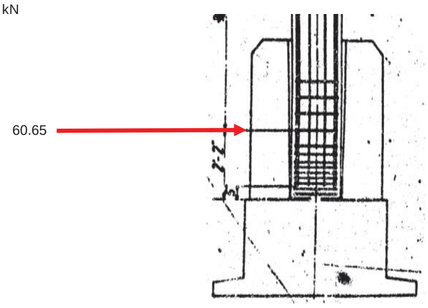
Bar Type	Bar Dia (")	Bar Dia (mm)	Area of Bar (mm^2)	Spacing (mm)	No. Bars	Area of Reinforcement (mm^2)
Longitduinal Reinforcement	5/8	15.88	197.93	457.2	3.00	593.80
Links	5/8	15.88	197.93	152.4	6.56	1298.77

Cover to reinforcement = 2 " = 50.8 mm

Effective depth = 914.4 - 50.8 - 15.88 = 847.73 mm

Loading on fender wall

Longitudinal Braking Force = 60.65 kN



Due to the articulation of the intermediate pier, there is no bending moment with the pier, therefore the base will not be assessed for bending

CS 455, 7.21

Shear resistance without shear reinforcement

The shear resistance of the base will be assessed under the more severe of the following conditions.

1) Shear along a vertical section extending across the full width of the base, at a distance equal to the effective depth from the face of the loaded area

2) Punching shear around the loaded area.

All the above will be calculated using the requirements of the shear resistance of slabs according to section 6 of CS 455

Shear along the vertical section

Equation 6.5 Shear resistance of concrete slabs more than 3d from a support

$$V_{uc} = \frac{0.27}{\gamma_{mv}} \xi_s \rho_s^{\frac{1}{3}} f_{cu}^{\frac{1}{3}} b_w d$$

$$\xi s = \left(\frac{500}{847.73}\right)^{0.25} = 0.88$$

$$Ps = \frac{100 \times 593.80}{914.4 \times 847.73} = 0.08 = \text{limited to} = 0.15$$

$$Vuc = \frac{0.27}{1.15} \times 0.88 \times 0.15^{1/3} \times 15.00^{1/3} \times 914.4 \times 847.73 = \#\ \text{kN}$$

Applied shear force per m

if the applied shear force is less than the resistance then the element passes =

Applied force	=	60.65	kN
Resistance	=	208.99	kN
Pass / Fail	=	PASS	

Hinge Assessment

Plan on Concrete Hinge Throat

The hinge will be assessed in accordance with CS 486 'Assessment of Freyssient Hinges'. The assessment will only focus on the rectangular throat as the hinge is not a true freussient hinge.

In accordance with CS 468 Cl 3.2 the hinge will only be assessd at serviceabilty limit state

Equation 3.14 compressive resistance of hinge throat

$$N < \frac{2a_1 b_1 f_{cu}}{\gamma_m}$$

where:

- N is the axial force (N) on the throat;
- a_1 is the effective width of the throat (mm);
- b_1 is the effective length of the throat (mm);
- f_{cu} is the characteristic or worst credible cube strength (N/mm²), not greater than 52.5 N/mm²; and,
- γ_m is partial factor for material strength in accordance with CS 455 [Ref 6.N].

N = 525150 N 525.15

a1 = 101.6 mm

b1 = 381 mm

fcu = 15.00 N/mm^2

Ym = 1.5

$$\frac{2a1 \times b1 \times fcu}{ym} = \frac{2 \times 101.6 \times 381 \times 15.00}{1.5} = 774192\ \text{N}$$

774.192 kN

Verification

Is N less than the compressive resistance = PASS

Utilisation = 67.83 %

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Material & Section Properties

Reinforced Concrete

Reference	Description	Symbol		Value	Unit
CS 455, Cl.3.1.3	Characteristic Strength of Concrete		=	15.00	N/mm^2
	Density of Reinforced Concrete		=	24	kN/m^3
CS 454 table 4.1.1a					

Steel Reinforcement

Reference	Description	Symbol		Value	Unit
CS455, Cl, 3.8.2	Mild steel reinforcement characteristic yield strength		=	230	N/mm^2
	Design Young's Modulus of steel reinforcement		=	210000	N/mm^2

Mass Concrete (Plain Concrete)

Reference	Description	Symbol		Value	Unit
CS 454 table 4.1.1a	Unit weight of plain concrete		=	23	kN/m^3

Fill - Miscellaneous

Reference	Description	Symbol		Value	Unit
CS 454 table 4.1.1a	Unit weight of miscellaneous fill		=	22	kN/m^3

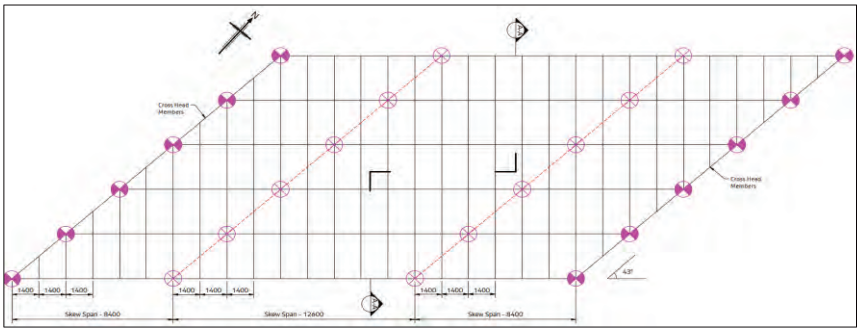
Carriageway & Footway Surfacing

Reference	Description	Symbol		Value	Unit
CS 454 table 4.1.1a	Unit weight of Bituminous Macadam (tar)		=	24	kN/m^3

Partial Factors

Reference	Description	Symbol		Value
CS 455 Table 2.13a	Partial Factor for reinforcement	γ_{ms}	=	1.15
CS 455 Table 2.13a	Partial Factor for Concrete	γ_{mc}	=	1.5
CS 455 Table 2.13a	Partial Factor for shear in concrete	γ_{mv}	=	1.15
CS 455 Table 2.13a	Partial Factor for Bond	γ_{mb}	=	1.25
CS 454 Table A.1	Partial Factor for Concrete Deadload		=	1.15
CS 454 Table A.1	Partial Factor for Surfacing		=	1.75
CS 454 Table A.1	Partial Factor for Fill		=	1.2
CS 454 3.9	Inaccurate assessment effects at ULS	$\gamma f3$	=	1.1

Grillage Model Span Dimmensions



MIDAS Grillage Model

Spans

Main Span 2 Skew Span	=	8.4	m
Main Span 3 Skew Span	=	12.6	m
Main Span 3 Skew Span	=	8.4	m

Tranverse slab effective widths

Longitudinal Direction	=	1400	mm
Transverse Direction	=	2500	mm

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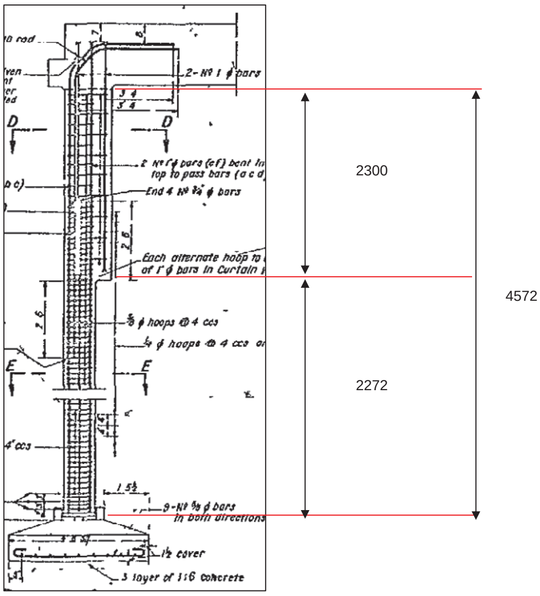
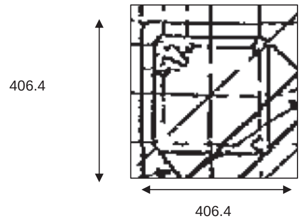
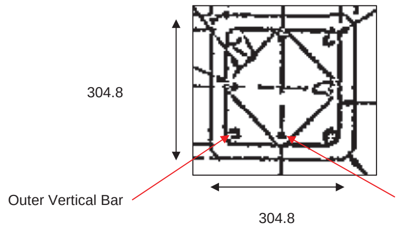
Abutment Column





Section ingnored as it is considering the curtain wall thickness

Section D-D



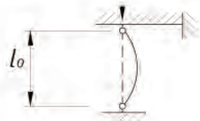
Section E-E

Dimensions above have been determined from the Survey drawing conducted on the 10/10/2023 and the As-Built drawing LC 5/3

Effective Height of the Column

The effective height of the column has been determined using CS 455 Table 7.2. The effective height is determining that the column is pinned at the top and bottom

CS 455, Table 7.2

Case 3	Top	Full	None	
	Bottom	Full	None	$1.0l_o$

Lo		=	2300	mm
Effective Height	Le	=	2300	mm

Slenderness criterion for short columns

$\frac{Le}{h} < 12 = \frac{4572}{304.8} = 15.00 < 12$ Slender

Forces applied onto the column

In accordance with CS 454 Equation 5.35.1 Longitudinal braking or traction Loads can be determined using the following equation

Equation 5.35.1 Longitudinal braking or traction load

$Q_L = \min(8L + 250, 750)$

where:

Q_L is the braking or traction load, in kN

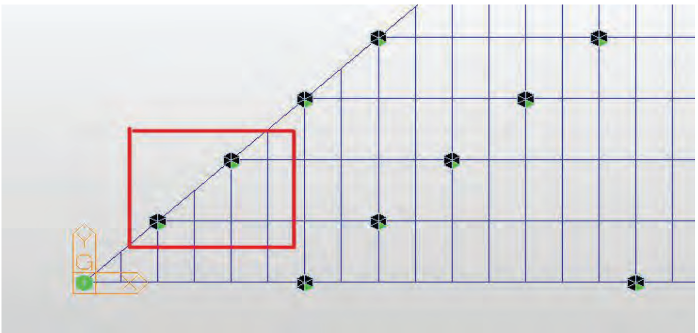
L is the loaded length, in m

L = Loaded Length = 29.4 m

Q_L = Minimim = ## < 750

Q_L = 485.2 kN

The braking load in accordance with CS 454 5.35.1 can only be applied to one lane. Therefore, it has been determined that the worse case loading will be applied to lane 1 and considering that 4 columns are working together to resist the braking force.



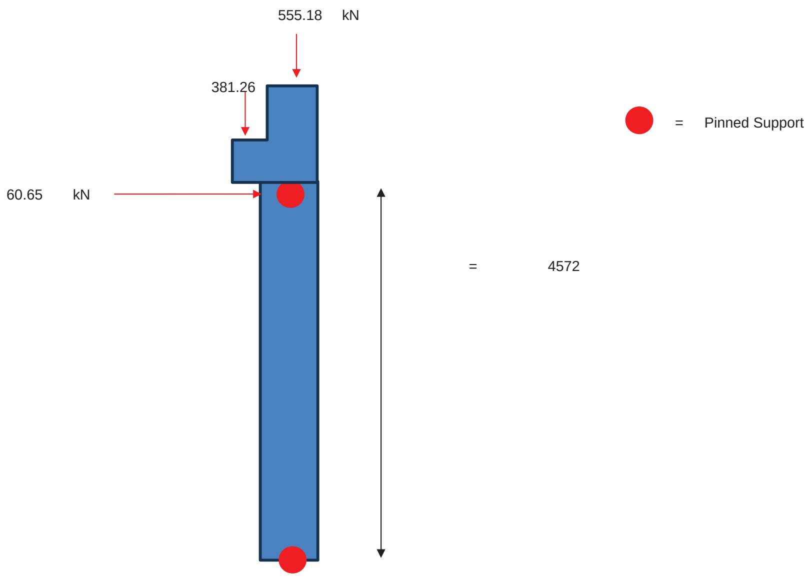
Midas Model showing worse case abutment column from braking

Braking forces distributed over 4 pier columns

$$\text{Force} = \frac{Q_L}{\text{No. Columns}} = \frac{485.2}{8} = 60.65 \text{ kN}$$

Worse case reaction due to live loading on deck	=	463.64	kN	(VehicleVa + SV80)	Node 66
Coexisting axial force due to deadload	=	91.54	kN		
Cway slab Deadload	=	33	kN		
Cway slab worse case reacton	=	381.26	kN	(Vehicle H2 + SV80)	

Idealised diagram of worse affected column (live loading)



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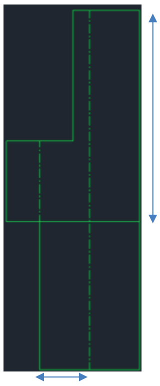
Bending Moment due to Eccentric loading from Carriageway Slab



698.5

0.152

Bending due to Eccentric load:

Force	=	381.26	kN					
e	=	0.152	m	(assumed acting around centre line of column)				
Moment =		381.26	x	0.152	=	57.95	kNm	(Unfactored)
Moment Factored for SV loading					=	63.75	kNm	(Factored)

Bending due to deadload:

Force	=	33	kNm					
e	=	0.152	m					
Moment =		33	x	0.152	=	5.02	kNm	(factored from Model)
Total Moment	=	63.75 +		5.02 =		68.76	kNm	

Due to the intermediate pier being pinned at the top of bottom, there is no bending in the column, therefore the column will be assessed for shear only.

Shear Resistance of the Intermediate Column

Internal Reinforcement

Bar Type	Bar Dia (")	Bar Dia (mm)	Area of Bar (mm^2)	Spacing (mm)	No. Bars	ea of Reinforcement (mn
Outer Hoop	3/8	9.525	71.26	101.6	1.00	71.26
Inner Link	3/4	19.05	285.02	101.6	1.00	285.02
Outer Vertical Bar	1	25.4	506.71	N/A	4	2026.83
Inner Vertical Bar	3/4	19.05	285.02	N/A	4	1140.09

Cover to reinforcement = 1 1/2 = 38.1 mm

Effective Depth, d = 261.94 mm

Area of the Pier = 155746.52 mm^2

Equation 5.6a Maximum Shear Based on Concrete Crushing

Equation 5.6a Maximum shear resistance based on concrete crushing

$$V_{\max} = 0.36\left(0.7 - \frac{f_{cu}}{250}\right)\left(\frac{f_{cu}}{\gamma_{mc}}\right)b_w d$$

b_W = is taken as the column diameter

$$V_{MAX} = 0.36 \times 0.7 - \frac{15.00}{250} \times \frac{15.00}{1.5} \times 304.8 \times 261.94 = \#\#\ \text{kN}$$

Equation 5.6b Shear resistance more than 3d from a support

$$V_{uc} = \frac{0.24}{\gamma_{mv}} \xi_s \rho_s^{\frac{1}{3}} f_{cu}^{\frac{1}{3}} b_w d$$

where:

γ_{mv} is the partial factor for shear defined in Section 2.

ξ_s is the depth factor, taken as $\xi_s = \left(\frac{500}{d}\right)^{0.25}$ but not less than 0.7

ρ_s is the ratio of longitudinal reinforcement $\rho_s = \frac{100A_s}{b_w d}$ but not less than 0.15, nor greater than 3.0

A_s is the area of longitudinal tension reinforcement that continues at least a distance d beyond the section being considered, and, for shear at supports, continues at least to the support.
Where top and bottom reinforcement are provided, the area in tension under the loading which produces the shear force is to be used.

$\xi s = \left(\dfrac{500}{261.94}\right)^{0.25} = 1.18$

Ps $= \dfrac{100 \times 1013.41}{304.8 \times 261.94} = 1.27$ = limited to 3.00

Shear resistance Vuc is enahanced by carrying out Equation 7.10.1

$$1+\frac{0.15N}{A_c}$$

where:

N is the ultimate axial load in Newtons, but not greater than $0.11 f_{cu} A_c$

A_c is the area of the entire concrete section, in units of mm^2

Ultimate axial load = 555.18 x 1.65 = 916.047 kN or = 916047 N

= 0.11 x 15.00 x 155746.52 = 256981.8 N

Ultimate axial load = min value of = 916047 or 256981.75 = 256981.8 N

Shear stress enhancement Factor = $1 + \dfrac{0.15 \times 256981.7545}{155746.52}$ = 0.2475

Vuc = $\dfrac{0.24}{1.15}$ x 1.18 x 1.27$^{1/3}$ x 15.00$^{1/3}$ x 304.8 x 261.94 = ## kN

Enchancement Factor = 52.30 x 0.2475 = 12.94 kN

Vuc = 65.24 kN

<u>**Ultimate resistance axial load in columns**</u>

Equation 7.4.1a Ultimate resistance axial load in columns

$$N_u = \left(\frac{0.6 f_{cu}}{\gamma_{mc}}\right) b d_c + \left(\frac{f_y}{\gamma_{ms}}\right) A'_{sl} + \sigma_{s2} A_{s2}$$

d' = 9.525 mm

d_c = taken as h = 304.8 mm d2 = 261.94 mm

A'_{sl} = $\dfrac{3166.92}{2}$ = 3166.92 mm^2 b = 304.8

σ_{s2} = from figure 3.13.1 = $\dfrac{230}{1.15}$ = 7.83 n/mm^2

A_{S2} = 3166.92 mm^2

h = 304.8

Nu = $\dfrac{0.6 \times 15.00}{1.5}$ x 304.8 x 304.8 = 557.42 kN

= $\dfrac{230}{1.15}$ x 3166.92 = 633.38 kN

= 7.83 x 3166.92 = 24.78 kN

Nu = 1215.59 kN

<u>**Ultimate bending resistance moment in columns**</u>

Equation 7.4.1b Ultimate resistance moment in columns

$$M_u = \left(\frac{0.3 f_{cu}}{\gamma_{mc}}\right) b d_c (h - d_c) + \left(\frac{f_y}{\gamma_{ms}}\right) A'_{sl}\left(\frac{h}{2} - d'\right) - \sigma_{s2} A_{s2}\left(\frac{h}{2} - d_2\right)$$

dc = 2 x d' due to looking at tension = 2 x 9.525 = 19.05 mm

$$Mu = \frac{0.3 \times 15.00}{1.5} \times 304.8 \times 19.05 \times 304.8 - 19 = 4.98 \text{ kNm}$$

$$= \frac{230}{1.15} \times 3166.92 \times \frac{304.8}{2} - 9.525 = 90.49 \text{ kNm}$$

$$= 7.83 \times 3166.92 \times \frac{304.8}{2} - 261.94 = -2.71 \text{ kNm}$$

Mu = 98.19 kNm

Summary Table

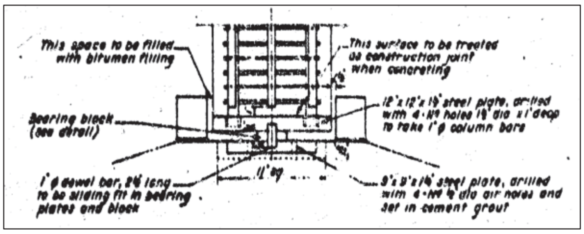
Element	Section Capacity	DI + SI Effects	Live Load Effects	Total Load Effect	Uitilsation	Pass / Fail	Remarks
Abutment Column	65.24	2.5	60.65	63.15	0.97	PASS	Shear
	98.19	5.02	63.75	68.76	0.70	PASS	Bending
	1215.59	91.54	510.00	601.54	0.49	PASS	Axial

Abutment Column Hinge Check

The abutment column hinge is located at the base of the abutment column just above the pad foundations. The hinge consists of a steel bearing block and dowels which hold down the colum to the pad foundation limiting its movement.

The check will consist of the following:

1) Shear capacity check of the dowel at the base of the column.
2) Concrete compression check at the base of the column



Section on Hinge

Dowel Shear Capacity Check

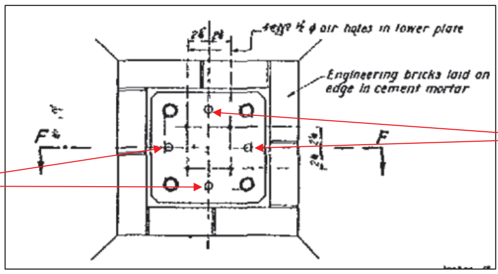
The shear capacity of one dowel, p_{sh} = $\frac{0.6 \times Fy \times Av}{\gamma_s}$

Av = Shear area of dowel

Dowel Diamter = 1" = 25.4 mm

$$Av = \frac{3.14 \times 25.42^2}{4} = 506.71 \text{ mm}^2$$

$$P_{sh} = \frac{0.6 \times 230 \times 506.71}{1.15} = 60.80 \text{ kN}$$



Plan on Hinge

It is assumed that the 4no. of dowels as shown above will be working together to resist the applied shear force.

Total dowel shear capacity = 60.80 x 4 = 243.22 kN

Applied shear force = 60.65 kN

Verification

if the applied shear force is less than the resistance then the element passes = PASS

Concrete compression check

Axial Load = 1053230 N

Width of Concrete in Compression = 304.8 mm

Length of Concrete in Compression = 304.8 mm

Compression in concrete due to axial load $= \frac{1053230}{304.8} = \frac{3455.48}{304.8} =$ 11.34 N/mm^2

CS 455 Cl 10.7 (2)

Permissible compression in the concrete $= \frac{1.5 \times 15.00}{1.5} =$ 15 N/mm^2

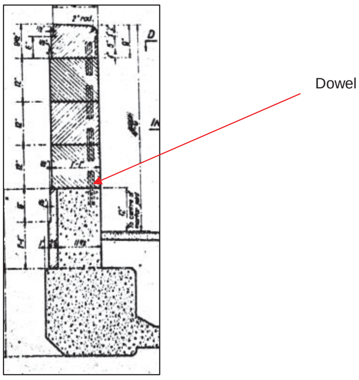
Verification

The permissible compression on the concrete must be greater than the stress generated from the axial load

Pass / Fail = PASS

Assessment of in service parapets

The parapets over the structure are of precast concrete construction. The parapets connected to the parapet beam by 1 1/2" dowels.



Parapet Connection

The parapet assessment will be conducted in accordance with CS 461 Section 3, Risk assessment of exisiting parapet site and prioritation of parapet upgrading.
Due to Station Way bridge spanning over LUL infastructure, the parapet will be assessed using figure 3.2b of CS 461.

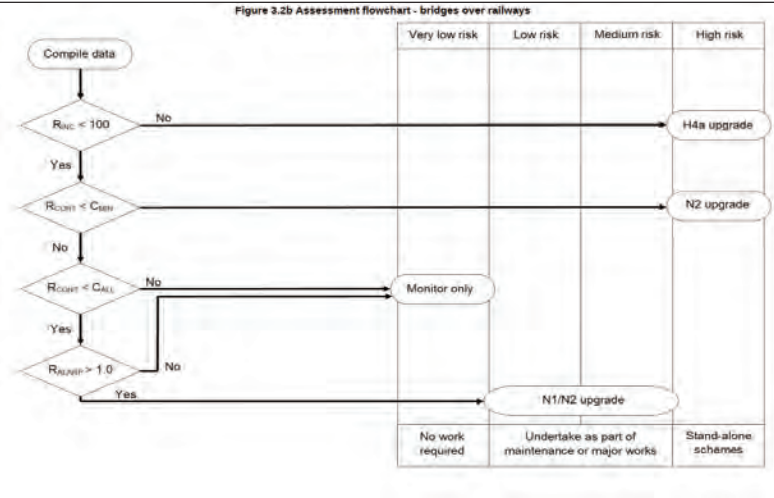


Figure 3.2b Assessment flowchart - bridges over railways

Table 3.5 Definition for parameters used in risk process flowcharts

Parameter	Definition	Notes
R_{ALARP}	ALARP-based risk ranking score	Refer to A2 and A4 of Appendix A.
R_{INC}	incursion risk ranking score for the highest scoring corner	Refer to Appendix B (also see sub-clauses 3.5.1 and 3.5.2).
R_{CONT}	remnant resistance of the parapet	expressed as a proportion of the required containment resistance C_{REQ} . Refer to Section 4 for guidance.
C_{ALL}	allowable resistance of the parapet	Refer to Equation 3.6.
C_{MIN}	minimum resistance of the parapet	Refer to Equation 3.7 and Table 3.8.
C_{REQ}	required containment resistance of the parapet	Refer to Equation 3.6 and Table 3.6.

Equation A.1 ALARP - based risk ranking score

$$R_{ALARP} = \frac{AADT \times F_1 \times F_2 \times F_3}{10000}$$

AADT = 8033 (taken from ECC Traffic Count dated 31.01.22 - 06.02.22)

F_1 = 5 (0%-33%)

F_2 = 1.12 (footway width = 2.5)

F_3 = 1.00 (new drilled anchors)

$$R_{ALARP} = \frac{8033 \times 5 \times 1.12 \times 1.00}{10000} = 4.50$$

Incursion Risk Ranking

Instructions: overall scoring and methodology

The overall score for a bridge is obtained by adding all 14 factors together.

As a guide, an increase of two in a score for any of the factors or for an overall risk score implies a doubling of the risk, so 6 is twice as bad as 4, and 12 is eight times worse than 6. This gives a wide range of risk values. A score of 90 implies that the risk is approximately a million times bigger than a score of 50.

The scoring regime assumes that no factor needs a score of zero, as even the best protection still allows a slim chance of a vehicle or debris, reaching the line.

Assessors are to rank bridges according to score, assessing the highest scoring bridges in more detail to see how they can be improved. As a guide, scores of 100 or more are significant and scores of 70 or more would suggest that highway authorities should at least consider the practicability of improvements. This does not rule out simple and cost-effective improvements at bridges that score less than 70. Mitigation action is not strictly required when:

1) for bridges carrying single carriageway roads that either score one for factor 1 (road approach containment) or score of 1 for factor 5 (site topography); and,
2) for bridges carrying motorway and dual carriageway roads that score one for factor 1 (road approach containment) combined with a score of 1 for factor 5 (site topography), and a score of two or less for factor 8 (vehicle parapet resilience).

For Mass Concrete Parapet
B1 factors:single carriageway over rail

Factors			Justification
f1	=	12	Medium / lightly wooded approaches / imperfect fencing
f2	=	1	Striaght
f3	=	2	Slight hump back
f4	=	3	30mph
f5	=	1	Vehicle / Debris unlikley to foul track
f6	=	5	Residential parking on approach to structure
f7	=	1	No obvious hazards
f8	=	5	Assumed as masonry parapet
f9	=	1	Footpath at least 2m on both sides
f10	=	1	signage / markings fit for purposes, clear and visible
f11	=	1	Local B class road
f12	=	1	Straight track up to 45mph
f13	=	11	Light rail
f14	=	12	Very heavily used route

Total 57 < 100

R_{CONT} = 34%-66% = 1.00

C_{REQ} = 1 N2 Therefore = 1 N2

C_{ALL} = $0.67C_{REQ}$ = 0.67 x 1 = 0.67 N2

C_{min} = 0.50 N2

Summary

R_{INC} = 57 < 100 **Yes**

R_{cont} = 1 N2 < C_{MIN} = 0.5 N2 = **No**

N1 / N2 Upgrade required

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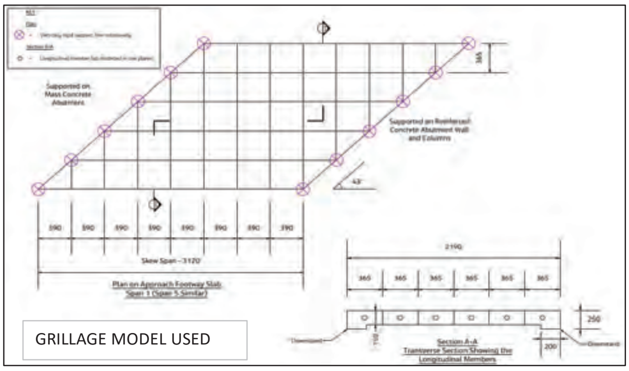
APPROACH SPAN (SPAN 1) - ORIGINAL FOOTWAY SLAB



Note: Accidental vehicle loading has not been considered as slab has been found to be only just adequate for permanent loading combined with pedestrian loading. By inspection, slab not adequate for any level of accidental vehicle loading.

RESULTS SUMMARY

Longitudinal Bending

Main Slab

ULS Bending

Loading / Effect	*Capacity (kN.m)	*DL+SDL (kN.m)	*LL (kN.m)	*Total BM (kN.m)	Pass / Fail	Utilisation (%)
Sagging (Pedestrian Loading)	3.6	2.6	0.8	3.5	PASS	97

ULS Shear

Loading / Effect	*Capacity (kN)	*DL+SDL (kN)	*LL (kN)	*Total SF (kN)	Pass / Fail	Utilisation (%)
Shear (Pedestrian Loading)	23.6	7.6	2.3	9.9	PASS	42

* 365mm wide member
Note: Shear capacity at 3d compared with applied shear force at support, therefore conservative.

Downstand

ULS Bending

Loading / Effect	**Capacity (kN.m)	**DL+SDL (kN.m)	**LL (kN.m)	**Total BM (kN.m)	Pass / Fail	Utilisation (%)
Sagging (Pedestrian Loading)	27.0	7.8	2.4	10.2	PASS	38

ULS Shear

Loading / Effect	**Capacity (kN)	**DL+SDL (kN)	**LL (kN)	**Total SF (kN)	Pass / Fail	Utilisation (%)
Shear @ d (Pedestrian Loading)	53.1	25.7	7.6	33.3	PASS	63
Shear @ 3d (Pedestrian Loading)	30.8	16.4	5.4	21.8	PASS	71

**203mm wide member (at bottom of section)

Transverse Bending - Main Slab

	Longitudinal Member (Main Slab)	Transverse Member (Main Slab)
Width of member	365mm	390mm
Sagging (bottom) reinforcement	3/8" dia at 6" centres	3/8" dia at 3"centres
Max ULS applied sagging bending moment (for width above)	3.5 kN.m	2.4 kN.m
Capacity	3.6 kN.m	> 3.6 (By inspection)
Conclusion	OK	OK (By Inspection)

REF	CALCULATION			OUTPUT
	MATERIAL PROPERTIES AND PARTIAL FACTORS			
	Reinforced Concrete			
AIP, Cl 3.10	Characteristic strength of concrete	= 15	N/mm^2	
CS 455, Cl 3.1.3	Density of reinforced concrete	= 24	kN/m^3	
	Steel Reinforcement			
AIP, Cl 3.10	Mild steel reinforcement characteristic yield strength	= 230	N/mm^2	
	Design Young's Modulus of steel reinforcement	= 200000	N/mm^2	
	Fill - Miscellaneous			
CS 454, CL4.1.1a	Unit weight of miscellaneous fill	= 22	kN/m^3	
	Carriageway & Footway Surfacing			
CS 454, CL4.1.1a	Unit weight of bituminous macadam (tar)	= 24	kN/m^3	
	Partial Factors			
CS 455, Tbl 2.13a	Partial factor for reinforcement	γ_{ms} = 1.15		
CS 455, Tbl 2.13a	Partial factor for concrete	γ_{mc} = 1.5		
CS 455, Tbl 2.13a	Partial factor for shear in concrete	γ_{mv} = 1.25		
CS 455, Tbl 2.13a	Partial factor for bond	γ_{mb} = 1.4		
CS 454 Tbl 3.4	Partial factor for concrete deadload	= 1.15		
CS 454 Tbl 3.4	Partial factor for surfacing	= 1.75		
CS 454 Tbl 3.4	Partial factor for fill	= 1.2		
CS 455, Cl 3.9	Inaccurate assessment effects at ULS	γ_{f3} = 1.1		
CS 454 Tbl 3.4	Partial factor for live loading	γ_{fL} = 1.5		

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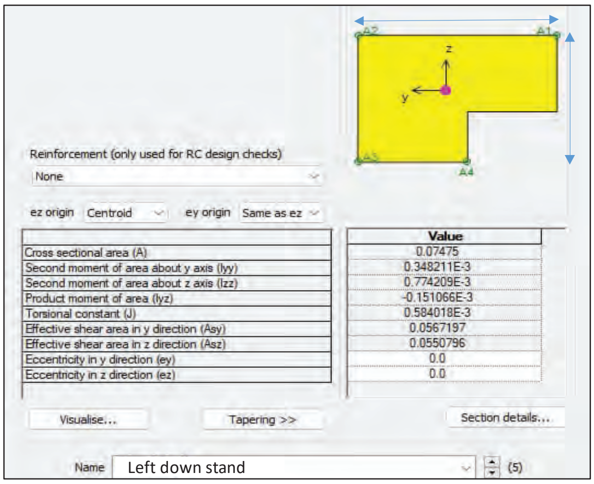
SECTION PROPERTIES

365 mm

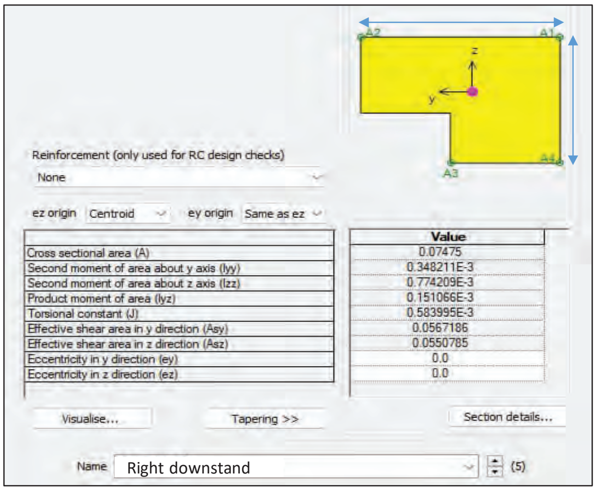
250 mm



	Value
Cross sectional area (A)	0.07475
Second moment of area about y axis (Iyy)	0.348211E-3
Second moment of area about z axis (Izz)	0.774209E-3
Product moment of area (Iyz)	-0.151066E-3
Torsional constant (J)	0.584018E-3
Effective shear area in y direction (Asy)	0.0567197
Effective shear area in z direction (Asz)	0.0550796
Eccentricity in y direction (ey)	0.0
Eccentricity in z direction (ez)	0.0

365 mm

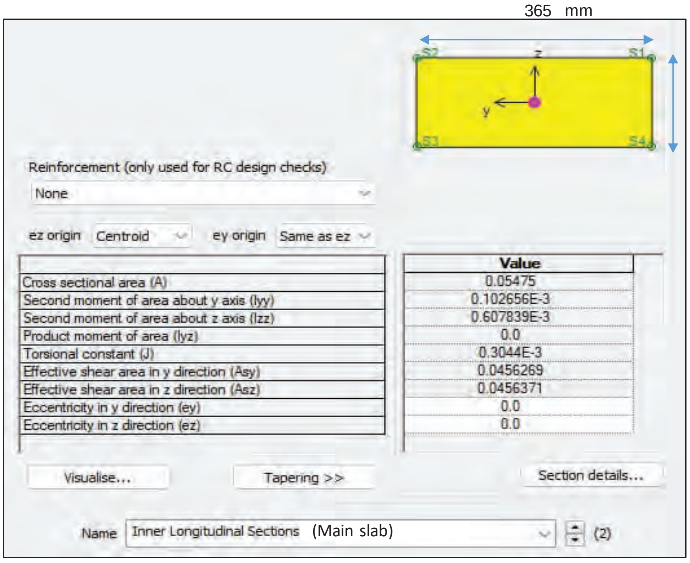
250 mm



	Value
Cross sectional area (A)	0.07475
Second moment of area about y axis (Iyy)	0.348211E-3
Second moment of area about z axis (Izz)	0.774209E-3
Product moment of area (Iyz)	0.151066E-3
Torsional constant (J)	0.583995E-3
Effective shear area in y direction (Asy)	0.0567186
Effective shear area in z direction (Asz)	0.0550785
Eccentricity in y direction (ey)	0.0
Eccentricity in z direction (ez)	0.0

365 mm

150 mm



	Value
Cross sectional area (A)	0.05475
Second moment of area about y axis (Iyy)	0.102656E-3
Second moment of area about z axis (Izz)	0.607839E-3
Product moment of area (Iyz)	0.0
Torsional constant (J)	0.3044E-3
Effective shear area in y direction (Asy)	0.0456269
Effective shear area in z direction (Asz)	0.0456371
Eccentricity in y direction (ey)	0.0
Eccentricity in z direction (ez)	0.0

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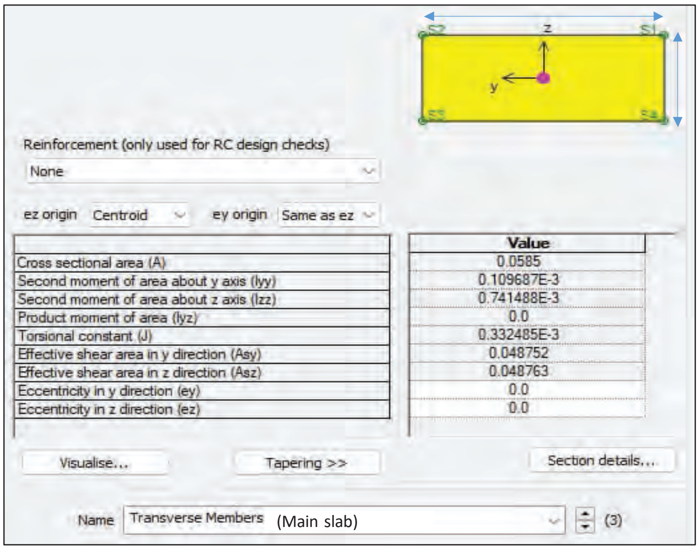
REF	CALCULATION	OUTPUT

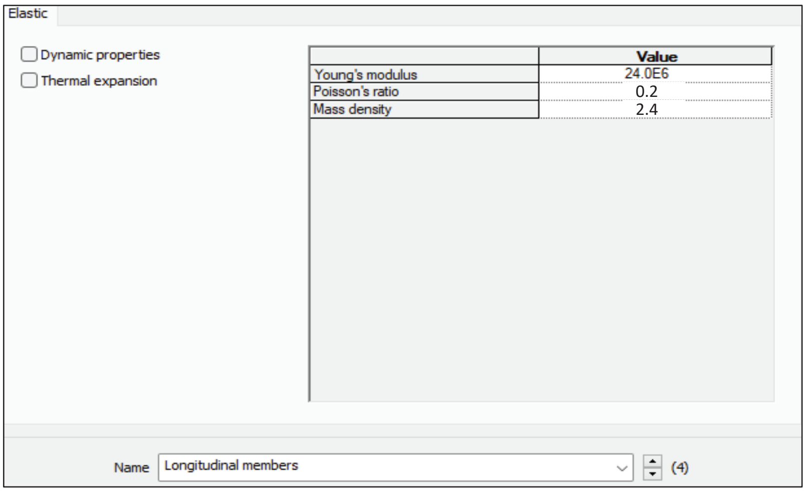
390 mm

150 mm



	Value
Cross sectional area (A)	0.0585
Second moment of area about y axis (Iyy)	0.109687E-3
Second moment of area about z axis (Izz)	0.741488E-3
Product moment of area (Iyz)	0.0
Torsional constant (J)	0.332485E-3
Effective shear area in y direction (Asy)	0.048752
Effective shear area in z direction (Asz)	0.048763
Eccentricity in y direction (ey)	0.0
Eccentricity in z direction (ez)	0.0

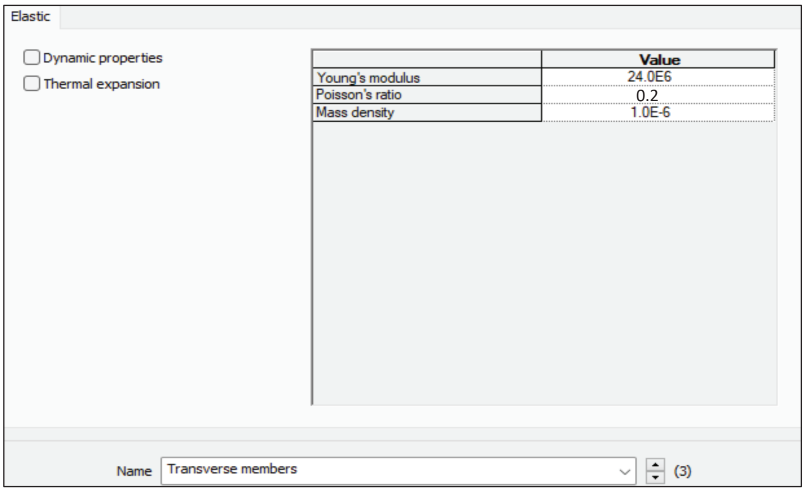
MATERIAL PROPERTIES



	Value
Young's modulus	24.0E6
Poisson's ratio	0.2
Mass density	2.4

kN/m2

t/m3



	Value
Young's modulus	24.0E6
Poisson's ratio	0.2
Mass density	1.0E-6

kN/m2

t/m3

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REF	CALCULATION	OUTPUT

LOADING

Self-weight

Width = 200 + 1790 + 200 = 2190

depth = 250 mm 150 mm 250 mm

Length = 3120 mm

Density = 24.0 kN/m3

Unfactored selfweight = 27.59 kN

Factored selfweight = 27.59 x 1.15 (γ_{fL}) x 1.1 (γ_{f3}) = 34.9 kN

Infill

Width = 2190 mm

depth = 550 mm

Length = 3120 mm

Density = 22.0 kN/m3

Unfactored infill loading = 82.7 kN

Factored Infill Loading = 82.7 x 1.2 (γ_{fL}) x 1.1 (γ_{f3}) = 109.1 kN

Concrete pavings

Width = 2190 mm

depth = 50.8 mm

Length = 3120 mm

Density = 21.6 kN/m3

Unfactored Con Paving Loading = 7.5 kN

Factored Con paving Loading = 7.5 x 1.2 (γ_{fL}) x 1.1 (γ_{f3}) = 9.9 kN

Surfacing

Width = 2190 mm

depth = 100 mm

Length = 3120 mm

Density = 24.0 kN/m3

Unfactored Surfacing Loading = 16.4 kN

Factored Surfacing Loading = 16.4 x 1.75 (γ_{fL}) x 1.1 (γ_{f3}) = 31.6 kN

Pedestrian Loading

Width = 2190 mm

Length = 3120 mm

Pedestrian Load = 5.0 kN/m2

Unfactored Ped Loading = 34.2 kN

Factored Ped Loading = 34.2 x 1.5 (γ_{fL}) x 1.1 (γ_{f3}) = 56.4 kN

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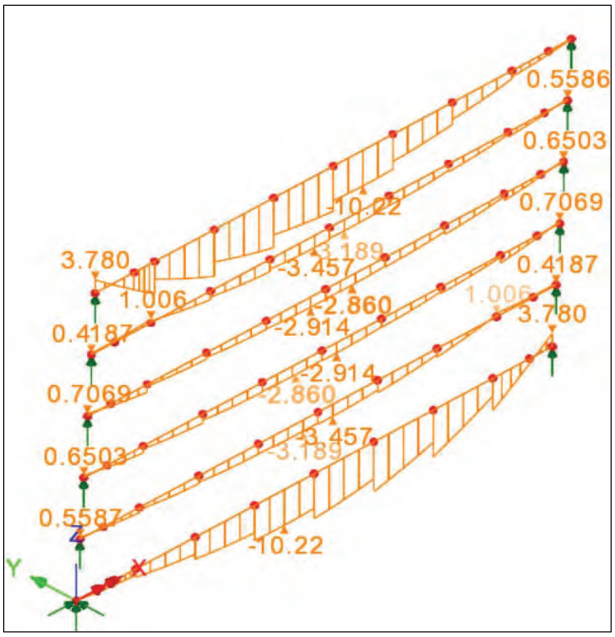
REF	CALCULATION	OUTPUT

APPLIED BENDING MOMENT - MAIN SLAB AND DOWNSTAND (OBTAINED FROM GRILLAGE MODEL)

7:Loadcase 7
Loading
1:Infill (x 1.2) x 1.1
2:Surfacing (x 1.75) x 1.1
3:Concrete paving (x 1.2) x 1.1
4:Pedestrain Loading (x 1.5) x 1.1
Gravity (x1.15) x 1.1

Longitudinal grillage members



Longitudinal Sagging bending (ULS)

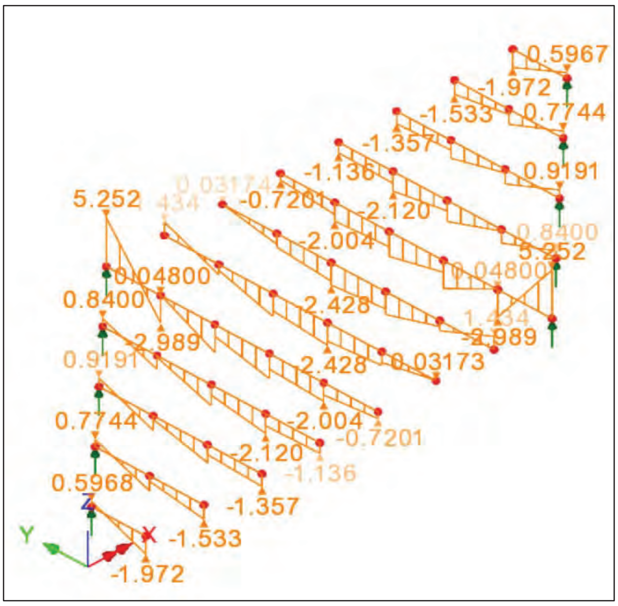
Max applied BM for main slab (365mm wide member) = 3.5 kN.m — (permanent loading + 5 kN/m2 Pedestrian live loading)

Capacity of slab (365mm wide member) = 3.6 kN.m, page ref: A 7 — ∴ OK

Maximum applied bending moment for downstand member = 10.2 kN.m — (permanent loading + 5 kN/m2 Pedestrian live loading)

Capacity of downstand member = 27.0 kN.m, page ref: A 8 — ∴ OK

Transverse grillage members



Transverse Bending (ULS)

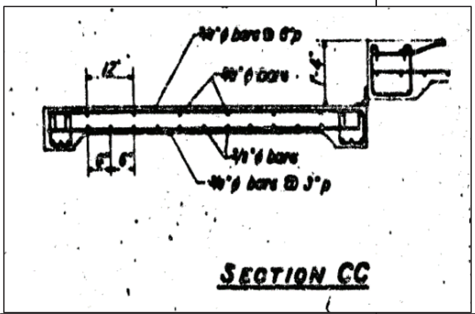
OK for permanent loading + pedestrian live loading by inspection (refer to summary table on page no. A 1

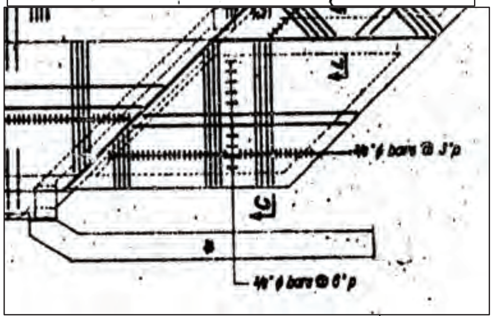
REF	CALCULATIONS	OUTPUT

LONGITUDINAL SAGGING BENDING CAPACITY - MAIN SLAB

Calculations of Effective Depth, d *(for reinforcement details refer to record drawings)*

REF						
	Slab height, h	=	152		mm	
	Nominal cover, c	=	38.0		mm	
	Transverse reinf.	=		9.53	mm bars	(Top & Bottom)
	Top layer (1)	=	1.2 x	9.53	mm	(3/8" at 12" c/c)
	Top layer (2)		0.0 x	0.00	mm bars	
A_{s2}	Bottom Layer (2)	=	0.0 x	0.00	mm	
A_{s1}	Bottom Layer (1)		2.75 x	9.53	mm bars	(11 x 3/8" bars in total for 4 no. members)





$d = h - y_1$

$y_1 = c + 9.5 \text{ (Trans rein.)} + 0.0 + 4.8 \text{ (Bott layer)}$

btm layer: $y_1 = 52.3$ mm

$A_{s1} = 196$ mm^2

$A_{s2} = 0$ mm^2

$d = 100.1$ mm

$b = 365.0$ mm

The position of the neutral axis to determine whether the compression reinforcement should be ignored:

CS 455 Eqn. 5.2.2d

$$\frac{f_y}{\gamma_{ms}} \times A_s = 0.6 \times \frac{f_{cu}}{\gamma_{mc}} \times bx + f_s' \times A_s'$$

f_y	=	230	N/mm^2	is the characteristic or worst credible strength of the reinforcement
γ_{ms}	=	1.15		is the partial factor for the reinforcement strength
A_s	=	196.0	mm^2	is the area of tension reinforcement
A_s'	=	85.3	mm^2	is the area of compression reinforcement
f_{cu}	=	15.0	N/mm^2	is the characteristic or worst credible cube strength of the concrete
γ_{mc}	=	1.50		is the partial factor for the concrete strength
f_s'	=	182		$\dfrac{f_y}{\gamma_{ms} + \dfrac{f_y}{2000}}$

$\therefore$ bx = 3946 mm^2

By the method of trial and error:

b = 365 mm

x = 10.8 mm

CS 455 Cl. 5.2.3

For d' < 0.429x

The compression reinforcement will be ignored

d' = 38.0 mm > 4.6 mm $\therefore$ **The compression reinforcement will be ignored.**

Cl. 5.2.2

$\therefore$ The moment resistance will be calculated using Equation 5.2.2a for sections without compression reinforcement.

Eqn. 5.2.2a

$$M_u = \min\left(\frac{f_y}{\gamma_{ms}} \times A_s \times z \quad \text{or} \quad 0.23 \times \frac{f_{cu}}{\gamma_{mc}} \times b \times d^2\right)$$

Eqn. 5.2.2b

$$z = \left(1 - \frac{0.84 \times \frac{f_y}{\gamma_{ms}} \times A_s}{\frac{f_{cu}}{\gamma_{mc}} \times b \times d}\right) \times d \leq 0.95 \times d$$

z = 91 mm < 95 mm

Eqn. 5.2.2a

M_u = **3.6** **kNm - ULS SAGGING moment resistance of main slab (365mm wide member)**

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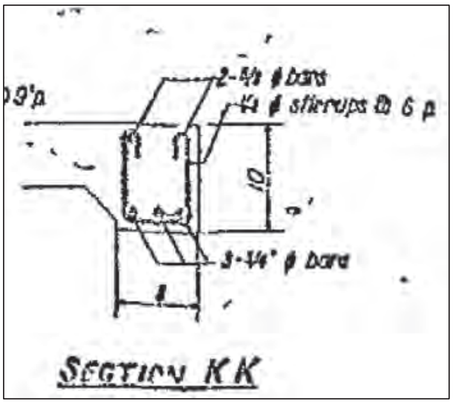
REF	CALCULATION	OUTPUT

LONGITUDINAL BENDING CAPACITY - DOWNSTAND

Calculations of Effective Depth, d *(for reinforcement details refer to record drawings)*

Downstand height = 254 mm
Nominal cover, c = 38.0 mm

REF							
A_{s1}	Top layer (1)	=	2.0	x	15.88	mm	(2 x 5/8")
A_{s2}	Top layer (2)		0.0	x	0.00	mm bars	
	Bottom Layer (2)	=	0.0	x	0.00	mm	
	Bottom Layer (1)		3.0	x	19.05	mm bars	(3 x 3/4")
	Shear links	=			9.53	mm links	



$d = h - y_1$

Trans rein. Shear links Bott layer

$y_1 = c + 0.0 + 9.5 + 9.5$

y_1 = 57.1 mm

A_{s1} = 855 mm^2

A_{s2} = 0 mm^2

d = 197.0 mm

b = 365.0 mm

The position of the neutral axis to determine whether the compression reinforcement should be ignored:

$$\frac{f_y}{\gamma_{ms}} \times A_s = 0.6 \times \frac{f_{cu}}{\gamma_{mc}} \times bx + f_s' \times A_s'$$

CS 455 Eqn. 5.2.2d

f_y	=	230	N/mm^2	is the characteristic or worst credible strength of the reinforcement
γ_{ms}	=	1.15		is the partial factor for the reinforcement strength
A_s	=	855	mm^2	is the area of tension reinforcement
A_s'	=	396	mm^2	is the area of compression reinforcement
f_{cu}	=	15.0	N/mm^2	is the characteristic or worst credible cube strength of the concrete
γ_{mc}	=	1.50		is the partial factor for the concrete strength
f_s'	=	182		$\dfrac{f_y}{\gamma_{ms} + \dfrac{f_y}{2000}}$

$\therefore$ bx = 16506 mm^2

By the method of trial and error:
b = 365 mm
x = 45.2 mm

For d' < 0.429x
The compression reinforcement will be ignored

CS 455 Cl. 5.2.3

d' = 47.5 mm > 19.4 mm $\therefore$ **The compression reinforcement will be ignored.**

Cl. 5.2.2 $\therefore$ The moment resistance will be calculated using Equation 5.2.2a for sections without compression reinforcement.

Eqn. 5.2.2a

$$M_u = \min\left(\frac{f_y}{\gamma_{ms}} \times A_s \times z \quad \text{or} \quad 0.23 \times \frac{f_{cu}}{\gamma_{mc}} \times b \times d^2\right)$$

Eqn. 5.2.2b

$$z = \left(1 - \frac{0.84 \times \frac{f_y}{\gamma_{ms}} \times A_s}{\frac{f_{cu}}{\gamma_{mc}} \times b \times d}\right) \times d \leq 0.95 \times d$$

z = 157.6 mm < 187 mm

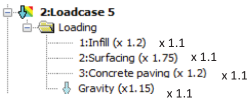
Eqn. 5.2.2a

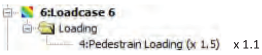
M_u = **27.0** **kNm - ULS SAGGING moment resistance of downstand**

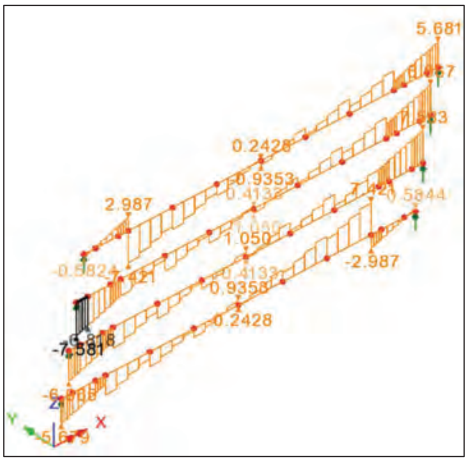
REF	CALCULATION	OUTPUT

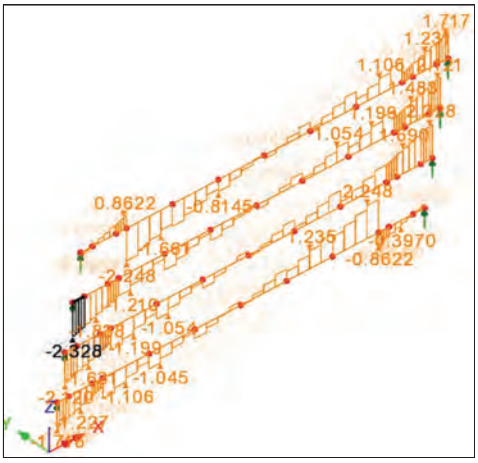
APPLIED SHEAR FORCE - MAIN SLAB

Main Slab Shear at Support









Maximum applied shear force due to ULS DL+SDL = 7.6 kN

Maximum applied shear force due to ULS Pedestrian loading = 2.3 kN

Total ULS applied loading = 9.9 kN **(Shear at Support)**

page ref: A 11 Shear capacity of main slab (365mm wide member) @ 3d from supp = 23.6 kN ∴ OK (See note below)

Note: Shear Capacity adjacent to support exceeds shear capacity at 3d. Therefore, above check is conservative.

JACOBS					CALCULATION SHEET
OFFICE	MANCHESTER	PAGE No.	A 10	CONT'N PAGE No.	A 11
JOB No. & TITLE	ESSEX COUNTY COUNCIL - STATION WAY BRIDGE ASSESSMENT	ORIGINATOR	MA	DATE	24/02/2025
SECTION	ASSESSMENT OF ORIGINAL FOOTWAY SLABS (SPAN 1 & 5)	CHECKER	CATII	DATE	06/03/2025

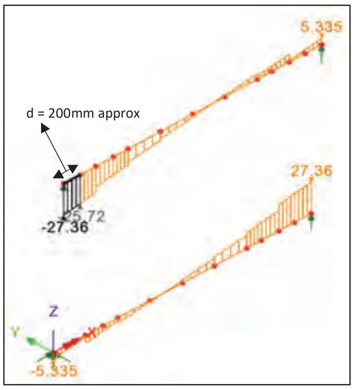
REF	CALCULATION	OUTPUT

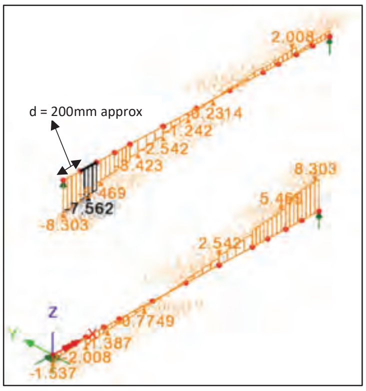
APPLIED SHEAR FORCE - DOWNSTAND

Downstand @ d from support

2:Loadcase 5
- Loading
  - 1:Infill (x 1.2) x 1.1
  - 2:Surfacing (x 1.75) x 1.1
  - 3:Concrete paving (x 1.2) x 1.1
  - Gravity (x1.15) x 1.1

6:Loadcase 6
- Loading
  - 4:Pedestrain Loading (x 1.5) x 1.1





Maximum applied shear force due to ULS DL+SDL = 25.7 kN

Maximum applied shear force due to ULS Pedestrian loading = 7.6 kN

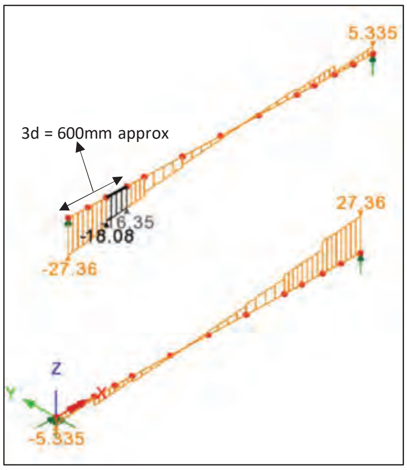
Total ULS applied loading = 33.3 kN

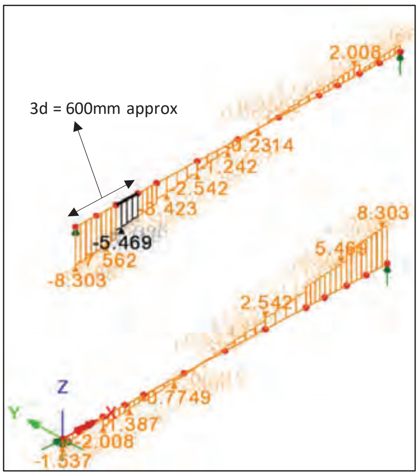
page ref: A 13 — Shear capacity of downstand @ d from supp = 53.1 kN ∴ OK

Downstand @ 3d from support

2:Loadcase 5
- Loading
  - 1:Infill (x 1.2) x 1.1
  - 2:Surfacing (x 1.75) x 1.1
  - 3:Concrete paving (x 1.2) x 1.1
  - Gravity (x1.15) x 1.1

6:Loadcase 6
- Loading
  - 4:Pedestrain Loading (x 1.5) x 1.1





Maximum applied shear force due to ULS DL+SDL = 16.4 kN

Maximum applied shear force due to ULS Pedestrian loading = 5.4 kN

Total ULS applied loading = 21.8 kN

page ref: A 12 — Shear capacity of downstand @ 3d from supp = 30.8 kN ∴ OK

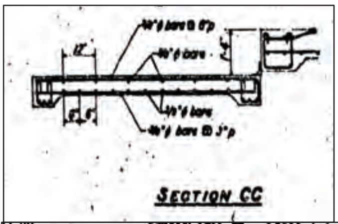
REF	CALCULATION	OUTPUT

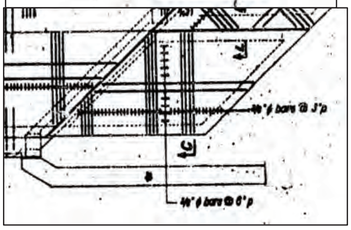
SHEAR CAPACITY - MAIN SLAB

For 365mm wide member:

Page ref. A 7 — As = 196.0 mm^2

Page ref. A 7 — d = 100.1 mm





Maximum shear Resistance based on concrete crushing

Eqn. 5.6a

$$V_{\max} = 0.36\left(0.7 - \frac{f_{cu}}{250}\right)\left(\frac{f_{cu}}{\gamma_{mc}}\right)b_w d$$

$$V_{max} = 0.36 \times \left(0.7 - \frac{15.0}{250}\right) \times \frac{15}{1.50} \times 365 \times 100 = 84.2 \text{ kN}$$

Shear Resistance more than 3d from a support

Eqn. 6.5

$$V_{uc} = \frac{0.27}{\gamma_{mv}}\xi_s \rho_s^{\frac{1}{3}} f_{cu}^{\frac{1}{3}} b_w d$$

$$\xi s = \frac{500}{100}^{0.25} = 1.49$$

ξ_s is the depth factor, taken as $\xi_s = \left(\frac{500}{d}\right)^{0.25}$ but not less than 0.7

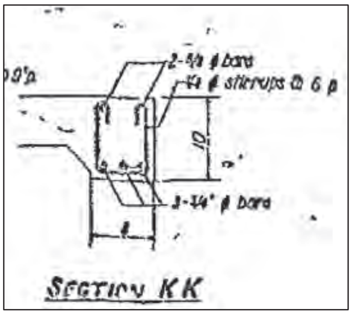
$$\rho_s = \frac{100 \times 196.0}{365 \times 100} = 0.54$$

$$V_{uc} = \frac{0.27}{1.25} \times 1.49 \times 0.54^{1/3} \times 15.0^{1/3} \times 365.0 \times 100$$

V_{uc} = 23.6 kN

Capacity at 3d (365mm wide member)

SHEAR RESISTANCE OF THE DOWNSTAND (CONSERVATIVELY IGNORE THE SHEAR REINFORCEMENT)



5.6 The assessment shear force shall not exceed:

1) V_{max} as defined in Equation 5.6a, anywhere;
2) V_{uc} as defined in Equation 5.6b, in regions more than $3d$ from a support; and,
3) V_u as defined in Equation 5.6c within $3d$ of a support.

Eqn. 5.6a $$V_{max} = 0.36 \times \left(0.70 - \frac{f_{cu}}{250}\right) \times \left(\frac{f_{cu}}{\gamma_{mc}}\right) \times b_w \times d$$

f_{cu} = 15 N/mm²

γ_{mc} = 1.50

b_w = 203 mm = 8" (at bottom of section)

Page ref. A 8 — d = 197 mm

∴ V_{max} = **92.2 kN**

Eqn. 5.6b $$V_{uc} = \frac{0.24}{\gamma_{mv}} \times \xi_s \times \rho_s^{1/3} \times f_{cu}^{1/3} \times b_w \times d$$

where:

γ_{mv} is the partial factor for shear

ξ_s is the depth factor

ρ_s is the ratio of longitudinal reinforcement

γ_{mv} = 1.25

$$\xi_s = \left(\frac{500}{d}\right)^{1/4} > 0.70$$

ξ_s = 1.26

$$\rho_s = \frac{100 \times A_s}{b_w \times d} > 0.15 \ \& < 3.00$$

A_s = 855 mm²

ρ_s = 2.14

∴ **V_{uc} = 30.8 kN** **Capacity at 3d**

Eqn. 5.6c $$V_u = \max\begin{cases} \frac{3d}{a_v}\Gamma V_{uc} \\ \frac{0.24}{\gamma_{mv}}\xi_s(0.15 f_{cu})^{\frac{1}{3}} b_w d \end{cases}$$

Eqn. 9.1b $$f_{ub} = \frac{k k_{cov} \beta \sqrt{f_{cu}}}{\gamma_{mb}}$$

k = 1.0 γ_{mb} = 1.4

CS 455, Tbl 9.1 — B = 0.39 (Plain bars in tension) ϕ = 19.05 mm

f_{cu} = 15.0 N/mm² K_{cov} = 1.0 (taken as 1)

f_{ub} = 1.08

$$F_{ub} = f_{ub} p L_a$$

P = 179.5 mm (For 3 no. bars)

L_a = 197 mm (conservative taken as d)

F_{ub} = 38.2 kN

Eqn. 5.6d

$$\Gamma = \min\begin{cases}\sqrt{\frac{z}{3d}\frac{F_{ub}}{V_{uc}}}\\ 1.0\end{cases}$$

Page ref. A 8

$z = 157.6$

$$\Gamma = \left(\frac{157.6}{590.9} \times \frac{38.2}{30.8}\right)^{1/2}$$

$$\Gamma = (0.27 \times 1.24)^{1/2}$$

$$\Gamma = 0.57$$

$$V_u = \frac{3d}{a_v}\Gamma V_{uc} = \frac{3 \times 197}{197} \times 0.575 \times 30.8 = 53.1 \quad \textbf{(at d)}$$

$$V_u = \frac{0.24}{\gamma_{mv}}\xi_s(0.15 f_{cu})^{\frac{1}{3}} b_w d = \frac{0.24}{1.25} \times 1.26 \times (0.15 \times 15.0)^{1/3} \times 203 \times 197 = 12.7 \text{ kN}$$

∴ $\boxed{V_u = 53.1 \text{ kN}}$ **Capacity at d**

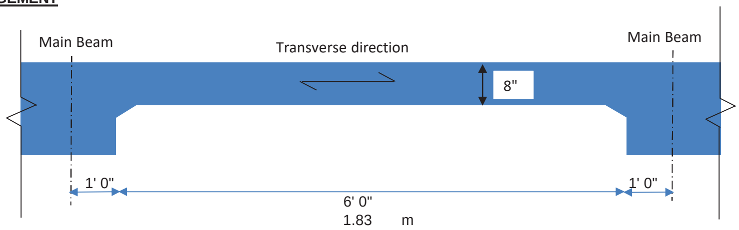
JACOBS **CALCULATION SHEET**

Office	Manchester	**Page No.**	B	1	**Calc No.**	
Job No. & Title	ECC - Assessment of Station Way Bridge	**Calcs by**	MA		**Date**	Mar-25
Section	Main Deck (Span 2 to 4) - Carriageway slab	**Checker**	CAT II		**Date**	Mar-25

CALCULATIONS FOR ASSESSMENT OF MAIN DECK SLAB (SPAN 2 – 4) – CARRIAGEWAY SLAB

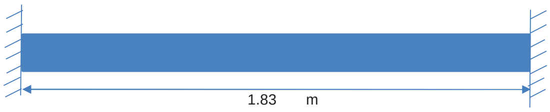
SLAB ARRANGEMENT

Record drgs:
LC5/2
LC5/6

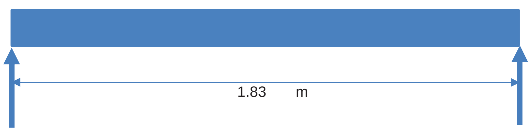


Idealised diagram

For Hogging:



For Sagging:



MATERIAL PROPERTIES AND PARTIAL FACTORS

Reinforced Concrete

Ref	Description	Symbol		Value	Unit
AIP, Cl 3.10	Characteristic Strength of Concrete		=	15.0	N/mm^2
CS 454 table 4.1.1a	Density of Reinforced Concrete		=	24	kN/m^3

Steel Reinforcement

Ref	Description	Symbol		Value	Unit
AIP, Cl 3.10	Mild steel reinforcement characteristic yield strength		=	230	N/mm^2
	Design Young's Modulus of steel reinforcement		=	200000	N/mm^2

Carriageway & Footway Surfacing

Ref	Description	Symbol		Value	Unit
CS 454 Table 4.1.1a	Unit weight of Bituminous Macadam (tar)		=	24	kN/m^3

Partial Factors

Ref	Description	Symbol		Value
CS 455 Table 2.13a	Partial Factor for reinforcement	γ_{ms}	=	1.15
CS 455 Table 2.13a	Partial Factor for Concrete	γ_{mc}	=	1.5
CS 455 Table 2.13a	Partial Factor for shear in concrete	γ_{mv}	=	1.25
CS 455 Table 2.13a	Partial Factor for Bond	γ_{mb}	=	1.4
CS 454, Tbl 3.4	Partial Factor for Concrete Deadload		=	1.15
CS 454, Tbl 3.4	Partial Factor for Surfacing		=	1.75
CS 454, Tbl 3.4	Partial Factor for nomral traffic, restricted traffic and footway loading	γ_{fL}	=	1.5
CS 454 Cl 3.9	Inaccurate assessment effects at ULS	γ_{f3}	=	1.1

Results Summary

Normal Traffic Loading

Bending

Loading / Effect	Capacity (kN.m/m)	DL+SDL (kN.m/m)	LL (kN.m/m)	Total BM (kN.m/m)	Pass / Fail	Utilisation (%)
44 Tonnes Sagging	45.1	4.5	34.4	38.8	PASS	86
44 Tonnes Hogging	-50.1	-3.0	-35.7	-38.7	PASS	77

Shear

Loading / Effect	Capacity (kN/m)	DL+SDL (kN/m)	LL (kN/m)	Total SF (kN/m)	Pass / Fail	Utilisation (%)
44 Tonnes Shear @ d	166.3	7.7	144.7	152.4	PASS	92
44 Tonnes Shear @ 3d	121.8	5.5	96.4	101.9	PASS	84

SV80 Loading

Bending

Loading / Effect	Capacity (kN.m/m)	DL+SDL (kN.m/m)	LL (kN.m/m)	Total BM (kN.m/m)	Pass / Fail	Utilisation (%)
SV80 Sagging	45.1	4.5	34.0	38.5	PASS	85
SV80 Hogging	-50.1	-3.0	-27.4	-30.4	PASS	61

Shear

Loading / Effect	Capacity (kN/m)	DL+SDL (kN/m)	LL (kN/m)	Total SF (kN/m)	Pass / Fail	Utilisation (%)
SV80 Shear @ d	166.3	7.7	105.3	113.0	PASS	68
SV80 Shear @ 3d	121.8	5.5	70.0	75.4	PASS	62

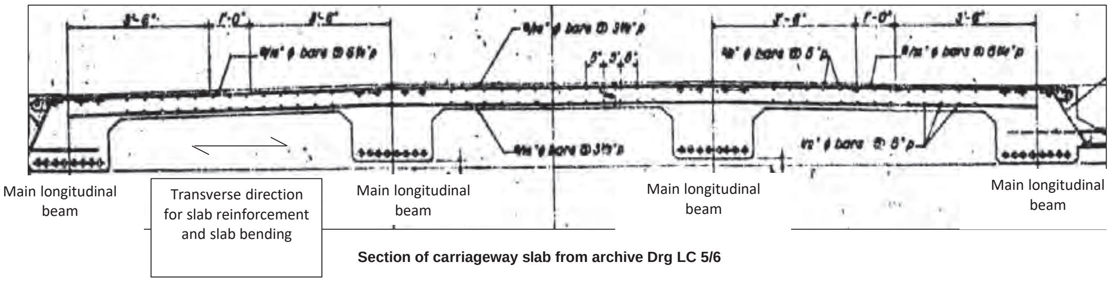
Notes:

1) The calculations are based on Pucher Charts and a line beam model assuming 1.0m wide carriageway slab.
2) For hogging moment, full fixity along the supported edges of each deck slab bay is assumed (conservative).
3) For sagging moment, no moment fixity along the supported edges is assumed (conservative).
4) Capacities are based on the defects identified in Appendix E of the AIP.
5) The global bending moments in the deck slab are deemed to be relatively low in magnitude. The combined global and local effects have been taken into account by the use of conservative values of the local bending moments (sagging and hogging).

JACOBS **CALCULATION SHEET**

Office	Manchester	**Page No.**	B 3	**Calc No.**	
Job No. & Title	ECC - Assessment of Station Way Bridge	**Calcs by**	MA	**Date**	Mar-25
Section	Main Deck (Span 2 to 4) - Carriageway Slab - Capacity	**Checker**	CAT II	**Date**	Mar-25

CARRIAGEWAY TRANSVERSE SLAB - BENDING CAPACITY (FULL SECTION WITHOUT DEFECTS)



Section of carriageway slab from archive Drg LC 5/6

LC 5/6 **Reinforcement**

Bar Type	Bar Dia (")	Bar Dia (mm)	Area of Bar (mm^2)	Spacing Inches	Spacing mm	No. Bars (per m width)	Area of Reinforcement (mm^2/m)
Transverse Bottom Bar	9/16	14.3	160.3	3 1/8"	79.4	12.6	2019.8
Longitudnal Bottom Bar	1/2	12.7	126.7	6"	152.4	6.6	831.2
Transverse Top Bar	9/16	14.3	160.3	3 1/8"	79.4	12.6	2019.8
Longitudinal Top Bar	1/2	12.7	126.7	6"	152.4	6.6	831.2

Moment Resistance for beams without compression reinforcement, Eq 5.2.2a

M_u = Minimum value dervied from the following equations $M_u = \min\left\{ \frac{f_y}{\gamma_{ms}} A_s z ,\ \frac{0.225}{\gamma_{mc}} f_{cu} b d^2 \right.$

Sagging (Transverse)

Cover to Reinforcement = 1 1/2" = 38 mm

Slab Depth = 203.2 mm

Effective Depth = 203.2 - 38.1 - 7.144 = 158 mm

z = 124.02 mm

$M_u = \frac{230}{1.15} \times 2019.85 \times 124.02 = 50.1$ kNm

$M_u = \frac{0.225 \times 15.00}{1.5} \times 1000 \times 158^2 = 56.14$ kNm

M_u = 50.1 kN.m/m	(without defects)

Hogging (Transverse)

Cover to Reinforcement = 1 1/2" = 38 mm

Slab Depth = 203.2 mm

Effective Depth = 203.2 - 38 - 7.1 = 158.1 mm

Z = 124.02 mm

$M_u = \frac{230}{1.15} \times 2019.85 \times 124.02 = 50.10$ kNm

$M_u = \frac{0.225 \times 15.00}{1.5} \times 1000 \times 158.1^2 = 56.21$ kNm

M_u = 50.1 kN.m/m	(without defects)

JACOBS				CALCULATION SHEET	
Office	Manchester	**Page No.**	B 4	**Calc No.**	
Job No. & Title	ECC - Assessment of Station Way Bridge	**Calcs by**	MA	**Date**	Mar-25
Section	Main Deck (Span 2 to 4) - Carriageway Slab - Capacity	**Checker**	CAT II	**Date**	Mar-25

CARRIAGEWAY TRANSVERSE SLAB - BENDING CAPACITY (ASSUMING 1MM SECTION LOSS TO TRANSVERSE BOTTOM BARS*)

Bar Type	Bar Dia (uncorroded)	Bar Dia (corroded) (mm)	Area of Bar (mm^2)	Spacing (mm)	No. Bars	Area of Reinforcement (corroded) (mm^2)
Transverse Bottom Bar	14.3	13.3	138.7	79.4	12.6	1747.0
Transverse Top Bar	No defects					

Sagging (Transverse)

Moment Resistance for beams without compression reinforcement, Eq 5.2.2a

M_u = Minimum value dervied from the following equations $M_u = \min\left\{\frac{f_y}{\gamma_{ms}} A_s z, \frac{0.225 f_{cu}}{\gamma_{mc}} b d^2\right.$

Cover to Reinforcement = 1 1/2" = 38 mm

Slab Depth = 203.2 mm

Effective Depth = 203.2 - 38 - 6.64 = 158.5 mm

z = 129.11 mm

$$M_u = \frac{230}{1.15} \times 1747.00 \times 129.11 = 45.11 \text{ kNm}$$

$$M_u = \frac{0.225 \times 15.00}{1.5} \times 1000 \times 158^2 = 56.49 \text{ kNm}$$

$$\boxed{M_u = 45.1 \text{ kN.m/m}}$$ (Allowing for corrosion)

* Refer to photo no. 3 in Appendix E of AIP - Allowance for corroded transverse reinforcement in soffit of the deck.

Defect is localised (5 no. transverse bars exposed). 1m width of slab includes approx. 12.5 no. transverse bars. Allow for corrosion of these 5 no. bars by assuming a 1mm loss of diameter to all bars within 1m strip.

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Job No. & Title	ECC - Assessment of Station Way Bridge	**Calcs by**	MA	**Date**	Mar-25	
Section	Main Deck (Span 2 to 4) - Carriageway Slab - DL& SDL	**Checker**	CAT II	**Date**	Mar-25	

APPLIED LOADING

DL & SID loading

	Depth (m)		Per metre width		Density		γ_{fl}		γ_{f3}				
Self-weight	0.2	x	1.0	x	24	x	1.0	x	1.0	=	4.8	kN/m	SLS
						x	1.15	x	1.1	=	6.07	kN/m	ULS
Surfacing	0.1	x	1.0	x	24	x	1.0	x	1.0	=	2.4	kN/m	SLS
						x	1.75	x	1.1	=	4.62	kN/m	ULS

Total DL & SID (Unfactored) 7.2 kN/m (SLS)

$\boxed{\text{Total DL \& SID (factored)} \quad 10.7 \text{ kN/m (ULS)}}$

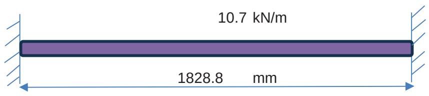
Bending Moment and Shear Force due to ULS DL & SDL

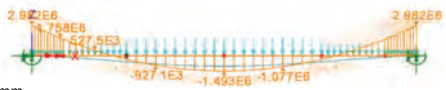
$M_{Sagging}$ = $wL^2/8$ = 4.5 kN.m (Assuming simply supported)

$M_{Hogging}$ = = 3.0 kN.m (From line beam model assuming fixed end supports)

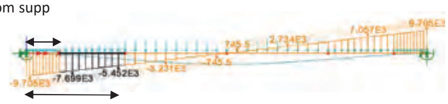
Shear at d = = 7.7 kN (From line beam model assuming fixed end supports)

Shear at 3d = = 5.5 kN (From line beam model assuming fixed end supports)





150mm
d from supp



450mm
3d from supp

JACOBS				**CALCULATION SHEET**			
Office	Manchester	**Page No.**	B 6	**Calc No.**			
Job No. & Title	ECC - Assessment of Station Way Bridge	**Calcs by**	MA	**Date**	Mar-25		
Section	Main Deck (Span 2 to 4) - Carriageway Slab - Live Loading	**Checker**	CAT II	**Date**	Mar-25		

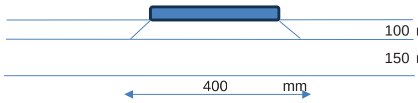
Live Loading (Normal Traffic - 44T)

CS 454 Table B.1

Axle load =	113	kN	(Primary wheel load)
Wheel load =	56.5	kN	Unfactored
Traffic flow factor =	0.9		
Impact factor =	1.8		
Lane factor =	1.0		
γ_{fl} =	1.5		
γ_{f3} =	1.1		
Wheel Load =	151.0	kN	Factored
Wheel Load =	**943.9**	**kN/m2**	Factored (For 0.4m x 0.4m contact area)

Contact area for 44T
300 mm

Wheel load dispersed (2V:1H) to the top of the slab.

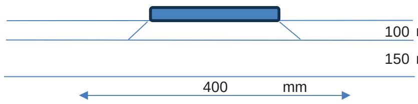


CS 454 Table B.1

Axle load =	74	kN	(Secondary wheel load)
Wheel load =	37	kN	Unfactored
Traffic flow factor =	0.9		
Impact factor =	1.0		Impact factor only applied to primary wheel load
Lane factor =	1.0		
γ_{fl} =	1.5		
γ_{f3} =	1.1		
Wheel Load =	54.9	kN	Factored
Wheel Load =	**343.4**	**kN/m2**	Factored (For 0.4m x 0.4m contact area)

Contact area for 44T
300 mm

Wheel load dispersed (2V:1H) to the top of the slab.



Transverse Sagging Moment due to Normal Traffic

Pucher charts are used to work out the sagging moment by assuming zero moment fixity along the supported edges of deck slab.

CALCULATION SHEET	No:	Rev:
Project: **Station Way Bridge**	By: MA	Date: 03/03/2025
Subject: **Assessment of carriageway slab - Simply supported case - 44T**	Check By: CATII	Date:

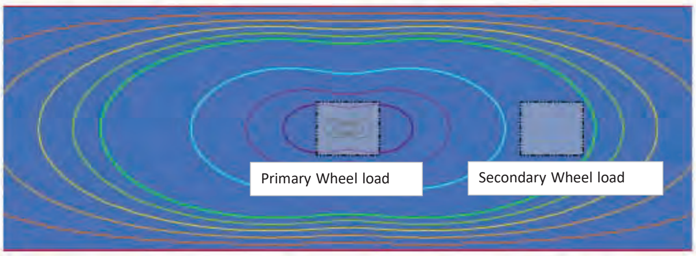
Calculation of Plate Moments According to Pucher

Instructions

Redraw

Chart 1 - mx - Influence surface for the centre of a plate strip with two supported edges

Span = 1.83 m
Contour resolution = 10
Area resolution = 10



Define rectangular patch load areas

Accidental | Redraw | Calculate

Patch	Centre x - m	Centre y - m	Width m	Height m	Pressure KN/m²	Volume (8π.β)	Moment KNm/m
1	0.000	0.000	0.400	0.400	944	0.2490	31.32
2	1.300	0.000	0.400	0.400	343.5	0.0662	3.03
3							
4							
5							
6							
7							
8							
9							
10							
					Total:	0.3152	34.35

Scales: 0.0, 0.2, 0.4, 0.6, 0.8, 1.0, 2.0, 3.0, 4.0, 5.0, 6.0, 7.0, 8.0
14
16
18
20

Note: Positions of loads are defined from the centre of the plate with x-negative being up in the above drawing and x-positive being down. Any areas that are off the edge of the plate are discounted in the calculation.
The volume calculated is the dimensionless volume, β.

JACOBS			**CALCULATION SHEET**			
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Section	Main Deck (Span 2 to 4) - Carriageway Slab - Live Loading	**Checker**	CAT II	**Date**	Mar-25

Transverse Hogging Moment due to Normal Traffic

Pucher charts are used to work out the hogging moment by assuming full moment fixity along the supported edges of deck slab. Sensitivity analysis have been carried out to work out the worst wheel position for the applied load effects for hogging moment. The worst wheels position are shown below,

CALCULATION SHEET | No: | Rev:

Project: **Station Way Bridge** | By: MA | Date: 03/03/2025

Subject: **Assessment of carriageway slab - Fixed end case - 44T** | Check By: CATII | Date:

Calculation of Plate Moments According to Pucher

Instructions

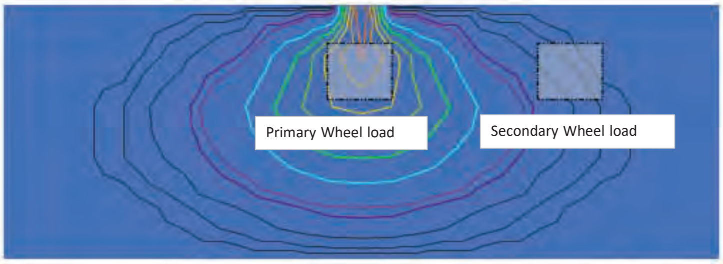
Chart 18. mx - support moment influence surface for the edge of a plate-strip with two restrained edge

Redraw

Span = 1.83 m

Contour resolution = 10

Area resolution = 10



Define rectangular patch load areas

Accidental | Redraw | Calculate

Patch	Centre x - m	Centre y - m	Width m	Height m	Pressure KN/m²	Volume (8π.β)	Moment KNm/m
1	0.000	-0.450	0.400	0.400	944	-0.2798	-35.20
2	1.300	-0.450	0.400	0.400	343.5	-0.0110	-0.51
3							
4							
5							
6							
7							
8							
9							
10							
					Total:	-0.2909	-35.70

Scales: -8.0, -7.0, -6.0, -5.0, -4.0, -3.0, -2.0, -1.0, -0.8, -0.6, -0.4, -0.2, -0.1

14
16
18
20

Note: Positions of loads are defined from the centre of the plate with x-negative being up in the above drawing and x-positive being down. Any areas that are off the edge of the plate are discounted in the calculation.
The volume calculated is the dimensionless volume, β.

Hogging moment with the wheel position at d from support,

CALCULATION SHEET | No: | Rev:

Project: **Station Way Bridge** | By: | Date: 11/03/2025

Subject: **Assessment of carriageway slab - Fixed end case - 44T Charts** | Check By: | Date:

Calculation of Plate Moments According to Pucher

Instructions

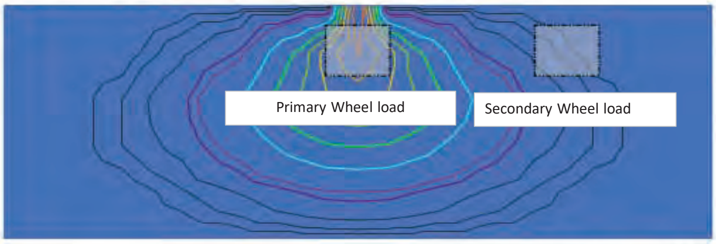
Chart 18. mx - support moment influence surface for the edge of a plate-strip with two restrained edge

Redraw

Span = 1.83 m

Contour resolution = 10

Area resolution = 10



Define rectangular patch load areas

Accidental | Redraw | Calculate

Patch	Centre x - m	Centre y - m	Width m	Height m	Pressure KN/m²	Volume (8π.β)	Moment KNm/m
1	0.000	-0.565	0.400	0.400	944	-0.2747	-34.56
2	1.300	-0.565	0.400	0.400	343.5	-0.0083	-0.38
3							
4							
5							
6							
7							
8							
9							
10							
					Total:	-0.2830	-34.94

Scales: -8.0, -7.0, -6.0, -5.0, -4.0, -3.0, -2.0, -1.0, -0.8, -0.6, -0.4, -0.2, -0.1

14
16
18
20

Note: Positions of loads are defined from the centre of the plate with x-negative being up in the above drawing and x-positive being down. Any areas that are off the edge of the plate are discounted in the calculation.
The volume calculated is the dimensionless volume, β.

Hogging moment with the wheel position at 3d from support,

CALCULATION SHEET

Project: **Station Way Bridge**

Subject: **Assessment of carriageway slab - Fixed end case - 44T Charts**

No: Rev: By: Date: 11/03/2025 Check By: Date:

Calculation of Plate Moments According to Pucher

Instructions

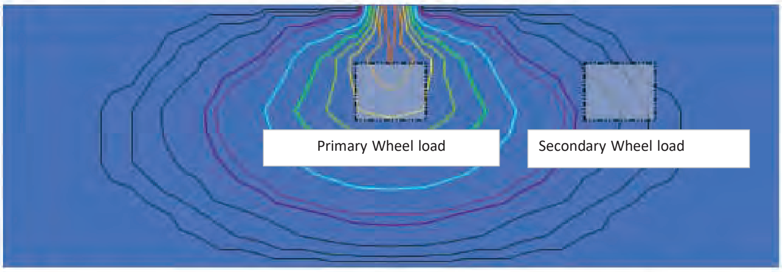
Redraw

hart 16 mx - support moment influence surface for the edge of a plate-strip with two restrained edge

Span = 1.83 m

Contour resolution = 10

Area resolution = 10



Define rectangular patch load areas

Accidental Redraw Calculate

Patch	Centre		Width	Height	Pressure	Volume	Moment
	x - m	y - m	m	m	KN/m²	($8\pi.\beta$)	KNm/m
1	0.000	-0.315	0.400	0.400	944	-0.2645	-33.27
2	1.300	-0.315	0.400	0.400	343.5	-0.0148	-0.68
3							
4							
5							
6							
7							
8							
9							
10							
					Total:	-0.2793	-33.95

Scales: -8.0, -7.0, -6.0, -5.0, -4.0, -3.0, -2.0, -1.0, -0.8, -0.6, -0.4, -0.2, -0.1

14
16
18
20

Note: Positions of loads are defined from the centre of the plate with x-negative being up in the above drawing and x-positive being down. Any areas that are off the edge of the plate are discounted in the calculation.

The volume calculated is the dimensionless volume, β.

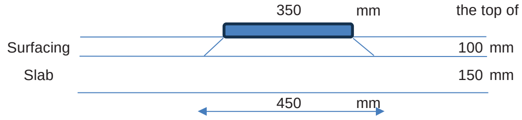
Live Loading (SV80)

CS 458 Cl. 3.8

Axle load =	130	kN	(Primary wheel load)
Wheel load =	65	kN Unfactored	
Overload factor =	1.2		
DAF =	1.16		
γ_{fl} =	1.1		
γ_{f3} =	1.1		
Wheel Load =	109.2	kN Factored	
Wheel Load =	539.3	kN/m2	Factored - (For 0.45m x 0.45m contact area)

Contact area for SV80

Wheel load dispersed (2V:1H) to the top of the slab.



CS 458 Cl. 3.8

Axle load =	130	kN	(Secondary wheel load)
Wheel load =	65	kN Unfactored	
Overload factor =	1.1		
DAF =	1.16		
γ_{fl} =	1.1		
γ_{f3} =	1.1		
Wheel Load =	100.1	kN Factored	
Wheel Load =	494.3	kN/m2	Factored - (For 0.45m x 0.45m contact area)

Transverse Sagging moment due to SV80

Pucher charts are used to work out the sagging moment by assuming zero moment fixity along the supported edges of deck slab.

CALCULATION SHEET | No: | Rev:

Project: **Station Way Bridge** | By: | Date: 03/03/2025

Subject: **Assessment of carriageway - Simply Supp case - SV80** | Check By: | Date:

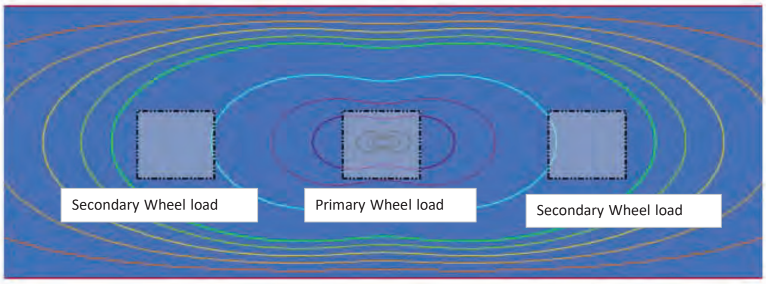
Calculation of Plate Moments According to Pucher

Instructions | Redraw

Chart 1 - mx - Influence surface for the centre of a plate strip with two supported edges

Span = 1.83 m

Contour resolution = 10 | Area resolution = 10



Define rectangular patch load areas

Accidental | Redraw | Calculate

Patch	Centre x - m	Centre y - m	Width m	Height m	Pressure KN/m^2	Volume (8π.β)	Moment KNm/m
1	-1.200	0.000	0.450	0.450	494.3	0.0942	6.21
2	0.000	0.000	0.450	0.450	539.3	0.3010	21.63
3	1.200	0.000	0.450	0.450	494.3	0.0942	6.21
4							
5							
6							
7							
8							
9							
10							
					Total:	0.4895	34.05

Scales: 0.0, 0.2, 0.4, 0.6, 0.8, 1.0, 2.0, 3.0, 4.0, 5.0, 6.0, 7.0, 8.0

14 16 18 20

Note: Positions of loads are defined from the centre of the plate with x-negative being up in the above drawing and x-positive being down. Any areas that are off the edge of the plate are discounted in the calculation.
The volume calculated is the dimensionless volume, β.

Transverse Hogging Moment due to SV80

Pucher charts have been used to work out the hogging moment by assuming full moment fixity along the supported edges of deck slab. Sensitivity analysis have been carried out to work out the worst wheel position for the applied load effects for hogging moment. The worst wheels position are shown below,

CALCULATION SHEET — No: — Rev:

Project: Station Way Bridge — By: MA — Date: 03/03/2025

Subject: Assessment of carriageway - Fixed end case - SV80 — Check By: CATII — Date:

Calculation of Plate Moments According to Pucher

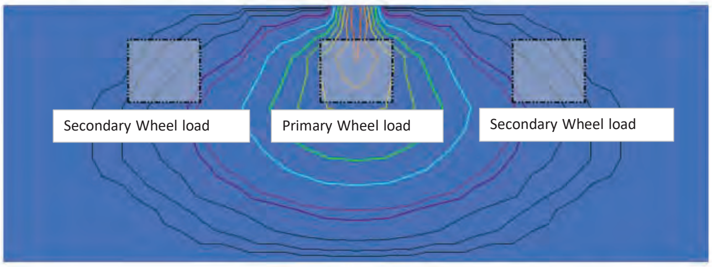
Instructions

Chart 16. mx - support moment influence surface for the edge of a plate-strip with two restrained edge

Redraw

Span = 1.83 m

Contour resolution = 10 — Area resolution = 10



Define rectangular patch load areas — Accidental — Redraw — Calculate

Patch	Centre x - m	Centre y - m	Width m	Height m	Pressure KN/m²	Volume (8π.β)	Moment KNm/m
1	-1.200	-0.450	0.450	0.450	494.3	-0.0203	-1.34
2	0.000	-0.450	0.450	0.450	539.3	-0.3445	-24.76
3	1.200	-0.450	0.450	0.450	494.3	-0.0203	-1.34
4							
5							
6							
7							
8							
9							
10							
					Total:	-0.3851	-27.43

Scales: -8.0, -7.0, -6.0, -5.0, -4.0, -3.0, -2.0, -1.0, -0.8, -0.6, -0.4, -0.2, -0.1

14
16
18
20

Note: Positions of loads are defined from the centre of the plate with x-negative being up in the above drawing and x-positive being down. Any areas that are off the edge of the plate are discounted in the calculation.
The volume calculated is the dimensionless volume, β.

Hogging moment with the wheel position at d from support,

CALCULATION SHEET — No: — Rev:

Project: Station Way Bridge — By: — Date: 11/03/2025

Subject: Assessment of carriageway slab - Fixed end case - SV80 — Check By: — Date:

Calculation of Plate Moments According to Pucher

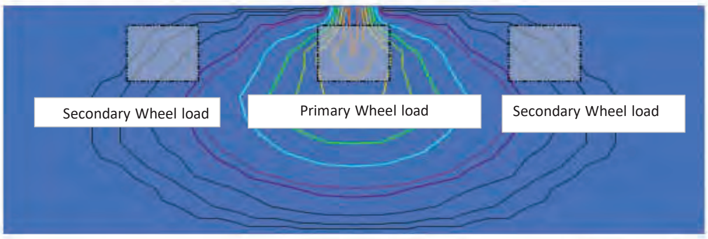
Instructions

Chart 16. mx - support moment influence surface for the edge of a plate-strip with two restrained edge

Redraw

Span = 1.83 m

Contour resolution = 10 — Area resolution = 10



Define rectangular patch load areas — Accidental — Redraw — Calculate

Patch	Centre x - m	Centre y - m	Width m	Height m	Pressure KN/m²	Volume (8π.β)	Moment KNm/m
1	-1.200	-0.540	0.450	0.450	494.3	-0.0162	-1.07
2	0.000	-0.540	0.450	0.450	539.3	-0.3381	-24.29
3	1.200	-0.540	0.450	0.450	494.3	-0.0162	-1.07
4							
5							
6							
7							
8							
9							
10							
					Total:	-0.3704	-26.43

Scales: -8.0, -7.0, -6.0, -5.0, -4.0, -3.0, -2.0, -1.0, -0.8, -0.6, -0.4, -0.2, -0.1

14
16
18
20

Note: Positions of loads are defined from the centre of the plate with x-negative being up in the above drawing and x-positive being down. Any areas that are off the edge of the plate are discounted in the calculation.
The volume calculated is the dimensionless volume, β.

Hogging moment with the wheel position at 3d from support,

CALCULATION SHEET

No: Rev:

Project: **Station Way Bridge** By: Date: 11/03/2025

Subject: **Assessment of carriageway slab - Fixed end case - SV80** Check By: Date:

Calculation of Plate Moments According to Pucher

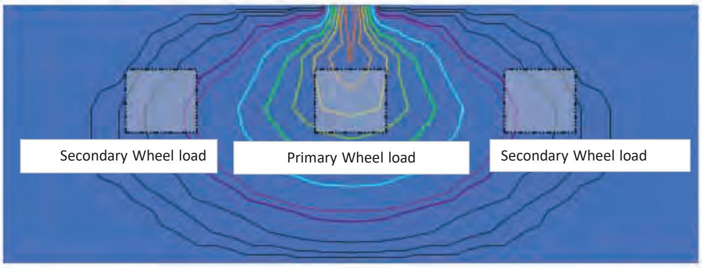
Instructions

hart 16. mx - support moment influence surface for the edge of a plate-strip with two restrained edge

Redraw

Span = 1.83 m

Contour resolution = 10 Area resolution = 10



Define rectangular patch load areas

Accidental Redraw Calculate

Patch	Centre x - m	Centre y - m	Width m	Height m	Pressure KN/m²	Volume (8π.β)	Moment KNm/m
1	-1.200	-0.240	0.450	0.450	494.3	-0.0291	-1.92
2	0.000	-0.240	0.450	0.450	539.3	-0.3127	-22.47
3	1.200	-0.240	0.450	0.450	494.3	-0.0291	-1.92
4							
5							
6							
7							
8							
9							
10							
					Total:	-0.3709	-26.30

Scales: -8.0, -7.0, -6.0, -5.0, -4.0, -3.0, -2.0, -1.0, -0.8, -0.6, -0.4, -0.2, -0.1

14
16
18
20

Note: Positions of loads are defined from the centre of the plate with x-negative being up in the above drawing and x-positive being down. Any areas that are off the edge of the plate are discounted in the calculation.
The volume calculated is the dimensionless volume, β.

SHEAR RESISTANCE OF THE CARRIAGEWAY SLAB

Bar Type	Bar Dia (")	Bar Dia (mm)	Area of Bar (mm^2)	Spacing (mm)	No. Bars	Area of Reinforcement (mm^2)
Transverse Bottom Bar	9/16	14.3	160.3	79.4	12.6	2019.8
Transverse Top Bar	9/16	14.3	160.3	79.4	12.6	2019.8

Maximum shear Resistance based on concrete crushing

Equation 5.6a Maximum shear resistance based on concrete crushing

V_{max} = $$V_{max} = 0.36\left(0.7 - \frac{f_{cu}}{250}\right)\left(\frac{f_{cu}}{\gamma_{mc}}\right)b_w d$$

$$V_{max} = 0.36 \times 0.7 - \frac{15.00}{250} \times \frac{15.00}{1.5} \times 1000 \times 158 = 363.9 \text{ kN}$$

(Per metre width)

Shear Resistance more than 3d from a support

Equation 6.5 Shear resistance of concrete slabs more than 3d from a support

V_{uc} = $$V_{uc} = \frac{0.27}{\gamma_{mv}}\xi_s \rho_s^{\frac{1}{3}} f_{cu}^{\frac{1}{3}} b_w d$$

$$\xi s = \left(\frac{500}{158}\right)^{0.25} = 1.33$$

$$Ps = \frac{100 \times 2019.85}{1000 \times 158} = 1.28$$

$$V_{uc} = \frac{0.27}{1.25} \times 1.33 \times 1.28^{1/3} \times 15.00^{1/3} \times 1000 \times 158$$

V_{uc} = 121.8 kN (per metre width @ 3d from support)

Shear Resistance within 3d of a support

Equation 5.6c Shear resistance within 3d of a support

$$V_u = \max\begin{cases} \frac{3d}{a_v}\Gamma V_{uc} \\ \frac{0.24}{\gamma_{mv}}\xi_s(0.15 f_{cu})^{\frac{1}{3}} b_w d \end{cases}$$

where:

- a_v is the distance of the section measured from the edge of a rigid bearing, the centre-line of a flexible bearing or the face of a support, where $d \le a_v \le 3d$
- Γ is the factor to account for short anchorage lengths, defined in Equation 5.6d

Equation 5.6d Factor to account for the effect of short anchorage lengths

$$\Gamma = \min\begin{cases} \sqrt{\frac{z}{3d}\frac{F_{ab}}{V_{uc}}} \\ 1.0 \end{cases}$$

where:

- F_{ab} is the total anchorage force that can be developed in the longitudinal tension reinforcing bars at the front face of the support according to Section 9, but not greater than $\frac{A_s f_y}{\gamma_{ms}}$
- z is the flexural lever arm at ULS at a position $3d$ from the support calculated from Equation 5.2.2b

Equation 9.1a Anchorage resistance

$$F_{ab} = f_{ab} p L_a$$

where:

- f_{ab} is the average anchorage bond strength over the effective anchorage length, given by Equation 9.1b;
- p is the effective perimeter, taken as
  $p = \pi\phi$ for a single bar, or
  $p = (1.2 - 0.2N)\sum(\pi\phi)$ for a bundled group of N bars, valid up to $N = 4$.
- ϕ is the nominal bar diameter
- L_a is the effective anchorage length at the position where the resistance is being determined.

Equation 9.1b Average anchorage bond strength

$$f_{ub} = \frac{k k_{cov} \beta \sqrt{f_{cu}}}{\gamma_{mb}}$$

	k	=	1.0		γ_{mb}	=	1.4	
CS 455, Tbl 9.1	B	=	0.39	(Plain bars in tension)	ϕ	=	14.3	mm
	f_{cu}	=	15.0	N/mm^2	K_{cov}	=	1.0	(taken as 1)

f_{ub} = 1.08

F_{ub} = $F_{ub} = f_{ub} p L_a$

P = 565.5 mm/m

L_a = 158.0 mm (conservative taken as d)

F_{ub} = 96.4 kN/m

Equation 5.6d Factor to account for the effect of short anchorage lengths

$$\Gamma = \min \begin{cases} \sqrt{\frac{z}{3d}\frac{F_{ub}}{V_{uc}}} \\ 1.0 \end{cases}$$

where:

F_{ub} is the total anchorage force that can be developed in the longitudinal tension reinforcing bars at the front face of the support according to Section 9, but not greater than $\frac{A_s f_y}{\gamma_{ms}}$

z is the flexural lever arm at ULS at a position $3d$ from the support calculated from Equation 5.2.2b

pg ref. B 3 d = 158 mm

Z = 124.0

$\Gamma = \left(\frac{124.0}{473.9} \times \frac{96.4}{121.8} \right)^{1/2}$

$\Gamma = (0.26 \times 0.79)^{1/2} = (0.207)^{1/2} = 0.455$

Vu = $\frac{3d}{a_v}\Gamma V_{uc}$ = $\frac{3 \times 158}{158} \times 0.455 \times 121.8$ = 166.3 kN/m at av = d

Vu = $\frac{0.24}{\gamma_{mv}}\xi_s(0.15 f_{cu})^{\frac{1}{3}} b_w d$ = $\frac{0.24}{1.25} \times 1.33 \times (0.15 \times 15.0)^{1/3} \times 1000 \times 158$ = 53.0 kN/m

$\boxed{\text{Vu} = 166.3 \text{ kN}}$ (per metre width at d from support)

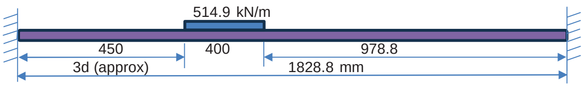
JACOBS **CALCULATION SHEET**

Office	Manchester	**Page No.**	B 14	**Calc No.**	
Job No. & Title	ECC - Assessment of Station Way Bridge	**Calcs by**	MA	**Date**	Mar-25
Section	Main Deck (Span 2 to 4) - Carriageway slab - Shear	**Checker**	CAT II	**Date**	Mar-25

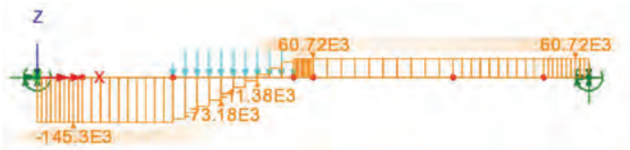
Shear - Applied Load - Normal Traffic

Live load effects using a line beam analysis @ 3d from support

Primary wheel load + Secondary wheel load: $\frac{151.0 + 54.9}{0.4}$ = 514.9 kN/m = Total ULS load due to both wheels



Shear force diagarm for above loading



Shear force at end and at 3d = 145.0 kN

Bending moment diagram for above loading

Bending moment at end = 53.8 kN.m

pg ref. B 8 Effective width of slab for hogging bending moment implied by pucher chart = $\frac{\text{Total bending moment given by beam model (kN.m)}}{\text{Bending per unit width given by Pucher chart for similar wheel arrangement (kN.m/m)}}$ = $\frac{53.8 \text{ kN.m}}{34.0 \text{ kN.m}}$ = 1.5 m

Shear Force = (per metre width) = $\frac{145.3}{1.5}$

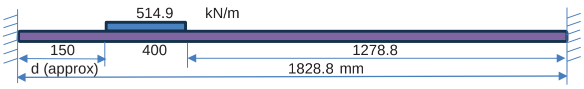
Shear Force $_{ULS}$ @ 3d	=	96.4	kN

(per metre width) - Normal Traffic - 44T

For this load arrangement, effective width for hogging bending moment at support has been applied to shear force at 3d. This is considered to be a reasonable approach.

Live load effects using a line beam analysis @ d from support

Primary wheel load + Secondary wheel load: $\frac{151.0 + 54.9}{0.4}$ = 514.9 kN/m = Total ULS load due to both wheels



Shear force diagarm for above loading

Shear force at end and at d = 184.0 kN

Bending moment diagram for above loading

Bending moment at end = 44.5 kN.m

pg ref. B 7 Effective width of slab for hogging bending moment implied by pucher chart = $\frac{\text{Total bending moment given by beam model (kN.m)}}{\text{Bending per unit width given by Pucher chart for similar wheel arrangement (kN.m/m)}}$ = $\frac{44.5 \text{ kN.m}}{35.0 \text{ kN.m}}$ = 1.3 m

Shear Force = (per metre width) = $\frac{184.0}{1.3}$

Shear Force $_{ULS}$ @ d	=	144.7	kN

(per metre width) - Normal Traffic - 44T

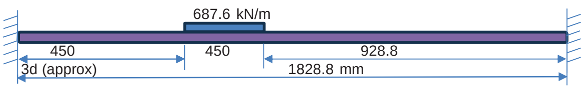
For this load arrangement, effective width for hogging bending moment at support has been applied to shear force at d. This is considered to be a reasonable approach.

JACOBS			CALCULATION SHEET			
Office	Manchester	**Page No.**	B 15	**Calc No.**	
Job No. & Title	ECC - Assessment of Station Way Bridge	**Calcs by**	MA	**Date**	Mar-25
Section	Main Deck (Span 2 to 4) - Carriageway slab - Shear	**Checker**	CAT II	**Date**	Mar-25

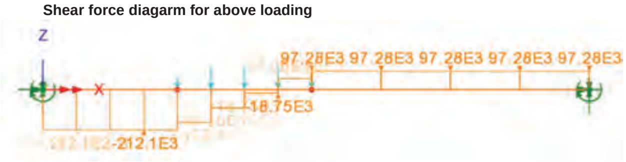
<u>Shear - Applied Loading - SV80</u>

Live load effects using a line beam analysis @ 3d from support

Primary wheel load		Secondary wheel load x 2				
109.2	+	200.2	=	687.6 kN/m		= Total ULS load due to both wheels
	0.45					

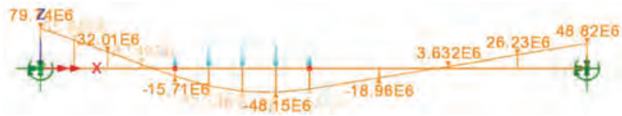


Shear force diagarm for above loading



Shear force at end and at 3d = 212.0 kN

Bending moment diagram for above loading



Bending moment at end = 79.7 kN.m

Effective width of slab for hogging bending moment implied by pucher chart = Total bending moment given by beam model (kN.m) / Bending per unit width given by Pucher chart for similar wheel arrangement (kN.m/m) = 79.7 kN.m / 26.3 kN.m = 3.0 m

pg ref. B 11

Shear Force = (per metre width) = 212 / 3.0

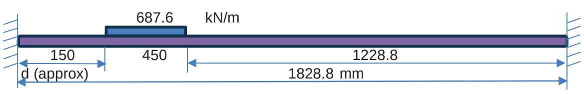
Shear Force $_{ULS}$ @ 3d	=	70.0	kN

(per metre width) - SV80

For this load arrangement, effective width for hogging bending moment at support has been applied to shear force at 3d. This is considered to be a reasonable approach.

Live load effects using a line beam analysis @ d from support

Primary wheel load		Secondary wheel load		
109.2	+	200.2	=	687.6 kN/m
	0.45			



Shear force diagarm for above loading

Shear force at end and at d = 271.0 kN

Bending moment diagram for above loading

Bending moment at end = 68.0 kN.m

Effective width of slab for hogging bending moment implied by pucher chart = Total bending moment given by beam model (kN.m) / Bending per unit width given by Pucher chart for similar wheel arrangement (kN.m/m) = 68.0 kN.m / 26.4 kN.m = 2.6 m

pg ref. B 10

Shear Force = (per metre width) = 271 / 2.6

Shear Force $_{ULS}$ @ d	=	105.3	kN

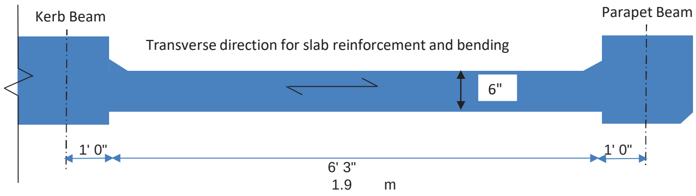
(per metre width) - SV80

For this load arrangement, effective width for hogging bending moment at support has been applied to shear force at d. This is considered to be a reasonable approach.

<u>CALCULATIONS FOR ASSESSMENT OF SERVICE BAY SLAB (SPAN 2 – 4)</u>

<u>SLAB ARRANGEMENT</u>

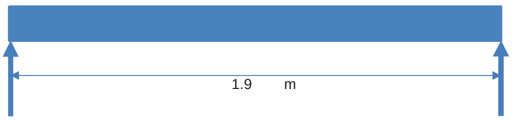
Record drgs: LC5/2 LC5/6



<u>Idealised diagram</u>

As shown on sheet C3, the top transverse reinforcement has half of the area of the bottom transverse reinforcement. Therefore, it appears that the original design intent was for the slab to be a simply supported element and it will be assessed on the same basis.

For Sagging: Free ends (simply supported)



Material Properties and Partial Factors

Reinforced Concrete

Ref	Description	Symbol		Value	Unit
AIP, Cl 3.10	Characteristic Strength of Concrete		=	15	N/mm^2
CS 454 table 4.1.1a	Density of Reinforced Concrete		=	24	kN/m^3

Steel Reinforcement

Ref	Description	Symbol		Value	Unit
AIP, Cl 3.10	Mild steel reinforcement characteristic yield strength		=	230	N/mm^2
	Design Young's Modulus of steel reinforcement		=	200000	N/mm^2

Fill - Miscellaneous

Ref	Description	Symbol		Value	Unit
CS 454 Table 4.1.1a	Unit weight of miscellaneous fill		=	22	kN/m^3

Carriageway & Footway Surfacing

Ref	Description	Symbol		Value	Unit
CS 454 Table 4.1.1a	Unit weight of Bituminous Macadam (tar)		=	24	kN/m^3

Partial Factors

Ref	Description	Symbol		Value
CS 455 Table 2.13a	Partial Factor for reinforcement	γ_{ms}	=	1.15
CS 455 Table 2.13a	Partial Factor for Concrete	γ_{mc}	=	1.5
CS 455 Table 2.13a	Partial Factor for shear in concrete	γ_{mv}	=	1.25
CS 455 Table 2.13a	Partial Factor for Bond	γ_{mb}	=	1.4
CS 454, Tbl 3.4	Partial Factor for Concrete Deadload		=	1.15
CS 454, Tbl 3.4	Partial Factor for Surfacing		=	1.75
CS 454, Tbl 3.4	Partial Factor for Fill		=	1.2
CS 454 Cl 3.9	Inaccurate assessment effects at ULS	γ_{f3}	=	1.1
CS 454, Tbl 3.4	Partial Factor for accidental vehicle loading and footway loading	γ_{fL}	=	1.5

RESULTS SUMMARY

AVL = Accidental vehicle loading (table B.1 of CS 454)

PLL = Pedestrian live loading (5 kN/m^2)

US Bending (Sagging)

Live Load Type	Bending Capacity (kN.m/m)	BM due to DL+SDL (kN.m/m)	BM due to LL (kN.m/m)	Total BM (kN.m/m)	Pass / Fail	Utilisation (%)
7.5T AVL	17.0	12.1	11.0	23.1	FAIL	136
3T AVL	17.0	12.1	4.0	16.1	PASS	95
PLL	17.0	12.1	3.7	15.8	PASS	93

ULS Shear At d

Live Load Type	Shear Capacity (kN/m)	SF due to DL+SDL (kN/m)	SF due to LL (kN/m)	Total SF (kN/m)	Pass / Fail	Utilisation (%)
44T, 26T & 18T AVL	95.9	22.5	102.6	125.0	FAIL	130
7.5T AVL	95.9	22.5	53.7	76.1	PASS	79
3T AVL	95.9	22.5	19.0	41.5	PASS	43
PLL	95.9	22.5	7.0	29.4	PASS	31

ULS Shear At 3d

Live Load Type	Shear Capacity (kN/m)	SF due to DL+SDL (kN/m)	SF due to LL (kN/m)	Total SF (kN/m)	Pass / Fail	Utilisation (%)
44T, 26T & 18T AVL	79.8	16.6	85.0	101.7	FAIL	127
7.5T AVL	79.8	16.6	44.5	61.1	PASS	77
3T AVL	79.8	16.6	15.8	32.4	PASS	41
PLL	79.8	16.6	5.1	21.8	PASS	27

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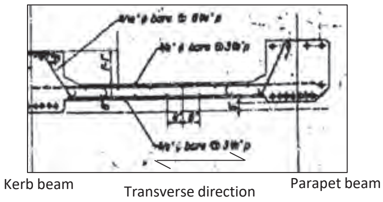
Section	Main Deck (Span 2 to 4) - Service Bay Slab - Bending Capacity	**Checker**	CAT II		**Date**	Mar-25

BENDING CAPACITY - SERVICE BAY TRANSVERSE SLAB



Section of carriageway slab from archive Drg LC 5/6

DRG LC 5/6

Reinforcement

Bar Type	Bar Dia (")	Bar Dia (mm)	Area of Bar (mm^2)	Spacing Inches	Spacing mm	No. Bars (per m width)	Area of Reinforcement (mm^2/m)
Transverse Bottom Bar	3/8	9.53	71.3	3 1/8"	79.4	12.6	897.7
Longitudinal Bottom Bar	3/8	9.53	71.3	12"	304.8	3.3	233.8
Transverse Top Bar	3/8	9.53	71.3	6 1/4"	158.8	6.3	448.9
Longitudinal Top Bar	3/8	9.53	71.3	12"	304.8	3.3	233.8

Moment Resistance for beams without compression reinforcement, Eq 5.2.2a

M_u = Minimum value dervied from the following equations $M_u = \min\left\{ \frac{f_y}{\gamma_m} A_s z,\ \frac{0.225 f_{cu}}{\gamma_m} b d^2 \right.$

ULS Sagging (Transverse)

A_s = 897.7 mm2/m (3/8" at 3 1/8")

Cover to Reinforcement = 1 1/2" = 38.1 mm

Slab Depth = 6" = 152.4 mm

Effective Depth = 152.4 - 38.1 - 4.76 = 109.5 mm

z = 94.5 mm

$M_u = \frac{230}{1.15} \times 897.7 \times 94.5 = 17.0$ kNm/m

$M_u = \frac{0.225 \times 15.0}{1.5} \times 1000 \times 110^2 = 27.0$ kNm/m

$\boxed{M_u = 17.0 \text{ kN.m/m}}$

APPLIED LOADING (FOR 1.0M WIDTH OF SLAB)

DL & SDL

	Depth (m)		Per metre width		Density		γ_{fl}		γ_{f3}				
Self-weight	0.152	x	1.0	x	24	x	1.0	x	1.0	=	3.7	kN/m	SLS
						x	1.15	x	1.1	=	4.6	kN/m	ULS
Surfacing	0.1	x	1.0	x	24	x	1.0	x	1.0	=	2.4	kN/m	SLS
						x	1.75	x	1.1	=	4.6	kN/m	ULS
Fill	0.6	x	1.0	x	22	x	1.0	x	1.0	=	13.2	kN/m	SLS
						x	1.2	x	1.1	=	17.4	kN/m	ULS
Total DL & SDL (Unfactored)											19.3	kN/m	SLS
Total DL & SDL (factored)											26.7	kN/m	ULS

Bending Moment and Shear Force due to DL & SDL (ULS)

Max $M_{Sagging}$ = $wL^2/8$ = 12.1 kN.m/m (Assuming simply supported)

Shear at support = 25.4 kN/m

Shear at d = = 22.5 kN/m

Shear at 3d = = 16.6 kN/m

Pg ref. C 3 — d = 110 mm

Bending Moment and Shear Force due to pedestrian loading (ULS)

Unfactored Load = 5.0 kN/m2

ULS UDL = 5.0 x 1.5 x 1.1 kN/m

= 8.25 kN/m (for 1.0m width of slab)

Max $M_{Sagging}$ = $wL^2/8$ = 3.7 kN.m/m (Assuming simply supported)

Shear at support = 7.9 kN/m

Shear at d = = 7.0 kN/m

Shear at 3d = = 5.1 kN/m

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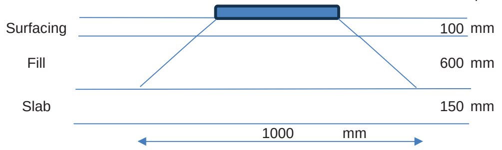
Section	Main Deck (Span 2 to 4) - Service Bay Slab - AVL	**Checker**	CAT II		**Date**	Mar-25	

Accidental Vehicle loading (44T & 26T)

CS 454 Table B.1	Axle load =	113	kN	(Primary load)
	Wheel load =	56.5	kN	Unfactored
AIP, Cl 4.13	Traffic flow factor =	0.9		
	Impact factor =	1.8		
	Lane factor =	1.0		
	γ_{fl} =	1.5		
	γ_{f3} =	1.1		
	Wheel Load =	151.0	kN	Factored (Primary wheel load)
	Wheel Load =	151.0	kN/m^2	Factored - (For 1000m x 1000m contact area)



CS 454 Table B.1	Axle load =	74	kN	(Secondary load)
	Wheel load =	37	kN	Unfactored
AIP, Cl 4.13	Traffic flow factor =	0.9		
	Impact factor =	1.0		Impact factor only applied to primary wheel load
	Lane factor =	1.0		
	γ_{fl} =	1.5		
	γ_{f3} =	1.1		
	Wheel Load =	54.9	kN	Factored (Secondary wheel load)
	Wheel Load =	54.9	kN/m^2	Factored - (For 1000m x 1000m contact area)

Axle spacing = 1.30 m

Transverse Sagging Moment due 44T & 26T AVL

Pucher charts are used to work out the sagging moment by assuming zero moment fixity along the supported edges of deck slab.

CALCULATION SHEET No: Rev:

Project: **Station Way Bridge** By: Date: 14/03/2026

Subject: **Service bay slab - Simply supp - 44T** Check By: Date:

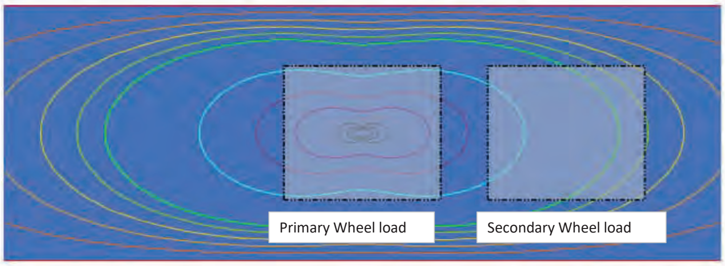
Calculation of Plate Moments According to Pucher Instructions

Chart 1 - mx - Influence surface for the centre of a plate strip with two supported edges

Redraw

Span = 1.9 m

Contour resolution = 10 Area resolution = 10



Define rectangular patch load areas Accidental Redraw Calculate

Patch	Centre x - m	Centre y - m	Width m	Height m	Pressure KN/m^2	Volume ($8\pi.\beta$)	Moment KNm/m
1	0.000	0.000	1.000	1.000	151	0.9472	20.54
2	1.300	0.000	1.000	1.000	54.9	0.3797	2.99
3							
4							
5							
6							
7							
8							
9							
10							
					Total:	1.3269	23.54

Scales: 0.0, 0.2, 0.4, 0.6, 0.8, 1.0, 2.0, 3.0, 4.0, 5.0, 6.0, 7.0, 8.0, 14, 16, 18, 20

Note: Positions of loads are defined from the centre of the plate with x-negative being up in the above drawing and x-positive being down. Any areas that are off the edge of the plate are discounted in the calculation.
The volume calculated is the dimensionless volume, β.

Max ULS Bending moment (44T & 26T)
= 23.5 kN.m/m

JACOBS				**CALCULATION SHEET**		
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Section	Main Deck (Span 2 to 4) - Service Bay Slab - AVL	**Checker**	CAT II		**Date**	Mar-25

Accidental Vehicle Loading - (Restricted Traffic - 18T)

CS 454 Table B.1 — Axle load = 113 kN (Primary load)

Wheel load = 56.5 kN Unfactored

AIP, Cl 4.13 — Traffic flow factor = 0.9

Impact factor = 1.8

Lane factor = 1.0

γ_{fl} = 1.5

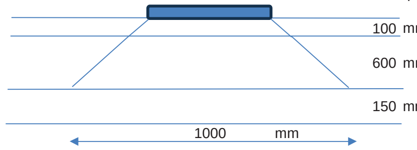
γ_{f3} = 1.1

Wheel Load = 151.0 kN Factored (Primary wheel load)

Wheel Load = 151.0 kN/m^2 Factored - (For 1000m x 1000m contact area)

Contact area for wheel load 300 mm

Wheel load dispersed (2V:1H) to the top of the slab.



Axle spacing = 3.00 m

Transverse Sagging Moment due 18T AVL

Pucher charts are used to work out the sagging moment by assuming zero moment fixity along the supported edges of deck slab.
Note: Secondary wheel load has been ignored as it has negligible effect on bending moment (axle spacing = 3.0 m)

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Project: **Station Way Bridge**

Subject: **Service bay slab - Simply supp - 18T**

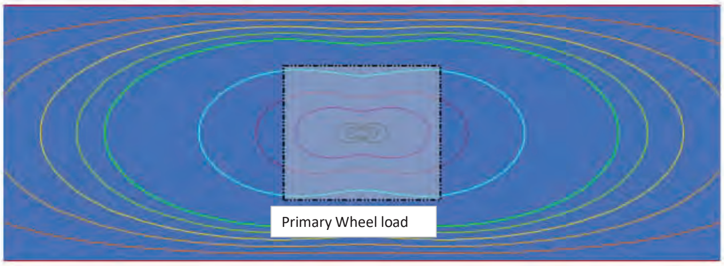
Calculation of Plate Moments According to Pucher

Instructions — Redraw

Chart 1 - mx - Influence surface for the centre of a plate strip with two supported edges

Span = 1.9 m

Contour resolution = 10 — Area resolution = 10



Define rectangular patch load areas — Accidental — Redraw — Calculate

Patch	Centre x - m	Centre y - m	Width m	Height m	Pressure KN/m^2	Volume (8π.β)	Moment KNm/m
1	0.000	0.000	1.000	1.000	151	0.9472	20.54
2							
3							
4							
5							
6							
7							
8							
9							
10							
					Total:	0.9472	20.54

Scales: 0.0, 0.2, 0.4, 0.6, 0.8, 1.0, 2.0, 3.0, 4.0, 5.0, 6.0, 7.0, 8.0, 14, 16, 18, 20

Note: Positions of loads are defined from the centre of the plate with x-negative being up in the above drawing and x-positive being down. Any areas that are off the edge of the plate are discounted in the calculation.
The volume calculated is the dimensionless volume, β.

Max ULS bending moment (18T AVL) = 20.5 kN.m/m

JACOBS | **CALCULATION SHEET**

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Section	Main Deck (Span 2 to 4) - Service Bay Slab - AVL	**Checker**	CAT II		**Date**	Mar-25

Accidental Vehicle Loading (Restricted Traffic - 7.5T)

CS 454 Table B.1

Axle load = 59 kN (Primary load)

Wheel load = 29.5 kN Unfactored

AIP, Cl 4.13

Traffic flow factor = 0.9

Impact factor = 1.8

Lane factor = 1.0

γ_{fl} = 1.5

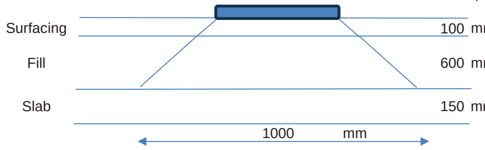
γ_{f3} = 1.1

Wheel Load = 78.9 kN Factored (Primary wheel load)

Wheel Load = 78.9 kN/m^2	Factored - (For 1000m x 1000m contact area)

Contact area for wheel load 300 mm

Wheel load dispersed (2V:1H) to the top of the slab.



CS 454 Table B.1

Axle load = 15 kN (Secondary load)

Wheel load = 7.5 kN Unfactored

AIP, Cl 4.13

Traffic flow factor = 0.9

Impact factor = 1.0 Impact factor only applied to primary wheel load

Lane factor = 1.0

γ_{fl} = 1.5

γ_{f3} = 1.1

Wheel Load = 11.1 kN Factored (Secondary wheel load)

Wheel Load = 11.1 kN/m^2	Factored - (For 1000m x 1000m contact area)

Axle spacing = 2.0 m

Transverse Sagging Moment due to 7.5T AVL

Pucher charts are used to work out the sagging moment by assuming zero moment fixity along the supported edges of deck slab.

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Calculation of Plate Moments According to Pucher

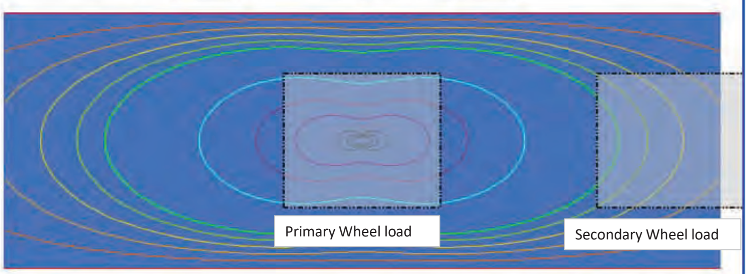
Instructions

Chart 1 - mx - Influence surface for the centre of a plate strip with two supported edges

Redraw

Span = 1.9 m

Contour resolution = 10 Area resolution = 10



Define rectangular patch load areas Accidental Redraw Calculate

Patch	Centre x - m	Centre y - m	Width m	Height m	Pressure KN/m^2	Volume $(8\pi\beta)$	Moment KNm/m
1	0.000	0.000	1.000	1.000	78.9	0.9472	10.73
2	2.000	0.000	1.000	1.000	11.1	0.1705	0.27
3							
4							
5							
6							
7							
8							
9							
10							
					Total:	1.1177	11.01

Scales: 0.0, 0.2, 0.4, 0.6, 0.8, 1.0, 2.0, 3.0, 4.0, 5.0, 6.0, 7.0, 8.0, 14, 16, 18, 20

Note: Positions of loads are defined from the centre of the plate with x-negative being up in the above drawing and x-positive being down. Any areas that are off the edge of the plate are discounted in the calculation.
The volume calculated is the dimensionless volume, β.

Max ULS bending moment (7.5T AVL) = 11.0 kN.m/m

JACOBS — CALCULATION SHEET

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Accidental Vehicle Loading (Restricted Traffic - 3T)

CS 454 Table B.1
Axle load = 21 kN (Primary load)

Wheel load = 10.5 kN Unfactored

AIP, Cl 4.13
Traffic flow factor = 0.9

Impact factor = 1.8

Lane factor = 1.0

γ_{fl} = 1.5

γ_{f3} = 1.1

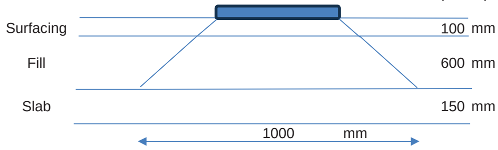
Wheel Load = 28.1 kN Factored (Primary wheel load)

Wheel Load =	28.1	kN/m^2

Factored - (For 1000m x 1000m contact area)

Contact area for wheel load
300 mm

Wheel load dispersed (2V:1H) to the top of the



CS 454 Table B.1
Axle load = 9 kN (Secondary load)

Wheel load = 4.5 kN Unfactored

AIP, Cl 4.13
Traffic flow factor = 0.9

Impact factor = 1.0 Impact factor only applied to primary wheel load

Lane factor = 1.0

γ_{fl} = 1.5

γ_{f3} = 1.1

Wheel Load = 6.7 kN Factored (Secondary wheel load)

Wheel Load =	6.7	kN/m^2

Factored - (For 1000m x 1000m contact area)

Axle spacing =	2.0 m

Transverse Sagging Moment due to 3T AVL

Pucher charts are used to work out the sagging moment by assuming zero moment fixity along the supported edges of deck slab.

CALCULATION SHEET

No: Rev:

Project: **Station Way Bridge** By: Date: 14/03/2025

Subject: **Service bay slab - Simply supp - 3T** Check By: Date:

Calculation of Plate Moments According to Pucher

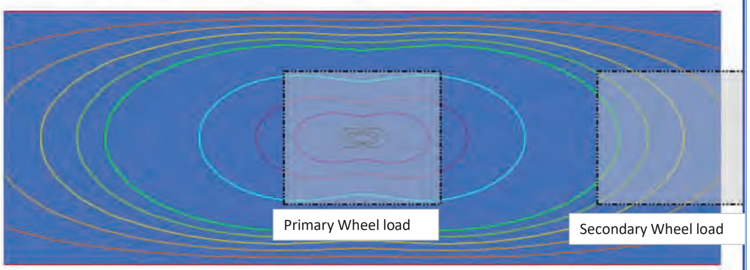
Instructions

Chart 1 - mx - Influence surface for the centre of a plate strip with two supported edges

Redraw

Span = 1.9 m

Contour resolution = 10 Area resolution = 10



Define rectangular patch load areas Accidental Redraw Calculate

Patch	Centre x - m	Centre y - m	Width m	Height m	Pressure KN/m^2	Volume ($8\pi.\beta$)	Moment KNm/m
1	0.000	0.000	1.000	1.000	28.1	0.9472	3.82
2	2.000	0.000	1.000	1.000	6.7	0.1705	0.16
3							
4							
5							
6							
7							
8							
9							
10							
					Total:	1.1177	3.99

Scales: 0.0, 0.2, 0.4, 0.6, 0.8, 1.0, 2.0, 3.0, 4.0, 5.0, 6.0, 7.0, 8.0, 14, 16, 18, 20

Note: Positions of loads are defined from the centre of the plate with x-negative being up in the above drawing and x-positive being down. Any areas that are off the edge of the plate are discounted in the calculation.
The volume calculated is the dimensionless volume, β.

Max ULS bending moment (3T AVL)
= 4.0 kN.m/m

SHEAR RESISTANCE OF THE SERVICE BAY SLAB

Bar Type	Bar Dia (")	Bar Dia (mm)	Area of Bar (mm^2)	Spacing (mm)	No. Bars (per m width)	Area of Reinforcement (mm^2/m)
Transverse Bottom Bar	3/8	9.53	71.3	79.4	12.6	897.7
Transverse Top Bar	3/8	9.53	71.3	158.8	6.3	448.9

Maximum shear Resistance based on concrete crushing

Equation 5.6a Maximum shear resistance based on concrete crushing

$$V_{max} = 0.36\left(0.7 - \frac{f_{cu}}{250}\right)\left(\frac{f_{cu}}{\gamma_{mc}}\right)b_w d$$

V_{max} = $0.36 \times \left(0.7 - \frac{15.0}{250}\right) \times \frac{15.0}{1.5} \times 1000 \times 110$ = 252.4 kN (per metre width)

Shear Resistance more than 3d from a support

Equation 6.5 Shear resistance of concrete slabs more than 3d from a support

$$V_{uc} = \frac{0.27}{\gamma_{mv}}\xi_s \rho_s^{\frac{1}{3}} f_{cu}^{\frac{1}{3}} b_w d$$

ξs = $\left(\frac{500}{110}\right)^{0.25}$ = 1.46

ρ_s = $\frac{100 \times 897.7}{1000 \times 110}$ = 0.82 (Based on bottom reinforcement)

V_{uc} = $\frac{0.27}{1.25} \times 1.46 \times (0.82)^{1/3} \times (15.0)^{1/3} \times 1000 \times 110$

V_{uc} = 79.8 kN (per m width - at 3d)

Shear Resistance within 3d of a support

Equation 5.6c Shear resistance within 3d of a support

$$V_u = \max\begin{cases} \frac{3d}{a_v}\Gamma V_{uc} \\ \frac{0.24}{\gamma_{mv}}\xi_s(0.15f_{cu})^{\frac{1}{3}}b_w d \end{cases}$$

Equation 5.6d Factor to account for the effect of short anchorage lengths

$$\Gamma = \min\begin{cases} \sqrt{\frac{z}{3d}\frac{F_{ub}}{V_{uc}}} \\ 1.0 \end{cases}$$

Equation 9.1a Anchorage resistance

$$F_{ub} = f_{ub} p L_a$$

Equation 9.1b Average anchorage bond strength

$$f_{ub} = \frac{k k_{cov} \beta \sqrt{f_{cu}}}{\gamma_{mb}}$$

	k	= 1.0		γ_{mb}	= 1.4	
CS 455, Tbl 9.1	B	= 0.39	(Plain bars in tension)	ϕ	= 9.53	mm
	f_{cu}	= 15.0	N/mm^2	K_{cov}	= 1.0	(taken as 1)

f_{ub} = 1.08

F_{ub} = $F_{ub} = f_{ub} p L_a$

P = 377.0 mm/m (Bottom reinforcement)

L_a = 110 mm (conservatively taken as d)

F_{ub} = 44.6 kN/m

Pg ref. C 3 d = 109.5 mm

Pg ref. C 3 z = 94.5

f_{ub} is the average anchorage bond strength over the effective anchorage length, given by Equation 9.1b;

p is the effective perimeter, taken as
$p = \pi\phi$ for a single bar, or
$p = (1.2 - 0.2N)\sum(\pi\phi)$ for a bundled group of N bars, valid up to $N = 4$.

ϕ is the nominal bar diameter

L_a is the effective anchorage length at the position where the resistance is being determined.

F_{ub} is the total anchorage force that can be developed in the longitudinal tension reinforcing bars at the front face of the support according to Section 9, but not greater than $\frac{A_s f_y}{\gamma_{ms}}$

z is the flexural lever arm at ULS at a position $3d$ from the support calculated from Equation 5.2.2b

Γ = $\left(\frac{94.5}{328.6} \times \frac{44.6}{79.8}\right)^{1/2}$

Γ = $(0.29 \times 0.56)^{1/2}$ = $(0.160)^{1/2}$ = 0.401

V_u = $\frac{3d}{a_v}\Gamma V_{uc}$ = $\frac{3 \times 110}{110} \times 0.401 \times 79.8$ = 95.9 kN/m (at av = d)

V_u = $\frac{0.24}{\gamma_{mv}}\xi_s(0.15f_{cu})^{\frac{1}{3}}b_w d$ = $\frac{0.24}{1.25} \times 1.46 \times (0.15 \times 15)^{1/3} \times 1000 \times 109.5$ = 40 kN/m

V_u = 95.9 kN (per metre width at d)

JACOBS

CALCULATION SHEET

Office	Manchester	**Page No.**	C	10	**Calc No.**	
Job No. & Title	ECC - Assessment of Station Way Bridge	**Calcs by**	MA		**Date**	Mar-25
Section	Main Deck (Span 2 to 4) – Service Bay Slab – Shear (AVL)	**Checker**	CAT II		**Date**	Mar-25

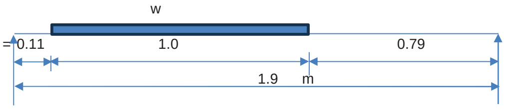
ULS SHEAR DUE TO ACCIDENTAL VEHICLE LOADING

All wheel loads applied to 1.0m x 1.0m area (allowing for dispersal through surfacing and fill).

Conservatively assume that shear due to primary wheel load is resisted by 1.0m width of slab.

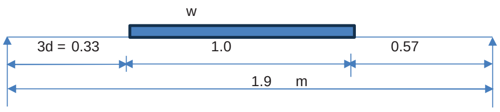
w = Primary wheel load (ULS)

Max Shear at d from support =



$$\text{Shear at d} = \frac{(1.9 - 0.11 - 0.5)}{1.9} w = 0.68\ w$$

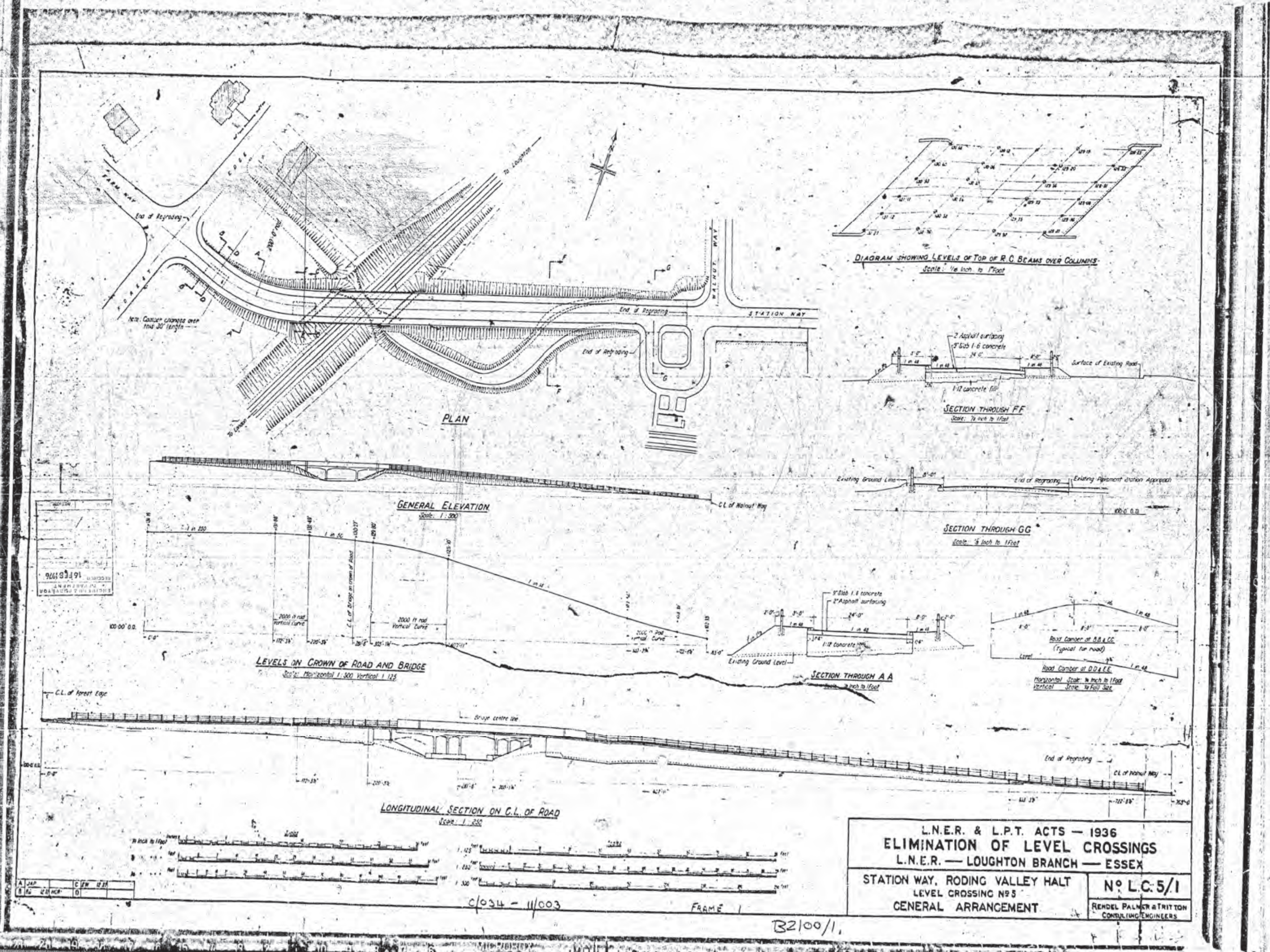
Max Shear at 3d from support =



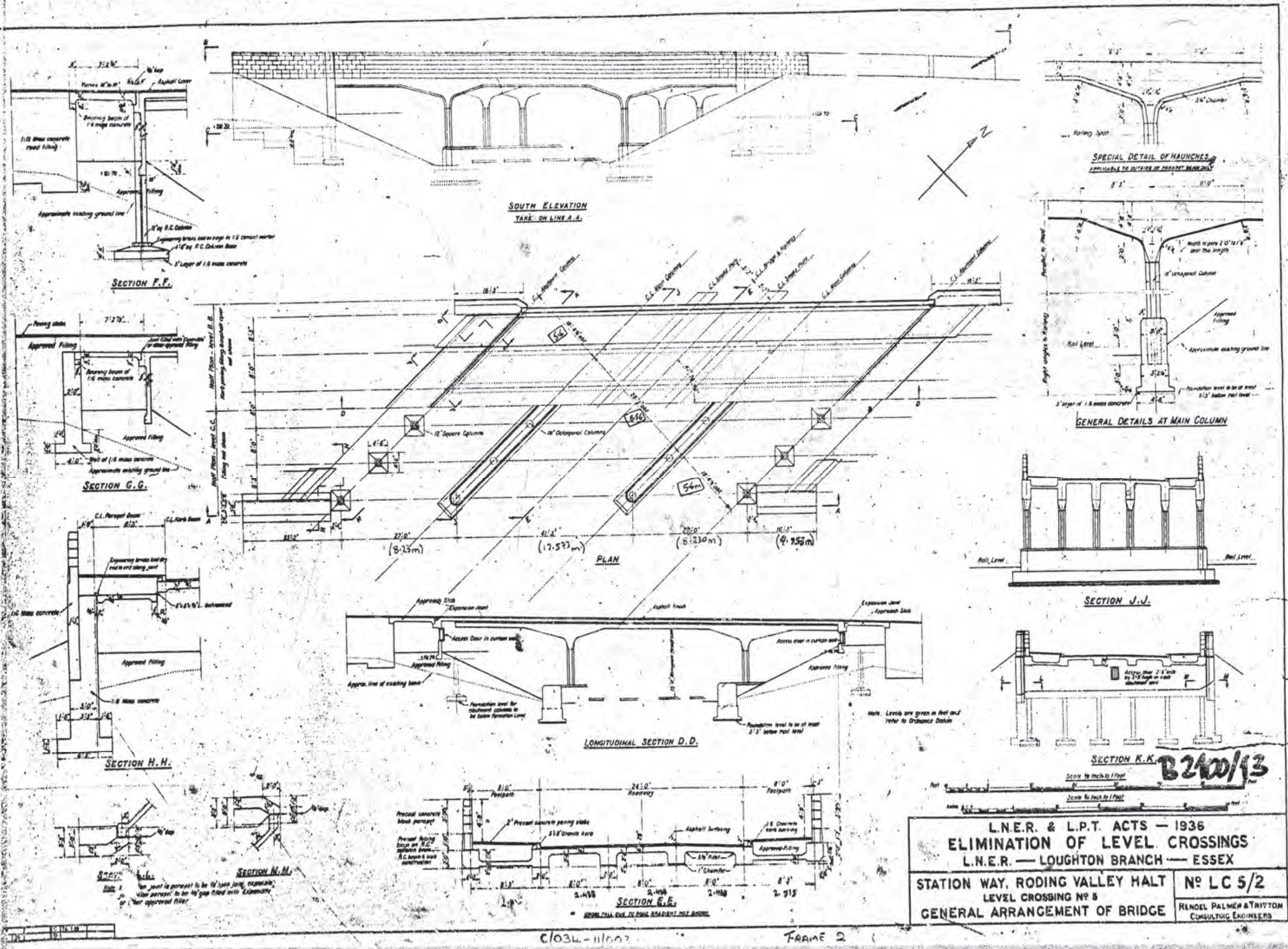
$$\text{Shear at 3d} = \frac{(1.9 - 0.33 - 0.5)}{1.9} w = 0.56\ w$$

Accidental Vehicle Load	w (kN)	ULS Shear (kN/m width of slab)	
		At d	At 3d
44T, 26T & 18T	151	102.6	85.0
7.5T	79	53.7	44.5
3T	28	19.0	15.8

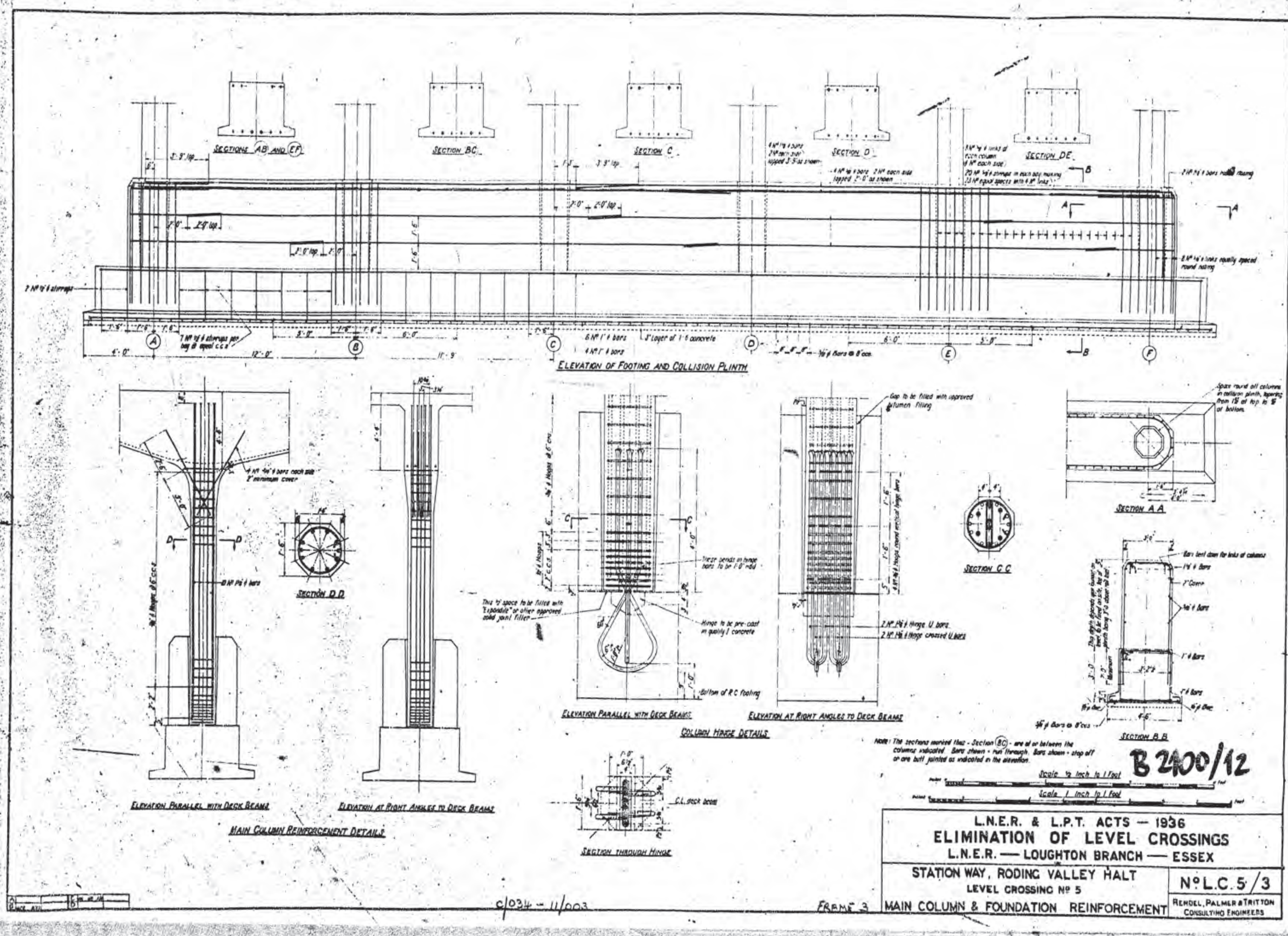
Appendix C. Record Drawings

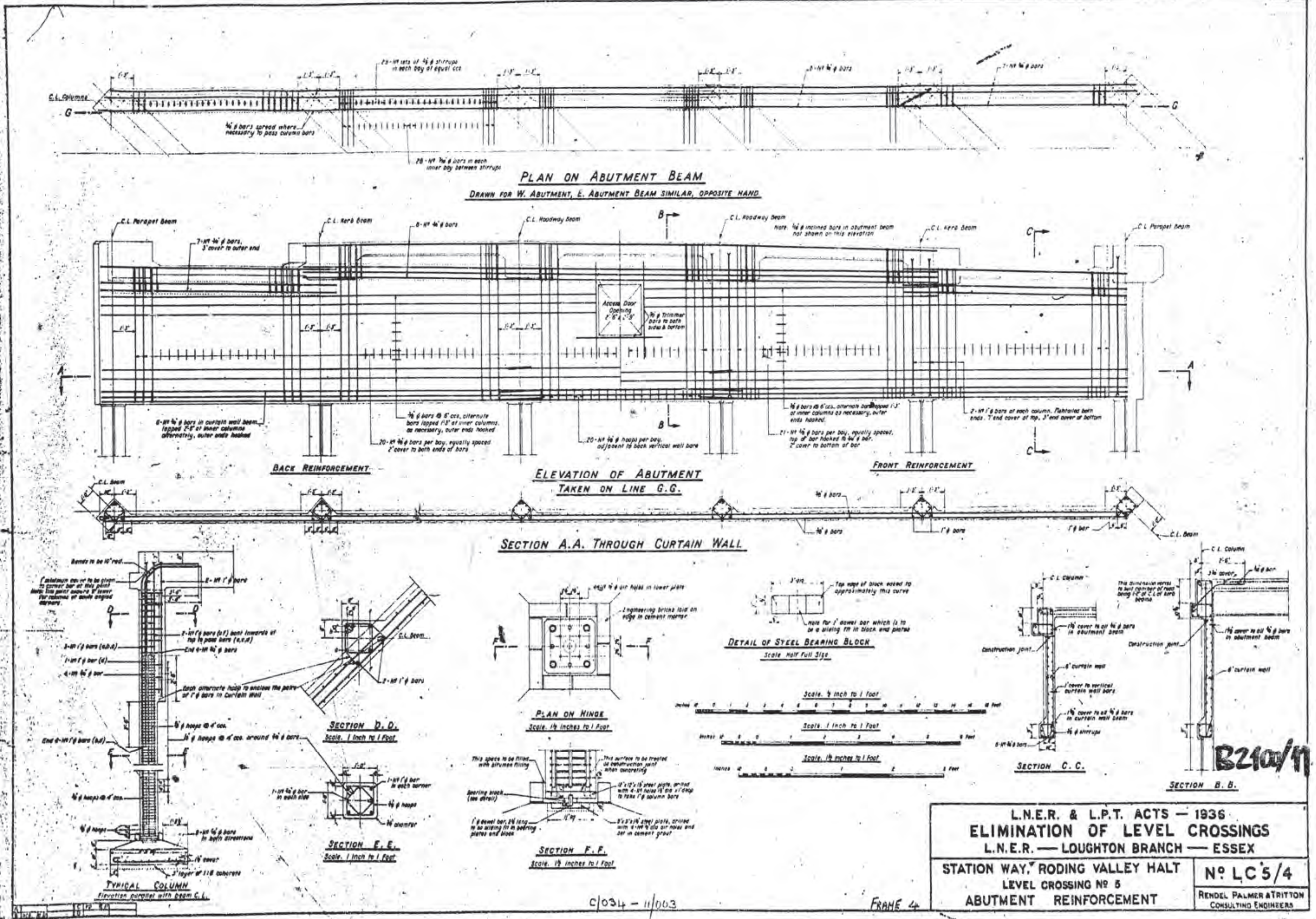


To Loughton
To London
FARM WAY
FOREST EDGE
WALNUT WAY
STATION WAY
End of Regrading
PLAN
DIAGRAM SHOWING LEVELS OF TOP OF R.C. BEAMS OVER COLUMNS
SECTION THROUGH FF
Surface of Existing Road
1:12 concrete fill
GENERAL ELEVATION
Scale: 1:500
C.L. of Walnut Way
Existing Ground Line
SECTION THROUGH GG
LEVELS ON CROWN OF ROAD AND BRIDGE
Scale: Horizontal 1:500 Vertical 1:125
2000 ft rad Vertical Curve
C.L. of Bridge on crown of Road
5" Slab 1:6 concrete
2" Asphalt surfacing
1:12 Concrete
Existing Ground Level
SECTION THROUGH A A
Road Camber at BB & CC (Typical for road)
Road Camber at DD & EE
C.L. of Forest Edge
Bridge centre line
LONGITUDINAL SECTION ON C.L. OF ROAD
Scale: 1:250
L.N.E.R. & L.P.T. ACTS — 1936
ELIMINATION OF LEVEL CROSSINGS
L.N.E.R. — LOUGHTON BRANCH — ESSEX
STATION WAY, RODING VALLEY HALT
LEVEL CROSSING No 5
GENERAL ARRANGEMENT
No L.C. 5/1
RENDEL PALMER & TRITTON CONSULTING ENGINEERS
C/034 - 11/003
FRAME 1
B2100/1

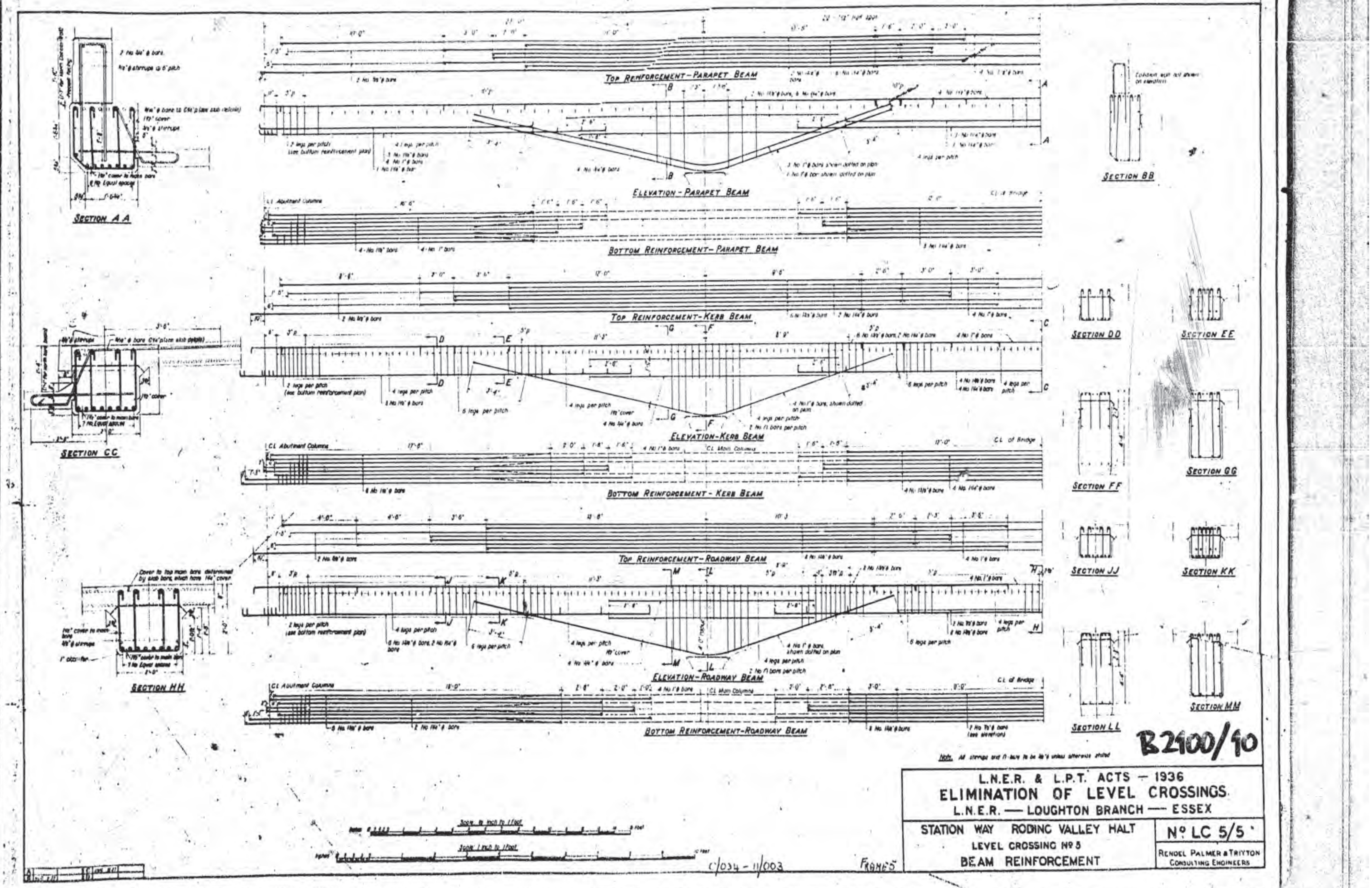


SOUTH ELEVATION
TAKEN ON LINE A.A.
SPECIAL DETAIL OF HAUNCHES
SECTION F.F.
PLAN
SECTION G.G.
GENERAL DETAILS AT MAIN COLUMN
SECTION J.J.
LONGITUDINAL SECTION D.D.
SECTION H.H.
SECTION K.K.
SECTION M.M.
SECTION E.E.
Note. Levels are given in feet and refer to Ordnance Datum
B2400/13
Scale 1/8 Inch to 1 Foot
L.N.E.R. & L.P.T. ACTS — 1936
ELIMINATION OF LEVEL CROSSINGS
L.N.E.R. — LOUGHTON BRANCH — ESSEX
STATION WAY, RODING VALLEY HALT
LEVEL CROSSING Nº 5
GENERAL ARRANGEMENT OF BRIDGE
Nº LC 5/2
RENDEL PALMER & TRITTON
CONSULTING ENGINEERS
C/034-11/007
FRAME 2

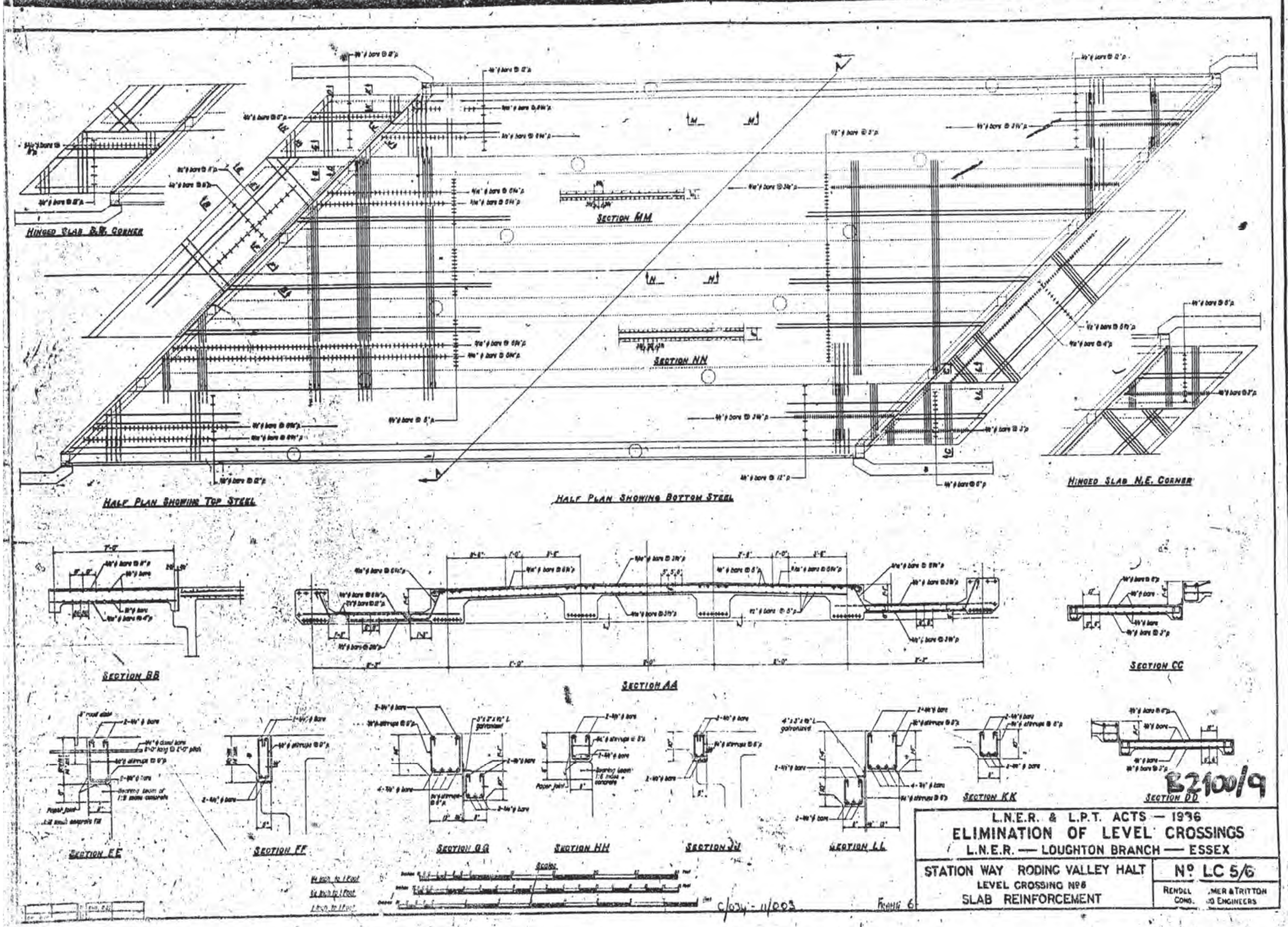

SECTIONS AB AND EF
SECTION BC
SECTION C
SECTION D
SECTION DE
ELEVATION OF FOOTING AND COLLISION PLINTH
SECTION D D
ELEVATION PARALLEL WITH DECK BEAMS
ELEVATION AT RIGHT ANGLES TO DECK BEAMS
MAIN COLUMN REINFORCEMENT DETAILS
SECTION C C
COLUMN HINGE DETAILS
SECTION THROUGH HINGE
SECTION A A
SECTION B B
Scale ½ inch to 1 Foot
Scale 1 inch to 1 Foot
B 2100/12
L.N.E.R. & L.P.T. ACTS — 1936
ELIMINATION OF LEVEL CROSSINGS
L.N.E.R. — LOUGHTON BRANCH — ESSEX
STATION WAY, RODING VALLEY HALT
LEVEL CROSSING Nº 5
MAIN COLUMN & FOUNDATION REINFORCEMENT
Nº L.C. 5/3
RENDEL, PALMER & TRITTON
CONSULTING ENGINEERS
c/034 – 11/003
FRAME 3

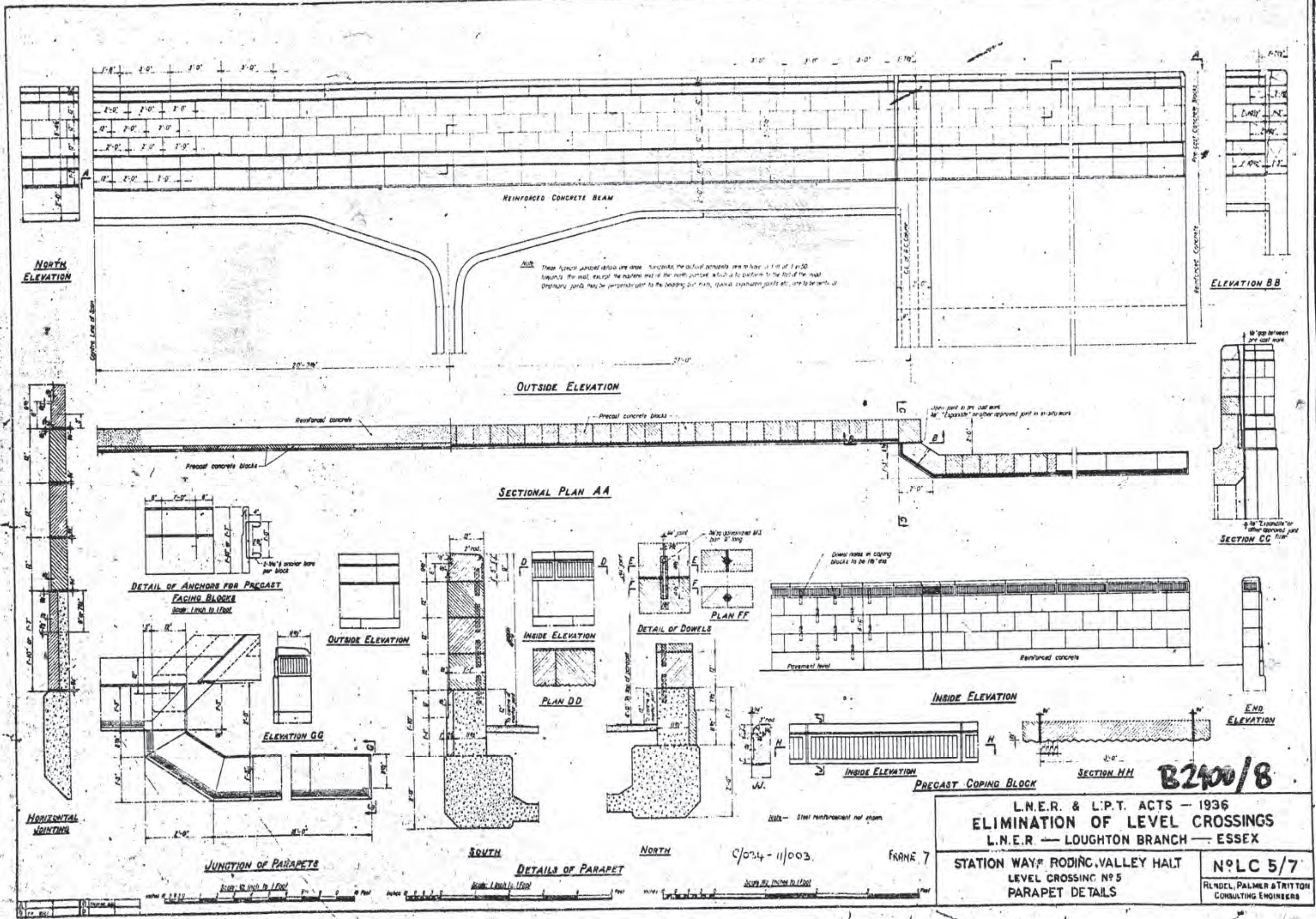


PLAN ON ABUTMENT BEAM
DRAWN FOR W. ABUTMENT, E. ABUTMENT BEAM SIMILAR, OPPOSITE HAND.
BACK REINFORCEMENT
FRONT REINFORCEMENT
ELEVATION OF ABUTMENT
TAKEN ON LINE G.G.
SECTION A.A. THROUGH CURTAIN WALL
TYPICAL COLUMN
Elevation parallel with beam C.L.
SECTION D.D.
Scale. 1 Inch to 1 Foot
SECTION E.E.
Scale. 1 Inch to 1 Foot
PLAN ON HINGE
Scale. 1½ Inches to 1 Foot
SECTION F.F.
Scale. 1½ Inches to 1 Foot
DETAIL OF STEEL BEARING BLOCK
Scale Half Full Size
Scale. ½ Inch to 1 Foot
Scale. 1 Inch to 1 Foot
Scale. 1½ Inches to 1 Foot
SECTION C.C.
SECTION B.B.
B210/11
L.N.E.R. & L.P.T. ACTS — 1936
ELIMINATION OF LEVEL CROSSINGS
L.N.E.R. — LOUGHTON BRANCH — ESSEX
STATION WAY, RODING VALLEY HALT
LEVEL CROSSING Nº 5
ABUTMENT REINFORCEMENT
Nº LC 5/4
RENDEL PALMER & TRITTON
CONSULTING ENGINEERS
C/034 - 11/003
FRAME 4

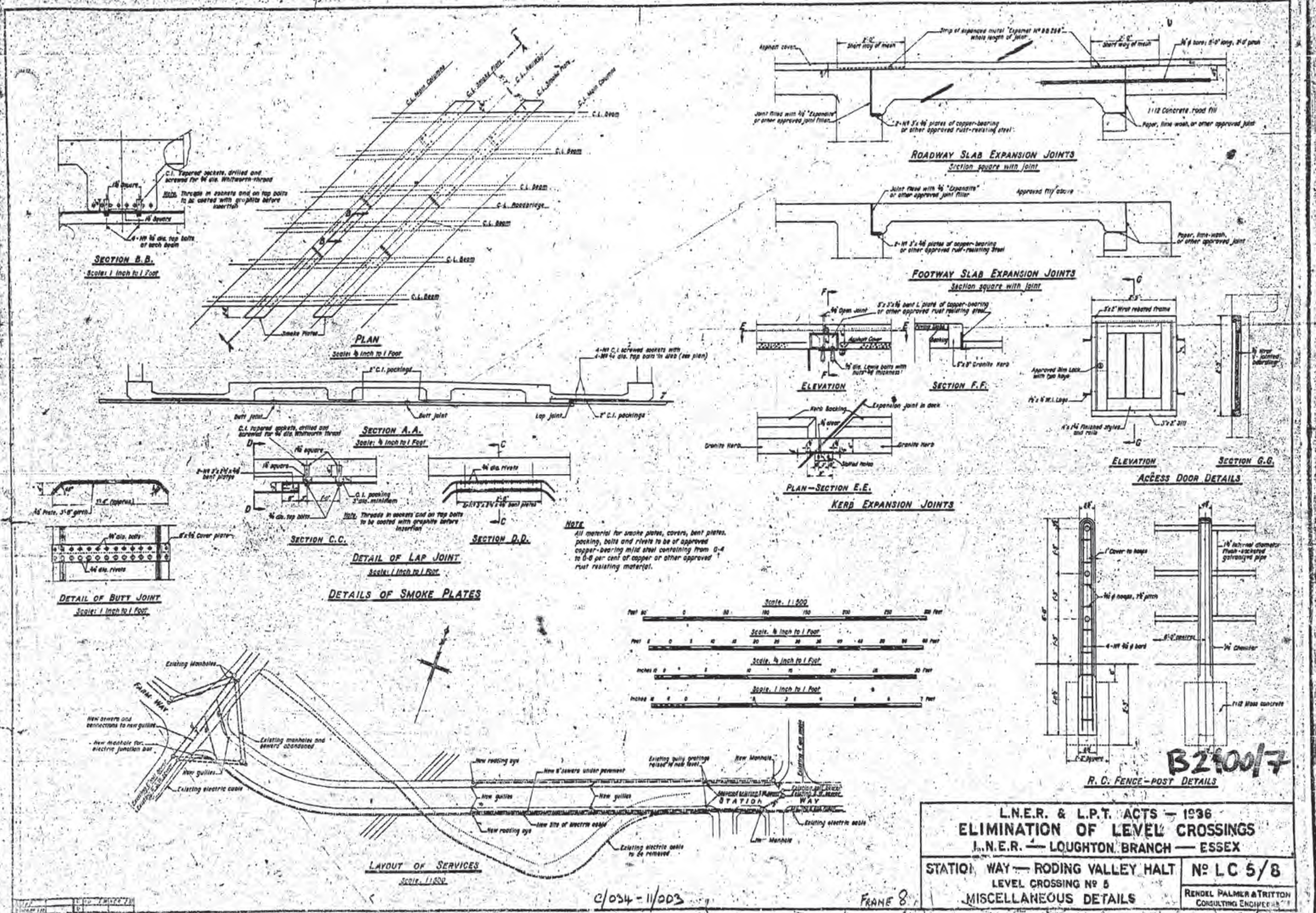


SECTION A A
TOP REINFORCEMENT - PARAPET BEAM
ELEVATION - PARAPET BEAM
SECTION BB
BOTTOM REINFORCEMENT - PARAPET BEAM
TOP REINFORCEMENT - KERB BEAM
SECTION CC
SECTION DD
SECTION EE
ELEVATION - KERB BEAM
BOTTOM REINFORCEMENT - KERB BEAM
SECTION FF
SECTION GG
TOP REINFORCEMENT - ROADWAY BEAM
SECTION JJ
SECTION KK
SECTION HH
ELEVATION - ROADWAY BEAM
BOTTOM REINFORCEMENT - ROADWAY BEAM
SECTION LL
SECTION MM
B2900/90
L.N.E.R. & L.P.T. ACTS — 1936
ELIMINATION OF LEVEL CROSSINGS.
L.N.E.R. — LOUGHTON BRANCH — ESSEX
STATION WAY RODING VALLEY HALT
LEVEL CROSSING Nº 5
BEAM REINFORCEMENT
Nº LC 5/5
RENDEL PALMER & TRITTON
CONSULTING ENGINEERS
C/034 - 11/003
FRAME 5


HINGED SLAB S.W. CORNER
SECTION MM
SECTION NN
HINGED SLAB N.E. CORNER
HALF PLAN SHOWING TOP STEEL
HALF PLAN SHOWING BOTTOM STEEL
SECTION BB
SECTION AA
SECTION CC
SECTION EE
SECTION FF
SECTION GG
SECTION HH
SECTION JJ
SECTION LL
SECTION KK
SECTION DD
B2100/9
Scales
L.N.E.R. & L.P.T. ACTS — 1936
ELIMINATION OF LEVEL CROSSINGS
L.N.E.R. — LOUGHTON BRANCH — ESSEX
STATION WAY RODING VALLEY HALT
LEVEL CROSSING Nº 8
SLAB REINFORCEMENT
Nº LC 5/6
RENDEL MER & TRITTON
CONS. ENGINEERS
C/034 - 11/003
FRAME 6



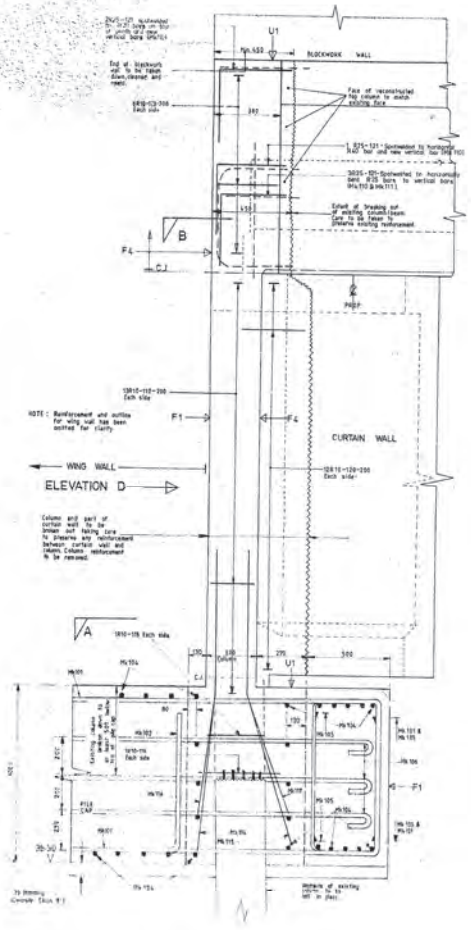
NORTH ELEVATION
REINFORCED CONCRETE BEAM
OUTSIDE ELEVATION
ELEVATION BB
SECTIONAL PLAN AA
Reinforced concrete
Precast concrete blocks
SECTION CC
DETAIL OF ANCHORS FOR PRECAST FACING BLOCKS
Scale: 1 inch to 1 Foot
OUTSIDE ELEVATION
INSIDE ELEVATION
PLAN DD
DETAIL OF DOWELS
PLAN FF
INSIDE ELEVATION
END ELEVATION
Pavement level
Reinforced concrete
ELEVATION GG
HORIZONTAL JOINTING
JUNCTION OF PARAPETS
SOUTH
NORTH
DETAILS OF PARAPET
INSIDE ELEVATION
PRECAST COPING BLOCK
SECTION HH
Note:- Steel reinforcement not shown
B2400/8
C/034-11/003.
FRAME 7
L.N.E.R. & L.P.T. ACTS — 1936
ELIMINATION OF LEVEL CROSSINGS
L.N.E.R. — LOUGHTON BRANCH — ESSEX
STATION WAY, RODING VALLEY HALT
LEVEL CROSSING Nº 5
PARAPET DETAILS
NºLC 5/7
RENDEL, PALMER & TRITTON
CONSULTING ENGINEERS



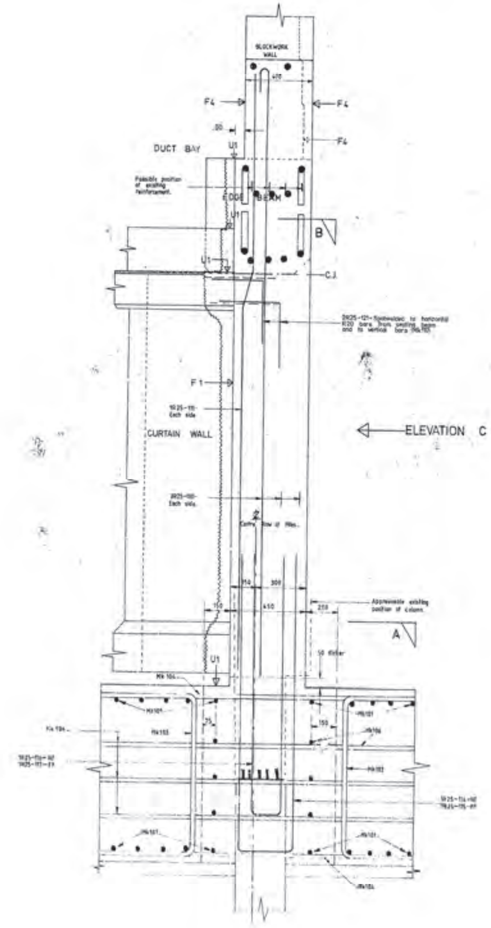
ROADWAY SLAB EXPANSION JOINTS
Section square with joint
FOOTWAY SLAB EXPANSION JOINTS
Section square with joint
SECTION B.B.
Scale: 1 inch to 1 Foot
PLAN
Scale: ¼ inch to 1 Foot
SECTION A.A.
Scale: ¼ inch to 1 Foot
SECTION C.C.
SECTION D.D.
DETAIL OF LAP JOINT
Scale: 1 inch to 1 Foot
DETAIL OF BUTT JOINT
Scale: 1 inch to 1 Foot
DETAILS OF SMOKE PLATES
NOTE
All material for smoke plates, covers, bent plates, packing, bolts and rivets to be of approved copper-bearing mild steel containing from 0·4 to 0·6 per cent of copper or other approved rust resisting material.
ELEVATION
SECTION F.F.
PLAN—SECTION E.E.
KERB EXPANSION JOINTS
ELEVATION
SECTION G.G.
ACCESS DOOR DETAILS
R.C. FENCE-POST DETAILS
Scale. 1:500
Scale. ⅛ inch to 1 Foot
Scale. ¼ inch to 1 Foot
Scale. 1 inch to 1 Foot
LAYOUT OF SERVICES
Scale. 1:500
STATION WAY
B2400/7
L.N.E.R. & L.P.T. ACTS — 1936
ELIMINATION OF LEVEL CROSSINGS
L.N.E.R. — LOUGHTON BRANCH — ESSEX
STATION WAY — RODING VALLEY HALT
LEVEL CROSSING Nº 8
MISCELLANEOUS DETAILS
Nº LC 5/8
RENDEL PALMER & TRITTON
CONSULTING ENGINEERS
C/034-11/003
FRAME 8

NOTES

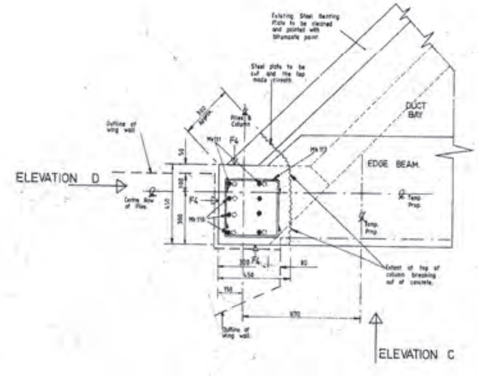
1. All dimensions are in millimetres unless otherwise stated.
2. Structural Concrete to be class 30/20 with a minimum sulphate resisting portland cement content of 3?? kg/m³ and a maximum water/cement ratio of 0.50.
3. Cover to reinforcement to be 40 mm.
4. Classes of Formed and Unformed Surfaces to concrete indicated thus: F1 ⟶
5. The temporary prop must not be removed until the concrete in the reconstructed column has gained a crushing strength of Ucr = 30 N/mm², but before 28 days after concrete column construction.
6. End bearing plate on temporary prop to be evenly bedded to the pile cap top and bearing plate to temporary prop must transmit the loading from the edge beam evenly onto the temporary prop as approved by the Engineer. The temporary prop arrangement shall be able to carry an axial load of 325 kN allowing no vertical movement to take place, and must be to the satisfaction of the Engineer.
7. The prop shall not transmit any load onto the pile cap until the concrete in the pile cap has gained a crushing strength of 20 N/mm².



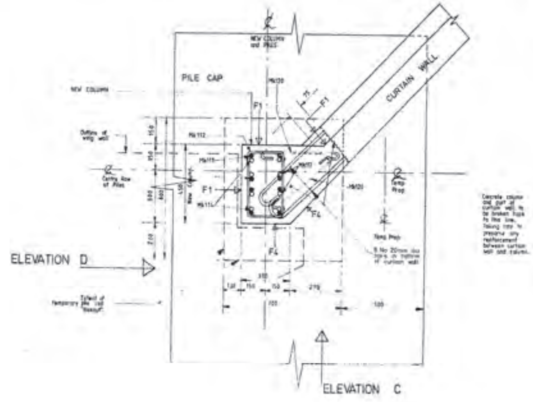
ELEVATION C



ELEVATION D



SECTION B-B



SECTION A-A

ESSEX COUNTY COUNCIL
HIGHWAYS DEPARTMENT

G. CARPENTER, C.Eng., F.I.C.E., F.I.Mun.E., F.Inst.H.E.
COUNTY SURVEYOR,
GLOBE HOUSE, NEW STREET, CHELMSFORD.
Telephone: Chelmsford 87131

SCALES: 1:10

	INITIALS	DATE
SURVEYED	T.H.	MARCH '79
LEVELLED		
DESIGNED	T.H.	APRIL '79
DRAWN	T.H.	APRIL '79
TRACED	J.M.	APRIL '79
CHECKED	R.D.C.	APRIL '79

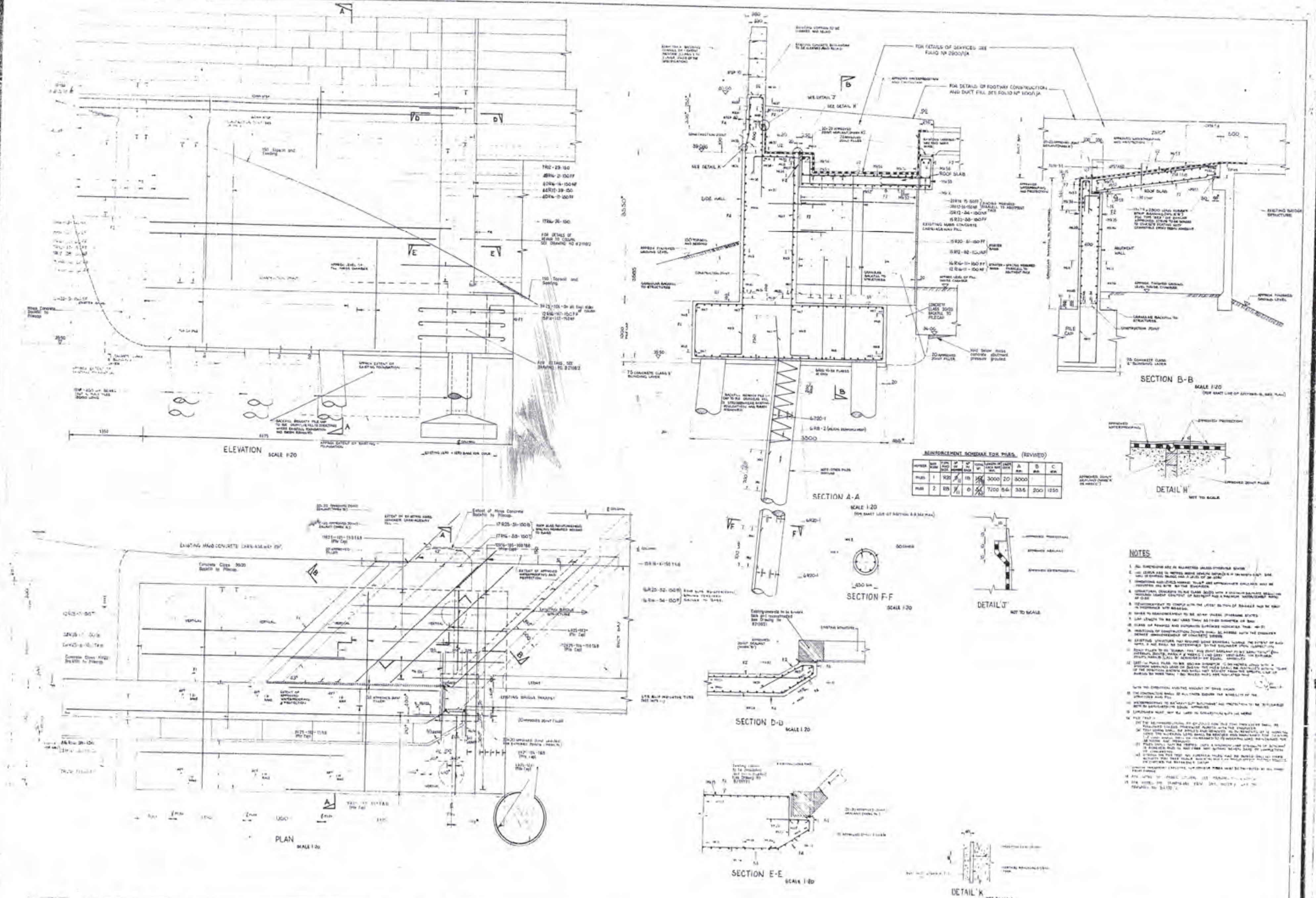
REVISIONS: AS BUILT DRAWING

REVISION LETTER	DATE
A	OCT '79

DESCRIPTION OF DRAWING: REPAIRS TO SOUTH WEST CORNER COLUMN

SCHEME TITLE: REPAIRS TO STATION WAY BRIDGE, BUCKHURST HILL

DRAWING No. B2100/2 A



PILES	1	R20	[illegible]	16	[illegible]	3000	20	3000		
PILES	2	R8	[illegible]	6	[illegible]	7200	64	336	200	1250

ESSEX COUNTY COUNCIL
HIGHWAYS DEPARTMENT

G. CARPENTER C.Eng. F.I.C.E. F.I.Mun.E. F.Inst.H.E.
COUNTY SURVEYOR
GLOBE HOUSE NEW STREET, CHELMSFORD
Telephone Chelmsford 67131

SCALES: 1:20

	INITIALS	DATE
SURVEYED		
LEVELLED		
DESIGNED		
DRAWN		
TRACED		
CHECKED		

REVISIONS
AS BUILT DRAWING

REVISION SUFFIX	DATE
A	April '79
B	OCT '79

DESCRIPTION OF DRAWING
DETAILS & REINFORCEMENT DRAWING

SCHEME TITLE
REPAIRS TO STATION WAY BRIDGE
BUCKHURST HILL

DRAWING No. B 2100/1B