



Station Way Bridge ECC Bridge No. 2100

Assessment Report May 2025





Jacobs

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

Main spans (spans 2 to 4)

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.

Approach spans (spans 1 and 5)

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	
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	47
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/mm² (clause 3.1.3 of CS 455)

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

Characteristic strength = 20 N/mm² (record drawing B2100/1B)

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

Characteristic strength = 230 N/mm² (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/m³)		
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. 11/2" (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	ElementLive LoadSectionDL + SDLLiveTotalUtilisationResultSectionCaseResistance*Effect*LoadLoad(%)Image: CaseImage: CaseSection(kNm)(kNm)Effect*Effect*Effect*Image: CaseImage: CaseSection(kNm)(kNm)KNm)(kNm)Image: CaseImage: CaseSection										
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	Element Live Load Section DL + SDL Live Total Utilisation** Result Section										
	Case	Resistance* (kN)	Effect* (kN)	Effect*	Load Effect*	(%)					
				(kN)	(kN)						
Main Slab	Pedestrian	23.6	7.6	2.3	9.9	42	Pass	Support (ULS shear)**			
	loading										

<u>Notes (Tables 3 & 4):</u>

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	ElementLive LoadSectionDL + SDLLiveTotalUtilisationResultSectionCaseResistance*Effect*LoadLoad(%)Image: CaseSection(kNm)(kNm)Effect*Effect*Effect*Image: CaseSection										
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	ElementLive LoadSectionDL + SDLLiveTotalUtilisationResultSectionCaseResistance*Effect*LoadLoad(%)(%)Effect*Effect*(%)(kN)(kN)Effect*Effect*(%)(%)(%)(%)(%)									
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 Dead load + Live Total Utilisation Result Section Section Load Load Load Load Load Load Load KNm) KN								Section					
	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)					
Carriageway Slab – Span 1	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				
	ALL Model 1	78	5	48	53	68	Pass	d from support				
Carriageway Slab – Span	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	Section Resistance* (kNm)	Dead load + Superimposed Dead load* (kNm)	Live Load Effect* (kNm)	Total Load Effect* (kNm)	Utilisation (%)	Result	Section						
	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)					
Carriageway Slab – Span 5	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	ALL Model 1	78	5	43	49	62	Pass	d from support			
5	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				
	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)				
South West Footway Slab – Span 1	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					
	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					
South West Footway Slab – Span 1	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 – Carri	ageway Beam (ef	fective width	= 1730mm)	(method of a	ssessment as des	cribed in section 5.4)			
Element	Load case	Section Reference	Section Resistance	Dead load + Superimposed	Live Load Effect	Total Load	Utilisation	Result	Section			
Assessm	nent		(kNm)	Dead load (kNm)	(kNm)	Effect (kNm)	(%)					
	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			
Carriageway Beam -	ALL 2	1	709	118	340	445	63	Pass	Max Sagging (Analysis 2) @ Mid Span of Span 3			
Main Spans 2 -4	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	Load case	Section Resistance	Dead load + Superimposed	Live Load	Total Load	Utilisation	Result	Section					
Assess	ment	(KIN)	(kN)	(kN)	(kN)	(70)							
	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)					
Main Spans 2	ALL Model 1	644	26	221	247	38	Pass	Shear @ 3d (1687mm) from end support (analysis 1)					
-4	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	Sectio n Defer	Section Resistance	Dead load + Superimpose	Live Load Effect	Total Load	Utilisation	Result	Section				
Asse	Assessment		(KNIII)	(kNm)	(KINITI)	(kNm)	(70)						
	Accidental Vehicle Loading (Normal Traffic)	1	709	215	191	406	57	Pass	Max Sagging (Analysis 1) @ Mid Spans of Span 3				
Main Spans 2 - 4	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 Be	am – Ulti	mate Limit State B	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	Load case	Section Resistance	Dead load + Superimposed	Live Load	Total Load	Utilisation	Result	Section					
Assessment		(kN)	Dead load (kN)	Effect (kN)	Effect (kN)	(%)							
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 Be	am – Ul	timate 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	Dead load + Superimposed	Live Load Effect	Total Load	Utilisation	Result	Section				
Assessi	ment		(kNm)	Dead load (kNm)	(kNm)	Effect (kNm)	(%)						
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				

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	Dead load + Superimposed	Live Load	Total Load	Utilisation	Result	Section				
Asse	ssment	(kN)	Dead load (kN)	Effect (kN)	Effect (kN)	(%)						
Main	Accidental Vehicle Loading (Normal Traffic)	685	105	184	289	42	Pass	Shear @ d (562mm) from end support (analysis 1)				
-4	Accidental Vehicle Loading (Normal Traffic)	1132	330	190	520	46	Pass	Shear @ d (1204mm) from Intermediate support (analysis 2)				

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	ElementLive LoadSectionDL + SDLLive LoadTotal LoadUtilisationResultSectionCaseResistanceEffectEffectEffect(%)(%)SectionSection(kNm/m)(kNm/m)(kNm/m)(kNm/m)(%)SectionSection												
	ALL model 1 (44t)	45.1	4.5	34.4	38.9	86	Pass	Midway between main beams (ULS sagging bending)					
Carriageway	ALL model 1 (44t)	-50.1	-3.0	-35.7	-38.7	77	Pass	At connection with main beam (ULS hogging bending)					
(spans 2-4)	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	ElementLive LoadSectionDL + SDLLive LoadTotal LoadUtilisationResultSectionCaseResistanceEffectEffectEffect(%)(kN/m)(kN/m)(kN/m)(kN/m)CaseCase												
	ALL model 1 (44t)	166.3	7.7	144.7	152.4	92	Pass	At d from connection with main beam (ULS shear)					
Carriageway slab (spans 2-4)	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			
	AVL (7.5t)	17.0	12.1	11.0	23.1	136	Fail				
Service bay slab (spans 2-4)	AVL (3t)	17.0	12.1	4.0	16.1	95	Pass	Midway between main beams (ULS sagging bending)			
	Pedestrian loading	17.0	12.1	3.7	15.8	93	Pass				

Notes:

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.

	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						
	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	At d from connection with main beam (ULS shear)					
	Pedestrian loading	95.9	22.5	7.0	29.5	31	Pass						

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

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				
	AVL (18t)	79.8	16.6	85.0	101.6	127	Fail					
Service bay	AVL (7.5t)	79.8	16.6	44.5	61.1	77	Pass					
slab (spans 2-4)	AVL (3t)	79.8	16.6	15.8	32.4	41	Pass	At 3d from connection with main beam (ULS shear)				
	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 Section case Resistance S			Dead load + Superimposed	Live Load	Total Load	Utilisation	Result	Section				
Assess	sment		Dead load	Епесс	Епест	(70)						
Main	ALL Model 1 + SV80	1216 kN	125 kN	929 kN	1053 kN	87	Pass	Axial				
Spans 2 -4	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	Dead load + Superimposed	Live Load	Total Load	Utilisation	Result	Section			
Assess	sment	(KN)	(kN)	(kN)	(kN)	(70)					
Main Spans 2	ALL Model 1 + SV80	2526	525	1190	1715	68	Pass	Axial			
-4	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)

, i i	Span 1 - Original Footway Slab – Main Slab – Transverse Bending											
	ULS Bending (Transverse) for 1.00m Width of Slab											
Element	ElementLive LoadSectionDL + SDLLiveTotalUtilisationResultCaseResistanceEffectLoadLoad(%)(kNm/m)(kNm/m)EffectEffectEffect(kNm/m)(kNm/m)(kNm/m)(kNm/m)											
Main slab	Main slab Pedestrian 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)*					

Notes (Tables 27 & 28):

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.

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

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

	Span 1 - Original Footway Slab – Downstand - Shear												
	ULS Shear												
Element	ElementLive LoadSectionDL + SDLLiveTotalUtilisationResultCaseResistance*Effect*LoadLoad(%)(kN)(kN)Effect*Effect*Effect*												
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					
	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)					
Carriageway Slab – Span	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)
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	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							
	ALL Model 1	78	5	48	52	67	Pass	d from support							
	ALL Model 2	78	9	41	51	64	Pass	d from support							
Carriageway Slab – Span 1	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						
	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)						
Carriageway Slab – Span	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)						

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 Effect* (kN) (kN) (kN) (kN) (kN) (kN) (kN) (kN)														
	ALL Model 1	78	5	48	53	68	Pass	d from support						
	ALL Model 2	78	9	41	51	65	Pass	d from support						
Carriageway Slab – Span 5	ALL Model 1 + SV80	78	5	51	56	71	Pass	d from support						

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						
	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)						
South West Footway Slab – Span 1	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)						

Table 36: Span	1 -	South-West	Footway	Slab –	Shear	(Cat II	Check)
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	Deck Information: Approach Span 1 – South West Footway Slab														
	Shear @ d from support														
Element	Load caseSection Resistance* (kN)Dead load + Superimposed Dead load* (kN)Live Load Effect* (kN)Total Load Effect* (kN)UtilisationResult(%)														
	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							
South West Footway Slab – Span 1	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							

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)

					Carriagew	ay Beam –	Ultimate L	imit State Bend	ing
				Bendi	ng — Carriage	way Beam (e	effective width	ו = 1730mm)	
Element	Load case	Section	Section	Dead load + Superimposed	Live Load	Total Load	Utilisation		
Assess	Assessment		(kNm)	Dead load (kNm)	(kNm)	Effect (kNm)	(%)	Result	Section
	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)
Carriageway Beam (Span 2-4)	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)

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

	Carriageway Beam – Ultimate Limit State Shear												
			Shear – Carriageway Beam	(effective width	n = 610mm)			-					
Element	Load case	Section Resistance (kN)	Dead load + Superimposed Dead load (kN)	Live Load Effect (kN)	Total Load Effect (kN)	Utilisation (%)	Result	Section					
Assessmer	nt					(,0)							
	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)					
Carriageway Beam (Span 2 -4)	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)						

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												
					Ber	nding – Kert	Beam (effec	tive widtł	n = 1730mm)				
Element	Load case	Section	Section	Dead load + Superimposed	Live Load	Total Load	Utilisation	Decult	Continu				
Asse	ssment	Reference	(kNm)	Dead load (kNm)	Effect (kNm)	Effect (kNm)	(%)	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)					

i able 40.	Spans 2 to	4 - Keib bea	anis – Shear (Ca	at ii che	CK)			
						Kerb Bea	am – Ult	imate Limit State Shear
						Shear – K	erb Beam	n (effective width = 610mm)
Element	Load case	Section	Dead load + Superimposed	Live Load	Total Load	Utilisation	Pocult	Section
Asse	ssment	(kN)	Dead load (kN)	Effect (kN)	Effect (kN)	(%)	Nesult	Section
	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)
Kerb Beam (Spans 2 -4)	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)

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

	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)						

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													
				Bendi	ng — Parapet	Beam (effec	tive width = 1	050mm)						
Element	Load case	Section	Section Resistance	Dead load + Superimposed	Live Load Effect	Total Load Effoct	Utilisation	Result	Section					
Assessment		Reference	(kNm)	(kNm)	(kNm)	(kNm)	(%)							
	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)					
Parapet	ALL Model 1	1	980	326.7	42.9	369.6	37.7	Pass	Sagging @ Mid Span of Span 3 (Analysis 1)					
(Spans 2 – 4)	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)					

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

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

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						
	ALL model 1 (44t)	44.8	4.5	37.7	42.2	94	Pass	Midway between main beams (ULS sagging bending)						
Carriageway	ALL model 1 (44t)	-50.4	-3.0	-37.2	-40.2	80	Pass	At connection with main beam (ULS hogging bending)						
(spans 2-4)	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.

able 44. Spans 2 to 4 - Carnageway Stab - Shear (Carn 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	ALL model 1 (44t)	121.0	5.0	97.0	102.0	84	Pass	At 3d from connection with main beam (ULS shear)						
(spans 2-4)	SV80	121.0	5.0	78.0	83.0	69	Pass	At 3d from connection with main beam (ULS shear)						

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

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					
	AVL (7.5t)	16.8	12.0	11.0	23.0	137	Fail						
Service bay slab (spans 2-4)	AVL (3t)	16.8	12.0	3.9	15.9	95	Pass	Midway between main beams (ULS sagging bending)					
	Pedestrian loading	16.8	12.0	3.7	15.7	93	Pass						

Notes:

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.

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		
	AVL (18t)	79.3	16.5	85.0	101.5	128	Fail			
Service bay	AVL (7.5t)	79.3	16.5	44.4	60.9	77	Pass			
(spans 2-4)	AVL (3t)	79.3	16.5	15.8	32.3	41	Pass	At 30 from connection with main beam (OLS shear)		
	Pedestrian loading	79.3	16.5	5.1	21.6	27	Pass			

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

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	Live Load	Total Load	Utilisation	Result	Section	
Assessr	ment		Dead load	Effect	Effect	(%)			
Abutmont	ALL Model 1 + SV80	1151	129.6	838.2	967.8	84.0	Pass	Axial (kN)	
Column	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	Dead load + Superimposed	Live Load	Total Load	Utilisation	Result	Section	
Assessment		(kN)	Dead load (kN)	Effect (kN)	Effect (kN)	(%)			
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:		
Name:		(Assessment Team Leader)
Engineering Qualifications:	BEng CEng MICE	_
Signed:		
Name:		
Position Held:	Associate Director	
Name of Organisation:	Jacobs	
Date:	30.05.2025	

Section 2

The certificate is agreed and submitted for acceptance

Signed:

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:

Name:

Position Held:

TAA:

Date:

Structures Manager

Essex County Council

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

Section 2

The certificate is agreed and submitted for acceptance

Signed:

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:

Name:

Position Held:

TAA:

Date:

Structures Manager

Essex County Council

Appendix A. Location Plan



Location Plan

Station Way Bridge

Grid Reference: TQ 414 929

Easting and Northing: 541460, 192900

Appendix B. Calculations

JAU	OBS	CALCULA	CALCULATION SHEET							
Office	Manchester First Street	Page No.	1	Calc No.						
ob No. & Title	ECC - Assessment of Station Way	Calcs by	JF	Date	May-2					
Section	Contents page	Checker	CAT II	Date	May-2					
	<u>Contents</u>	Page no.								
	Pages not used	2 - 17	(Pages r	emoved ()4/25)					
	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 r	emoved ()4/25)					
	Intermediate Pier Columns	92								
	Abutment Columns	100								
	Parapet Assessment	107								
	Original Footway Slab (Spans 1 and 5)	A1 - A13	(Pages a	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 a	added 04/	25)					



JAC	COBS C.							ALCULATION SHEET			
Office	Manchester First Street							Page No.	18	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
	Material & Section Properties										
	Reinforced Concrete										
CS 455,	Characteristic Strength of Concrete				=	15.00	N/mm ²				
CI.3.1.3 CS 454	Density of Reinforced Concrete				=	24	kN/m ³				
table 4.1.1a	Steel Reinforcement										
CS455, CI,	Mild steel reinforcement characteristic yield strength	n					=	230	N/mm ²		
3.8.2	Design Young's Modulus of steel reinforcement						=	210000	N/mm ²		
	Mass Concrete (Plain Concrete)										
CS 454	Unit weight of plain concrete				=	23	kN/m ³				
table 4.1.1a	Fill - Miscellaneous										
CS 454	Linit weight of miscellaneous fill				_	22	kN/m ³				
table 4.1.1a					-	22	KIN/III				
CS 454	Unit weight of Bituminous Macadam (tar)				_	24	kN/m ³				
table 4.1.1a	Partial Factors				-	24	KIN/III				
CS 455	Partial Factors					1 1 5					
Table 2.13a			/ms		-	1.15					
Table 2.13a	Partial Factor for Concrete		Ŷmc		=	1.5					
CS 455 Table 2.13a	Partial Factor for shear in concrete		γ̈́mv		=	1.15					
CS 455 Table 2.13a	Partial Factor for Bond		γmb		=	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			γf3	=	1.1					
	Material Thicknesses These dimensions have be scaled of drawing LC 5/	2 in inche	s and converte	ed to m	neters	and mil	imeters				
	Asphalt Cover (depth assumed)			=	4	"		= 101.6	mm		
	Slab Dimensions used in MIDAS				·			- 101.0			
	Skew Snan of Slab	_	3.12 m								
	Square Span of Slab	=	2.13 m								
	Skew of Slab	=	43 °								
	width of Slab	-	11.575 11								

	OBS										CA	CALCULATION SHEET							
се	Manchester First Street											Page No.	19	Calc No.					
& Title	ECC - Assessment of St	tation V	Vay									Calcs by	JF	Date	May-24				
ion	Approach Span 5 - East	Carria	geway Slab									Checker	CAT II	Date	May-24				
d e	Member dimensio																		
	Section	on			A (m)		-	B (m)		Area	ı (m²)								
	Longitudin	al Sla	ıb		0.4564			0.25		0.1	141								
	Self Weight of Sect Total Reaction from	tion n MID.	AS	<u> </u>			=		3.15 167.6	kN/m (a kN	ipplied ir	n MIDAS)							
	Deadload calculat	tions	10 Serrives 10 Serrives 11 Serrives 12 Serrives 12 Serrives 12 Serrives 12 Serrives 13 Serrives 13 Serrives 13 Serrives 13 Serrives 135 Serrives 136 Serrives 137 Serrives 138 Serrives 139 Serrives 139 Serrives 141 Serrives 142 Serrives 143 Serrives 144 Serrives 145 Serrives 145 Serrives 147 Serrives 150 Serrives 151 Serrives 152 Serrives 153 Serrives 154 Serrives 155 Serrives 155 Serrives 1		c 000000 c 000000	3 34490 3 34490 3 34490 3 34490 3 34490 3 324701 3 34490 3 34970 3 34970 1 3 3470 1 3 34700 1 3 34700 1 3 34700 1 3 347000 1 3 347000 1 3 347000 1 3 3	0 000000 0 0000												
	To check the mode is a demonstration	el accu of the	uracy com procedur	pared to re.	the as-built	constru	ction of	the str	ucture the	e following									
Skew Span Sqaure Span Width of Slab Skew Angle Depth of slab			= = = =	3.12 2.13 11.62 43 0.25	m m ° m														
	Area of slab on plar Volume of Slab	n		= =	24.725 6.18125	m² m³	2												

	DBS (CALCULATION SHEET					
e	Manchester First Street								Page No.	20	Calc No.			
Title	ECC - Assessment of Stati	on Way							Calcs by	JF	Date	Ν		
on	Approach Span 5 - East Ca	rriageway Slab							Checker	CAT II	Date	Ν		
	Verification													
	MIDAS calculated		= 167.6		kN									
	As built calculated		= 170.6025		kN									
	Percentage difference	9	= 1.7	7	%	(OK)								
	Superimposed Deac	lload												
	There is no fill over th dimensions for the as	e carriageway	/ slab. The only epth has been a	material assumed	which is pr and is sho	resent on wn below	top of th v.	e is asphalt. The	re is no exact					
	Asphalt Depth	=	100	mm	or		0.1 m	(assumed)						
	Ashphalt Width Asphalt above	=	0.4564 0.04564	m m ³										
	Total weight of	=	1.92	kN/m	(applied li	n MIDAS))							
		_	102.16	kN	(
		-	·····											
	Superimposed Dead	lioad Calcula	tion using Arci	live Drav	wings									
	Area of slab on plan Depth of surfacing		= 24.7 = 0.1	25 I	m ² m ³									
	Total reactions		= 103.	85	kN									
		Noc	le Load FX (kN)	FY (kN)	FZ (kN)	MX (kN*m)	MY (kN*m)	MZ (kN*m)						
			17 Superimp 0.00000 18 Superimp 0.00000 19 Superimp 0.00000	0 0.0000 0 0.0000 0 0.0000	00 2.044396 00 2.041476 00 2.047795	0.000000 0.000000 0.000000	0.000000 0.000000 0.000000	0.000000 0.000000 0.000000						
			20 Superimp 0.00000 21 Superimp 0.00000 22 Superimp 0.00000	0 0.0000	00 2.044770 00 2.038790 00 2.042169	0.000000 0.000000 0.000000	0.000000 0.000000 0.000000	0.000000 0.000000 0.000000						
			23 Superimp 0.00000 24 Superimp 0.00000 25 Superimp 0.00000 26 Superimp 0.00000	0.0000 0.0000 0.0000 0.0000 0.0000	00 1.991853 00 2.051654 00 0.292800	0.000000 0.000000 0.000000	0.000000 0.000000 0.000000	0.000000 0.000000 0.000000						
			26 Superimp 0.00000 31 Superimp 0.00000 32 Superimp 0.00000 33 Superimp 0.00000	0.0000	00 6.289510 00 0.290134 00 2.052473	0.000000	0.000000	0.000000 0.000000 0.000000						
			34 Superimp 0.00000 35 Superimp 0.00000 36 Superimp 0.00000	0.0000	00 1.993982 00 2.035703 00 2.042036	0.000000	0.000000 0.000000 0.000000	0.000000						
			37 Superimp 0.00000 38 Superimp 0.00000 39 Superimp 0.00000	0.0000	00 2.044108 00 2.044361 00 2.044237	0.000000 0.000000 0.0000000000000000000	0.000000 0.000000 0.0000000000000000000	0.000000 0.000000 0.000000						
			40 Superimp 0.00000 41 Superimp 0.00000 42 Superimp 0.00000	0 0.0000 0 0.0000 0 0.0000	00 2.044106 00 2.044080 00 2.044192	0.000000 0.000000 0.000000	0.000000 0.000000 0.000000	0.000000 0.000000 0.000000						
			43 Superimp 0.00000 44 Superimp 0.00000 45 Superimp 0.000000	0.0000	00 2.044415 00 2.044614 00 2.044315	0.000000 0.000000 0.000000	0.000000 0.000000 0.000000	0.000000 0.000000 0.000000						
			46 Superimp 0.00000 47 Superimp 0.00000 48 Superimp 0.00000 48 Superimp 0.00000	0.0000 0.0000 0.0000 0.0000 0.0000	00 2.042078 00 2.034411 00 2.015072	0.000000 0.000000 0.000000	0.000000 0.000000 0.000000	0.000000 0.000000 0.000000						
				0.0000	00 1.976064 00 1.926268 00 1.872966 00 1.816339	0.000000	0.000000	0.000000						
			52 Superimp 0.00000 53 Superimp 0.00000 54 Superimp 0.00000 55 Superimp 0.00000	0.0000	00 1.016552 00 1.466562 00 1.466569	0.000000	0.000000	0.000000						
			56 Superimp 0.00000 60 Superimp 0.00000	0 0.0000 0 0.0000 0 0.0000	00 0.013071 00 1.818843 REACTION FORCES	0.000000 0.000000 0.000000	0.000000	0.000000						
			Load (kN) Superimp 0.00000	FY (kN) 0 0.0000	FZ (kN) 00 102.158523									
				MIDAS	S Compari	son								
	Vertification													
	MIDAS Calculated		= 102.16		kN									
	As Built Calculated		= 103.85		kN									
	Percentage Differenc	е	= 1.64		%	(OK)								
	<u>Capacity</u>													
	Moment Resistance	of Carriagew	ay Slab											
	Bar Type	Bar Dia (")	Bar Dia (n	nm)	Area of	Bar (mm	1 ²) S	pacing (") Centre Centre	to Spacing	g (mm) C Centre	entre to]		
L	Longitudinal Bottom											-		

JAC	OBS							CAI			SHEE	T
Office	Manchester First Street								Page No.	21	Calc No.	
Job No. & Title	ECC - Assessment of Stati	ion Way							Calcs by	JF	Date	May-24
Section	Approach Span 5 - East Ca	arriageway Slab							Checker	CAT II	Date	May-24
	Grillage Model	Dims										
	Note: Only the	250mm deep s	slab cons	idered for M	u							
	Width of section		=		456.00	mm						
	Height of section	ı	=		250	mm						
	Cross Sectional	Area	=		114000	mm ²						
	Total number of section width	tension bars p	er	=	4.49							
	Area of tension r section width	einforcement p	per	=	720.83	mm²						
	Cover to reinford	ement	=	1 1/2	2 "	=	38 mm	(Assumed)				
	Longitudinal Di	rection										
	Effective Depth,	d	=		204.85	mm						
CS 455, Eq	Z = z=($\left(1-rac{0.84rac{f_y}{\gamma_n}}{rac{f_{cubd}}{\gamma_{mc}}} ight)$	$\left(\frac{A_s}{d}\right) d$		=	178.2	9 mm					
	Should be less th Take Z	han 0.95d			= =	194.6 178.2	1 mm (OK 9 mm)				
	Moment resistan	ice equation										
CS 455, Eq	$M_u = mi$	$n \left\{ \begin{array}{c} \frac{f_y}{\gamma_{ms}} A_s \\ \frac{0.225 f_{cu}}{\gamma_{mc}} l \end{array} \right.$	z bd^2									
	M _u =	25.70	kNm									
	M _u =	43.05	kNm									
	Therefore mome	ent reistance		=	25.7	0 kNm						
	Transverse Directio	n										
	Bar Type	Bar Dia (")	Bar	Dia (mm)	Ara of	Bar (mm ²)	Spacing	g (") Centre to Centre	o Spacing (mm) C Centre		entre to	
	Transverse Bottom Bar	1/2		12.7	1	26.68		5.5		139.7		
	Grillage Model Dims	5										
	Width of section			=	42	5 mm						
	Height of section			=	25	i0 mm						
	Cross sectional area	of section		=	10625	i0 mm²						
	Total number of tens	ion bars per se	ection	=	3.04	or	3 bai	rs				
	Area of tension reinfo	prcement per s	ection	=	385.38	mm ²						
	Cover to reinforceme	ent		=	1 1/2"	"	= 52	2.3 mm				
	Effective depth_d			_	185	mm	- 52					
				-	100							

JAC	OBS	С	CALCULATION SHEET									
Office	Manchester First Street		Page No.	22	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						
\$ 455, Eq 5.2.	$Z = z = \left(1 - \frac{0.84 \frac{f_y A_s}{\gamma_{ms}}}{\frac{f_{cubd}}{\gamma_{mc}}}\right) d = 169.76613 \text{ mm}$											
	Should be less than 0.95d = 175.75 mm (OK)										
	Moment Resistance Equation											
CS 455, Eq	$M_u = \min \left\{ egin{array}{c} rac{f_y}{\gamma_{ms}}A_s z \ rac{0.225 f_{cu}}{\gamma_{mc}}bd^2 \end{array} ight.$											
	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											
	$V_{\text{max}} = 0.36 \left(0.7 - \frac{f_{cu}}{250} \right) \left(\frac{f_{cu}}{\gamma_{mw}} \right) b_w d$											
	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:			20	64.0							
	γ_{mv} is the partial factor for shear defined in Section 2.			3D =	010 1010							
	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 $\rho_s = \frac{100A_s}{100A_s}$ but not less than 0.15, nor greater than 3.0											
	As is the area of longitudinal tension reinforcement that continues at least a dis beyond the section being considered, and, for shear at supports, continues support.	stance <i>d</i> at least to the										
	Where top and bottom reinforcement are provided, the area in tension und which produces the shear force is to be used.	er the loading										
	$\xi_{\rm s}$ = 1.239 < 0.7 (OK) As	= 642.4	mm²									
	ps = 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 $V_u \equiv \max \begin{cases} \frac{3d}{a_v} \Gamma V_{uc} \\ \frac{0.24}{\gamma_{mv}} \xi_s (0.15 f_{cu})^{\frac{1}{3}} b_w d \end{cases}$											
	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										
	a, = 204.85 mm or 3d											

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title ECC - Ass	essment of Station Way						Calcs by	JF	Date	May-2
Approach \$	Span 5 - East Carriageway Slab						Checker	CAT II	Date	May-2
T =	$\sqrt{\frac{z}{3d}\frac{F_{ub}}{V_{uc}}}$									
fub	$f_{ub} = \frac{kk_{cov}\beta\sqrt{f_{cu}}}{\gamma_{mb}}$									
К =	1	acon	=	0.40						
β =	0.39	с	=	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 <mark>39.71</mark>	kN		144.166079	Limiting value				
Г =	$\Gamma = \min \left\{ \begin{array}{c} \sqrt{\frac{z}{3d} \frac{F_{ub}}{V_{uc}}} \\ 1.0 \end{array} \right.$	= 0.429	>	1	0.47					
V _u =	$\frac{3d}{2}\Gamma V_{uc}$	= 77.88	kN							
=	$\frac{a_p}{a_p}$ $\frac{1}{3}b_n d$	= 31656.79	N							
	Amb 20(1.1.2 Cal am	31.66	kN							
Shear R	esistance - Transverse	Direction								
Maximur	n shear resistance based		Ishina							
Equ	lation 5.6a Maximum she	ar resistance ba	sed on	concrete c	rushing					
$V_{ m m}$	$u_{\rm x} = 0.36 \left(0.7 - \frac{f_{cu}}{250} \right) \left(0.7 - \frac{f_{cu}}{250} \right)$	$\left(\frac{f_{cu}}{\gamma_{mc}}\right)b_w d$								
wh	ere:									
	b_w is the breadth of the	e section, taken	as the w	eb width fo	r flanged beam	s				
V _{Max}	= 181.1	5								
Shear re	sistance more than 3d fro	om the support								
Equa	tion 6.5 Shear resista	nce of concre	te slab	s more t	han 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 i	n Sectio	on 2.						
$\xi_{\bar{s}}$	is the depth factor, tak $c = (\frac{500}{2})^{0.25}$ but not	en as loss than 0.7								
ρ_s	$\zeta_s = \left(\frac{-d}{d}\right)^{-1}$ but not is the ratio of longitudi $\rho_s = \frac{100A_s}{b-d}$ but not les	nal reinforceme s than 0.15, no	nt r greate	r than 3.0						
A_s	is the area of longitudi beyond the section be support. Where top and bottom	nal tension rein ing considered, n reinforcement	forceme and, fo are pro	ent that co r shear at ovided, the	ntinues at lea supports, con	st a distance <i>d</i> tinues at least t	o the			
	which produces the sh	ear torce is to b	e used							



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Section	Approach Span 5 - East C	arriageway Slab				Checker	CAT II	Date	May-24
	Length (mm)	Length (m)	W (kNm)	Med (kNm)	MIDAS Results	Section Res	istance	Posu	+
	Length (mm)	Length (III)			(kNm)	(kNm))	Nesu	

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

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Deadload Capacity Check - Shear Longitudinal Direction

Maximum Shear Load due to deadload Ved 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

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Section	Approach Span 5 - East Car	riageway Slab				Checker	CAT II	Date	May-2
	Live Loading - All Mo	del 1							
	Note: ALL Model 2 will	not be appli	ed to the carriageway slab due to the	e element complying with	section 5.	6 of CS 454			
	Impact Factors and L	ane Widths.							
		Impact	factor applied to critical axle	Notional lane width	Maxim	num lateral	spacing I	between	wheel
	Single vehicle in		Poor Road Surface	(m)	Ce	entres of ad	ljacent ve	ehicles (m	1)
	each lane		1.8	3			1.2		
	in each lane		1	2.5			0.7		
	Traffic Flow Factors								
	Traffic Flow Cat	tegory	Traffic Flow Factor]					
	Low		0.9						
	Lane Factors for ALL	. Model 1							
	Lane		Lane Factor]					
	Lane 1		1	-					
	Lane 3		0.5	-					
	Lane 4 and		0.4						

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Section	Approach Span 1 - West Carriageway Slab							Checker	CAT II	Date	May-24
	Material & Section Properties										
	Reinforced Concrete										
CS 455,	Characteristic Strength of Concrete				=	15.00	N/mm ²				
CS 454	Density of Reinforced Concrete				=	24	kN/m ³				
(able 4.1.1a	Steel Reinforcement										
CS455, Cl,	Mild steel reinforcement characteristic yield streng	gth					=	230	N/mm ²		
0.0.2	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										
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		γms		=	1.15					
CS 455 Table 2 13a	Partial Factor for Concrete		γmc		=	1.5					
CS 455	Partial Factor for shear in concrete		γ _{mv}		=	1.15					
CS 455	Partial Factor for Bond		γmb		=	1.25					
CS 454	Partial Factor for Concrete Deadload				=	1.15					
CS 454	Partial Factor for Surfacing				=	1.75					
CS 454	Partial Factor for Fill				=	1.2					
CS 454 3.9	Inaccurate assessment effects at ULS			γf3	=	1.1					
	Material Thicknesses These dimensions have be scaled of drawing LC	5/2 in inch	es and converte	ed to m	neters	and mili	meters				
	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 0.25 m								
	Skew of Slab	=	43 °								
	Width of Slab	=	11.375 m								

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ection	Approach Span 1 - West	t Carria	geway Slab									Checker	CAT II	Date	May-24
cad age	Member dimensio	ons													
əl	Longitudinal Edge S	Slab N	<u>/lember</u>				1								
			C)	•										
	Section	on			A (m)			B (m)		A	rea (m²)				
	Longitudin	al Sla	b		0.4564			0.25			0.1141				
	Self Weight of Sect	tion				·	=		3.15	kN/m	(applied	in MIDAS)			
	Total Reaction from	n MID/	AS				=		167.6	kN					
			17 Serr Weg 18 Serr Weg 19 Serr Weg 20 Serr Weg 21 Serr Weg 23 Serr Weg 23 Serr Weg 23 Serr Weg 24 Serr Weg 25 Serr Weg 13 Serr Weg 13 Serr Weg 13 Serr Weg 13 Serr Weg 13 Serr Weg 14 Serr Weg 15 Serr Weg	0.00000 0.000000 <td< th=""><th>0.00000 0.000000 0.0000000 0.0000000 0.000000 0.00000</th><th>3.344097 3.34927 3.359664 3.354701 3.354701 3.344890 3.354701 3.344890 3.344890 3.344890 3.344890 3.344890 3.347844 3.357844 3.35964 3.367846 3.367846 3.367846 3.367846 3.359646 3.3538569 3.354109 3.3538569 3.3538569 3.3538569 3.354109 3.3538569 3.354109 3.3538569 3.354109 3.3538569 3.354109 3.3538569 3.354109 3.3538569 3.354109 3.35421000000000000000000000000000000000000</th><th>0.000000 0.0000000 0.0000000 0.0000000 0.0000000 0.0000000 0.0000000 0.0000000 0.0000000 0.0000000 0.0000000 0.00000000 <tr< th=""><th></th><th>0.00000 0.0000 0.0000 0.0000 0.0000 0.00000 0.000</th><th></th><th></th><th></th><th></th><th></th><th></th></tr<></th></td<>	0.00000 0.000000 0.0000000 0.0000000 0.000000 0.00000	3.344097 3.34927 3.359664 3.354701 3.354701 3.344890 3.354701 3.344890 3.344890 3.344890 3.344890 3.344890 3.347844 3.357844 3.35964 3.367846 3.367846 3.367846 3.367846 3.359646 3.3538569 3.354109 3.3538569 3.3538569 3.3538569 3.354109 3.3538569 3.354109 3.3538569 3.354109 3.3538569 3.354109 3.3538569 3.354109 3.3538569 3.354109 3.35421000000000000000000000000000000000000	0.000000 0.0000000 0.0000000 0.0000000 0.0000000 0.0000000 0.0000000 0.0000000 0.0000000 0.0000000 0.0000000 0.00000000 <tr< th=""><th></th><th>0.00000 0.0000 0.0000 0.0000 0.0000 0.00000 0.000</th><th></th><th></th><th></th><th></th><th></th><th></th></tr<>		0.00000 0.0000 0.0000 0.0000 0.0000 0.00000 0.000						
				MII	DAS Comp	<u>arison</u>									
	Deadload calculat	ions f	from the a	archive c	drawings										
	To check the mode is a demonstration	el accu of the	iracy com procedur	pared to e.	the as-buil	t constru	ction of	f the stru	ucture the	e following	g				
	Skew Span Sqaure Span Width of Slab Skew Angle Depth of slab			= = = =	3.12 2.13 11.62 43 0.25	m m ° m									
	Area of slab on plan	n		=	24.725	m	2								

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Section	Approach Span 1 - West Carriag	geway Slab							Checker	CAT II	Date	May-24
	Verification											
	MIDAS calculated	=	167.6	k	N							
	As built calculated	=	170.6025	k	N							
	Percentage difference	=	1.77	7 9	6	(OK)						
	Superimposed Deadload	<u>d</u>										
	There is no fill over the ca dimensions for the aspha	arriageway slab. It so the depth h	The only r as been a	naterial w ssumed a	which is prea	sent onto n below.	op of the	e is asphalt. T	here is no exact			
	Asphalt Depth Ashphalt Width	= (100 0.4564	mm c m	r		0.1 m	(assumed)				
	Asphalt above	= 0	.04564	m°								
	Total weight of	=	1.92	kN/m (applied lin	MIDAS)						
	MIDAS Reactions	= 1	02.16	kN								
	Superimposed Deadload	d Calculation u	sing Arch	ive Draw	ings							
	Area of slab on plan Depth of surfacing	=	24.72 0 1	25 r	n ² n ³							
	Total reactions	=	103.8	35 k	N							
		Node Load	FX (kN)	FY (kN)	FZ (kN)	MX (kN*m)	MY (kN ^a m)	MZ (kN*m)				
		17 Superim 18 Superim 19 Superim	0.000000	0.000000 0.000000 0.000000	2.044396 2.041476 2.047795	0.000000	0.000000	0.000000 0.000000 0.000000				
		20 Superim 21 Superim 22 Superim	0.000000	0.000000 0.000000 0.000000	2.044770 2.038790 2.042169	0.000000 0.000000 0.000000	0.000000 0.000000 0.000000	0.000000 0.000000 0.000000				
		23 Superim 24 Superim 25 Superim	0.000000	0.000000	1.991853 2.051654 0.292800	0.000000	0.000000	0.000000				
		25 Superim 26 Superim 131 Superim	0.000000	0.000000	6.287396 6.289510	0.000000	0.000000	0.000000				
		132 Superim 133 Superim 134 Superim	0.000000 0.000000 0.000000	0.000000 0.000000 0.000000	0.290134 2.052472 1.993982	0.000000 0.000000 0.000000	0.000000 0.000000 0.000000	0.000000 0.000000 0.000000				
		135 Superim 136 Superim 137 Superim	0.000000 0.000000 0.000000	0.000000 0.000000 0.000000	2.035703 2.042036 2.044108	0.000000 0.000000 0.000000	0.000000 0.000000 0.000000	0.000000 0.000000 0.000000				
		138 Superim 139 Superim 140 Superim	0.000000 0.000000 0.000000	0.000000 0.000000 0.000000	2.044361 2.044237 2.044106	0.000000 0.000000 0.000000	0.000000 0.000000 0.000000	0.000000 0.000000 0.000000				
		141 Superim 142 Superim 143 Superim	0.000000	0.000000	2.044080 2.044192 2.044415	0.000000	0.000000	0.000000				
		143 Superim 144 Superim 145 Superim	0.000000 0.000000 0.000000	0.000000	2.044415 2.044614 2.044315	0.000000	0.000000	0.000000				
		146 Superim 147 Superim 148 Superim	0.000000 0.000000 0.000000	0.000000 0.000000 0.000000	2.042078 2.034411 2.015072	0.000000 0.000000 0.000000	0.000000 0.000000 0.000000	0.000000 0.000000 0.000000				
		149 Superim 150 Superim 151 Superim	0.000000 0.000000 0.000000	0.000000 0.000000 0.000000	1.978064 1.926268 1.872966	0.000000 0.000000 0.000000	0.000000 0.000000 0.000000	0.000000 0.000000 0.000000				
		152 Superim 153 Superim 154 Superim	0.000000	0.000000	1.816339 1.706552	0.000000	0.000000	0.000000				
		154 Superim 155 Superim 156 Superim	0.000000 0.000000 0.000000	0.000000	1.466562 1.106599 0.013071	0.000000	0.000000	0.000000				
		160 Superim	0.000000 FX	0.000000 SUMMATION OF RE FY	1.818843 ACTION FORCES PRI FZ	0.000000 NTOUT	0.000000	0.000000				
		Superim	(KN) 0.000000	(kN) 0.000000	(kN) 102.158523							
	Vertification			MIDAS	Comparise	<u>on</u>						
	MIDAS Calculated	_	102 16	L	N							
		=	102.10	K L	N							
	Rereantage Difference	_	1.64	0								
	Fercentage Difference	=	1.04	7	0	(UK)						

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	<u>Capacity</u> Moment Resistance	of Carriagew	ay Slab								•	•
	Bar Type	Bar Dia (")	Bar Dia	(mm)	Area o	of Bar (mm ²)	Spacing (") C	Centre to	Spacing	g (mm) C	entre to]
	Longitudinal Bottom	9/16"	14.3	3		160.61	4	e		101.6		-
	Bar	<u> </u>										J
	Grillage Model	Dims										
	Note: Only the 2	250mm deep s	alab consider	ed for M _u								
	Width of section		=	4	156.00	mm						
	Height of section	ı	=		250	mm						
	Cross Sectional	Area	=	1	14000	mm ²						
	Total number of section width	tension bars pe	er =		4.49							
	Area of tension r section width	einforcement p	er =	721		mm²						
	Cover to reinford	cement	=	1 1/2	"	=	38 mm (A	Assumed)				
	Effective Depth	d	_	2	04 95	mm						
CS 455 Eq		u		2	104.05							
US 433, LY	z =	$\left(1-rac{0.84rac{f_{y\cdot}}{\gamma_n}}{rac{f_{cubd}}{\gamma_{mc}}} ight)$	$\left(\frac{A_s}{a_s}\right)d$		=	178.4	h mm					
	Should be less ti Take Z	han 0.95d			= =	194.61 178.4	mm (OK) mm					
	Moment resistar	ice equation										
CS 455, Eq	$M_u = \mathrm{mi}$	$\ln \left\{ \begin{array}{c} \frac{f_y}{\gamma_{ms}} A_s \\ \frac{0.225 f_{cu}}{\gamma_{mc}} b \end{array} \right.$	z pd^2									
	M _u =	26 kNi	m									
	M _u =	43.05	kNm									
	Therefore mome	ent reistance		=	26	kNm						
	Transverse Directio	n										
1		Bar Dia (")	Bar Dia	(mm)	Ara o	f Bar (mm²)	Spacing (") C	Centre to	Spacing	g (mm) C	entre to]
	Bar Type	Dui Dia ()					Centi	e	1	Centre		

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Section	Approach Span 1 - West Carriageway Slab		Checker	CAT II	Date	May-24		
	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 m	าฑ					
	Effective depth, d	= 191.7 mm						
\$ 455, Eq 5.2	Z = $\left(1 - \frac{0.84 rac{f_y A_s}{\gamma_{ms}}}{rac{f_{cubd}}{\gamma_{mc}}} ight)d$	= 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						
	$V_{\text{max}} = 0.36 \left(0.7 - \frac{f_{cu}}{2} \right) \left(\frac{f_{cu}}{2} \right) b_{ud}$							
	$(250)(\gamma_{mc})^{-\omega}$							
	b_w is the breadth of the section, tak	en as the web width for flanged beams						
	$V_{max} = 215.22 \text{ kN}$							
	Shear resistance more than 3d from the sur	nort						
	Equation 5.66 Shear resistance more t 0.24 ± 1	han 3d from a support						
	$V_{uc} = \frac{1}{\gamma_{mv}} \xi_s \rho_s^3 f_{cu}^3 b_{w} d$							

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	Where: γ_{mv} ξ_s $ ho_s$	is the partial is the depth $\xi_s = \left(\frac{500}{d}\right)^{c}$ is the ratio $\rho_s = \frac{100A_s}{b_w d}$	I factor for s factor, take 0.25 but not le of longitudin but not less	hear d n as ess tha al reinf than (efined in S in 0.7 orcement 0.15, nor g	Section	n 2. than 3.0	0		3D =	618 mm				
	A_s	is the area beyond the support. Where top which produ	of longitudin section beir and bottom uces the she	al tens ig cons reinfor ar forc	ion reinfor sidered, ar cement ar se is to be	ceme nd, for re prov used.	nt that c shear a ⁄ided, th	ontinues at least a distance of t supports, continues at least e area in tension under the lo	d t to the pading						
	ξ _s	=	1.25		<	0.7	(OK)								
	ρs	=	0.69												
	V _{uc}	=	27.93	k	N										
	Shear resis	stance within 3d	of the support												
	$V_u = \mathbf{r}$ where	$\max \begin{cases} \frac{3}{a} \\ \frac{0.24}{\gamma_{mv}} \xi_s(0) \end{cases}$	$\frac{d}{v}\Gamma V_{uc}$ 0.15 f_{cu}) ^{$\frac{1}{3}$} $b_w d$ nce of the secti earing or the fa r to account for	on meas ce of a s short ar	ured from the upport, wher ichorage leng	e edge e $d \leq a$ gths, de	of a rigid b $_{y} \leq 3d$ fined in Ec	earing, the centre-line of quation 5.6d							
	a _v =	614.55 r	nm	or 3	b										
	T =	$\sqrt{\frac{z}{3d}\frac{F_{ub}}{V_{uc}}}$													
	fub	$f_{ub} = \frac{kk_a}{k_a}$	$rac{coveta\sqrt{f_{cu}}}{\gamma_{mb}}$												
	K =	1		a	con	=	0.40								
	β =	0.39		С		=	52.3	mm							
	fcu =	15 N	√mm²	ф		=	14.3	mm							
	γ _{mb} =	1.25		k	cov	=	1								
	fub =	1.21													
	Fub =	$F_{ub} = f_{ub}p.$	L_{α}												
	p =	$p=\pi\phi$ for a	single bar,	= 4	4.92	mm									
	L _a =	Length taken as	3d / 2	= 3	07.275	mm									
	Fub =	16680.67	4	or 1	6.68	kN									
	Г =	$\Gamma = \min \bigg\{$	$ \sqrt{\frac{z}{3d} \frac{F_{ub}}{V_{uc}}} \\ 1.0 $	=	7388816583 0.086	>	1								

G	<u>ob</u>	5				C	ALCULA	ATION	SHEE	=
e	Mancheste	r First Street					Page No.	34	Calc No.	
Title	ECC - Asse	essment of Station W	Vay				Calcs by	JF	Date	May-2
on	Approach S	Span 1 - West Carria	geway Slab				Checker	CAT II	Date	May-2
	V _u =	$rac{3d}{a_v} \Gamma V_{uc}$		2.40	kN					
	=	$\frac{0.24}{2}\xi_s(0.15f_{cu})$	$\frac{1}{3}b_wd =$	31929.48	Ν					
		ine		31.93	kN					
	Shear R	esistance - Trai	nsverse Dire	ction						
	Maximun	n shear resistan	ce based on (concrete crus	hing					
	Equ	ation 5.6a Maxir	mum shear re	sistance bas	ed on concrete crushing					
	$V_{ m ma}$	x = 0.36(0.7 -	$-\frac{f_{cu}}{250}\left(\frac{f_{cu}}{\gamma}\right)$	$b_w d$						
	wh	ere:	250) (me							
		b_w is the bre	eadth of the se	ction, taken a	s the web width for flanged beams					
	V _{Max}	=	187.71							
	Shear re	sistance more th	nan 3d from th	ne support						
	Equation	on 5.6b Shear	resistance	more than	3d from a support					
	$V_{uc} =$	$\frac{0.24}{\gamma_{mv}}\xi_s \rho_s^{\frac{1}{3}} f_{cu}^{\frac{1}{3}} b$	$b_w d$							
	Where:									
	γ_{mv}	is the partial f	actor for she	ar defined in	Section 2.					
	$\xi_{\bar{s}}$	is the depth fa $\xi_s = \left(\frac{500}{d}\right)^{0.25}$	actor, taken a ⁵ but not less	than 0.7						
	ρ_s	is the ratio of $100A$ by	longitudinal r	einforcemen	t greater than 3.0					
	As	$p_s = \frac{1}{b_w d}$ of	longitudinal t	ension reinfo	provident that continues at least a di	istance d				
		beyond the se support. Where top ar which produce	ection being (nd bottom rei es the shear	considered, a nforcement a force is to be	and, for shear at supports, continues are provided, the area in tension und a used.	s at least to the der the loading				
	ξs	=	1.27	<	0.7 OK					
	ρs	=	0.42							

JAC	OBS 0	ALCULA	TION	SHEE	T
Office	Manchester First Street	Page No.	35	Calc No.	
Job No. & Title	ECC - Assessment of Station Way	Calcs by	JF	Date	May-24
Section	Approach Span 1 - West Carriageway Slab	Checker	CAT II	Date	May-24

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

JAC	OBS				C	ALCULA		SHEE	т
Office	Manchester First Street					Page No.	36	Calc No.	
Job No. & Title	ECC - Assessment of Stat	ion Way		Calcs by	JF	Date	May-24		
Section	Approach Span 1 - West C	Carriageway Slab		Checker	CAT II	Date	May-24		
	Length (mm)	Length (m)	W (kNm)	Med (kNm)	MIDAS Results (kNm)	Section Res (kNm	istance)	Resu	lt
	425.6	0.4256	0.5	12.10)	PASS	S		
	851.1	0.8511	1.92	0.17	0.8	12.10)	PASS	6

0.39

0.69

1.00

1.00

PASS

PASS

PASS

12.10

12.10



MIDAS Result Screenshot

Load Combination 1 - Deadload and Superimposed Deadload

1.2767

1.7023

1.92

1.92

1276.7

1702.3

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

JAC	OBS C	ALCULA	TION	SHEE	Т
Office	Manchester First Street	Page No.	37	Calc No.	
Job No. & Title	ECC - Assessment of Station Way	Calcs by	JF	Date	May-24
Section	Approach Span 1 - West Carriageway Slab	Checker	CAT II	Date	May-24

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

Office	UD5				CALCULA	ATION	SHEF			
onice	Manchester First Street				Page No.	38	Calc No.			
o No. & Title	ECC - Assessment of Station	n Way			Calcs by	JF	Date	May-2		
Section	Approach Span 1 - West Car	riageway Slab			Checker	CAT II	Date	May-		
	Live Loading - All Mo	<u>del 1</u>								
	Note: ALL Model 2 will	not be applie	ed to the carriageway slab due to th	he element complying with	section 5.6 noted 4 o	f CS 454.				
	Impact Factors and L	ane Widths	0,							
				Net and lane width	M					
	Load Situation	Impact	Poor Road Surface	(m)	centres of a	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 Cat	egory	Traffic Flow Factor	7						
	Low		0.9							
	Lane Factors for ALL	Model 1								
	Lane		Lane Factor							
	Lane 1		1	_						
F	Lane 3		0.5	-						
	Lane 4 and		0.4							

JA	COBS						CALCULA	TION S	HEET	
Office	Manchester First Street						Page No.	39	Calc No.	
Job No. & Title	ECC - Assessment of Station Way						Calcs by	JF	Date	May-24
Section	South West Footway Slab (Span 1)						Checker	CAT II	Date	May-24
	Material & Section Properties									
	Reinforced Concrete									
CS 455,	Characteristic Strength of Concrete				=	20 N/	/mm ²			
CI.3.1.3 CS 454 table	Density of Reinforced Concrete				=	24 ki	N/m ³			
4 1 1a	Steel Reinforcement									
CS455,	Mild steel reinforcement characteristic yield strer	igth					= 250	N/mm ²		
CI, 3.8.2	Design Young's Modulus of steel reinforcement						= 210000	N/mm ²		
	Mass Concrete (Plain Concrete)									
CS 454	Unit weight of plain concrete				=	23 kN/m ³				
table	Fill - Miscellaneous									
US 454 table	Unit weight of miscellaneous fill				=	22 kN/m ³				
4119	Carriageway & Footway Surfacing									
CS 454 table	Unit weight of Bituminous Macadam (tar)				=	24 kN/m ³				
4 1 1a	Partial Factors									
Table	Partial Factor for reinforcement			γms	=	1.15				
2.13a Table	Partial Factor for Concrete			γmc	=	1.5				
2.13a Table	Partial Factor for shear in concrete			γmv	=	1.15				
2.13a CS 454	Partial Factor for Concrete Deadload				=	1.15				
Table A.1 CS 454	Partial Factor for Surfacing				=	1.75				
Table A.1 CS 454	Partial Factor for Fill				=	1.2				
Table A.1	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 Height of fill min	us surfacing			= 40.05 = 39.95	m m		
			Height of top of Average depth of	upstand of upstand			= 39.085 = 0.095	m m		
			Thickness of Fill				= 0.960	m		
	Slab Dimensions									
	Skew Span =	3.12	m							
	Square Span = Slab Depth =	2.13 0.25	m m							
	Skew of slab =	43	0							
		2.34	m							
	× 40 40 47 47 47				ũ	2340]	
	A			Unstand	390 39	0 390 3	90 390 390	linend		
	Supported and Market Science S			same of the second seco		+ +	+ +			
	Togethis or Restroyed			250 to	0 0	0 0				
	A A A A A A A A A A A A A A A A A A A				S Trans	Section A-A	in the			
						ongitudinal Member	5			
	428 435 433 433 536 536									
	See Sport - 3120 Plan on South Sport - 100x01 Fourture State									
	<u>.</u>									

Anchester First Street C - Assessment of Station Way uth West Footway Stab (Span 1) ember Dimensions. natitudinal Edge Memebers Self Weight Edge Member If weight of section section waction due to self weight natitudinal Member & Self Weight	A (m) 0.345		D B (m) 0.390 2.91 3.35 10.44	kN/m kN/m kN/m	C (m) 0.250 (unfactored) (factored) (factored)	D (m) 0.250 per full	Page No. Calcs by Checker Area (m²) 0.12125	40 JF CAT II	Calc No. Date Date	May- May-
C - Assessment of Station Way uth West Footway Slab (Span 1) ember Dimensions naitudinal Edge Memebers Self Weight Edge Member If weight of section H weight of section vaction due to self weight naitudinal Member & Self Weight	A (m) 0.345	=	D B (m) 0.390 2.91 3.35 10.44	kN/m kN/m kN/m	C (m) 0.250 (unfactored) (factored) (factored)	D (m) 0.250 per full	Calcs by Checker	JF CAT II	Date	May- May-
uth West Footway Slab (Span 1) ember Dimensions natitudinal Edge Memebers Self Weight Edge Member If weight of section If weight of section waction due to self weight natitudinal Member & Self Weight	A (m) 0.345	A = =	D B (m) 0.390 2.91 3.35 10.44	kN/m kN/m kN/m	C (m) 0.250 (unfactored) (factored) (factored)	D (m) 0.250 per full	Area (m ²) 0.12125	CAT II	Date	May-
Amber Dimensions natitudinal Edge Memebers Self Weight Edge Member If weight of section weight of section waction due to self weight natitudinal Member & Self Weight	A (m) 0.345		D B B (m) 0.390 2.91 3.35 10.44	kN/m kN/m kN	C (m) 0.250 (unfactored) (factored) (factored)	D (m) 0.250 per full	Area (m²) 0.12125			
Section Edge Member If weight of section If weight of section If weight of section ueight of section vaction due to self weight naitudinal Member & Self Weight	A (m) 0.345	=	D B (m) 0.390 2.91 3.35 10.44	kN/m kN/m kN/m	C (m) 0.250 (unfactored) (factored) (factored)	D (m) 0.250 per full	Area (m²) 0.12125			
Section Edge Member If weight of section If weight of section eaction due to self weight ngitudinal Member & Self Weigh	<u>A (m)</u> 0.345	=	B (m) 0.390 2.91 3.35 10.44	kN/m kN/m kN	C (m) 0.250 (unfactored) (factored) (factored)	D (m) 0.250 per full	Area (m²) 0.12125			
Section Edge Member	A (m) 0.345	=	B (m) 0.390 2.91 3.35 10.44	kN/m kN/m kN	C (m) 0.250 (unfactored) (factored) (factored)	D (m) 0.250 per full	Area (m ²) 0.12125			
Section Edge Member	<u>A (m)</u> 0.345 <u>t</u>	=	B (m) 0.390 2.91 3.35 10.44	kN/m kN/m kN	C (m) 0.250 (unfactored) (factored) (factored)	D (m) 0.250 per full	Area (m ²) 0.12125			
Eagle Member	<u>t</u>	= =	2.91 3.35 10.44	kN/m kN/m kN	(unfactored) (factored) (factored)	0.250 per full	length of longitudinal membe			
If weight of section If weight of section vaction due to self weight ngitudinal Member & Self Weigh	<u>t</u>	= =	2.91 3.35 10.44	kN/m kN/m kN	(unfactored) (factored) (factored)	per full	length of longitudinal membe			
If weight of section action due to self weight naitudinal Member & Self Weigh	<u>t</u>	=	3.35 10.44	kN/m kN	(factored) (factored)	per full	length of longitudinal membe			
action due to self weight ngitudinal Member & Self Weigh	<u>t</u>	=	10.44	kN	(factored)	per full	length of longitudinal membe			
ngitudinal Member & Self Weigh	<u>t</u>							er		
				В						
				\bigcirc						
		A		\bigcirc						
			↓ L							
	L									
Section	A (m)	<u> </u>	B (m)		Area (m ²)	1				
Edge Member	0.250		0.390		0.098					
If weight of section		=	2.34	kN/m	(unfactored)	I				
If weight of section		=	2 69	kN/m	(factored)					
tal Reportion due to colf weight			2.00	LNI	(factored)	por full	longth of longitudinal mambe			
tal Reaction due to self weight		=	8.40	KIN	(factored)	per full	length of longitudinal membe	er		
tal reactions per member grou	up for self	weight	_							
ngitudinal Upstand Member		=	20.88 kN							
ngitudinal Member		=	33.58 kN							
tal Rections		_	54.47 kN							
		-	54.47 KN							
das Reactions		-	54.48 kN							
n nt d	Section Edge Member f weight of section al Reaction due to self weight al reactions per member grou gitudinal Upstand Member gitudinal Member al Reactions as Reactions	Section A (m) Edge Member 0.250 f weight of section	Section A (m) Edge Member 0.250 f weight of section = al Reaction due to self weight = al reactions per member group for self weight igitudinal Upstand Member = al Rections = al Rections =	Section A (m) B (m) Edge Member 0.250 0.390 f weight of section = 2.34 f weight of section = 2.69 al Reaction due to self weight = 8.40 al reactions per member group for self weight = 8.40 al reactions per member group for self weight = 33.58 kN igitudinal Upstand Member = 54.47 kN as Reactions = 54.48 kN	Section A (m) B (m) Edge Member 0.250 0.390 f weight of section = 2.34 kN/m f weight of section = 2.69 kN/m al Reaction due to self weight = 8.40 kN al reactions per member group for self weight = 20.88 kN igitudinal Upstand Member = 33.58 kN al Reactions = 54.47 kN as Reactions = 54.48 kN	Section A (m) B (m) Area (m ²) Edge Member 0.250 0.390 0.098 f weight of section = 2.34 kN/m (unfactored) if weight of section = 2.69 kN/m (factored) al Reaction due to self weight = 8.40 kN (factored) al reactions per member group for self weight = 20.88 kN gitudinal Upstand Member = 33.58 kN al Rections = 54.47 kN as Reactions = 54.48 kN	Section A (m) B (m) Area (m ²) Édge Member 0.250 0.390 0.098 f weight of section = 2.34 kN/m (unfactored) f weight of section = 2.69 kN/m (factored) al Reaction due to self weight = 8.40 kN (factored) per full al reactions per member group for self weight = 20.88 kN gitudinal Upstand Member = 23.58 kN agitudinal Member = 33.58 kN aa Reactions = 54.47 kN as Reactions = 54.48 kN	Section A (m) B (m) Area (m ²) Édge Member 0.250 0.390 0.098 f weight of section = 2.34 kN/m (unfactored) if weight of section = 2.69 kN/m (factored) al Reaction due to self weight = 8.40 kN (factored) per full length of longitudinal member al reactions per member group for self weight = 20.88 kN gitudinal Upstand Member = 30.58 kN al Rections = 54.47 kN sa Reactions = 54.48 kN	Section A (m) B (m) Area (m') Edge Member 0.250 0.390 0.098 Iweight of section = 2.34 kN/m (unfactored) iweight of section = 2.69 kN/m (factored) al Reaction due to self weight = 8.40 kN (factored) preactions per member group for self weight = 20.88 kN gludinal Upstand Member = 33.58 kN al Reactions = 54.47 kN as Reactions = 54.48 kN	Section A (m) B (m) Area (m) Édge Member 0.250 0.390 0.098 Iweight of section = 2.34 kN/m (unfactored) Iweight of section = 2.69 kN/m (factored) al Reaction due to self weight = 8.40 kN (factored) gludinal Upstand Member = 20.88 kN as Reactions = 54.47 kN al Reactions = 54.48 kN Hertorian Hertorian Hertorian

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Ma	anchester First Stre	et							Pag	ge No.	41	Calc No.
& EC	CC - Assessment of	Station Way							Cal	lcs by	JF	Date
So	outh West Footway	Slab (Span 1))						Ch	ecker	CAT II	Date
			Node Load 1 Deadload 2 Deadload 3 Deadload 4 Deadload 5 Deadload 6 Deadload 43 Deadload 44 Deadload 45 Deadload 46 Deadload 47 Deadload 48 Deadload	FX (kN) 0.000000 0.000000 0.000000 0.000000 0.000000	FY (kN) 0.000000 0.000000 0.000000 0.000000 0.000000	FZ (kN) 1.925334 3.615398 3.602300 3.963500 -0.182375 14.145091 0.057596 3.93980 3.9398135 3.592573 1.915174 4.CTOLE CE S2	MX (kl*m) 0.000000 0.000000 0.000000 0.000000 0.000000	MY (kN*m) 0.000000 0.000000 0.000000 0.000000 0.000000	MZ (kN*m) 0.000000 0.000000 0.000000 0.000000 0.000000			
			Load	FX (kN)	FY (kN)	FZ (kN)						
			Deadload	0.000000	0.000000	54.480311						
						Midas Reaction	Output					
			250mm		2400mr Section three	n pugh Span		<u>J</u>				
3		UPSTAND	1500 July 1500 J	Mp 53	2400mr Section three	n pough Span		N7 N6 V7 W6 V 7 D7 A6 D5 A	N5 V5 W4 5 D5 A4 D	N4 N3 W3 4 A3 D3	N2 W2 A2 D2 :	NI WI D
	pstand Area - T	UP\$74N0 152 1850 16 Tapering S	250mm	Mp 53 250 Stab BB F2 +Mk51	2400mr	n bugh Span		N7 N6 V7 W6 V 7 D7 A6 D5 A	N5 V5 W4 5 D5 A4 D	N4 N3 W3 4 A3 D3	N2 W2 4 A2 D2	
22 Up	pstand Area - 1 Node	UPSTAND MSS JBSD IC Tapering S	250mm U2 U2 U2 U2 U2 U2 U2 U2 U2 U2 U2 U2 U2	HL 53 250 Stab B F2Mk51 Wid	2400mr Section three	n bough Span		N7 N6 W7 W6 V 7 D7 A6 D5 A Density of Cc	N5 V5 W4 5 D5 A4 D	N4 N3 W3 4 A3 D3 Unit We	N2 W2 A2 D2 eight (kN/	
	pstand Area - 1 Node	UP\$TANO M357 1850 Tapering S	250mm	HL-53 250 Stab B F2 -Mk51 Wid	2400mr Section three Section three th (m) 250	n bugh Span	N8 D8 A	N7 N6 N7 W5 V 7 D7 A6 D5 A Density of Cc 24 24	N5 W4 5 D5 A4 D	N4 N3 W3 4 A3 D3 Unit We	N2 W2 A2 D2 eight (kN/ 0.165 0.495	
	pstand Area - T Node 1 2 3	UP\$TANO	100 100 100 100 100 100 100 100 100 100	HL 53 250 Stab B F2Mk51 0.0.0.0.0.0.0.0.0.0.0.0.0.0.0.0.0.0.0.	2400m/ Section thro Section thr	n bugh Span Area (n 0.0068 0.02067 0.02067 0.02087	1) 1) 1) 1) 1) 1) 1) 1) 1) 1)	N7 N6 V7 W6 V 7 D7 A6 D5 A Density of Cc 24 24 24	NS VS W4 5 DS A4 D	N4 N3 4 A3 D3 Unit W	N2 W2 A2 D2 eight (kN/ 0.165 0.495 0.789	N1 A1 D
	pstand Area - 1 2 3 4	UP\$7440	250mm	Wid Wid Wid 0. 0. 0.	2400m/ Section three	n bugh Span	N8 V D8 A A A A A A A A A A A A A A A A A A	N7 N6 V7 W6 V 7 D7 A6 D5 A Density of Cc 24 24 24 24 24	v5 W4 5 D5 A4 D	N4 N3 4 A3 D3 Unit W	N2 W2 A2 D2 eight (kN/ 0.165 0.495 0.495 0.789 1.062	N1 A1 D
	230 23 250 23 250 23 250 23 250 24 25 250 24 25 250 24 25 25 25 25 25 25 25 25 25 25 25 25 25	UP\$7AND M552 16 Tapering S	250mm 100 100 100 100 100 100 100	Ma 53 250 Stab Ba F2 - Mik51 Concentration (Concentration) (Co	2400m/ Section three Section three th (m) 250 250 250 250 250	Area (n 0.0068 0.02063 0.02063 0.02063 0.02063 0.02063 0.02063 0.02063 0.02063 0.02063 0.02063 0.02063 0.02063 0.02063	NB DB A 75 75 75 75 75 75 75 75 75 75	N7 N6 V7 V6 V 7 D7 A6 D5 A Density of Cc 24 24 24 24 24	v5 V4 5 D5 A4 D	N4 N3 W3 4 A3 D3 Unit W	N2 N2 A2 D2 eight (kN/ 0.165 0.495 0.789 1.062 1.335	N1 A1 D
	233 24 24 24 24 24 24 24 24 24 24 24 24 24	UP\$TAN0	190 190 190 190 190 190 190 190	Mk 53 250 St ab B F2 Mk51 0.0.0.0.0.0.0.0.0.0.0.0.0.0.0.0.0.0.0.	2400m/ Section three Section three th (m) 250 250 250 250 250 250 250	Area (n 0.0068 0.02062 0.0328 0.0442 0.05562 0.0688	N8 V D8 A 75 25 75 5 25 75 5 25	N7 N6 V7 V6 V 7 D7 A6 D5 A Density of Cc 24 24 24 24 24 24 24	vs w4 5 D5 A4 D	N4 N3 4 A3 D3 Unit W	N2 W2 A2 D2 eight (kN/ 0.165 0.495 0.789 1.062 1.335 1.605 4.927	N1 A1 D
	250 1 pstand Area - 1 Node 1 2 3 4 5 6 7 8	UPSTAND VISTO	1990 1990	HL 53 250 St ab F2 Wid 0. 0. 0. 0. 0. 0. 0. 0. 0. 0.	2400mr Section three Section t	Area (n 0.0068 0.0328 0.0442 0.0556 0.0685 0.0685 0.0685 0.0685 0.0685 0.0685 0.0685 0.0685 0.0685 0.0685 0.07811	N8 V D8 75 75 75 75 75 75 75 75 75 75 75 75 75	N7 N6 V7 V6 V 7 D7 A6 D5 A Density of Cc 24 24 24 24 24 24 24	N5 W4 5 D5 A4 D	N4 N3 W3 4 A3 D3 Unit W	N2 W2 A2 D2 eight (kN/ 0.165 0.495 0.789 1.062 1.335 1.605 1.875	N1 A1 D
	255 6 7 8 aximum loads	UPSTAND VIPSTAN	250mm 250mm 400 400 400 400 400 400 400	Wid B F2 Wid 0.0 0.0 0.0 0.0 0.0 0.0 0.0 0.0 0.0 0.0 0.0 0.0 0.0 0.0 0.0 0.0 0.0 0.0 0.0	2400mr Section three Section t	Area (n 0.0068 0.0206 0.03287 0.04683 0.02066 0.02066 0.02066 0.02066 0.02066 0.02066 0.02066 0.06883 0.07813 Σ	N8 V D8 75 75 75 75 75 75 75 75 75 75 75 75 75	N7 N6 V7 W6 V 7 D7 A6 D5 A Density of Cc 24 24 24 24 24 24 24 24 24 24	vs w4 s D5 A4 D	N4 N3 4 A3 D3	N2 W2 A2 D2 eight (kN/ 0.165 0.495 0.789 1.062 1.335 1.605 1.875	N1 M1 D
	pstand Area - 1 Node 1 2 3 4 5 6 7 8 aximum loads Node	UP\$TANO VP\$TA	250mm 250mm 400 407 407 407 407 407 407 407	Wa 53 250 51 ab 250 51 ab F2 Wid 0.0 0.0 0.0 0.0 0.0 0.0 0.0 0.0 0.0 0.0 0.0 0.0 0.0 0.0 0.0 0.0 0.0 0.0 0.0	2400m/ Section three Section t	Area (n 0.0068 0.02066 0.02066 0.0328 0.038 0.0328	N8 V D8 A 75 25 75 25 75 25 0.30525	N7 N6 V V7 V6 V 7 07 A6 D5 A Density of Cc 24 24 24 24 24 24 24 24 24 24	vs w4 5 D5 A4 D oncrete	N4 N3 4 A3 D3 Unit W	N2 W2 A2 D2 eight (kN/ 0.165 0.495 0.789 1.062 1.335 1.605 1.875 e of upstar	N1 A1 D
	pstand Area - 1 Node 1 2 3 4 5 6 7 8 aximum loads Node 1	UPSTAND MISS2 ISP50 IC IC IC IC IC IC IC IC IC IC	1990 1990	Wid B3 F2 Wid 0.0 0.0 0.0 0.0 0.0 0.0 0.0 0.0 0.0 0.0 0.0 0.0 0.0 0.0 0.0 0.0 0.0 0.0 0.0 0.0 0.0 0.0 0.0 0.0 0.0 0.0 0.0	2400mr Section three Section t	Area (n 0.042 0.0422 0.0422 0.0556: 0.0685: 0.0781: Σ ering T	1) 1) 10 10 10 10 10 10 10 10 10 10	N7 N6 V7 W6 V 7 D7 A6 D5 A Density of Cc 24 24 24 24 24 24 24 24 24 24 24 24 24	vs vs vv s ps A4 p oncrete	N4 N3 W3 4 A3 D3 Unit W	N2 W2 A2 D2 eight (kN/ 0.165 0.495 0.789 1.062 1.335 1.605 1.335 1.605 1.335 2.69	N1 A1 D
	pstand Area - 1 Node 1 2 3 4 5 6 7 8 aximum loads Node 1 3 5 6 7 8 aximum loads	UPSTAND VPS	250mm 250mm 250mm 250mm 250mm 250mm 250mm 270m 200m	Wid B F2 Wid 0.0	2400mr Section thro Section thr	Area (n 0.0068 0.02062 0.02062 0.02062 0.02063 0.02063 0.02063 0.02063 0.02063 0.02063 0.02063 0.05565 0.06683 0.07813 Σ ering T	N8 V D8 X 75 5 25 75 5 25 0.30525	N7 N6 V7 W6 V 7 D7 A6 D5 A Density of CC 24 24 24 24 24 24 24 24 24 24 24 24 24	vs w4 s bs A4 b	N4 N3 4 A3 D3 Unit Weight	N2 W2 A2 D2 eight (kN/ 0.165 0.495 0.789 1.062 1.335 1.605 1.875 t of upstar 2.69 3.44 4.07	M1 A1 D
	pstand Area - 7 Node 1 2 3 4 5 6 7 8 aximum loads Node 1 3 5 8	UP\$TAND Tapering S	250mm 250mm 250mm 250mm 250mm 250mm 250mm 250mm 200 200 200 200 200 200 200	Wid B F2 Wid 0.0	2400m/ Section thro Section thro Section thro th (m) 250 250 250 250 250 250 250 250 250 250	Area (n 0.0068 0.0206 0.0206 0.0206 0.0206 0.0206 0.0206 0.0328 0.0442 0.0566 0.0568 0.0668 0.07812 Ering T	1) 1) 1) 1) 1) 1) 1) 1) 1) 1)	N7 N6 V7 V5 V 7 D7 A6 D5 A Density of Cc 24 24 24 24 24 24 24 24 24 24	vs w4 s DS A4 D	N4 N3 4 A3 D3 Unit Weight	N2 W2 A2 D2: A2 D2: eight (kN/ 0.165 0.495 0.789 1.062 1.335 1.605 1.875 2.69 3.44 4.07 5.00	m)


COB	5								CAL	COLAI	ION 3	HEEI	
Manchester First Stre	et								Paç	ge No.	43	Calc No.	
ECC - Assessment of	Station Way								Cal	lcs by	JF	Date	Ма
South West Footway	Slab (Span 1)								с	hecker	CATII	Date	Ма
Due to the varyin	gof the footway s	slab below is a si	ummary o	of the levels and	weight of fill	for superimposed o	deadload						
Element	Surface	Level (m)		Level at To	p of Member	' (m)	Depth	of Fill (r	n)	Unit We	eight (kN/n	n ³)	toro
South Upstand	39.	.9225		3	39.085		().7375			7.59		
Member	39.	.9225		:	38.99		().8325			8.57	Fac	tore
North Upstand	39.	.9225		3	39.741		(0.0815			0.84	Fac	tore
Surfacing													
Element	Surface	Level (m)		Width of Sec	tion	Unit Weight (I	kN/m³)]_					
South Upstand	(0.1		0.390		1.638		Factored	b				
Longitudinal Member	(0.1		0.390		1.638		Factored	b				
North Upstand	(0.1		0.390		1.638		Factored	d				
Total Superimpo	sed Deadload t	o be applied to	each m	ember		1		J					
Flement	Total	ad (kN/m ³)											
South Unstand		au (KN/M [*])	Factore	ed									
Longitudinal			Factore	ed									
Member	10	0.21	Factore	he									
÷	2	2.48	1 actore	50									
Additional Dead Note: Only the ca Elem North U	load from Carria rriageway downs nent pstand	ageway Slab or stand taken into Depth 0.406	n the Nor	rth Upstand ation for the cont <u>Width</u> 1	tribution to de	adload. Weight kN/m 9.744	Fact	ored Uni 1	t Weig h 1.21	nt (kN/m)			
Additional Dead Additional Dead Note: Only the ca Elem North U Section Capaciti	load from Carria mriageway downs nent pstand	ageway Slab or stand taken into Depth 0.406	n the Nor	r th Upstand ation for the conf <u>Width</u> 1	tribution to de	adload. Weight kN/m 9.744	Fact	ored Uni 1	t Weigh 1.21	<u>nt (kN/m)</u>			
North Upstand Additional Dead Note: Only the ca Elem North U Section Capaciti Moment Resistar	load from Carria rriageway downs nent pstand ies icce of Reinforced	ageway Slab or stand taken into Depth 0.406	n the Nor considera	rth Upstand ation for the conf Width 1	tribution to de	adload. Weight kN/m 9.744	Fact	ored Uni 1	t Weigh 1.21	<u>nt (kN/m)</u>			
Additional Dead Additional Dead Note: Only the ca Elen North U Section Capaciti Moment Resistar	load from Carria miningeway downs ment pstand ies nce of Reinforced ype	ageway Slab or stand taken into Depth 0.406 I Concrete longit Bar Dia (mm)	n the Nor considera tudinal me	rth Upstand ation for the conf Width 1 embers (longitud	tribution to de Unit	eadload. 9.744 9.744	(mm)	ored Uni 1	<u>t Weigh</u> 1.21	nt (kN/m)			
North Upstand Additional Dead Note: Only the ca Elen North U Section Capaciti Moment Resistar Bar t Top Compre	load from Carria mriageway downs nent pstand ies ice of Reinforced sype	ageway Slab or stand taken into Depth 0.406 I Concrete Iongit Bar Dia (mm) 16	n the Nor considera tudinal me	rth Upstand ation for the conf 1 embers (longitud Bar (mm ²) 201.06	tribution to de Unit linal direction Spacinç	eadload. 9.744 9.744 9.745 9.744	(mm)	ored Uni 1	<u>t Weigh</u> 1.21	<u>it (kN/m)</u>			
Additional Dead Additional Dead Note: Only the ca Elen North U Section Capaciti Moment Resistar Bart Top Compre- Bottom Ter	load from Carria miningeway downs ment pstand ies ies ince of Reinforced type ission Bars ission Bars	ageway Slab or stand taken into 0.406 I Concrete longit Bar Dia (mm) 16 25	n the Nor	rth Upstand ation for the conf 1 ambers (longitud Ear (mm ²) 201.06 490.87	tribution to de Unit tinal direction Spacing	eadload. 9.744 9.744 1 <u>0 Centre to Centre to Centre 150</u> 150	(mm)	ored Uni 1	<u>t Weigh</u> 1.21	<u>it (kN/m)</u>			
Additional Dead Additional Dead Note: Only the ca North U Section Capaciti Moment Resistar Bart Top Comprese Bottom Ter Grillage Model I	load from Carris minageway downs ment pstand ies ace of Reinforced yppe assion Bars assion Bars Dims	ageway Slab or stand taken into 0.406 I Concrete longit Bar Dia (mm) 16 25	tudinal me	rth Upstand ation for the cont <u>Width</u> 1 embers (longitud (Bar (mm ²) 201.06 490.87	tribution to de Unit	eadload. Weight kN/m 9.744 1 Centre to Centre 150 150	(mm)	ored Uni 1	t Weigh 1.21	<u>st (kN/m)</u>			
North Upstand Additional Dead Note: Only the ca Elen North U Section Capacit Moment Resistar Bar t Top Compre Bottom Ter Grillage Model I Width of section	load from Carria mriageway downs nent pstand ies ace of Reinforced ype pe pssion Bars nsion Bars Dims	ageway Slab or stand taken into 0.406 I Concrete longit Bar Dia (mm) 16 25 =	tudinal me	rth Upstand ation for the conf 1 ambers (longitud Ear (mm ²) 201.06 490.87 mm	tribution to de Unit tinal direction Spacinç	eadload. 9.744 9.744 1 <u>Centre to Centre</u> 150 150	(mm)	ored Uni 1	<u>t Weigh</u> 1.21	<u>st (kN/m)</u>			
North Upstand Additional Dead Note: Only the ca Elen North U Section Capacit Moment Resistar Bart 1 Top Comprese Bottom Ter Grillage Model I Width of section Height of Section	load from Carris miningeway downs ment pstand ies ies ince of Reinforced type ission Bars ission Bars ission Bars	ageway Slab or stand taken into 0.406 I Concrete longit Bar Dia (mm) 16 25 = =	tudinal me Area of 390 250	rth Upstand ation for the conf 1 1 embers (longitud Ear (mm ²) 201.06 490.87 mm mm	tribution to de Unit	eadload. Weight kN/m 9.744 1 Centre to Centre 150 150	(mm)	ored Uni 1	t Weigh 1.21	<u>ıt (kN/m)</u>			
Additional Dead Additional Dead Note: Only the ca Elen North U Section Capacit Moment Resistar Bart Top Compre Bottom Ter Grillage Model I Width of section Height of Section Cross Sectional A	load from Carria mriageway downs nent pstand ies ice of Reinforced type ession Bars insion Bars Dims	ageway Slab or stand taken into 0.406 I Concrete longit Bar Dia (mm) 16 25 = = =	tudinal me Area of 390 250 97500	rth Upstand ation for the conf 1 1 embers (longitud Ear (mm ²) 201.06 490.87 mm mm mm	tribution to de	eadload. 9.744 9.744 1 <u>0 Centre to Centre 150</u> 150	(mm)	ored Uni 1	t Weigh	<u>st (kN/m)</u>			
North Upstand Additional Dead Note: Only the ca Elen North U Section Capacit Moment Resistar Top Compre Bottom Ter Bottom Ter Grillage Model I Width of section Height of Section Cross Sectional A Total Number Co per section width	load from Carria miningeway downs ment pstand ies ince of Reinforced sype assion Bars insion Bars Dims	ageway Slab or stand taken into 0.406 I Concrete longit Bar Dia (mm) 16 25 = = = =	tudinal me Area of 390 250 97500 2.6	rth Upstand ation for the conf 1 1 embers (longitud Ear (mm [×]) 201.06 490.87 mm mm mm mm ² or	tribution to de Unit dinal direction Spacing	eadload. 9.744 9.744 9.744 9.744 9.744 9.744 150 150 150 8ars	(mm)	ored Uni 1	t Weigh	<u>st (kN/m)</u>			
North Upstand Additional Dead Note: Only the ca Elen North U Section Capacit Moment Resistar Top Compre Bottom Ter Grillage Model I Width of section Height of Section Height of Section Cross Sectional / Total Number Co per section width	load from Carris miningeway downs ment pstand ies nee of Reinforced yppe pession Bars nsion Bars Dims Area mpression Bars psion Bars	ageway Slab or stand taken into (Depth 0.406 Concrete longit Bar Dia (mm) 16 25 = = = = = = = =	an the Nor considera tudinal me Area of Area of 390 250 97500 2.6 2.6	rth Upstand ation for the cond 1 ambers (longitud Bar (mm ²) 201.06 490.87 mm mm ² or	tribution to de Unit tinal direction Spacing 2.5 2.5	eadload. Weight kN/m 9.744 1 Centre to Centre of 150 150 Bars Bars	(mm)	ored Uni 1	t Weigh	<u>st (kN/m)</u>			
North Upstand Additional Dead Note: Only the ca Elen North U Section Capacit Moment Resistar Top Compre Bottom Ter Bottom Ter Bottom Ter Grillage Model I Width of Section Height of Section Height of Section Cross Sectional A Total Number Co per section width Total Number Ter section width Area of compress reinforcement per	Ioad from Carris miningeway downs ment pstand ies ies ince of Reinforced type assion Bars ision Bars Dims Area mpression Bars pr sion Bars per sion Bars per	ageway Slab or stand taken into 0.406 I Concrete longit Bar Dia (mm) 16 25 = = = = = = =	In the Nor consideration tudinal me Area of 390 250 97500 2.6 2.6 502.65	rth Upstand ation for the cont 1 1 embers (longitud Ear (mm ²) 201.06 490.87 mm mm mm or or or or	tribution to de Unit dinal direction Spacing 2.5 2.5	eadload. Weight kN/m 9.744 1 Centre to Centre 150 150 Bars Bars Bars	(mm)	ored Uni 1	t Weigh	<u>st (kN/m)</u>			
North Upstand Additional Dead Note: Only the ca Elen North U Section Capacit Moment Resistar Bar Top Compression Bottom Ter Grillage Model I Width of section Height of Section Cross Sectional A Total Number Coper section width Area of compress reinforcement per Area of tension reper section	Ioad from Carria rriageway downs rent pstand ies ace of Reinforced ype assion Bars hsion Bars Dims Area mpression Bars pression Bars pression Bars hsion Bars per sion r section	ageway Slab or stand taken into (0.406 I Concrete longit Bar Dia (mm) 16 25 = = = = = = = = = = = = =	In the Nor considera tudinal me Area of 390 250 97500 2.6 502.65 1276.27	rth Upstand ation for the cond width 1 ambers (longitud Ear (mm ²) 201.06 490.87 490.87 mm mm mm ² or or or mm ² mm ²	tribution to de Unit tinal direction Spacing 2.5 2.5	eadload. 9.744 9.744 150 150 Bars Bars	(mm)	ored Uni	<u>t Weig</u>	tt (kN/m)			

JA	CO	BS									CAL	CULA	TION S	HEET	
Office	Manchester F	irst Street									Pa	age No.	44	Calc No.	
Job No. & Title	ECC - Assess	ment of Static	on Way								C	alcs by	JF	Date	May-24
Section	South West Fo	ootway Slab (Span 1)								0	Checker	CAT II	Date	May-24
	Bending e E	Equation 5.2 Equation 5.2 $M_{\mu} = \frac{0.6f_c}{\gamma_{mc}}$ where: A'_a is	2.2c Momen $\frac{u}{d}bx(d-0)$ is the area of	compression r tresistance for b $(5x) + f'_s A'_s (d -$ t compression rein	einforceme eams with d) forcement	ent compression n	einforcement								
		d is f _n is	s the depth t s equal to $\frac{1}{7}$ s the neutral	to the compression $\frac{f_0}{\frac{f_0}{2\pi m}}$ (axis depth, which	may be deter	ent from the extr	reme fibre in comp	pression							
	x d' f's d	= = = =	125 46 196.08 220 78574400	mm (re mm mm mm	ectangular s or	section) 78.57	7 kN/m								
	Bending E Equation 5 $M_u = \min$ where: M_u f_y	Equation Co 5.2.2a Moment $n \begin{cases} \frac{fy}{2}, fy$	considering ent resistant s^2 bd^2 ment resistart acteristic or	g Tension Reinf nce for beams wi nce worst credible str	orcement (thout comp ength of the	Donly. pression reinfo reinforcement	rcement)								
	M_u is the moment resist f_y is the characteristic a γ_{ms} is the partial factor for λ_s is the area of tension z is the lever arm, dett f_{eu} is the characteristic a γ_{mc} is the characteristic a γ_{mc} is the partial factor for b is the varial factor for d is the varial factor f			einforcement mined from Equat worst credible cul the concrete strer ion in compressio of the tension reinf eams without cor	ion 5.2.2b, b be strength i ngth n at the leve orcement fro	but no greater th of the concrete of the neutral om the extreme einforcement	ian 0.95 <i>d</i> axis fibre in compress	ion							
	$z = \left(1 - \right)$	$\frac{0.84\frac{f_yA_s}{\gamma_{ms}}}{\frac{f_{cu}bd}{\gamma_{mc}}}$	d												
	d	=	208	mm											
	z	=	162.68	mm											
	Mu	=	250 1.15	x	1276.27	x	162.68	=	45.14	kNm 2					
	Mu Mu	=	0.225	x 1.5 kNm	20	х	390	х	208	-	=	50.38	kNm		
	Moment R	esistance	of Reinfor	ced Concrete lo	ngitudinal	members (Tr	ansverse direc	tion)							
B2100/1B		Bar type		Bar Dia (mm)	Area of F	Bar (mm ²)	Spacing C	entre to Cer	otre (mm)	٦					
D2100/1D	Top C	Compression	n Bars	16	/ lica of l	201.06	opacing o	150	itte (min)						
	Botto	om Tension	Bars	25		490.87		150		_					
	Crillono M	adal Dima													
	Note: There considetation	e is two sec on of the m	ction width oment cap	is the transverse acity.	direction w	which need to b	be calculated. Th	e Minimum	width of 420	mm will	be taker	n into			
	Width of Se	ection		=	420	mm									
	Depth of se	ection		=	250	mm									
	Cross Sect	ional Area		=	105000	mm ²									
	Total Numb bars	per of comp	ression	=	2.8	or	2	Bars							

	_																			•
Manche	ster Fi	rst Stree	t													Pa	age No.	45	Calc No.	·
ECC - A	ssessr	ment of S	Station W	у												с	alcs by	JF	Date	
South W	est Fo	ootway S	lab (Spar	1)												C	hecker	CAT II	Date	
Total N Area o	lumb f.com	er of To	ension on	ars		=		2.8		or	2		Bars							
reinfor	cmen	it per s	ection			=	5	62.97		mm ²										
Area o per se	f Ten	ision re	inforce	nent		=	9	81.75		mm²										
Cover	to rei	nforcer	ment			=		30	mm											
Bendi	ng E	quatio	n using	comp	oressi	on rein	forcer	nent	_											
	E	quation	n 5.2.2c	Nome	nt resi	stance	for bea	ms with	compr	ression i	einforceme	ent								
	1	$I_u = \frac{0}{2}$	7mc	d = 0	l.5 <i>x</i>) ⊣	$f'_s A'_s$	d - d)												
	M	A'.	is the	area o	of comp	pression	reinfor	cement												
		ď	is the	depth	to the	compre	ssion re	inforcem	ent from	m the ext	reme fibre i	n compre	ession							
		Ĭ.	is eq	al to -	1. 1. 1. 1. 1. 1. 1. 1. 1. 1. 1. 1. 1. 1															
	0	x	is the	neutra	l axis i	depth, w	hich m	ay be det	ermine	ed from E	quation 5.2.	2d								
x	-	=	1	25	mm		(recta	angular s	sectior	ר)										
d' f's	-	=	217.5	6 63043	mm 3 mm															
d	-	=	2	20	mm															
Mu		=	87	5732	6 kN/ı	nm	or		87.46	5	kN/m									
Bendi Equat M_u =	ng Eo tion ! = min	quation 5.2.2a I n $\left\{ \underbrace{0.1}{0.1} \right\}$	n using Momen <u>Jy</u> A _s z 225 _{fre} be 7mc	tensi resis	on rei tance	nforce for be	ment o ams w	only /ithout c	compr	ession	reinforcen	nent)								
Bendin Equation $M_u =$ where M_u f_y γ_{ms} A_s z f_{cu} γ_{mc} b d Equation $z = \left(\left(\right) \right)$	ng Ed tion ! - min e:	quation $5.2.2a$ I 10^{-1} is the $12.2b$ Lec 1	n using Momen $\frac{f_{y}}{225f_{rm}}A_{y}z_{y}$ $\frac{g_{225}}{7mc}bd$ momer charac partial area of a charac partial i area of a charac partial width o effectiv we arm $\frac{f_{y}}{2}A_{y}z_{y}$	tensi resis 2 t resis eristic actor f tensic m, dei eristic actor f the s dept	on rei tance or wo for the or wo for the ection h of th aams w	nforce for be rst cree reinfor forcem rst crea concre concre in com in com in thout c	ment c ams w dible st cement n Equa dible cu te stre pressi on rein compre	only vithout of rength o it strength tion 5.2. ube strer ngth on at the forceme ssion rel	compr f the n th 2b, bu ngth of e level ent fror	einforce at no gre f the con of the n m the ex ement	ment ater than 0 crete eutral axis treme fibre	.95d	pression							
Bendii Equat $M_u =$ wher M_u f_y γ_{ms} A_s z f_{cu} γ_{mc} b d Equat $z = \left(\begin{array}{c} \\ \\ \\ \\ \end{array} \right)$	ng Edion ! = min e: 1	quation $5.2.2a$ I a a a a b a a b b a b b a b	n using Momen $\frac{f_u}{225}A_vz^2$ momer bit momer characc partial area of lever a charac partial width o effectiv ver arm $\frac{f_u}{2}$ d	tension resist 2 t resist eristic actor f the s actor f actor f the s actor f actor f ac	tance or wo for the n rein or wo for the ection h of th ams w	nforce a for be rst crea reinfor forcem red fron rst crea concre in com in com in com	ment c ams w dible st cement 1 Equa tible cc the stree pression on rein compre	only vithout of rength o tion 5.2. tion 5.2.	f the n f the n 2b, bu ngth of e level ent fror	einforce at no gre of the cor of the n m the ex	ment ater than 0 crete eutral axis treme fibre	.95d	pression							
Bendin Equation $M_u =$ where M_u f_y γ_{ms} A_s z f_{cu} γ_{mc} b d Equation $z = \begin{pmatrix} \\ \\ \\ \\ \\ \\ \\ \\ \\ \\ \\ \\ \\ \\ \\ \\ \\ \\$	ng E(ion ! e: 1	quation 5.2.2a I $1 \left\{ \underbrace{0.1}_{0.1} \\ \underbrace{0.1}_{0.2} \\ \underbrace{0.1}_{0.$	n using Momen $\frac{f_u}{225}A_{u22}$ $\frac{f_{u225}f_{u22}}{7mc}bd$ momer charac partial area of lever a charac partial vidth c effectiv ver arm $\frac{f_u}{d}$ d 18 18 15 15 15 15 15 15 15 15 15 15	tensic resis 2 t resis eristic actor f tensic m, det eristic actor f the s a dept for be 2.5	on rei tance or wo for the or wo for the ection h of th ams w mm	nforce a for be rst crea reinfor forcem red fron rst crea concre in com in com in com i thout a	ment c ams w lible st cement 1 Equa lible cc ette stre pressi on rein on rein	only vithout of rength o tion 5.2. tion 5.2. tion 5.2. tion 5.2. tion 5.2. stion 5.2. stion for forceme ssion rel	compr f the n th 2b, bu ngth of e level ent fror	einforce at no gre of the cor of the n m the ex	ment ater than 0 crete eutral axis treme fibre	.95d	pression							
Bendii Equal $M_u =$ wher M_u f_y γ_{ms} A_s z f_{cu} γ_{mc} b d Equat z = d z Mu	ng Ed tion : = min e: 1	quation 5.2.2a I 1 $\left\{ \underbrace{0.1}_{0.1} \\ \underbrace{0.1}_{0.$	n using Momen $\frac{f_{u}}{2}A_{u}z^{2}$ momer charac partial area of lever a charac partial area of lever a charac partial width c effectiv ver arm $\frac{f_{u}}{2}$ d 156 2 1.56	tension resist 2 t resisteristic actor 1 tensicom, det eristic actor 1 the s a dept for be 2.5 .49	tance or wo for the n rein termin or wo for the ection h of th ams w mm mm	nforce a for be rest creat reinfor forcem in com ret creat concre in com ret ensi ithout c	ment c ams w lible st cement 1 Equa lible cc ette stree pression rein sompre	only vithout of rength o t streng tion 5.2. tion 5.2. tion 5.2. tion 5.2. streng th on at the forceme ssion rel	f the ri th 2b, bu gth of e level inforce	einforce at no gre of the or of the n m the ex ment	ment ater than 0 crete eutral axis treme fibre	nent) .95d in com	pression	32.12	kNm					
Bendin Equal $M_u =$ wher M_u f_y γ_{ms} A_s z f_{cu} γ_{mc} b d Equat z = d Z Mu	ng Edition : = min e: 1	quation 5.2.2a I is the is the constant of the is the is t	n using Momen $\frac{f_{u}}{225f_{m}}A_{u}zz$ $\frac{f_{u}}{225f_{m}}bd$ momer charac partial area of lever a charac charac partial width c effectiv ver arm $\frac{f_{u}}{2}$ d 18 150 2 1. 0.	tension resister actor 1 tension tensi	on rei tance or wo for the or wo for the ection h of th ams w mm	nforce a for be rest creater reinfor forcem rest creater concreater in com rest creater concreater in com rest creater in com rest c	ment c ams w lible st cemen n Equa lible cu te stree pressis on rein sompre	only /ithout of rength o ti streng tion 5.2. ube stren forceme ssion rel 81.75	f the n th 2b, bu ngth of e level ent fror	einforce tho gre the cor of the n n the ex- ment x x	ment ater than 0 crete eutral axis treme fibre 150.4 220.0	9 0	pression ' = X	32.12	kNm 2	-	41.97	kNm		
Bendii Equal $M_u =$ wher M_u f_y γ_{ms} A_s z f_{cu} γ_{mc} b d Equat z = d z d z Mu Mu	ng Ed tion ! = min e: 1	quation 5.2.2a I $n \left\{ \underbrace{0.1}_{0.1} \\ \underbrace{0.1}_{0.1}$	n using Momen $\frac{f_{u}}{225} A_{v2}z$ momer charac partial area of lever a charac partial area of lever a charac partial width c effectiv ver arm $\frac{f_{u}}{2}$ d 150 150 2 1.50 2 1.50 2 2 2 2 2 2 2 2 2 2 2 2 2	tension resister actor 1 tension tension the s a dept for be 2.5 .49 .5 .25 .25	tance or wa for the n rein termin or wa for the ection h of th ams w mm mm	nforce for be rst creat reinfor forcem reinfor forcem in com rst creat concre in com rst creat concre rst concre rst concre concre rst concre conc concre concre concre conc conc conc conconconc conc conc co	ment c ams w lible st cement 1 Equa lible cc ette stre pressi on rein sompre	only vithout of rength o tistrengi tion 5.2. ube strer ngth on at the forceme ssion rei 81.75 20	f the n th 2b, bu ngth of e level ent fror	einforce the ogree of the or of the n the example the example th	reinforcen ment ater than 0 crete eutral axis treme fibre 150.43 420.00	nent) .95d in com	pression = x	32.12 182.5	kNm 2	=	41.97	kNm		
Bendii Equat $M_u =$ wher M_u f_y γ_{ms} A_s z f_{cu} γ_{mc} b d Equat z = (d Z Mu Mu	ng Ed tion ! = min e: 1 -	quation 5.2.2a I is the is	n using Momen $\frac{f_u}{225}A_vz$ momer bi momer bi momer bi $\frac{f_u}{225}f_{cm}bi$ how bi charac charac partial i lever a charac charac charac partial i lever a charac charac partial i lever a charac c	tensional resistic actor fi tensic actor fi tensic actor fi tensic actor fi tensic actor fi tensic actor fi tensic actor fi tensic for be 2.5 .49 .00 .15 .25 .25 .25 .25 .25 .25 .25 .25 .25 .2	on rei tance or wo for the or wo for the ection h of th ams w mm	nforce for be rst cree reinfor forcem rst cree concre in com re tensi ithout c x x 1.5	ment c ams w dible st cemen ent n Equa tible cu ete stre pressi on rein on rein sompre	only vithout of rength o t streng tion 5.2. ube stren ngth on at the forceme ssion rei 81.75 20	f the n th 2b, bu ngth of e level ent fror	einforce the or of the n the exemption the exemp	reinforcen ment ater than 0 crete eutral axis treme fibre 150.49 420.00	99 0	pression = x	32.12 182.5	kNm 2		41.97	kNm		
Bendin Equal $M_u =$ wher M_u f_y γ_{ms} A_s z f_{cu} γ_{mc} b d Equat z = d Z Mu Mu Mu	ng Ed tion ! = min e: 1	quation 5.2.2a I is the is the is the is the is the is the is the is the is the is the one $\frac{1}{2} \frac{1}{\sqrt{2}} \frac{1}{$	n using Momen $\frac{f_u}{225f_{rm}}A_vz$ momer partial area of lever a charac partial i area of lever a charac or area of lever a $\frac{f_u}{25} \int_{rm} bd$ lever a $\frac{f_u}{2} \int_{rm} bd$ 18 15(1) 19 10 10 10 10 10 10 10 10 10 10	tensi resis 2 t resiseristic actor 1 the sid control the sid the sid control the sid control the sid the sid control the sid c	on rei tance or wo for the n rein termin or wo for the ection h of th ams w mm mm	nforce a for be rest creater reinfor forcem rest creater concreater in com rest creater concreater in com rest creater in com rest creater concreater in com rest creater in com rest com	ment c ams w lible st cement n Equa lible cc te stre pressi ible co te stre pressi on rein compre	only /ithout of rength o t streng tion 5.2. ube stren forceme ssion rel 81.75 20	f the n th 2b, bu ngth of e level ent fror inforce	einforce the or of the norm of the example. where the example of	ment ater than 0 crete eutral axis treme fibre 150.43 420.00	9 9 0	pression = x	32.12 182.5	kNm 2	Ξ.	41.97	kNm		
Bendii Equal $M_u =$ wher M_u f_y γ_{ms} A_s z f_{cu} γ_{mc} b d Equat z = d z Mu Mu Mu	ng Ed tion ! = min e: 1	quation 5.2.2a I $1\left\{\begin{array}{c}0\\0\\0\\\end{array}\right\}$ is the is the is the is the is the is the	n using Momen $\frac{f_u}{225}A_vz^2$ momer charac partial area of lever a charac partial area of lever a charac partial width c effectiv ver arm $\frac{f_u}{2}$ d 156 $\frac{f_u}{2}$ d 156 $\frac{f_u}{2}$ d 156 $\frac{f_u}{2}$	tension resister actor 1 tension tensi	tance or wo for the n rein termin or wo for the ection h of th ams w mm mm	nforce a for be rst creating forcem reinfor forcem in com retension in com retension com retension in com re	ment c ams w lible st cemen n Equa lible cu te stree pression on rein .compre	only vithout of rength o tistrengi tion 5.2. ube strer ngth on at the forceme ssion rei 81.75 20	f the n th 2b, bu ngth of e level ent fror	einforce at no gre the cor of the n n the ex oment x x	ment ater than 0 crete eutral axis treme fibre 150.49 420.00	nent) .95d in com	pression ' = x	32.12 182.5	kNm 2	-	41.97	kNm		
Bendii Equat $M_u =$ wher M_u f_y γ_{ms} A_s z f_{cu} γ_{mc} b d Equat z = (d z Mu Mu	ng Ed tion : : : : : : : : : : : : :	quation 5.2.2a I is the is	n using Momen $\frac{f_u}{225}A_{u2}z$ momercharacc partial area of lever a charac partial iever a charac charac charac charac charac set area of lever a charac charac set $\frac{f_u}{2}$	tension resistic actor f tensic m, deferistic actor f tensic m, deferistic actor f tensic actor f tensic tensic actor f tensic tensic tensic for be 2.5 .49 .0 .12	on rei tance or wo for the ection h of th ams w mm mm	nforce a for be rst crec reinfor forcem ed from rst crec concre in com in com in com in thout of x x x 1.5	ment c ams w lible st cement n Equa lible cu ete stre pressi on rein on rein sompre	only vithout of rength o t t streng tion 5.2. ube stren ngth on at the forceme ssion rei 81.75 20	f the n th 2b, bu ngth of e level ent fror	einforce It no gree of the cor of the n m the exement X	reinforcen ment ater than 0 crete eutral axis treme fibre 150.4 420.0	99 0	pression = x	32.12 182.5	kNm 2		41.97	kNm		

	COBS								CALCULA	TION S	HEET	
M	anchester First Street								Page No.	46	Calc No.	
& E0	CC - Assessment of Station Way								Calcs by	JF	Date	May-24
Sc	outh West Footway Slab (Span 1)								Checker	CAT II	Date	May-24
м	Ioment Canacity of edge mem	hore								1		1
_	Destance	Der Die (mm)	Area of Por	(mm ²)	Creation	Contro to Con	tara (mama)	1				
	Top Compression Bars	Bar Dia (mm) 16	20 ²	1.06	Spacing	150	ure (mm)	-				
\vdash	Bottom Tension Bars	25	490	0.87		150						
	nillene Medel Dime]				
		_	200	~~~								
		-	550		Assumed by	inht to motch (
	leight of section (max depth)	=	550 I	2	Assumed ne	ignt to match t	JATII					
С	ross Sectional Area	=	172500 n	nm²								
	250											
	↑											
	550											
		▲										
			250									
	↓	 +										
	390	C										
т	otal Number of bars per section	=	2.6	or	2.50	bars						
A	rea of tension reinfocement	=	1276.27	mm ²								
с	cover to Reinforcement	=	30	mm								
E	ffective depth. d	=	507.5	mm								
I E	$M_{u} = \min \begin{cases} \frac{0.225T_{ch}}{2mc}bd^{2} \\ \frac{0.225T_{ch}}{2mc}bd^{2} \end{cases}$ where: $M_{u} \qquad \text{is the moment resista} \\ f_{y} \qquad \text{is the characteristic of} \\ \gamma_{ms} \qquad \text{is the characteristic of} \\ A_{s} \qquad \text{is the partial factor fo} \\ A_{s} \qquad \text{is the area of tension} \\ z \qquad \text{is the lever arm, dete} \\ f_{cu} \qquad \text{is the characteristic of} \\ \gamma_{mc} \qquad \text{is the characteristic of} \\ \gamma_{mc} \qquad \text{is the characteristic of} \\ b \qquad \text{is the partial factor fo} \\ b \qquad \text{is the partial factor fo} \\ b \qquad \text{is the partial factor fo} \\ b \qquad \text{is the effective depth} \\ \hline cutoff 1 = \left(1 - \frac{0.84 \frac{f_{y,ms}}{f_{ymc}}}{f_{ymc}}\right)d \\ cutoff 2 = \left(1 - \frac{0.84 \frac{f_{y,ms}}{f_{ymc}}}{d}\right)d \end{cases}$	ance or worst credible s r the reinforcement reinforcement armined from Equ or worst credible o r the concrete str ction in compress of the tension rei ms without compr	strength of th ent strength ation 5.2.2b, cube strength ength sion at the le inforcement ession reinfo	te reinforcer but no grea h of the cond vel of the ne from the ext procement	nent tter than 0.95, crete utral axis reme fibre in	compression						
Z M	= 462.68 1u = <u>250</u>	mm x1	1276.27	×	462.68	=	128.37	kNm				
N.4	1.15	v	20	×	300.00	~	163 69	_	250 47 KNm			
	- <u>U.22</u> 3	1.5	20	^	330.00	~	702.00	-	LUU.TI NIVIII			
M	lu = 128.37	kNm										
1												

	82							CALCULA	TION S	HEET
Manchester Firs	st Street							Page No.	47	Calc No.
ECC - Assessme	ent of Station Way							Calcs by	JF	Date
South West Foo	tway Slab (Span 1)							Checker	CAT II	Date
Shear Reista	ance without she uation 5.6a Maxi $_{ m max} = 0.36 \Big(0.7$ - here: b_w is the br	ar reinforcement $\left(\frac{f_{cu}}{250}\right)\left(\frac{f_{cu}}{\gamma_{mc}}\right)$ eadth of the sec	it - Longitudin istance base $b_w d$ tion, taken as	al Direction	te crushir h for flange	g ed beams				
V _{max}	=	259358.4	N or	259.36	kN					
Ec V.	quation 5.6b She $w = \frac{0.24}{\gamma_{mv}} \xi_s \rho_s^{\frac{1}{3}} f$ where: γ_{mv} is the p ξ_s is the p ξ_s is the p $\rho_s = \frac{1}{2}$ A_s is the a beyond suppor Where which	ear resistance $\frac{1}{cu}b_w d$ partial factor for lepth factor, tak $\frac{500}{d}$) ^{0.25} but not atio of longitudi $\frac{00A_x}{b_x d}$ but not les area of longitudi d the section be t. a top and bottor produces the sh	more than 30 shear defined en as less than 0.7 nal reinforcen is than 0.15, r inal tension re ing considere n reinforceme tear force is to	I from a su I in Section hent hor greater t inforcement d, and, for s nt are provie b be used.	pport 2. han 3.0 that conti hear at su ded, the ar	nues at lea oports, con ea in tensio	st a distance <i>d</i> tinues at least to the on under the loading			
ξs ps	; = ; =	1.25 1.58								
V ur	c =	66.48 H								
	$V_u = \max \begin{cases} \frac{3}{2} $	constructed within 3c $\frac{1}{2} \nabla V_{uc}^{c}$ $\frac{1}{bc} \Gamma V_{uc}^{c}$ $\frac{1}{bc} \Gamma f_{cu}^{c})^{\frac{1}{2}} b_{uc} d$ Ince of the section meaning or the face of r to account for shor to account for the construction of the sup anchorage force that front face of the sup	easured from the e a support, where e t anchorage length offect of short and t can be developed port according to 5	dge of a rigid b $l ≤ a_{\nu} ≤ 3d$ s, defined in Eq :horage length lin the longitudi section 9, but no	earing, the cer uation 5.6d s nal tension re	ntre-line of nforcing $\frac{\Delta f_A}{2m}$				
	a is the flexu 5.2.2b	ral lever arm at ULS	at a position 3d fro	m the support o	alculated from	Equation				
av	=	623 r 162.68 r	nm nm							
Z T fub	=	$f_{ub} = \frac{kk_{cov}\beta}{\gamma_n}$	$\frac{d\sqrt{f_{cu}}}{db}$							
Z T fub K	= = _	$f_{ub} = \frac{kk_{cov}\beta}{\gamma_n}$	$B\sqrt{f_{cu}}$ ab	=	0.40	mm				
Z T fub K β		$f_{ub} = \frac{kk_{cov}\beta}{\gamma_n}$ 1 0.39 20	$\frac{B\sqrt{f_{eu}}}{ab}$ acon	= = =	0.40 30 25	mm				

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tle	ECC - Assessment of Station Way	Calcs by	JF	Date	May-
ction	South West Footway Slab (Span 1)	Checker	CAT II	Date	May-
	fub = <u>1.25</u>				
	Fub = $F_{ub} = f_{ub}pL_a$				
	P = $p = \pi \phi$ for a single bar, = 157.08 mm ²				
	La = Length taken as d = 623 mm				
	Fub = <u>121.82 kN</u>				
	$\Gamma = \min \left\{ \begin{array}{ccc} \sqrt{\frac{z}{3d} \frac{F_{ub}}{V_{uc}}} &= 0.40 &< 1\\ 1.0 & \Gamma &= 0.40 \end{array} \right.$				
	Equation 5.6c Shear resistance within 3d of a support				
	$V_u = \max \begin{cases} \frac{3d}{a_v} \Gamma V_{uc} \\ \frac{0.24}{\gamma_{mv}} \xi_s (0.15 f_{cu})^{\frac{1}{3}} b_w d \end{cases}$				
	$Vu = \frac{3d}{a_v} \Gamma V_{uc} = \frac{79.68}{\text{kN}}$				
	Vu $\frac{0.24}{\gamma_{mv}}\xi_{v}(0.15f_{cu})^{\frac{1}{3}}b_{w}d = 30.35$ kN				
	Edge Section Shear Resistance				
	$V = 0.26 \left(0.7 J_{cu} \right) \left(J_{cu} \right)_{L}$				
	$v_{\text{max}} = 0.35 \left(\frac{0.7 - 250}{250} \right) \left(\frac{\gamma_{mc}}{\gamma_{mc}} \right)^{b_w d}$ where: b_w is the breadth of the section, taken as the web width for flanged beams				
	$v_{\text{max}} = 0.30 \left(0.7 - \frac{250}{250} \right) \left(\frac{\gamma_{mc}}{\gamma_{mc}} \right) b_w a$ where: b_w is the breadth of the section, taken as the web width for flanged beams. $V_{\text{MAX}} = 589.02 \text{ kN}$				
	$V_{\text{max}} = 0.30 \left(0.7 - \frac{250}{250} \right) \left(\frac{1}{\gamma_{mc}} \right)^{b_w d}$ where: b_w is the breadth of the section, taken as the web width for flanged beams $V_{\text{MAX}} = 589.02 \text{ kN}$ Equation 5.6b Shear resistance more than 3d from a support $V_{uc} = \frac{0.24}{\xi_s} \rho_s^{\frac{1}{3}} f_{cu}^{\frac{1}{3}} b_w d$				
	$V_{\text{max}} = 0.30 \left(0.t - \frac{250}{250} \right) \left(\frac{\gamma_{mc}}{\gamma_{mc}} \right) b_w dt$ where: b_w is the breadth of the section, taken as the web width for flanged beams $V_{\text{MAX}} = 589.02 \text{ kN}$ Equation 5.6b Shear resistance more than 3d from a support $V_{uc} = \frac{0.24}{\gamma_{mv}} \xi_s \rho_s^{\frac{1}{4}} f_{cu}^{\frac{1}{4}} b_w dt$ where:				
	$V_{max} = 0.30 \begin{pmatrix} 0.t - \frac{1}{250} \end{pmatrix} \left(\frac{1}{\gamma_{mc}} \right)^{b_w d}$ where: b_w is the breadth of the section, taken as the web width for flanged beams $V_{MAX} = 589.02 \text{ kN}$ Equation 5.6b Shear resistance more than 3d from a support $V_{uc} = \frac{0.24}{\gamma_{mv}} \xi_s \rho_s^{\frac{1}{3}} f_s^{\frac{1}{4}} b_w d$ where: γ_{mv} is the partial factor for shear defined in Section 2.				
	$V_{max} = 0.30 \left(0.7 - \frac{250}{250} \right) \left(\frac{\gamma_{mc}}{\gamma_{mc}} \right)^{b_w d}$ where: b_w is the breadth of the section, taken as the web width for flanged beams $V_{MAX} = 589.02 \text{ kN}$ 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 $\xi = (\frac{500}{\gamma_{mv}})^{0.25}$				
	$V_{max} = 0.30 \left(0.t - \frac{250}{250} \right) \left(\frac{\gamma_{mc}}{\gamma_{mc}} \right)^{0} w^{d}$ where: b_w is the breadth of the section, taken as the web width for flanged beams $V_{MAX} = 589.02 \text{ kN}$ 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 $\rho_s = \frac{190\Delta_s}{100}$ but not less than 0.15, nor greater than 3.0				
	$V_{max} = 0.30 \left(0.t - \frac{250}{250} \right) \left(\frac{\gamma_{mc}}{\gamma_{mc}} \right)^{b_w d}$ where: b_w is the breadth of the section, taken as the web width for flanged beams $V_{MAX} = 589.02 \text{ kN}$ 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 $\rho_s = \frac{100A}{b_w d}$ 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			
	$V_{max} = 0.30 \left(0.7 - \frac{250}{250} \right) \left(\frac{\gamma_{me}}{\gamma_{me}} \right)^{b_w d}$ where: $b_w \text{ is the breadth of the section, taken as the web width for flanged beams}$ $V_{MAX} = 589.02 \text{ kN}$ Equation 5.6b Shear resistance more than 3d from a support $V_{uc} = \frac{0.24}{\gamma_{mv}} \xi_s \rho_s^{\frac{1}{2}} f_{cu}^{\frac{1}{2}} b_w d$ where: $\gamma_{mv} \text{ is the partial factor for shear defined in Section 2.}$ $\xi_s \text{ is the depth factor, taken as}$ $\xi_s = \left(\frac{500}{d}\right)^{0.25} \text{ but not less than 0.7}$ $\rho_s \text{ is the ratio of longitudinal reinforcement}$ $\rho_s = \frac{100A}{b_w d} \text{ but not less than 0.15, nor greater than 3.0}$ $A_s \text{ is the area of longitudinal tension reinforcement that continues at least a distance d}$ beyond the section being considered, and, for shear at supports, continues at least a support. Where top and bottom reinforcement are provided, the area in tension under the low which produces the shear force is to be used.	to the ading			
	$V_{max} = 0.30 \left(0.t - \frac{250}{250} \right) \left(\frac{\gamma_{me}}{\gamma_{me}} \right)^{0_w d}$ where: b_w is the breadth of the section, taken as the web width for flanged beams $V_{MAX} = 589.02 \text{ kN}$ Equation 5.6b Shear resistance more than 3d from a support $V_{ucc} = \frac{0.24}{\gamma_{mv}} \xi_s \rho_s^4 f_{cu}^4 b_{wc} d$ where: γ_{mv} is the partial factor for shear defined in Section 2. ξ_s is the depth factor, taken as $\xi_s = \left(\frac{500}{(d^0)}\right)^{0.25}$ but not less than 0.7 ρ_s is the ratio of longitudinal reinforcement $\rho_s = \frac{100A}{b-d}$ 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 <i>d</i> beyond the section being considered, and, for shear at supports, continues at least support. Where top and bottom reinforcement are provided, the area in tension under the low which produces the shear force is to be used. $\xi_s = 1.00$ $\rho_s = 0.64$	to the ading			
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	$V_{max} = 0.30 \left(0.7 - \frac{250}{250} \right) \left(\frac{\gamma_{mc}}{\gamma_{mc}} \right) b_w d$ where: b_w is the breadth of the section, taken as the web width for flanged beams $V_{MAX} = 589.02 \text{ KN}$ Equation 5.6b Shear resistance more than 3d from a support $V_{ucc} = \frac{0.24}{\gamma_{mc}} \xi_{a} p_{a}^{\frac{1}{2}} f_{a}^{\frac{1}{2}} b_w d$ where: γ_{mv} is the partial factor for shear defined in Section 2. ξ_* is the depth factor, taken as $\xi_* = \left(\frac{200}{0}\right)^{0.25}$ but not less than 0.7 p_* is the ratio of longitudinal reinforcement $p_s = \frac{1000}{b_w d}$ but not less than 0.15, nor greater than 3.0 As 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 $\xi_s = (\frac{100}{b_w d}) = 0.64$ Vuc = 96.51 KN	to the ading			
	$V_{max} = 0.50 \left(0.7 - \frac{250}{250} \right) \left(\frac{\gamma_{me}}{\gamma_{me}} \right)^{0} w^{d}$ where: $b_{w} \text{ is the breadth of the section, taken as the web width for flanged beams}$ $V_{Max} = 589.02 \text{ kN}$ Equation 5.6b Shear resistance more than 3d from a support $V_{wax} = \frac{0.24}{\gamma_{mv}} \xi_{\mu} p_{\pi}^{4} f_{vu}^{1} b_{wd}$ where: $\gamma_{mv} \text{ is the partial factor for shear defined in Section 2.}$ $\xi_{\nu} \text{ is the dapth factor, taken as}$ $\xi_{\nu} = \left(\frac{500}{ct}\right)^{0.25} \text{ but not less than 0.7}$ $P_{\nu} \text{ is the ratio of longitudinal reinforcement}$ $p_{\mu} = \frac{100A}{bcd} \text{ but not less than 0.15, nor greater than 3.0}$ $A_{\nu} 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 a distance d beyond the section being considered, and, for shear at supports, continues at least at least at distance d beyond the section being considered, and, for shear at supports, continues at least at distance d beyond the section being considered, and, for shear at supports, continues at least $	to the ading			
	$V_{max} = 0.50 \left(0.7 - \frac{250}{250} \right) \left(\frac{\gamma_{me}}{\gamma_{me}} \right)^{0} w^{d}$ where: b_{w} is the breadth of the section, taken as the web width for flanged beams. $V_{Max} = 589.02 \text{ KN}$ Equation 5.6b Shear resistance more than 3d from a support $V_{wax} = \frac{0.24}{\gamma_{mv}} \xi_{x} \rho_{x}^{4} \frac{1}{2} \frac{1}{2}$	to the ading			

	OB2								C	ALCULA	TION S	HEET	
ice Manches	ster First Street									Page No.	49	Calc No.	
lo. & ECC - A	ssessment of St	tation Way								Calcs by	JF	Date	May-
tion South W	/est Footway Sla	ab (Span 1)								Checker	CAT II	Date	May-
	Equation f	5 6c Shear recist	tance within 3d of	a sunnort									
	V = max	$\int \frac{3d}{a_v} \Gamma V$	uc	a support									
	where:	$\int \frac{0.24}{\gamma_{mu}} \xi_s(0.15)$	$(c_u)^{\frac{1}{3}} b_w d$										
	a _n	is the distance of a flexible bearing	of the section measu g or the face of a su	ured from the en	dge of a rigid be $\leq u_{-} \leq 3d$	aring, the centr	e-line ot	1					
	F	is the factor to a	account for short and	chorage lengths	s, defined in Equ	ation 5.6d							
	Equation 5	5.6d Factor to ac	count for the effec	t of short and	horage lengths								
	$\Gamma = \min s$	$\begin{cases} \sqrt{\frac{3}{3d}V_{ar}} \\ -1.0 \end{cases}$											
	where:	is the total anch	orage force that car) be developed	in the longitudir	al tension reinf	orcina						
		bars at the front	face of the support	according to S	ection 9, but not	greater than $\frac{A}{\gamma}$	<u>fa</u> ma						
	7	is the flexural le 5.2.2b	ver arm at ULS at a	position 3d from	n the support ca	alculated from E	quation						
		_	_				_						
	av	=	507.5 mi	m									
	Z 	=	462.68 mi	m									
	Т		$\sqrt{\frac{z}{3d}\frac{F_{ub}}{V_{uc}}}$										
	<i>.</i> .		kkB	F									
	fub	=	$f_{ub} = \frac{\gamma_{mb}}{\gamma_{mb}}$	cu									
	к	=	1	acon	=	0.40							
	β	=	0.39	с	=	30							
	fcu	=	20	φ	=	25							
	γmb	=	1.4	kcov	=	1							
	fub	=	1.25										
	Fub	=	$F_{ub} = f_{ub} p L$	a									
	Р	=	$p = \pi \phi$ for a sin	ngle bar,	=	78.540	mm						
	La	=	length taken as	s d	=	507.5	mm						
				_									
	Fub	=	49.66 kN	1									
	г	$\Gamma = \min$	$\left\{ \sqrt{\frac{z}{3d} \frac{F_{ub}}{V_{uc}}} = \right.$	0.40	<	1.0							
			(1.0	Г	=	0.40							
	Equation 5	5.6c Shear I	resistance wi	thin 3d of	a support								
		($\frac{3d}{2}\Gamma V_{uc}$		10 200 8 20 1 0								
	$V_u = \max$	$\frac{0.24}{\gamma_{mv}}\xi_s(0)$	$(0.15f_{cu})^{\frac{1}{3}}b_w d$	l									
		3 8 -1 / 2007.											
	Vu :	=	$\frac{3d}{\Gamma}V$	=	114.49	kN							
1			$a_v - uc$										
			0.24 ¢ (0 15 f)	b d =	83.70	kN							
	Vu	=	~ \$s(0.10 Jeu)	- West									







and calcular field should be applied to the transverse members. angular Distribution $x = 0.744$ x 1.4 = 1 Unit Weight $x = 0.93$ x 22 = 20 Cross Section Area of fill $x = 0.744$ x 1.25 = 1 Unit weight per m for mangle $x = 20.46$ $x = 1.25$ $x = 1$ Unit Weight $x = 0.93$ x 22 = 20 20.46 $x = 1.25$ $x = 1$ Unit Weight $x = 0.93$ x 22 = 20 20.46 $x = 1.25$ $x = 1$ Unit Weight $x = 0.93$ x 22 = 20 20.46 $x = 1.25$ $x = 1$ Unit Weight $x = 0.93$ x 22 = 20 20.46 $x = 1.25$ $x = 1$ Unit Weight $x = 0.93$ x 22 = 20 20.46 $x = 1.25$ $x = 1$ Unit Weight $x = 0.93$ x 22 = 20 20.46 $x = 1.25$ $x = 1$ Unit Weight $x = 0.93$ x 22 = 20 20.46 $x = 1.25$ $x = 1$ Unit Weight $x = 0.93$ x 22 = 20 20.46 $x = 1.25$ $x = 1$ Unit Weight $x = 0.93$ x 22 = 20 20.46 $x = 1.25$ $x = 1$ Unit Weight $x = 0.93$ $x = 22$ $x = 1$ Unit Weight $x = 0.93$ $x = 22$ $x = 20$ 20.46 $x = 1.25$ $x = 1$ Cross section area of fill $x = 0.744$ $x = 1.25$ $x = 1$ Unit Weight $x = 0.93$ $x = 22$ $x = 20$ 20.46 $x = 1.25$ $x = 1$ 21.5 Cross section area of fill $x = 0.744$ $x = 1.45$ $x = 1$ Unit Weight $x = 0.93$ $x = 22$ $x = 20$ 22.46 23.5 24.5 25.5	at the superflow of the superflowed deadtoad will be applied to the transverse members. Label 1.25 m $\frac{1}{2}$ 1	Nuclear for final Page fac. S2 Call R0 Not close 2 - 4 Gain by Jr Gain by <	Interview Page No. 10 Deck No. Decket with the second of the standard of the s	10	OBS						С	ALC	ULAT	ION SI	HEET	
C - Assessed at Balance Way $is general index of Hill in the same principles above, the superfragead dedicaded will be applied to the transverse members: angular Distribution i = 0.744 \times 1.4 = 1i = 1.25 i = 0.755 \times 244 = \frac{3}{2} = 2 MVmi = 1.0416 \times 125 = 1i = 0.744 \times 1.4 = 1i = 0.744 \times 1.25 = -1i = 0.744 \times 1.4 = -1i = 0.74$	$\frac{12 \text{ does twy}}{12 \text{ does } \text{ Wy}} \underbrace{ \text{ Decker } \\ 0.71 \text{ Bask} \\ 0.7$	The COLOR sharement of States Way U_{12} $U_{$	$\frac{10}{125} + \frac{12}{125} + 1$	се	Manchester First Street							Pa	age No.	53	Calc No.	
in Score 2:-4 angular distribution on span ends	ibution on span ends ibution on span ends 1.25 m 1.25 m 1	$\frac{1}{2}$ two figure 2 · 4	Normal Space 2.4 Undext Currols Observe Currols Res Item Traingular distribution on span ends	Title	ECC - Assessment of Station Way							Ca	alcs by	JF	Date	Ма
angular distribution on span ends $\int_{1.5 \text{ m}} Cross \operatorname{Section Area of fil} = 0.744 \times 1.25 = 1$ $\int_{1.5 \text{ m}} Unt \operatorname{Weight} = 0.33 \times 22 = 20$ $\int_{2} 2.48 = -1$ $\int_{2} 2.$	buttor on span ends 1.25 m 1.25 m	Traingular distribution on span ends $\int 25 \text{ m} + \frac{1}{125} $	Traingular distribution on span ends $\int Cross Section Area of Til = 0.744 \times 1.25 = 1 \text{ in } n^2$ Unit weight per m for triangle = 0.033 x 22 = 20 Mem. Unit weight per m for triangle = 0.744 x 1.25 = 1 \text{ in } n^2 Unit weight per m for triangle = 0.744 x 1.25 = 1 \text{ in } n^2 Unit weight per m for triangle = 0.744 x 1.25 = 1 \text{ in } n^2 Unit weight per m for triangle = 0.744 x 1.25 = 1 \text{ in } n^2 Unit weight per m for triangle = 0.744 x 1.25 = 1 \text{ in } n^2 Unit weight per m for triangle = 0.744 x 1.25 = 1 \text{ in } n^2 Unit weight per m for triangle = 0.744 x 1.25 = 1 \text{ in } n^2 Unit weight per m for triangle = 0.744 x 1.25 = 1 \text{ in } n^2 Unit weight per m for triangle = 0.744 x 1.25 = 1 \text{ in } n^2 Unit weight = 0.933 x 22 = 2 0 Mem. Area 2 Cross section area of fill = 0.744 x 1.25 = 1 \text{ in } n^2 Unit Weight = 0.93 x 1.22 = 2 0 Mem. Normal Square Section Cross section area of fill = 0.744 x 1.4 = 1 \text{ in } n^2 Unit Weight = 0.93 x 1.22 = 0 0 Mem. 2.5 Cross section area of fill = 0.744 x 1.4 = 1 \text{ in } n^2 Unit Weight = 0.93 x 1.22 = 0 Mem. Unit Weight = 0.93 x 1.22 = 0 Mem. Cross section area of fill = 0.744 x 1.4 = 1 \text{ in } n^2 Unit Weight = 0.93 x 1.22 = 0 Mem. Cross section area of fill = 0.744 x 1.4 = 1 \text{ in } n^2 Unit Weight = 0.93 x 1.22 = 0 Mem. Area 1 Cross section area of fill = 0.744 x 1.4 = 1 \text{ in } n^2 Unit Weight = 0.940 members. Cross section area of fill = 0.744 x 1.4 = 1 \text{ in } n^2 Unit Weight = 0.941 x 1.4 = 0 \text{ in } members.	'n	Main Spans 2 - 4							C	hecker	CAT II	Date	Ma
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Unit weight per m for triangle $=$ $\frac{20.46}{2}$ $=$ 10 $\frac{2}{2}$ $=$ 20 $\frac{2}{2}$	Unit weight per m for triangle = $\frac{20.46}{2}$ = 10 kN/m acute corners Area 1 Cross section area of fill = $0.744 \times 1.25 = 1 \text{ m}^2$ 1.25 Unit weight per m for triangle = $\frac{20.46}{2}$ = 10 kN/m Area 2 Cross section area of fill = $0.744 \times 1.25 = 1 \text{ m}^2$ Unit weight per m for triangle = $\frac{20.46}{2}$ = 10 kN/m Area 2 Cross section area of fill = $0.744 \times 1.25 = 1 \text{ m}^2$ Unit Weight = $0.93 \times 22 = 20 \text{ kN/m}$ 1.25 Cross section area of fill = $0.744 \times 1.4 = 1 \text{ m}^2$ Unit Weight = $0.93 \times 22 = 20 \text{ kN/m}$ 2.5 Cross sectional area of fill = $1.0416 \times 22 = 2 \text{ m} \text{ m} \text{ kN/m}$ 2.5 Cross sectional area of fill = $1.0416 \times 22 = 2 \text{ m} \text{ m} \text{ kN/m}$ 2.5 Cross sectional area of fill = $1.0416 \times 22 = 2 \text{ m} \text{ m} \text{ kN/m}$ 2.5 Cross sectional area of fill = $2.5 \text{ m} \text$	Unit weight per m for triangle $=$ $\frac{20.46}{2}$ $=$ 10 kMm $\frac{2}{2}$ $=$ 20 kMm $\frac{2}{2}$ $=$ 20 kMm $\frac{2}{2}$ $=$ 20 kMm $\frac{2}{2}$ $=$ 2 kMm $\frac{2}{2$	Unit weight per m for triangle $=$ $\frac{20.46}{2}$ $=$ 10 kNm $\frac{20.46}{2}$ $=$ 10 kNm $\frac{20.46}{2}$ $=$ 10 kNm $\frac{20.46}{2}$ $=$ 10 kNm $\frac{20.46}{2}$ $=$ 1 m ² $\frac{20.46}{2}$ $=$ 2 m ² $\frac{20.4}$			1.25 m	Unit weight			=	0.93	х	22	=	20 kN/r	n
$\int_{1,396} \int_{1,396} \int_{1,396} \int_{1,25} \int_{1,25$	acute corners Area 1 Cross section area of fill = $0.744 \times 1.25 = 1 \text{ m}^2$ Unit Weight = $0.33 \times 22 = 20 \text{ kN/m}$ 1.25 1.2	$\int_{1,360} \frac{1}{1,360} + \frac{1}{1,360} + \frac{1}{1,360} + \frac{1}{1,25} + 1$	$\mathbf{Area 1}$ $\mathbf{Area 1}$ $\mathbf{Area 1}$ $\mathbf{Area 1}$ $\mathbf{Area 2}$ $\mathbf{Area 2}$ $\mathbf{Area 2}$ $\mathbf{Area 3}$ $\mathbf{Area 3}$ $\mathbf{Area 4}$ $\mathbf{Area 4}$ $\mathbf{Area 2}$ $\mathbf{Cross section area of III = 0.744 \times 1.25 = 1 \text{ in}^{2}$ $\mathbf{Area 2}$ $\mathbf{Cross section area of III = 0.744 \times 1.25 = 1 \text{ in}^{2}$ $\mathbf{Area 2}$ $\mathbf{Cross section area of III = 0.744 \times 1.25 = 1 \text{ in}^{2}$ $\mathbf{Area 2}$ $\mathbf{Cross section area of III = 0.744 \times 1.4 = 1 \text{ in}^{2}$ $\mathbf{Mrem st Sqaure Section}$ $\mathbf{Area 5}$ $\mathbf{Cross section area of III = 0.744 \times 1.4 = 1 \text{ in}^{2}$ $\mathbf{Mrem st Sqaure Section}$ $\mathbf{Cross section area of III = 0.744 \times 1.4 = 1 \text{ in}^{2}$ $\mathbf{Mrem st Sqaure Section}$ $\mathbf{Cross section area of III = 0.744 \times 1.4 = 1 \text{ in}^{2}$ $\mathbf{Mrem st Sqaure Section}$ $\mathbf{Cross section area of III = 0.744 \times 1.4 = 1 \text{ in}^{2}$ $\mathbf{Mrem st Sqaure Section}$ $\mathbf{Cross section area of III = 0.744 \times 1.4 = 1 \text{ in}^{2}$ $\mathbf{Mrem st Sqaure Section}$ $\mathbf{Cross section area of III = 0.744 \times 1.4 = 1 \text{ in}^{2}$ $\mathbf{Mrem st Sqaure Section}$ $\mathbf{Cross section area of III = 0.744 \times 1.4 = 1 \text{ in}^{2}$ $\mathbf{Mrem st Sqaure Section}$ $\mathbf{Cross section area of III = 0.744 \times 1.4 = 1 \text{ in}^{2}$ $\mathbf{Mrem st Sqaure Section}$ $\mathbf{Cross section area of III = 0.744 \times 1.4 = 1 \text{ in}^{2}$ $\mathbf{Mrem st Sqaure Section}$ $\mathbf{Cross section area of III = 0.744 \times 1.4 = 1 \text{ in}^{2}$ $\mathbf{Mrem st Sqaure Section}$ $\mathbf{Cross section area of III = 0.744 \times 1.4 = 1 \text{ in}^{2}$ $\mathbf{Mrem st Sqaure Section}$ $\mathbf{Cross section area of III = 0.744 \times 1.4 = 1 \text{ in}^{2}$ $\mathbf{Mrem st Sqaure Section}$ $Mrem st$				Unit weight pe	er m for triangle		=		20.46 2		_=	10 kN/r	n
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And a divergence of the superimode of declaration of the same principles above, the superimode of declaration will be applied to the transverse members. anguar Distribution $= \begin{array}{c} Area 1 \\ 1.25 \\ Area 2 \\ 1.25 \end{array}$ Area 1 Cross section area of fill $= \begin{array}{c} 0.744 \times 1.25 = 1 \\ 0.033 \times 22 = 2 \\ 2 \\ 0.033 \times 22 = 2 \\ $	actic corrers Area 1 Cross section area of fill = 0.744 x 1.25 = 1 f r^2 Unit Weight = 0.933 x 22 = 20 kN/m Area 2 Cross section area of fill = 0.744 x 1.25 = 1 f r^2 Unit Weight = 0.833 x 22 = 20 kN/m Area 2 Cross section area of fill = 0.744 x 1.25 = 1 f r^2 Unit Weight = 0.833 x 22 = 20 kN/m 1.25 Cross sectional area of fill = 0.744 x 1.4 = 1 m ² Unit Weight = 1.0416 x 22 = 4% kN/m 2.5 Cross sectional area of fill = 1.0416 x 1.4 = 1 m ² Unit Weight = 1.0416 x 22 = 4% kN/m 2.5 Cross sectional area of fill = 1.0416 x 1.4 = 1 m ² Unit Weight = 1.0416 x 22 = 4% kN/m 2.5 Cross sectional area of fill = 1.0416 x 1.4 = 1 m ² Unit Weight = 1.0416 x 1.4 = 1 m ² 1.4 kN/m 1.4	Deadloading at acute cornersArea 1Image: construction of the superimposed deadload will be applied to the transverse members.Image: construction of the superimposed deadload will be applied to the transverse members.Image: construction of the superimposed deadload will be applied to the transverse members.Image: construction of the superimposed deadload will be applied to the transverse members.Image: construction of the superimposed deadload will be applied to the transverse members.Image: construction of the superimposed deadload will be applied to the transverse members.Image: construction of the superimposed deadload will be applied to the transverse members.Image: construction of the superimposed deadload will be applied to the transverse members.Image: construction of the superimposed deadload will be applied to the transverse members.Image: construction of the superimposed deadload will be applied to the transverse members.Image: construction of the superimposed deadload will be applied to the transverse members.Image: construction of the superimposed deadload will be applied to the transverse members.Image: construction of the superimposed deadload will be applied to the transverse members.Image: construction of the superimposed deadload will be applied to the transverse members.Image: construction of the superimposed deadload will be applied to the transverse members.Image: construction of the superimposed deadload will be applied to the transverse members.Image: construction of the superimposed deadload will be applied to the transverse members.Image: construction of the superimposed deadload will be applied to the transverse members.Image: construction of the superimposed deadload will be applied to the transverse members.	Deadloading at acute cornersArea 1Image: construction of the superimode deadload will be applied to the transverse members.Cross section area of fill= 0.744 x 1.25 = 1 m^2 Image: construction of the superimode deadload will be applied to the transverse members.Cross section area of fill= 0.744 x 1.4 = 1 m^2 Image: construction of the superimode deadload will be applied to the transverse members.Cross section area of fill= 0.744 x 1.4 = 1 m^2 Image: construction of the superimode deadload will be applied to the transverse members.Cross section area of fill= 0.744 x 1.4 = 1 m^2 Image: construction of the superimode deadload will be applied to the transverse members.Cross section area of fill= 0.744 x 1.4 = 1 m^2 Image: construction of the superimode deadload will be applied to the transverse members.Cross section area of fill= 0.744 x 1.4 = 1 m^2 Image: construction of the superimode deadload will be applied to the transverse members.Cross section area of fill= 0.744 x 1.4 = 1 m^2 Image: construction of the superimode deadload will be applied to the transverse members.Cross section area of fill= 0.744 x 1.4 = 1 m^2 Image: construction of the superimode deadload will be applied to the transverse members.Cross m^2 m^2 $m^$		1.396											
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$\int_{1.25} \int_{1.25} \int$	$\frac{1.25}{1.25}$ Unit Weight per m for triangle = $\frac{0.33}{2} \times \frac{22}{2} = 20$ kN/m $\frac{1.25}{1.25}$ Unit Weight per m for triangle = $\frac{20.46}{2} = 10$ kN/m $\frac{1.25}{1.25}$ $\frac{1.25}{1.25}$	Unit Weight = $0.33 \times 22 = 20$ NVm 1.25 Unit Weight per m for triangle = $\frac{20.46}{2} = 10$ NVm Area 2 Cross section area of fill = $0.744 \times 1.25 = 1$ fm ² Unit Weight = $0.93 \times 22 = 20$ NVm Area 2 Cross section area of fill = $0.744 \times 1.4 = 1$ m ² Unit Weight = $1.0416 \times 22 = 100$ NVm Area 2 Cross sectional area of fill = $0.744 \times 1.4 = 1$ m ² Unit Weight = $1.0416 \times 22 = 100$ NVm 2.5 Cross sectional area of fill = $0.744 \times 1.4 = 1$ m ² Unit Weight = $1.0416 \times 22 = 100$ NVm Area 2 Carriageway Super Imposed Deadload Using the same principles above, the superimposed deadload will be applied to the transverse members. Trangular Distribution = $\frac{Area}{1.25} \times \frac{Density}{24} = \frac{3}{2} = 2 MVm$ Acute Corners = $0.125 \times 24 = \frac{3}{2} = 2 MVm$	$\int_{1,25} \int_{1,25} \int$				Cross section	area of fill		=	0.744	x	1.25	=	1 m ²	
125 125	1.25 1.25	1.25 1.25	125 125				Unit Weight			=	0.93	x	22	=	20 kN/r	n
$\int_{125} \int_{125} \int_{1$	A1 A1 A2 A2 A2 A2 A2 A2 A2 A2 A2 A2	$Area 2$ $Area 2$ $Cross section area of fill = 0.744 \times 1.25 = 1 m^2$ $Unit Weight = 0.93 \times 22 = 20 \text{ NVm}$ $Area 2$ $Cross section area of fill = 0.744 \times 1.4 = 1 m^2$ $Unit Weight = 1.0416 \times 22 = \# \text{ KNm}$ $Cross sectional area of fill = 0.744 \times 1.4 = 1 m^2$ $Unit Weight = 1.0416 \times 22 = \# \text{ KNm}$ $Cross sectional area of fill = 0.744 \times 1.4 = 1 m^2$ $Unit Weight = 1.0416 \times 22 = \# \text{ KNm}$ $Cross sectional area of fill = 0.744 \times 1.4 = 1 m^2$ $Unit Weight = 1.0416 \times 22 = \# \text{ KNm}$ $Cross sectional area of fill = 0.744 \times 1.4 = 1 m^2$ $Unit Weight = 1.0416 \times 22 = \# \text{ KNm}$ $Cross sectional area of fill = 0.744 \times 1.4 = 1 m^2$ $Unit Weight = 1.0416 \times 22 = \# \text{ KNm}$ $Cross sectional area of fill = 0.744 \times 1.4 = 1 m^2$ $Cross sectional area of fill = 0.744 \times 1.4 = 1 m^2$ $Unit Weight = 1.0416 \times 22 = \# \text{ KNm}$ $Cross sectional area of fill = 0.744 \times 1.4 = 1 m^2$ $Cros $	$\frac{1}{2} + \frac{1}{2} + \frac{1}$			1.25	Unit weight pe	er m for triangle		_		20.46		=	10 kN/r	n
$\int_{125} \int_{125} \int_{1$	A2 A2 A2 A2 A2 A2 A2 A2 A2 A2	$\int_{125} \int_{125} \int_{1$	$\int_{125} \int_{125} \int_{1$		A1		Area 2	in tor thangle				2		_	10 140	
$\begin{array}{c} \text{Cross section area of init} = 0.744 \times 1.25 = 1\\ \text{Unit Weight} = 0.93 \times 22 = 20\\ 1.25\\ $	$\begin{array}{c} \text{Cross section area of nin} & = & 0.744 \times 1.25 & = & 1 \text{ Im} \\ \text{Unit Weight} & = & 0.93 \times 22 & = & 20 \text{ kN/m} \\ 1.25 & & & & & & & \\ 1.4 & & & & & & & \\ \hline 1.4 & & & & & & & \\ \hline 1.4 & & & & & & & \\ \hline 1.4 & & & & & & & \\ \hline 1.4 & & & & & & & \\ \hline 1.4 & & & & & & & \\ \hline 1.4 & & & & & & & \\ \hline 1.4 & & & & & & & \\ \hline 1.4 & & & & & & & \\ \hline 1.4 & & & & & & & \\ \hline 1.4 & & & & \\ \hline 1.4 & & & & & \\ \hline 1.4 & & & & \\ 1.4 & & & & \\ \hline 1.4 & & & \\ \hline 1.4 & & & \\ \hline 1.4 & & & \\ \hline $	$\begin{array}{c} \text{Closs section area of min} \qquad = 0.744 \text{x} 1.25 \qquad = 1 \text{ min} \\ \text{Unit Weight} \qquad = 0.93 \text{x} 22 = 20 \text{ kN/m} \\ 125 \qquad 1.25 $	$\begin{array}{c} \text{Cross sector area or init} & = & \text{U}_{1}\text{-4} \times 1.25 & = & 1 \text{ iff} \\ \text{Unit Weight} & = & 0.33 \times 22 & = & 20 \text{ kN/m} \\ 1.25 & & 1.25 & & 0.33 \times 22 & = & 20 \text{ kN/m} \\ \text{Normal Square Section} \\ \hline \\ \text{Normal Square Section} \\ \hline \\ \text{Cross sector al area of fill} & = & 0.744 \times 1.4 & = & 1 \text{ m}^2 \\ \text{Unit Weight} & = & 1.0416 \times 22 & = & \# \text{ kN/m} \\ 2.5 & & 0.116 \times 22 & = & \# \text{ kN/m} \\ \text{Carriageway Super Imposed Deadload} \\ \text{Using the same principles above, the supertimposed deadload will be applied to the transverse members.} \\ \hline \\ \text{Trangular Distribution} & = & 0.125 \times 244 & = & \frac{3}{2} & = & 2 \text{ kN/m} \\ \text{Acute Comers} & = & 0.125 \times 244 & = & \frac{3}{2} & = & 2 \text{ kN/m} \\ \hline \\ \text{Acute Comers} & = & 0.125 \times 244 & = & \frac{3}{2} & = & 2 \text{ kN/m} \\ \hline \\ \text{Acute Comers} & = & 0.125 \times 244 & = & \frac{3}{2} & = & 2 \text{ kN/m} \\ \hline \\ \text{Area 2} & \text{Donsity} \\ 0.125 \times 244 & = & 3 & \text{kN/m} \\ \hline \\ \text{Normal Square Section} & = & \frac{\text{Area}}{0.14} \times \frac{224}{24} & = & 3.36 & \text{kN/m} \\ \hline \end{array}$			Κ	Cross soction	area of fill		_	0 744	Y	1 25	_	1 m ²	
$1.25 \qquad \qquad$	A2 1.25 1.4 Section Cross sectional area of fill = 0.744 x 1.4 = 1 m ² Unit Weight = 1.0416 x 22 = ## kN/m 2.5 2.5 2.5 2.5 2.5 2.5 2.5 2.5	$\begin{array}{c c c c c c c c c c c c c c c c c c c $	1.25 1.25				Unit Maiaht	area UI IIII		-	0.744	^ V	1.20	-	20 6814	n
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$\frac{1.4}{1.4}$	1.4 Deer Imposed Deadload principles above, the superimposed deadload will be applied to the transverse memebers. pution = $\begin{array}{c} \mathbf{Area} & \mathbf{Density} \\ 24 & = & 3 \\ 2 & 2 \end{array} = 2 \text{ kN/m} \\ \hline \mathbf{Area 1} & \mathbf{Density} \\ = & 0.125 & x & 24 \\ 24 & = & 3 \\ 2 & 2 \end{array} = 2 \text{ kN/m} \end{array}$	$\begin{array}{c} & & & \\ & & \\ \hline \hline \\ \hline & & \\ \hline \hline \hline \\ \hline \hline & & \\ \hline \hline \hline & & \\ \hline \hline \hline \hline$	$\begin{array}{c} & & & \\ & & & \\ \hline & & & \\ \hline & & & \\ \hline \hline & & \\ \hline \hline & & \\ \hline & & \\ \hline & & \\ \hline & & \\ \hline \hline \hline & & \\ \hline \hline & & \\ \hline \hline \hline & & \\ \hline \hline \hline & & \\ \hline \hline \hline \\ \hline \hline \hline \hline$													
1.4 arriageway Super Imposed Deadload sing the same principles above, the superimposed deadload will be applied to the transverse memebers. angular Distribution = $\frac{Area}{2}$ ute Corners = 0.125 x 24 = $\frac{3}{2}$ = 2 kN/m Area 1 Density - - 2 = 2 kN/m Area 2 Density - - 2 = 2 kN/m	1.4 ber Imposed Deadload principles above, the superimposed deadload will be applied to the transverse memebers. pution = $\begin{array}{ccc} \mathbf{Area} & \mathbf{Density} \\ = & 0.125 & x & 24 & = & \frac{3}{2} & = & 2 \text{ kN/m} \\ = & 0.125 & x & 24 & = & \frac{3}{2} & = & 2 \text{ kN/m} \\ \end{array}$	1.4Carriageway Super Imposed DeadloadUsing the same principles above, the superimposed deadload will be applied to the transverse memebers.Triangular Distribution= $\begin{array}{cccc} Area \\ 0.125 \\ x \end{array}$ $\begin{array}{cccccccccccccccccccccccccccccccccccc$	1.4Carriageway Super Imposed DeadloadUsing the same principles above, the superimposed deadload will be applied to the transverse memebers.Triangular Distribution= $\begin{array}{ccc} Area \\ 0.125 \\ x \end{array}$ $\begin{array}{ccc} Density \\ 24 \\ z \end{array}$ = $\begin{array}{ccc} 3 \\ 2 \\ z \end{array}$ =2 kN/mAcute Corners= $\begin{array}{ccc} 0.125 \\ x \end{array}$ $\begin{array}{ccc} 24 \\ z \end{array}$ = $\begin{array}{ccc} 3 \\ z \end{array}$ =2 kN/mAcute Corners= $\begin{array}{ccc} 0.125 \\ x \end{array}$ $\begin{array}{ccc} 24 \\ z \end{array}$ = $\begin{array}{ccc} 3 \\ z \end{array}$ =2 kN/mNomral Square Section= $\begin{array}{ccc} 0.14 \\ 0.14 \end{array}$ $\begin{array}{ccc} X \\ z \end{array}$ $\begin{array}{ccc} 24 \\ z \end{array}$ $\begin{array}{ccc} 3 \\ z \end{array}$ $\begin{array}{ccc} kN/m \end{array}$		+											
Arriageway Super Imposed Deadload sing the same principles above, the superimposed deadload will be applied to the transverse memebers. angular Distribution = $Area$ Density ute Corners = 0.125 x 24 = 3 = 2 kN/m ute Corners = 0.125 x 24 = 3 = 2 kN/m Area 1 Density 2 = 3 = 2 kN/m Area 2 Density = 3 = 2 kN/m	ber Imposed Deadload principles above, the superimposed deadload will be applied to the transverse memebers. pution = $\begin{array}{ccc} \mathbf{Area} & \mathbf{Density} \\ 0.125 & x & 24 & = \\ & & & & & \\ \mathbf{Area 1} & \mathbf{Density} \\ = & 0.125 & x & 24 & = \\ & & & & & & \\ \mathbf{Area 2} & & & & & \\ \mathbf{Area 2} & & & & & \\ \mathbf{Area 3} & & & & \\ \mathbf{Area 4} & & \\ \mathbf{Area 4} & & & \\ Area $	Carriageway Super Imposed Deadload Using the same principles above, the superimposed deadload will be applied to the transverse memebers. Triangular Distribution = Area 1 Density = 2 kN/m Acute Corners = 0.125 x 24 = 3 = 2 kN/m Acute Corners = 0.125 x 24 = 3	Carriageway Super Imposed DeadloadUsing the same principles above, the superimposed deadload will be applied to the transverse memebers.Triangular Distribution=		1.4											
Area Density angular Distribution = 0.125 x 24 = 3 = 2 kN/m ute Corners = 0.125 x 24 = 3 = 2 kN/m Area 1 Density 2 2 = 2 kN/m Area 2 Density = 3 = 2 kN/m	principles above, the superimposed deadload will be applied to the transverse memebers. $\begin{array}{c c} \mathbf{Area} & \mathbf{Density} \\ \mathbf{Density} \\ = & 0.125 & \mathbf{x} & 24 & = & \frac{3}{2} \\ = & 0.125 & \mathbf{x} & 24 & = & \frac{3}{2} \\ \mathbf{Area 1} & \mathbf{Density} \\ = & 0.125 & \mathbf{x} & 24 & = & \frac{3}{2} \\ \mathbf{Area 2} & \mathbf{Density} \\ \mathbf{Area 2} & \mathbf{Density} \\ \mathbf{Area 2} & \mathbf{Density} \\ \mathbf{Area 3} & \mathbf{Density} \\ \mathbf{Area 4} & \mathbf{Area 4} \\ \mathbf{Area 5} & \mathbf{Area 6} \\ \mathbf{Area 6} & \mathbf{Area 6} \\ \mathbf{Area 7} & \mathbf{Area 6} \\ \mathbf{Area 7} & \mathbf{Area 7} \\ \mathbf{Area 8} & \mathbf{Area 7} \\ \mathbf{Area 8} & \mathbf{Area 8} \\ \mathbf{Area 8} & $	Using the same principles above, the superimposed deadload will be applied to the transverse memebers. Triangular Distribution = $0.125 \times 24 = 3 = 2 \text{ kN/m}$ Acute Corners = $0.125 \times 24 = 3 = 2 \text{ kN/m}$ Acute Corners = $0.125 \times 24 = 3 = 2 \text{ kN/m}$ Area 2 Density $0.125 \times 24 = 3 \text{ kN/m}$	Using the same principles above, the superimposed deadload will be applied to the transverse memebers. Triangular Distribution = $\begin{array}{cccc} Area & Density \\ - & 2 & 2 \\ - & 2 & - \\ - & -$		Carrianeway Super Imposed Deadles											
AreaDensitylangular Distribution= 0.125 x 24 = 3 =2 kN/mute Corners= 0.125 x 24 = 3 =2 kN/mArea 1Density24= 3 =2 kN/mArea 2Density22=2 kN/m	principles above, the superimposed deadload will be applied to the transverse memebers. $\begin{array}{cccccccccccccccccccccccccccccccccccc$	Using the same principles above, the superimposed deadload will be applied to the transverse memebers. Triangular Distribution = $\begin{pmatrix} Area \\ 0.125 \\ x & 24 \\ x & 24 \\ x & 25 \\ x & $	Using the same principles above, the superimposed deadload will be applied to the transverse memebers. Triangular Distribution = $\begin{pmatrix} Area \\ 0.125 \\ x & 24 \\ z & 24 \\ 0.125 \\ x & 24 \\ z & 24 \\$		Samageway Super Imposed Deadioad											
AreaDensityiangular Distribution= 0.125 x 24 = 3 =2 kN/mute Corners= 0.125 x 24 = 3 =2 kN/mArea 2Density	bution $ \begin{array}{cccc} \mathbf{Area} & \mathbf{Density} \\ = & 0.125 & \mathbf{x} & 24 & = & \underline{3} & 3$	AreaDensityTriangular Distribution= 0.125 x 24 = 3 = 2 kN/mAcute Corners= 0.125 x 24 = 3 = 2 kN/mAcute Corners= 0.125 x 24 = 3 = 2 kN/mArea 2Density 24 = 3 kN/m	Triangular Distribution=		Using the same principles above, the sup	erimposed deadload	will be applied to t	the transverse me	mebers.							
$\begin{array}{cccccccccccccccccccccccccccccccccccc$	Area 1 Density = 0.125 x 24 = 3 = 2 kN/m Area 2 Density	Acute Corners=0.125x24= 3 =2N/mAcute Corners=0.125x24= 3 =2kN/mArea 2Density 0.125x24= 3 kN/m	Acute Corners=0.125x24= 3 =2 kN/mAcute Corners=0.125x24= 3 =2 kN/mArea 2Density 0.125x24= 3 kN/mArea 2Density 0.125x24= 3 kN/mNomral Square Section=0.14x24= 3.36 kN/m		Triangular Distribution –	Area	Density	=	3		=	2 kN	/m			
sute Corners = 0.125 x 24 = 3 = 2 kN/m Area 2 Density	= 0.125 x 24 $= 3$ $= 2$ kN/m	Acute Corners = 0.125 x 24 = 3 = 2 kN/m Area 2 Density 0.125 x 24 = 3 kN/m	Acute Corners= 0.125 x 24 = 3 = 2 kN/mArea 2Density 0.125x 24 = 3 kN/mAreaDensity 2.4 24 = 3.36 kN/m			Area 1	Density		2			2 N.N				
Area 2 Density	Area 2 Density	Area 2 Density 0.125 x 24 = 3 kN/m	Area 2Density 0.125 x 24 = 3 kN/mAreaDensityNomral Square Section= 0.14 x 24 = 3.36 kN/m			0.125 x	24	=	3		_=	2 kN	/m			
0.125 x 24 = 3 kN/m	$\begin{array}{cccc} & & & & \\ 0.125 & x & 24 & = & 3 & & \\ kN/m & & & \end{array}$		Area Density Nomral Square Section = 0.14 x 24 = 3.36 kN/m		Acute Corners =		Density	=	-		kN/m					
Area Density		Area Density	Nomral Square Section = 0.14 x 24 = 3.36 kN/m		Acute Corners =	Area 2 0.125 ¥	24		-							
ameral Square Section - 0.14 y 0.4 0.00 https://www.		Nomral Square Section = 0.14 x 24 = 3.36 kN/m			Acute Corners =	Area 2 0.125 x Area	24 Density									
0.125 x 24 = 3 kN/m	0.125 x 24 = 3 kN/m	Area Density	Nomral Square Section = 0.14 x 24 = 3.36 kN/m		Triangular Distribution =	Area 0.125 x Area 1 0.125 x	Density 24 Density 24 Density	=	3 2 3 2 3		_= _= kN/m	2 kN 2 kN		I/m I/m	l/m l/m	l/m l/m
אווימו טייןעמויפ טפיטוטוו = ט. וייא א א א א א א א א א א א א א א א א א	Section = 0.14 x 24 = 3.36 kN/m				Acute Corners = Nomral Square Section =	Area 2 0.125 x Area 0.14 x	24 Density 24	=	3.36		kN/m					
אווומו טקעמוד שבינווטוו = ט. ויז א א א א א א א א א א א א א א א א א א א	Section = 0.14 x 24 = 3.36 kN/m				Acute Corners = Nomral Square Section =	Area 2 0.125 x Area 0.14 x	24 Density 24	=	3.36		kN/m					
אווימי טקעמיד שבעווטוו = ט. ויז א א א א א א א א א א א א א א א א א א א	Area Density Section = 0.14 x 24 = 3.36 kN/m				Acute Corners = Nomral Square Section =	Area 2 0.125 x Area 0.14 x	24 Density 24	-	3.36		kN/m					
אווימן טקעמוד שרעווטוו = 0.14 ג 24 = 3.35 KN/M	Section = 0.14 x 24 = 3.36 kN/m				Acute Corners = Nomral Square Section =	Area 2 0.125 x Area 0.14 x	24 Density 24	-	3.36		kN/m					
אווימן טקעמוד שלעווטדו = 0.14 ג 24 = 3.36 KN/M	Section = 0.14 x 24 = 3.36 kN/m				Acute Corners = Nomral Square Section =	Area 2 0.125 x Area 0.14 x	24 Density 24	-	3.36		kN/m					
אווומו טקעמוד שרעווטוו = ט. וא א אלא א א א א א א א א א א א א א א א	Section = 0.14 x 24 = 3.36 kN/m				Acute Corners = Nomral Square Section =	Area 2 0.125 x Area 0.14 x	24 Density 24	-	3.36		kN/m					
אווומן טקעמוד שרעווטון = 0.14 ג 24 = 3.30 KN/M	Section = 0.14 x 24 = 3.36 kN/m				Acute Corners = Nomral Square Section =	Area 2 0.125 x Area 0.14 x	24 Density 24	-	3.36		kN/m					
אווומן טקעמוד שרעווטון = 0.14 ג 24 = 3.30 KN/M	Section = 0.14 x 24 = 3.36 kN/m				Acute Corners = Nomral Square Section =	Area 2 0.125 x Area 0.14 x	24 Density 24	-	3.36		kN/m					



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Job No. & Title	ECC - Assessment of Station	n Way					Calcs by	JF	Date	May-24
Section	Main Spans 2 - 4						Checker	CAT II	Date	May-24
	Reinforced Concrete	Beams - Moment	Resistance of Bean	ns						
	Section 1 - Carriagew	ay Beam 610 dep	oth - Sagging - Mids	oan (720)						
				1730						
						- The second s	↑			
		1.2.8:43	n '		Π			200		
		1 mars	تنغير والالالال	in a surfai	-	in a state and a state of the				
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								410		
				i de la fil	-					
			5	aba		<u>.</u>	•			
			·	610		I				
	Internal Reinforceme	nt for sagging - 6	10mm depth							
	Bar Type	Bar Dia (")	Bar Dia (mm)	Area of Bar (mm2)	Numer of Bars	Area of Rei (mr	nforceme n ²)	nt	
	Longitudinal Bottom Bar	1.375	34.93	957.99		8	7663	3.95		
	Longitudinal Top Bar	1.375	34.93	957.99		4	383	1.98		
	Stirrups	0.375	9.525	71.26		N/A	N/	'A		
CS455 Eq	Using Equation 5.2.4,	Moment of resis	tance of flanged bea	ms - Sagging						
5.2.4	M _u = Minimum val	ue dervied from the	e following equations	$\min \left\{ \begin{array}{c} \frac{-\overline{\gamma}}{\gamma} \\ \frac{0.6}{\gamma_{t}} \end{array} \right.$	$\frac{f_y}{ms}A_s\Big(\frac{f_{ss}}{f_{cu}}bh_f\Big)$	$\left(d-rac{h_f}{2} ight) \left(d-rac{h_f}{2} ight)$				
	Cover to reinforcement	t =	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/mn	1 ²						
	230	x	7663.95 x	562.38	-	200=	708.72	kNm		
	0.6 15	¥	1730 ×	200	x	ے 562 38	200	_	## 1/N1-	m
	1.5	X	1750 X	200	~	502.30 -	200	_=	## KINI	
	Mu = 708.72	kNm								
	Internal Reinforceme	nt for sagging - 6	10mm depth with 2n	nm section loss						
	BarTures	Bar Dia /"\	Bor Die (mm)	Area of Port	mm ²⁾	Numer of Pere	Area of Rei	nforceme	nt	
	Dar Type	dar Dia (*)	⊳ar ∪ia (mm)	Area of Bar (I	um∠)		(mr	n²)		
	Bar	1.375	32.93	851.42		8	681	1.32		
	Longitudinal Top Bar	1.375	34.93	957.99		4	383	1.98		
	Stirrups	0.375	9.525	71.26		N/A	N/	A		
	Using Equation 5.2.4,	Moment of resis	tance of flanged bea	ms - Sagging			·			
	M _u = Minimum valu	ue dervied from the	e following equations	$\int \frac{1}{2}$	$\frac{f_y}{ns}A_s(a)$	$\left(-\frac{h_f}{2}\right)$				
			6	$\lim_{n\to\infty} \left\{ \frac{0.6j}{\gamma_n} \right\}$	$\frac{f_{ou}}{de}bh_f$	$d - \frac{\dot{h_f}}{2}$				



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& Title	ECC - Assessment of Station	Way				Calcs by	JF	Date	May
n	Main Spans 2 - 4					Checker	CAT II	Date	Мау
	Section 3 - Carriagew	av Beam 916 dep	th - Sagging and Hogo	ling					
	oconon o ournagen	•		1730					
	l 🛉		ΦΦΦ	00000					
	200								
	Î Î								
	825								
			0	000					
	+								
	Internal Reinforceme	<u>nt</u>		610					
	Bar Type	Bar Dia (")	Bar Dia (mm)	Area of Bar (mm2)	Numer of Bars	Area of Rei (mr	nforcemer m ²)	nt	
	Longitudinal Bottom Bar	1.375	34.93	957.99	4	383	1.98		
	Longitudinal Top Bar	1.375	34.93	957.99	8	7663	3.95		
	Stirrups	0.375	9.53	71.26	1	71.	.26		
	Z = 782.18	mm							
	Hogging Bending Cap	pacity							
	<u>230</u> x	7663.95 x	782.18 =	1198.91 kNm					
	1.15	15.00	010	000545.0	4054.00				
	0.225 x 1.5	<u>15.00</u> x	610 x	986545.6 =	1354.03 kNm				
	M								
	MU = 1198.91	KNM							

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(mm²)

3831.98

7663.95

71.26

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

7663.95 x 929.18 = 1424.23 kNm

<u>15.00</u> x 610 x 1300170 = 1784.48 kNm]

d = 1140.25 mm

Z = 929.18 mm

Hogging Bending Capacity Mu

230 x 1.15

0.225 x 1.5

Mu = 1424.23 kNm

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Job No. & Title	ECC - Assessment of Station	n Way				Calcs by	JF	Date	May-24
Section	Main Spans 2 - 4					Checker	CAT II	Date	May-24
Section	Main Spans 2 - 4 200 1120 Bar Type Longitudinal Bottom Bar	Bar Dia (") 1.375	Bar Dia (mm) 34.93	1730 1730 1730 1730 0 0 0 0 0 0 1730 0	Numer of Bars 4	Area of Reii (mr 383)	nforceme n²) 1.98	nt	May-24
	Longitudinal Top Bar	1.375	34.93	921.99	ŏ	/663	5.95		
	Stirrups	0.375	9.53	71.26	1	71.	26		
	Cover to top hogging b d = 1288.25 Z = 1077.18 Hogging Bending Cap 230 x 1.15 0.225 x 1.5 Mu = 1651.09	aars = mm pacity Mu 7663.95 x 15.00 x kNm	1 1/4 = 1077.18 = 610 x	31.75 mm 1651.09 kNm 1659588 =	2277.78 kNm				

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Job No. & Title	ECC - Assessment of Statio	n Way				Calcs by	JF	Date	May-24
Section	Main Spans 2 - 4					Checker	CAT II	Date	May-24
	Section 6 - Carriagev	vay Beam 768mm	depth - Sagging and Ho	ogging 1730					
	200	↓ ↑		• • • • • • • • •	Φ				
	726			0000000					
		•		€10					
	Bar Type	Bar Dia (")	Bar Dia (mm)	Area of Bar (mm2)	Numer of Bars	Area of Rei (mr	nforcemen m ²)	t	
	Longitudinal Bottom Bar	1.375	34.93	957.99	8	766	3.95		
	Longitudinal Top Bar	1.375	34.93	957.99	8	766	3.95		
	Stirrups	0.375	9.53	71.26	1	71.	26		
	Using Equation 5.2.4	, Moment of resist	ance of flanged beams	- Sagging & Hogging		·			
	M _u = Minimum val	ue dervied from the	e following equations 1 1/4" =	$M_u = \min \left\{ egin{array}{c} rac{f_1}{0.22} \ rac{0.22}{\gamma_u} \end{array} ight.$ 31.75 mm	$\frac{1}{2} \frac{A_s z}{b_{ss}} b d^2$				
	d = 894.25 Z = 683.18	mm							
	230 x 1.15	раситу Ми 7663.95 х	683.18 =	1047.17 kNm					
	0.225 x 1.5	15.00 x	610 x	799683.1 =	1097.57 kNm				
	Mu = 1047.17	kNm							

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Job No. & Title	ECC - Assessment of Statio	n Way				Calcs by	JF	Date	May-24
Section	Main Spans 2 - 4					Checker	CAT II	Date	May-24
	Section 7 - Carriagev	vay Beam 1025mr	n depth - Sagging and H	logging			_		
				1730					
					→				
	200	Î			DΦ				
		¥ I							
	923								
		*							
				€ 10	→				
	Bar Type	Bar Dia (")	Bar Dia (mm)	Area of Bar (mm2)	Numer of Bars	Area of Rei	nforceme	nt	
	Longitudinal Bottom	1 275	24.02	057.00	4	202	1 09		
	Bar	1.575	34.33	337.33	+		1.50		
	Longitudinal Top Bar	1.375	34.93	957.99	8	766	3.95		
	Stirrups	0.375	9.53	71.26	1	71	.26		
	Using Equation 5.2.4	, Moment of resis	tance of flanged beams	- Sagging & Hogging					
	M _u = Minimum val	lue dervied from the	e following equations	$M_u = \min \left\{ \begin{array}{c} \frac{J_u}{2\pi s} \end{array} \right.$	1, z				
				Inc	#bd*				
	Cover to top hogging b	bars =	1 1/4" =	31.75 mm					
	d = 1091.25	mm							
	Z = 880.18	mm							
	Hogging Bending Ca	pacity Mu							
	230 x	7663.95 x	880.18 =	1349.127 kNm					
	1.15								
	0.225 x	15.00 v	610 x	1190827 -	1634.41 kNm				
	1.5	<u>10.00</u> X	010 X	1130027 -	100 1.1 1 KM				
	Mu = 1349.13	kNm							
<u> </u>	<u> </u>								

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Job No. & Title	ECC - Assessment of Station Wa	у										Calcs by	JF	Date	May-24
Section	Main Spans 2 - 4											Checker	CAT II	Date	May-24
	Section 8 - Carriageway	Beam 12	22mm	depth - Sag	gging and	Hogging									
							1730								
		•	-				~ ~ ~	•							
	200														
		Ť													
	1120														
	(increased)														
							0 0								
		•				4	610								
	Using Equation 5.2.4, Mo	oment of	resista	ance of flan	ged beam	s - Sagging &	Hogging								
	M _u = Minimum value c	dervied fro	om the	following eq	uations	min	$\frac{f_y}{\gamma_{ms}}A_s\Big($	$d-\frac{h_f}{2}$							
						($\frac{0.6f_{ou}}{\gamma_{mc}}bh_f$	$\left(d-\frac{h_f}{2}\right)$							
	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										
	230 1.15	х		7663.95	х	1246.98	-		20	200	=	1758.07	kNm		
	0.6 15	x		1730	x	200	x		1246.98		-	200	=	2381.1	kNm
	1.5											2	-		
	Mu = 1758.07 kNr	n													
	Hogging														
	Cover to top bars		=	1 1/4"	=	31.75 г	nm								
	Effective Depth,d		=	1320	-	31.75	-		34.93	=	1253.33	mm			
	Depth of Flange, hf		=	200	mm										
	230 1.15	х		7663.95	х	1253.33	-		20	200		= 1767.81	kNm		
	0.6 15	x		1730	х	200	x	1	253.33		200	=	2394.3	kNm	
	1.5										2				
	Mu = 1767.81 kNr	n													

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					-

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

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ffice	Manchester First Stree	t											Page No.	64	Calc No.	
o. & Title	ECC - Assessment of	Station Way											Calcs by	JF	Date	May-24
on	Main Spans 2 - 4												Checker	CAT II	Date	May-24
	Shear Capacity f	or Carriageway Bea	<u>m</u>													
				1259	698	698	698	698	698	1119						
					Casting	Gardian		Carolina (C							
	-	Section 1		Section 2	3	4	5	8	7	Section 6		Section 1				
					3353	3			251	5						
	3d = 1687	More than 3	3d	3	d = 361	4			3	d = 3614		More	than 3d			
						Shear z	ones									
	South Carriagew	ay Beam Shear														
	The carriageway	beam will be checked	in 5 location	s. As seen a	bove, th	e locatio	ons have	e been	defined	by 3d from s	support and r	nore than	n 3d from su	pport		
	with the d being d	efined as the effective	e depth of the	e beam. Due	to the s	tructure	being s	ymmetr	ical, onl	y one taper	will be check	ed.				
	Equation 5.7.1 M	inimum effective sh	ear reinforc	ement												
	Equation 5.7.1 M	linimum effective shea	r reinforcem	ent												
	$\frac{\alpha_{sv}}{s_v b_w} (\sin \alpha + c \alpha)$	$\left(\frac{Jyv}{\gamma_{ms}}\right) \ge 0.2 MF$	Pa													
	where				direction and											
	A _{sv} is the a is the	cross-sectional area of angle of the shear reinfo	shear reinforc	ement at a pa the longitudi	nal axis c	ross-sec of the bea	am									
	s _v is the	spacing of the shear rei	nforcement al	ong the memb	per											
	f_{gv} is the than 5	characteristic, or worst o i00 MPa	credible, stren	igth of the she	ar reinfo	rcement	but not g	reater								
	b _w is the	breadth of the cross-sec	ction												Try w	vithout
	95	7.99 ×	6.12574	E-17 +		1	х		230	=	2.47	32 >	0.2mpa	=	reinfor	cement
	127.00 x	610							1.15						ħ	rst
	Shear at within 3	d from the intermed	iate suppor	t calculatior	IS											
	3d = 3	х	1288.25	=	38	365 r	nm									
	Bar Type	Bar Dia (")	Bar I	Dia (mm)		Spacing	(mm)		Area of	Bar (mm2)	Nu	mer of B	ars (3d)	Area o	of Reinfor	cement
	Main top longitud	inal 1.375		34 93		127	00		9	57 99		8.00)		7663.95	5
	bars	1.070		-1.00		127.	00		0	01.00		0.00	,		1000.00	,
	Shear reinforcer	nent anywhere, Equa	ation 5.6a, V	мах												
	Equation 5.6a	Maximum shear res	sistance bas	sed on con	crete cr	ushing										
	V = 0.36	$0.7 - \frac{f_{cu}}{f_{cu}} \left(\frac{f_{cu}}{f_{cu}} \right)$	bed													
	- max wide	250 / (Yme)	Jewa													
	V =	0.36	07	-	15	00	x		15.00	x	610	×	1288.25	5 =	kN	
	MAX -	0.00			2	50	~		1.5	^	010	~	1200.20	, _		
	Shear Resistanc	e more than 3d from	a support,	Equation 5.	6b											
	$V_{uc} = \frac{0.24}{\xi_s \rho_s^4} \xi_s \rho_s^4 f_c^4$,b _w d														
	Yme Sore Jo															
	mr is the pa	rtial factor for shear define	ed in Section 2.													
	ξ_s is the de $\xi_s = \left(\frac{5}{2}\right)$	$\frac{10}{10}^{0.25}$ but not less than 0.	7													
	ρ_s is the ratio $\rho_s = rac{10}{b}$	tio of longitudinal reinforce $\frac{3A_a}{d}$ but not less than 0.15,	ment nor greater tha	in 3.0												
	A _x is the ar beyond	ea of longitudinal tension r the section being consider	einforcement th ed, and, for she	nat continues at ear at supports,	least a d continue	istance d s at least	to the									
	Where which p	op and bottom reinforcem oduces the shear force is	ent are provide to be used.	d, the area in te	ension une	der the lo	ading									

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	$\gamma mv = 1.15$ $\xi s = 500 \\ d 0.25 = 500 \\ 1288.25 \\ 0.25 = 0.79$				
	$\rho s = \underbrace{100 \times As}_{bw \times d} = \underbrace{100 \times 7663.95}_{610} = 0.98$ $1/3 \qquad 1/3$				
	$Vuc = \underbrace{0.24}_{1.15} x 0.79 x 0.98 x 15.00 x 610$	x 1288.25			
	Vuc = <u>316.59</u> kN				
	Shear resistance within 3d of intermeidate support				
	Equation 5.6c Shear resistance within 3d of a support				
	$V_u = \max \left\{ egin{array}{c} rac{3d}{a_w} \Gamma V_{uc} \ rac{0.24}{7 m v} \xi_s (0.15 f_{cu})^rac{1}{s} b_w d \end{array} ight.$				
	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				
	$\Gamma = \min \left\{ egin{array}{l} \sqrt{rac{z}{3d} rac{F_{ub}}{V_{uc}}} \ 1.0 \end{array} ight.$				
	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 $\frac{A_s f_s}{\gamma_{me}}$				
	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_u 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				
	$B = 0.39 \qquad \phi = 34.93$				
	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 = 1288.25 mm				
	Fub = <u>1366.40</u> kN <u>7663.95 x 230</u> = <u>1532.79</u> kN 1.15	(limiting value)			

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	Manchester First Street							Page	No.	66	Calc No
tle	ECC - Assessment of Station	Way						Calc	s by	JF	Date
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	Fub = 1366.40 I Equation 5.60 Factor $\Gamma = \min \left\{ \sqrt{\frac{z}{3d} \frac{F_{th}}{v_{wc}}} \right.$ where: F_{ub} is the total is bars at the z is the flexue 5.2.2b Z = 1077.18 I $\Gamma = 0.613$	kN to account for the anchorage force tha front face of the sup ral lever arm at ULS mm < 1	effect of sho at can be dev at a position (ok)	ort anchora eloped in th ng to Sectio 3 <i>d</i> from the	nge lengths ne longitudinal tensio n 9, but not greater e support calculated	n reinforcing than <u>4 fy</u> 7mr from Equation	n				
	$Vu = \frac{3a}{a_v} \Gamma V_1$ $\frac{0.24}{\gamma_{mv}} \xi_v (0.15 f_c)$ $Vu = \frac{582.47}{3} I_1$ Shear at within 3d from 3d = 3	$a_{w}^{k} = a_{w}^{k} a_{w}^{k} a_{w}^{k} = a_{w}^{k}$ kN m the end suppor	582.47 169.62 t calculation 562.38	kN kN ns =	1687.125 mm						
	Bar Type	Bar Dia (")	Bar Dia	a (mm)	Area of Bar (m	m2)	Numer of Bars	Area	of Reinforc	ement (r	mm ²)
	Leastudiael Dettern	Bai Bia ()	Bai Bia	. ()	7	/		7404 0			,
	where: a_v is the dista a flexible b Γ is the factor	ance of the section n bearing or the face o	neasured fror f a support, v	the edge $d \leq a$ where $d \leq a$	of a rigid bearing, the $v \leq 3d$	e centre-line o	ſ				
	where: a_v is the distance of the distance	unce of the section n bearing or the face o or to account for sho to account for the	neasured fror f a support, v rt anchorage effect of sho	n the edge $d \le a$, where $d \le a$, lengths, de	of a rigid bearing, the $v \leq 3d$ fined in Equation 5.6 age lengths	e centre-líne a	я				
	where: a_v is the dista a flexible t Γ is the factor Equation 5.6d Factor $\Gamma = \min \left\{ \sqrt{\frac{z}{3d} \frac{F_{ub}}{V_{uv}}} \right.$ where: F_{ub} is the total bars at the z is the flexu 5.2.2b	ance of the section n bearing or the face o or to account for sho to account for the anchorage force the front face of the su aral lever arm at ULS	neasured fror f a support, v rt anchorage effect of sho at can be dev pport accordi S at a positior	In the edge of where $d \le a$, lengths, de ort anchore reloped in th ng to Section	of a rigid bearing, the $v \leq 3d$ fined in Equation 5.6 age lengths he longitudinal tensio on 9, but not greater i e support calculated	e centre-line o 3d on reinforcing than $\frac{A_{s}f_{s}}{\gamma_{me}}$ from Equation	1				
	where: a_v is the dista a flexible t Γ is the factor Equation 5.6d Factor $\Gamma = \min \left\{ \sqrt{\frac{z}{3d} \frac{F_{ab}}{V_{ad}}} \right\}$ where: F_{ab} is the total bars at the z is the flexu 5.2.2b Equation 9.1a Anchor $F_{ab} = f_{ab}pL_a$ where:	ance of the section n bearing or the face o or to account for sho to account for the anchorage force the front face of the su and lever arm at ULS rage resistance	neasured fror f a support, v rt anchorage effect of sho at can be dev pport accordi S at a positior	n the edge where $d \le a$, lengths, de ort anchore reloped in th ng to Section n 3 <i>d</i> from the	of a rigid bearing, the $v \leq 3d$ fined in Equation 5.6 age lengths the longitudinal tensio on 9, but not greater is a support calculated	e centre-line o 3d on reinforcing than As <i>f</i> ₄ $\frac{Asf_4}{\gamma_{me}}$ from Equation	af				
	where: a_v is the dista a flexible t Γ is the factor Equation 5.6d Factor $\Gamma = \min \left\{ \sqrt{\frac{z}{z_u} \frac{E_{ub}}{V_{ua}}} \right.$ $r = \min \left\{ \sqrt{\frac{z}{z_u} \frac{E_{ub}}{V_{ua}}} \right\}$ $r = \min \left\{ \sqrt{\frac{z}{z_u} \frac{E_{ub}}{V_{ua}} \right\}$ $r = \max \left\{ \frac{$	ance of the section n bearing or the face o or to account for the to account for the anchorage force the front face of the sup ral lever arm at ULS rage resistance age anchorage bond .1b; twe perimeter, taker a single bar, or $0.2N \ge (\pi \phi)$ for a nal bar diameter twe anchorage leng	neasured fror f a support, v rt anchorage effect of sho at can be dev pport accordi S at a position S at a position d strength ov- n as bundled grou th at the pos	In the edge $d \le a$, lengths, de ort anchore reloped in th ng to Section of <i>S</i> d from the er the effect up of <i>N</i> bars itton where	of a rigid bearing, the $v \leq 3d$ fined in Equation 5.6 age lengths be longitudinal tension on 9, but not greater to e support calculated tive anchorage lengt s, valid up to $N = 4$. the resistance is bei	e centre-line o id in reinforcing than $\frac{A \circ F_{2}}{\gamma me}$ from Equation th, given by	of n				
	where: a_v is the dista a flexible t Γ is the factor Equation 5.6d Factor $\Gamma = \min \left\{ \sqrt{\frac{z}{Md} \frac{F_{ab}}{V_{ad}}} \right\}$ where: F_{ab} is the total bars at the z is the flexu 5.2.2b Equation 9.1a Anchor $F_{ab} = f_{ab}pL_a$ where: f_{ab} is the averation 9 p is the effect $p = \pi\phi$ for $p = (1.2 - \phi)$ ϕ is the nominities L_a is the effect Equation 9.1b $f_{ab} = \frac{kk_{cov}\beta_V}{\gamma_{mb}}$	ance of the section n bearing or the face o or to account for sho to account for the anchorage force the front face of the su tral lever arm at ULS rage resistance age anchorage bond 1b; tive perimeter, take a single bar, or $0.2N \sum (\pi \phi)$ for a nal bar diameter tive anchorage leng Average anch	neasured fror f a support, v rt anchorage effect of sho at can be dev pport accordi S at a position d strength ov- n as bundled grou th at the pos	In the edge $d \le a$, lengths, de ort anchora reloped in th ng to Section an $3d$ from the er the effect up of N bars itton where nd stren	of a rigid bearing, the $v \leq 3d$ fined in Equation 5.6 age lengths the longitudinal tensio on 9, but not greater to e support calculated tive anchorage lengt s, valid up to $N = 4$. the resistance is bei	e centre-line o 3d In reinforcing than $\frac{A_{2}F_{2}}{2me}$ from Equation th, given by	a n				
	where: a_v is the dista a flexible t Γ is the facto Equation 5.6d Factor $\Gamma = \min \left\{ \sqrt{\frac{x}{3d} \frac{F_{ub}}{V_{ua}}} \right.$ where: F_{ub} is the total bars at the z is the flexu 5.2.2b Equation 9.1a Anchor $F_{ub} = f_{ub}pL_a$ where: f_{ub} is the avera Equation 9 p is the effector $p = \pi\phi$ for $p = (1.2 - \phi)$ is the effector L_u is the effector Equation 9.1b $f_{ub} = \frac{kk_{cov}\beta_v}{\gamma_{mb}}$	ance of the section n bearing or the face o or to account for sho to account for the anchorage force tha front face of the su irral lever arm at ULS rage resistance age anchorage bond .1b; tive perimeter, taker a single bar, or 0.2λ) $\sum_{(\pi\phi)} for anal bar diametertive anchorage lengAverage anch$	neasured fror f a support, v rt anchorage effect of sho at can be dev pport accordi 5 at a position d strength ow n as bundled grou th at the pos norage bo	In the edge $d \le a$, lengths, de ort anchora reloped in the ng to Section in $3d$ from the er the effect up of N bars itton where nd stren	of a rigid bearing, the $v \leq 3d$ fined in Equation 5.6 age lengths the longitudinal tension 9, but not greater is a support calculated tive anchorage length s, valid up to $N = 4$. the resistance is beingth 1.25	e centre-line o 3d In reinforcing than $\frac{A - f_{\rm M}}{7 m e}$ from Equation th, given by	of n				
	where: a_v is the dista a flexible t Γ is the facto Equation 5.6d Factor $\Gamma = \min \left\{ \sqrt{\frac{x}{3d} \frac{F_{ub}}{V_{uc}}} \right.$ where: F_{ub} is the total bars at the z is the flexu 5.2.2b Equation 9.1a Anchor $F_{ub} = f_{ub}pL_a$ where: f_{ub} is the averation 9 p is the effector $p = \pi \phi$ for $p = \pi \phi$ for $p = \pi \phi$ for L_a is the effector $p = \pi \phi$ for L_a is the effector $p = \pi \phi$ for L_a is the effector $f_{ub} = \frac{kk_{cov}\beta\gamma}{\gamma_{mb}}$ k = 1	ance of the section n bearing or the face o or to account for sho to account for the anchorage force tha front face of the suj tral lever arm at ULS rage resistance age anchorage bond .1b; tive perimeter, taket a single bar, or $0.2N \sum (\pi \phi)$ for a nal bar diameter tive anchorage leng Average anch	neasured fror f a support, v rt anchorage effect of sho at can be deve pport accordi S at a position d strength ov n as bundled grou th at the pos norage bo ymb	In the edge $d \le a$, lengths, de ort anchora reloped in thing to Section in 3 <i>d</i> from the er the effect up of <i>N</i> bars ition where nd stren	of a rigid bearing, the $v \leq 3d$ fined in Equation 5.6 age lengths the longitudinal tension 9, but not greater is a support calculated tive anchorage lengt s, valid up to $N = 4$. the resistance is beingth 1.25 34.93	e centre-line o 3d n reinforcing than <u>Arfy</u> from Equation th, given by	n d.				
	where: a_v is the dista a flexible t Γ is the facto Equation 5.6d Factor $\Gamma = \min \left\{ \sqrt{\frac{x}{2d} \frac{F_{bb}}{V_{vac}}} \right.$ where: F_{ub} is the total bars at the z is the flexu 5.2.2b Equation 9.1a Anchor $F_{ub} = f_{ub}pL_a$ where: f_{ub} is the averative f_{ub} is the averative f_{ub} is the effective $p = \pi \infty \delta r n$ $p = \pi \infty \delta r n$ L_a is the effective Equation 9.1b $f_{ub} \equiv \frac{kk_{cov}\beta \gamma}{\gamma_{mb}}$ k = 1 B = 0.39	ance of the section n bearing or the face o or to account for sho to account for the anchorage force tha front face of the su tral lever arm at ULS rage resistance age anchorage bond .1b; tive perimeter, take a single bar, or $0.2N \sum (\pi \phi)$ for a nal bar diameter tive anchorage leng Average anch $\sqrt{f_{cu}}$	neasured fror f a support, v rt anchorage effect of sho at can be deve pport accordi S at a position d strength ov n as bundled grou th at the pos norage bo ymb ¢	In the edge $d \le a$, lengths, de ort anchora reloped in the ng to Section in 3 <i>d</i> from the er the effect up of <i>N</i> bars ition where nd stren = =	of a rigid bearing, the $v \leq 3d$ fined in Equation 5.6 age lengths he longitudinal tension on 9, but not greater 1 e support calculated tive anchorage lengt s, valid up to $N = 4$. the resistance is beingth 1.25 34.93	e centre-line o d m reinforcing than $\frac{A_{Fy}}{2me}$ from Equation th, given by	n d.				

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ECC - Assessme Main Spans 2 - 4	Street					Page No.	67	Calc No.	
Main Spans 2 - 4	ent of Station Way					Calcs by	JF	Date	May-24
	Ļ					Checker	CAT II	Date	May-24
rub = Fub = F_u p = 8 L _a = take Fub = 5 Fub = Equation 5.6e $\Gamma = \min \left\{ \begin{array}{c} \\ \\ \\ \\ \\ \\ \\ \\ \\ \\ \\ \\ \\ \\ \\ \\ \\ \\ \\$	1.21 1.21	562.38 mm <u>7663.95 x</u> 1.15 effect of short anchora at can be developed in th pport according to Sections S at a position 3 <i>d</i> from the 271.46 kN 271.46 kN	<u>230</u> = age lengths ne longitudinal tension reinfo on 9, but not greater than $\frac{4}{2\pi}$ e support calculated from Ec	1532.79 kN	(limiting Val	Le)	CAT II	Date	May-24
Vu =	271 46 kN								
Shear calcul Equation 5.7 Equation 5.7. $\frac{A_{sv}}{s_v b_w}(\sin \alpha +$ where A_{sv} is t s_v is t f_{yv} is t b_w is t 127.00	ations based upon the sh .1 Minimum effective sheat 1 Minimum effective shear reli- $\cos \alpha i \left(\frac{f_{ge}}{\gamma_{ms}} \right) \ge 0.2 MPa$ the cross-sectional area of shea- he angle of the shear reinforcer he spacing of the shear reinforcer in 500 MPa he breadth of the cross-section $\frac{285.02}{x}$ x 610	ear reinforcement ar reinforcement inforcement r reinforcement at a particu ment from the longitudinal a ement along the member ble, strength of the shear ro 6.12574E-17 +	lar cross-section ixis of the beam	<u></u>	0.74	> 0.2mpa	_	ΡΑ	SS
Shear calcul Equation 5.7 Equation 5.7: $\frac{A_{sv}}{s_u b_w} (\sin \alpha +$ where A_{sv} is t s_v is t f_{yv} is t f_{yv} is t 127.00 Shear at with	ations based upon the sh .1 Minimum effective sheat 1 Minimum effective shear reid $\cos \alpha \left(\frac{f_{ye}}{\gamma_{ms}} \right) \ge 0.2 MPa$ the cross-sectional area of sheat he angle of the shear reinforcer he spacing of the shear reinforcer he characteristic, or worst credit in 500 MPa he breadth of the cross-section $\frac{285.02}{x}$ 610 x	ear reinforcement ar reinforcement inforcement reinforcement at a particul ment from the longitudinal a mement along the member ble, strength of the shear re 6.12574E-17 + ate support calculation	lar cross-section ixis of the beam einforcement but not greater 1 x	<u>230</u> = 1.15	0.74	> 0.2mpa	=	PA	SS
Shear calcul Equation 5.7 Equation 5.7: $\frac{A_{sv}}{s_{u}b_{w}}(\sin \alpha +$ where A_{sv} ist s_{v} ist f_{yv} ist f_{yv} is t 127.00 Shear at with 3d =	ations based upon the sh .1 Minimum effective sheat 1 Minimum effective sheat reit $\cos \alpha \left(\frac{f_{W}}{\gamma_{ms}}\right) \ge 0.2 MPa$ the cross-sectional area of sheat he angle of the sheat reinforcer he spacing of the sheat reinforcer he characteristic, or worst credit in 500 MPa he breadth of the cross-section $\frac{285.02}{x}$ 610 x in 3d from the intermediat 3 x De Bar Dia (")	ear reinforcement ar reinforcement inforcement reinforcement at a particul ment from the longitudinal a sement along the member ble, strength of the shear re 6.12574E-17 + te support calculation 1288.25 = Bar Dia (mm)	lar cross-section ixis of the beam ainforcement but not greater 1 x 138 3865 mm	<u>230</u> = 1.15 =	0.74 Numer	> 0.2mpa	=	PA	SS

a Name No N	AC	OBS C	ALCULATI	ON SH	IEET	
The UC Converse dataset by $\frac{1}{1000}$ and $\frac{1}{1000}$ by	ffice	Manchester First Street	Page No.	68	Calc No.	
in Description Description <thdescription< th=""> <thdes< th=""><th>o. & Title</th><th>ECC - Assessment of Station Way</th><th>Calcs by</th><th>JF</th><th>Date</th><th>May</th></thdes<></thdescription<>	o. & Title	ECC - Assessment of Station Way	Calcs by	JF	Date	May
<equation-block>Note that the number of th</equation-block>	ction	Main Spans 2 - 4	Checker	CAT II	Date	May
$ \begin{aligned} & \left(\begin{array}{c} - \frac{1}{90} - \frac{1}{90} + \frac{1}{90} - \frac{1}{900} + \frac{1}{1900} - \frac{1}{1900} + \frac{1}{190$		Equation 5.6b Shear resistance more than 3d from a support $V_{w} = \frac{0.24}{7\pi w} \frac{2}{6} \mu^3 f_A^3 h_R d$ where: The is the partial factor for shear defined in Section 2. Similar is the partial factor for shear defined in Section 2. Similar is the partial factor for shear defined in Section 2. Similar is the partial factor for shear defined in Section 2. Similar is the partial factor for shear defined in Section 2. Similar is the partial factor for shear defined in Section 3. Similar is the partial factor for shear defined in Section 3. Similar is the partial factor for shear defined in Section 3. Similar is the partial factor for shear defined in Section 3. Similar is the partial factor for shear defined in Section 3. Similar is the partial factor for the shear 3. Similar is the partial factor for the shear 3. Similar is the shear 3. Similar				
<form>$\begin{aligned}$</form>		$\xi s = \underbrace{500}_{d} 0.25 = \underbrace{500}_{128825} 0.25 = 0.79$				
$ \begin{array}{cccccccccccccccccccccccccccccccccccc$		$\rho s = \frac{100 \text{ x As}}{\text{bw x d}} = \frac{100 \text{ x 7663.95}}{610 \text{ x 1288.25}} = 0.98$				
But the set of the		$V_{UC} = 245.75 \text{ kN}$				
<equation-block>$\begin{aligned}$</equation-block>		Equation 5.9b Component of shear resistance provided by effective shear reinforcement				
$ \begin{array}{cccccccccccccccccccccccccccccccccccc$		$V_{ys} = d rac{A_{sv}}{s_b} (\sinlpha + \coslpha) rac{f_{yv}}{\gamma_{ms}}$				
<section-header> Yet 242 3 623 9 NA Secretarization within 3 of support Secretarization within 3 of support Control term and the second in the data expected expects as defined in Equator 5. Control term and the second in the data expected expects as defined in Equator 5. Control term and the second in the data expected expects as defined in Equator 5. Control term and the second in the data expected expects as defined in Equator 5. Control term and the second in the data expected expects as defined in Equator 5. Control term and term and the data expected expects as defined in the data expected expects as defined in the data on the data expected expects and the data expect and the data expect and the data ex</section-header>		Vus = 1288.25 x <u>285.02</u> x 1 + 6.12574E-17 x <u>230</u> 127.00 1.15	=	578.24		
<section-header> Shark Resistance within glid of support Support</section-header>		Vu = 245.75 + 578.24 = 823.99 kN				
Function for these relations within fact on support $f = f_{ab}^{b} = \frac{1}{2}$. The function of the states is accurate the index of the states is a states of the states is a state of the states i		Shear Resistance withing 3d of support				
$ \begin{aligned} & \qquad \qquad$		Emission 5 0s Sheatr registered within 24 of a support				
Write 1 and 1 also characterized the table characterized particular bits 1 and 1 also characterized particular bits 1 and 1 also characterized particular bits 1 also characterized particular bits 1 and 1 also characterized particular bits 1 and 1 also characterized particular bits 1 also characterized particular bits 1 and 1 and 1 also characterized particular bits 1 and 1 and 1 also characterized particula		Equation s.e. shear resistance within at or a support $V_{n} = \Gamma\left(\frac{3d}{V_{nr}} + V_{nr}\right)$				
P = Product number is account for that account for the deg of a single linear at the deg of a		(a,				
• the detection of the success region were $d \leq u_{n} \leq u_{n}$ Example 1 . A subscription of the success region were $d \leq u_{n} \leq u_{n}$ Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form Form 		$\Gamma_{\rm }$ is the reduction factor to account for short anchorage lengths, as defined in Equation 5.6 d				
Figure 1. An Anchorage maintainse $F_{m} = f_{m} F_{m} = f_{m} F_{m}$ is the sector point into the effective anticompoint in the effective anticompoint is the effective anticompoint in the effective anticompoint in the effective anticompoint is the effective anticompoint in the effective anticompoint is the effective anticompoint is the effective anticompoint in the effective anticompoint is theready anticompoint is the		a_{\parallel} 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 $d \le a_{\perp} \le 3d$				
$F_{ub} = \int_{u}^{u} \int_{u}$		Equation 9.1a Anchorage resistance				
where $\int_{C_{1}}^{V_{1}} dte accesson and choose based states and the set of the effective analysis of the Set of the $		$F_{ub} = f_{ub}pL_a$				
The Equation 3.15. For Equation 3.15. For the electron protocols and any protocols and the position values that up to N = 4. For the electron according both at a dual dual group of N bars, valid up to N = 4. For the electron according both at a dual end of the position values the resolution is being determined. Equation 3.15 A vertage anchorage bond strength $f_{ab} = \frac{kk_{acc}^2 J_{ab}^2}{\gamma_{abb}}$ k = 1 ymb = 1.25 B = 0.39 ϕ = 34.00 for $= 15.00$ K cov = 0.65 Fub = 0.79 Fub = $76b$ $f_{ab} f_{ab} f_{ab} f_{ab}$ p = 854.5132 mm ² L_a = taken as d from support = 1140.25 mm ² Fub = 765.30 KN $\underline{7663.95 \times 230}$ = 1532.79 kN (Emiting value)		where:				
$\int_{-\infty}^{\infty} e^{-x} \cos x \sin x$		Final and the second				
⁹ is the neutronage length at the position values the resistance is being determined: Equation 9.1b Average anchorage bond strength $f_{10} = \frac{bk_{end}^{2} \sqrt{f_{m}}}{T_{mb}}$ $k = 1 \text{ymb} = 1.25$ $B = 0.39 \phi = 34.00$ $fcu = 15.00 \text{Kcov} = 0.65$ $Rub = -0.79$ $Fub = -F_{ub} = f_{ub} p L_{u}$ $p = 854.5132 \text{mm}^{2}$ $L_{u} = \text{taken as d from support} = 1140.25 \text{mm}^{2}$ $Fub = -765.30 \text{KN} \qquad -7663.95 x 230 \text{s} = 1532.79 \text{ KN} (\text{limiting value})$		p is the electric permetative permetation, taken as $p = \pi \delta$ for a single bar, or $p = (1.2 - 0.2N) \sum (\pi \phi)$ for a bundled group of N bars, valid up to $N = 4$.				
Equation 9.1b Average anchorage bond strength $f_{ub} = \frac{kk_{vm}\beta\sqrt{f_{ub}}}{\gamma_{mb}}$ k = 1 mmb = 1.25 B = 0.39 ϕ = 34.00 fou = 15.00 Koov = 0.65 kub = 0.79 Fub = $f_{ub}pL_{u}$ p = 854.5132 mm ² L _a = taken as d from support = 1140.25 mm ² Fub = 765.30 kN <u>7663.95 x 230</u> = 1532.79 kN (imiting value) 1.15		 is the nominal bar diameter is the effective anchorane length at the position where the resistance is being determined. 				
Equation 9.1b Average anchorage bond strength $f_{ab} = \frac{kk_{cond}8\sqrt{f_{con}}}{\gamma_{parba}}$ $k = 1 ymb = 1.25$ $B = 0.39 \phi = 34.00$ fou = 15.00 Kcov = 0.65 $hb = 0.79$ $Fub = -0.79$ $Fub =0.79$		L_0 is the effective ant-totage rengel at the position where the resistance is being betermined.				
$f_{ub} = \frac{h_{mult}}{\gamma_{mb}}$ $k = 1 \text{ymb} = 1.25$ $B = 0.39 \phi = 34.00$ fou = 15.00 Koov = 0.65 fub = 0.79 Fub = $F_{ub} = f_{ub} p L_{u}$ $\rho = 854.5132 \text{mm}^2$ $L_a = \text{taken as drom support} = 1140.25 \text{mm}^2$ Fub = 765.30 kN <u>7663.95 x 230</u> = 1532.79 kN (limiting value) 1.15		Equation 9.1b Average anchorage bond strength				
$k = 1 \text{ymb} = 1.25$ $B = 0.39 \phi = 34.00$ $fcu = 15.00 \text{Kcov} = 0.65$ $fub = 0.79$ $Fub = F_{ub} = f_{ub}pL_a$ $p = 854.5132 \text{mm}^2$ $L_a = \text{taken as d from support} = 1140.25 \text{mm}^2$ $Fub = 765.30 \text{kN} \qquad \underline{7663.95 x 230} = 1532.79 \text{kN} (\text{limiting value})$ 1.15		$f_{ub} = \frac{\kappa \kappa_{con} \beta \sqrt{f_{cu}}}{\gamma_{mb}}$				
B = 0.39 ϕ = 34.00 fcu = 15.00 Kcov = 0.65 fub = 0.79 Fub = $F_{ub} = \frac{1}{f_{ub}pL_a}$ p = 854.5132 mm ² L _a = taken as d from support = 1140.25 mm ² Fub = 765.30 kN <u>7663.95 x 230</u> = 1532.79 kN (limiting value) 1.15		k = 1 γmb = 1.25				
fcu = 15.00 Kcov = 0.65 fub = 0.79 Fub = $F_{ub} = f_{ub}pL_a$ p = 854.5132 mm ² L _a = taken as d from support = 1140.25 mm ² Fub = 765.30 kN <u>7663.95 x 230</u> = 1532.79 kN (limiting value) 1.15		$B = 0.39 \phi = 34.00$				
fub = 0.79 Fub = $F_{ub} = f_{ub} p L_a$ p = 854.5132 La = taken as d from support = 1140.25 mm² Fub = 765.30 KN		fcu = 15.00 Kcov = 0.65				
fub = 0.79 Fub = $F_{ub} = f_{ub}pL_u$ p = 854.5132 mm² La = taken as d from support = 1140.25 mm² Fub = 765.30 kN 7663.95 x 230 = 1532.79 kN (limiting value) 1.15						
$Fub = F_{ub} = f_{ub}pL_a$ $p = 854.5132 mm^2$ $L_a = taken as d from support = 1140.25 mm^2$ $Fub = 765.30 kN $ $\frac{7663.95 x 230}{1.15} = 1532.79 kN (limiting value)$		fub = 0.79				
$p = 854.5132 \text{ mm}^2$ $L_a = \text{taken as d from support} = 1140.25 \text{ mm}^2$ Fub = 765.30 kN <u>7663.95 x 230</u> = 1532.79 kN (limiting value) 1.15		Fub = $F_{ub} = f_{ub}pL_a$				
$L_a = taken as d from support = 1140.25 mm^2$ Fub = 765.30 kN <u>7663.95 x 230</u> = 1532.79 kN (limiting value) 1.15		p = 854.5132 mm ²				
Fub = 765.30 kN 230 = 1532.79 kN (limiting value) 1.15 1.15		$L_a = taken as d from support = 1140.25 mm^2$				
		Fub = 765.30 kN 7663.95 x 230 - 1532.70	kN (limiting value)		
		1.15		•		

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Ma	ain Spans 2 - 4					Checker	CAT II	Date	Ma
I	Equation 5.60 Factor $S = \min \begin{cases} \sqrt{\frac{3}{3} \frac{F_{\mu\mu}}{V_{el}}} \\ \sqrt{\frac{3}{3} \frac{V_{el}}{V_{el}}} \\ \frac{1}{3} \sqrt{\frac{3}{2}} \frac{1}{3} \sqrt{\frac{3}{2}} \\ \frac{1}{3} \sqrt{\frac{3}{2}} \sqrt{\frac{3}{2}} \sqrt{\frac{3}{2}} \sqrt{\frac{3}{2}} \\ \frac{1}{3} \sqrt{\frac{3}{2}} \sqrt{\frac{3}{2}} \sqrt{\frac{3}{2}} \sqrt{\frac{3}{2}} \\ \frac{1}{3} \sqrt{\frac{3}{2}} \sqrt{\frac{3}{2}} \sqrt{\frac{3}{2}} \\ \frac{1}{3} \sqrt{\frac{3}{2}} \sqrt{\frac{3}{2}} \sqrt{\frac{3}{2}} \\ \frac{1}{3} \sqrt{\frac{3}{2}} \sqrt{\frac{3}{2}} \sqrt{\frac{3}{2}} \sqrt{\frac{3}{2}} \\ \frac{1}{3} \sqrt{\frac{3}{2}} \sqrt{\frac{3}{2}} \sqrt{\frac{3}{2}} \sqrt{\frac{3}{2}} \\ \frac{1}{3} \sqrt{\frac{3}{2}} \sqrt{\frac{3}{2}} \sqrt{\frac{3}{2}} \\ \frac{1}{3} \sqrt{\frac{3}{2}} \sqrt{\frac{3}{2}} \sqrt{\frac{3}{2}} \\ \frac{1}{3} \sqrt{\frac{3}{2}} \sqrt{\frac{3}{2}} \sqrt{\frac{3}{2}} \sqrt{\frac{3}{2}} \\ \frac{1}{3} \sqrt{\frac{3}{2}} \sqrt{\frac{3}} \frac{3$	I anchorage force the formation of the second	e effect of short ancho hat can be developed in upport according to Sect S at a position 3d from t	rage lengths the longitudinal tension re ion 9, but not greater thar he support calculated fror	inforcing As fu Time n Equation				
r	= <u>929.18</u> = <u>929.177</u> 3420.75	= (2 of section 4	765.30 = 245.75	3.114168312	г = 0.92			
	Equation 5.9c Sf $V_u = \Gamma igg(rac{3d}{a_v} V_{uc} igg)$	hear resistance $+V_{us}$	within 3d of a supp	ort					
s	u = 0.920	×	<u>3421</u> x 1288.25	245.75 +	578.24 =	1131.99 kN			
s	hear at within 3d fr	om the intermedi	ate support calculatio	ns					
30	d = 3	x	562.38 =	1687 mm					
_	Bar Type	Bar Dia (")	Bar Dia (mm)	Spacing (mm)	Area of Bar (mm2)	Numer of Bars (3d)	Area c	of Reinfor	cer
	0.1				=			(mm²)	
E 1	quation 5.6b Shear resistance $y_{\rm acc} = \frac{0.24}{\gamma_{\rm acc}} \xi_s \rho_s^{\frac{1}{2}} f_{rn}^{\frac{1}{2}} b_w d$	more than 3d from a suppo	et						
	where: γ_{mn} is the partial factor for ξ_{n} is the depth factor, in $\xi_{n} = (\frac{2\pi}{3})^{1/3}$ but in the $\rho_{n} = \frac{4\pi^{2}}{3}$ is the ratio of longitur $\rho_{n} = \frac{4\pi^{2}}{3}$ but not le λ_{n} is the ratio of longitur beyond the section b support. Where top and botto which produces the s	r shear defined in Section 2. ken as t less than 0.7 final reinforcement so than 0.15, nor greater than anal tension reinforcement that eng considered, and, for shear m reinforcement are provided hear force is to be used.	3.0 It continues at least a distance of at supports, continues at least to th the area in tension under the feading	e 1					
v	where: $\gamma_{\rm em}$ is the partial factor in $\xi_{\rm e} = (\frac{e_{\rm p}}{2})^{-22}$ but not $k_{\rm e} = (\frac{e_{\rm p}}{2})^{-22}$ but not $\mu_{\rm e} = (\frac{e_{\rm p}}{2})^{-22}$ but not $k_{\rm e} = (e_{\rm p})^{-22}$ but not beyond the sectors in where top and botto which produces the s	r shear defined in Section 2. ken as (rises than 0.7 Staal preforcement) as than 0.15, nor greater than a land tension reinforcement tha eng considered, and, to shea ne reinforcement are provided hear force is to be used.	3.0 t continues at locat a distance of or at supports, continues at locat to th the area in tension under the loading						
γr ξε	where: $\begin{array}{rcl} & & & & & & & & & & & & & & & & & & &$	r shear defined in Section 2. Kern as thes than 0.7 That predictoremont is than 0.1 show 0.1 show 0.1 that predictoremont is than 0.1 show 0.1 show m reinforcement are provided hear force is to be used. 	3.0 If continues at least a distance of at supports, continues at least to th , the artes in tension under the leasting $\underline{-500} 0.$	25 = 0.97					
γη ξε νι	where: There is the depth factor is $\xi_{-} = (\frac{\pi}{2})^{-1/3}$ but not $k_{-} = \frac{\pi}{2} = (\frac{\pi}{2})^{-1/3}$ but not $k_{-} = \frac{\pi}{2} = \frac{\pi}{2}$ but not $k_{-} = \frac{\pi}{2} = \frac{\pi}{2} = \frac{\pi}{2}$ $k_{-} = \frac{\pi}{2} = $	r shear defined in Section 2. ken as ties than 0.7 Brail renforcement is than 0.1, nor greater than is than 0.1, nor greater than in renforcement are provided thear force is to be used. 	3.0 If continues at least a distance of an supports, continues at least to th , the arts in tension under the leasting $\frac{500}{562.38}$ 0. $\frac{100}{610}$ x	25 = 0.97 7663.95 = 562.38 =	2.23				







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	Traingular distribution on span end	ds											
		∕ .		Cross Section	Area of fill		_	0 744	×	1.25	_	0.02 m ²	
				Linit weight	Area or fill		=	0.744	x	1.20	=	0.93 11	/~~
			1.25 m	Unit weight			=	0.93	X	22	= .	20.46 KIN	/m
				Unit weight pe	er m for triangle		=		20.46		_=	10.23 KIN	/m
		_ +											
	1.396	→											
	Deadloading at acute corners			Area 1									
		1		Cross section	area of fill		=	0.744	x	1.25	=	0.93 m ²	
				Unit Weight			=	0.93	х	22	= :	20.46 kN	/m
			1.25	Unit weight pe	er m for triangle		=		20.46		=	10.23 kN	/m
	A1			Area 2					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
	A2 1.25		1.25										
	1.396	-											
	Normal Sqaure Section												
		1		Cross section	al area of fill		=	0.744	х	1.396	= 1	.0386 m ²	
				Unit Weight			=	1.038624	х	22	= 3	22.85 kN	/m
			2.5										
	← →	•											
	1.396												
	Carriageway Super Imposed Deadlor	ad											
	Using the same principles above, the	e superimp	oosed deadload w	ill be applied to the	transverse mem	ebers.							
	Triangular Distribution	=	Area	Density	=	3		=	2 kN	Im			
		_	Area 1	Density		2		-	- 10				
	Acute Corners	=	0.125	x 24	=	3		=	2 kN	lm			
			Area 2	Density	_	2		kNm					
			0.120 2	Doncing	-	3		ADUIT					
1		_	0.1396	x 24	=	3.3504		kNm					
	Nomral Square Section	-											
	Nomral Square Section	-											
	Nomral Square Section	=											
	Nomral Square Section	=											
	Nomral Square Section	=											
	Nomral Square Section	=											
	Nomral Square Section	-											
	Nomral Square Section	-											



JAC	OBS	CALCULATION SHEET													
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Section	Main Spans 2 - 4 - Parapet Beam	Checker	CAT II	Date	Apr-24										
CS455 Eq 5.2.2a	Using Equation 5.2.2a, Moment of resistance of beams without compression reinforcement - Sagging $M_u = Minimum value dervied from the following equations M_a = \min \begin{cases} \frac{h_a A_{gs} z}{2 \frac{2 (2 M_{cab} b d)^2}{\gamma_{m_a}}} \end{cases}$														
	Cover to reinforcement = 1 1/2 " = 38 mm Effective Depth, d = 1170 - 38 - 9.525 =	1122.38	mm												
	$F_{cu} = 15 \text{ N/mm}^2$														
	Mu = <u>230</u> x 6705.96 x 397.67 = 533.35 kNm 1.15 2	0055.40													
	$Mu = 0.225 \times 15 \times 610 \times 1122.38 = 1.15$ $Mu = 533.35 \text{ kNm}$	2255.18													
	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 = <u>230</u> = 181.82 N/mm 1.15 + <u>230</u> 2000														
	Mu = <u>0.6 x 15</u> x 610 x 270.00 x 1122.38 - <u>1.5</u>	386.16	= 7.	27.52 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) = $560 \times 290 = 162400 \text{ mm}^2$														
	Area 2 (Beam) = 610 x 610 = 372100 mm ²														
	Area Total = 534500 mm ²														
	Σ_{ay} = 372100 x <u>610</u> + 162400 x 610	+ 560 2	= 2	E+08 n	1m2										
	ybtm = <u>212554780</u> = 397.67 mm 534500														
	y top = 610 + 560 - 397.67 = 772.33	mm													
	Sher Capacity for parapet beam with reduced section	e than 3d													

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	Parapet Beam with 2mm section loss - shear calculations														
	Equation 5.7.1 Minimum effective shear reinforcement														
	Equation 5.7.1 Minimum effective shear reinforcement														
	$rac{A_{sv}}{s_v b_w} (\sinlpha + \coslpha) iggl(rac{J_{yv}}{\gamma_{ms}}iggr) \geq 0.2$ MPa														
	where A _m is the cross-sectional area of shear reinforcement at a particular cross-section														
	$_{lpha}$ is the angle of the shear reinforcement from the longitudinal axis of the beam														
	 sty is the spacing of the shear reinforcement along the member is the characteristic, or worst credible, strength of the shear reinforcement but not greater 														
	b _m is the breadth of the cross-section														
-	851.42 127.00 x	610 ×	6.12574E-17 +	1 x	<u>230</u> = 1.15	2.198052 > 0.2mpa	 Try without shear reinforcement firs 								
	Shear at within 3d from	the end support c	562 38 -	1687 mm											
		~	-	1007 1111											
	Bar Type	Bar Dia (")	Bar Dia (mm) 2mm section loss	Spacing (mm)	Area of Bar (mm2)	Numer of Bars (3d)	Area of Reinforcement (mm								
	Main top longitudinal bars	1.375	32.93	127.00	851.42	8.00	6811.32								
	$\begin{array}{ll} \hline \gamma_{me} \\ \mbox{where:} \\ \gamma_{mu} & \mbox{ is the partial factor} \\ \xi_{s} & \mbox{ is the epsilic factor} \\ \xi_{s} & \mbox{ (20)}^{(20)} \mbox{ is the actor} \\ \eta_{s} & is$	r for shear defined in S , taken as not less than 0.7 itudinal reinforcement tt less than 0.15, nor gr itudinal tension reinfor n being considered, an attom reinforcement ar te shear force is to be to	section 2. reater than 3.0 cement that continues at least id, for shear at supports, contin e provided, the area in tension used.	a distance <i>d</i> uues at least to the under the loading											
	γmv = 1.15														
	$\zeta s = \frac{500}{d}$	0.25 =	<u>500</u> 0.25 562.38	= 0.97											
	ρs = <u>100 x As</u>	=	100 x	6811.32 =	1.99										
	DW X d		610 X	562.38	1/0										
	Vuc = <u>0.24</u> 1.15	х	0.97 x	1.99 x	15.00 x	610 x 562.38									
	Vuc = 215490.2	N/mm													
	Vuc = 215.49 kN														
Shear reinforcement															
ł	Bar Type	Bar Dia (")	Bar Dia (mm) 2mm section loss	Spacing (mm)	Area of Bar (mm2)	Numer of Bars (3d)	Area of Reinforcement (mm								
-	Stirrups	3	7.53	76.20	44.47	22.14	984.68								
-	apo					1									

_	OB:	5									CALCU	JLAT	ION SH	IEET	
e	Manchester Fir	st Street									Page N	No.	77	Calc No.	
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'n	Main Spans 2 ·	4 - Parapet Bean	ı								Check	ker	CAT II	Date	Ap
	Equation 5	9h Component	of shear resis	tance pr	ovided by effective	e shear reinforcemer									
	L. A.			tunee pro	ovided by encent	aneur rennoreenner									
	$v_{us} = a \frac{1}{s_i}$	$-(\sin \alpha + \cos \alpha)$	Yrus												
	Vus =	562.38	x		228.6	x 1	+	6.12574E-17	х	230	= 337	7.425	kN		
					76.20					1.15	_				
	Vu =	215.49	+	3	337.425	= 552.92	kN								
	Shear Resistance within 3d of support														
	Equation 5.9c Shear resistance within 3d of a support														
	$V_u = \Gamma\left(\frac{1}{a_v}V_{uv} + V_{us}\right)$														
	where:			0.5.4.2		ALCONE COL									
	 as we resolution nation to account for short an intra account generations as defined in Equation 5.0 d as its the distance of the section measured from the edge of a right bearing, the centre-line 														
	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: fue is the average anchorage bond strength over the effective anchorage length, given by Environce 3 by														
	p is	the effective perin	ieter, taken as												
	p p	$= \pi \omega \text{ for a single } \\ = (1.2 - 0.2N) \sum$	$(\pi \phi)$ for a bundle	ed group of	f N bars, valid up to P	$\vec{v} = 4$,									
	φ is L _a is	the nominal bar di the effective anch	ameter orage length at t	he position	where the resistance	is being determined.									
	and the and an and a second to be a second to the second time to pairly defending to														
	Equation 9.1b Average anchorage bond strength														
	$f_{ub} = \frac{k}{k}$	$k_{cov}\beta\sqrt{f_{cu}}$													
		Imb													
	k =	1	γmb	=	1.25										
	B =	0.39	φ	=	32.93										
	fcu =	15	Kcov	=	0.67										
	fub =	0.81													
	Fub =	$F_{ub} = f_{ub}pL_a$													
	_		2												
	P =	827.50	mm-												
	La =	taken as d fro	om support	=	562.38 mn	n									
	Fub -	277.02	LNI												
	Equation 5.6d	Factor to account	for the effect of :	short anch	orage lengths										
	$\Gamma = \min \left\{ \Lambda \right\}$	$\sqrt{\frac{2}{3d}\frac{F_{ab}}{V_{ac}}}$													
	where:	1.0													
	Fut is ba	he total anchorage is at the front face o	force that can be c f the support acco	leveloped in rding to See	n the longitudinal tensio ction 9, but not greater	n reinforcing than $\frac{A_{a}f_{a}}{2m}$									
	= is)	he flexural lever arr	n at ULS at a posit	tion 3d from	the support calculated	from Equation									
	Z =	468.56	mm												
	Г =	0.70	<	1	OK										
	Equation	5.9c Shear	resistance	within	3d of a suppo	ort									
	1	3d)												
	V - D	$\overline{a_v}^{V_{uc}} + V$	us)												
	$V_u = \Gamma($							337 425	_	605.00	kN				
	$V_u = \Gamma ($	0.70	Y		1697	V 215.40	T	337.423	=	005.00	NIN				
	$V_u = \Gamma \Big($ Vu =	0.70	x		1687 562.38	x 215.49									
	$V_u = \Gamma \Big($ Vu =	0.70	x		<u>1687</u> 562.38	x 215.49									
	$V_u = \Gamma \Big($ Vu =	0.70	х		<u>1687</u> 562.38	x 215.49									
	$V_u = \Gamma \Big($ Vu =	0.70	x		1687 562.38	x 215.49									
	$V_u = \Gamma \Big($ Vu =	0.70	x		<u>1687</u> 562.38	x 215.49									
	$V_u = \Gamma \Big($ Vu =	0.70	x		<u>1687</u> 562.38	x 215.49									
	$V_u = \Gamma \Big($	0.70	x		<u>1687</u> 562.38	x 215.49									
	$V_u = \Gamma \Big($ Vu =	0.70	x		<u>1687</u> 562.38	x 215.49									
	$V_u = \Gamma \Big($ Vu =	0.70	x		<u>1687</u> 562.38	x 215.49									


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	Material & Section Properties				
	Reinforced Concrete				
CS 455,	Characteristic Strength of Concrete = 15.00 N	/mm ²			
CI.3.1.3	Density of Reinforced Concrete – 24 k	N/m ³			
CS 454					
table 4.1.1a					
	Steel Reinforcement				
CS455, CI,	Mild steel reinforcement characteristic yield strength	= 230) N/mm ²		
3.8.2	Design Young's Modulus of steel reinforcement	= 210000	N/mm ²		
	Mass Concrete (Plain Concrete)				
CS 454	Unit weight of plain concrete = 23 kN/m ³				
table 4.1.1a	Fill - Miscellaneous				
CS 454	I Init weight of miscellaneous fill – 22 LNU-3				
table 4.1.1a					
CS 454	Carriageway & rootway Surracing				
table 4.1.1a					
	Partial Factors				
CS 455 Table 2.13a	Partial Factor for reinforcement $\gamma_{ms} = 1.15$				
CS 455 Table 2.13a	Partial Factor for Concrete $\gamma_{mc} = 1.5$				
CS 455 Table 2.13a	Partial Factor for shear in concrete $\gamma_{mv} = 1.15$				
CS 455	Partial Factor for Bond $\gamma_{mb} = 1.25$				
CS 454	Partial Factor for Concrete Deadload = 1.15				
CS 454	Partial Factor for Surfacing = 1.75				
CS 454	Partial Factor for Fill = 1.2				
CS 454 3.9	Inaccurate assessment effects at ULS yf3 = 1.1				
	Grillage Model Span Dimmensions				
		Ð			
	MIDAS Grillage Model Spans Tranverse slab effective widths				
	Main Span 2 Skew Span = 8.4 m Main Span 3 Skew Span = 12.6 m Longitudinal Direction	= 1400	mm		
	Main Span 3 Skew Span = 8.4 m Transverse Direction	= 2500	mm		





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n	Intermeidate Columns					Checker	CAT II Dat	e May-24				
	Shear Resistance of the Section Properties of the	Section Properties of the Octogonal Column										
	179.6											
		457.2	Ļ		203.2							
	Internal Reinforcement		•	457.2	→							
ŀ	Bar Type	Bar Dia (")	Bar Dia (mm)	Area of Bar (mm ²)	Spacing (mm)	No. Bars	ea of Reinforcement (n	nr				
	Ноор	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					
ſ	Cover to reinforcement	=	1 1/2 =	38.1 mm				_				
	Effective Depth, d	=	409.58 mm									
			455740.50 mm ²									
	Area of the Pier	=	155746.52 mm									
	Equation 5.6a Maximum	Shear Based on Co	oncrete Crushing	amables								
	Equation 5.64 Maximu	r v 7 F v	ce based on concrete	crusning								
	$V_{\rm max} = 0.36 \left(0.7 - \frac{1}{2} \right)$	$\left(\frac{J_{cu}}{250}\right)\left(\frac{J_{cu}}{\gamma_{mc}}\right)b_w d$										
	X	/ X Ime /										
	b _W = is taken as	the column diamete	r									
	V _{MAX} =	0.36 x	0.7 -	15.00 x	15.00 x	457.2 x 409.58	= 431.44	kN				
	Equation 5.6b Shear res	istance more than 3	d from a support	200								
	0.24		a nom a support									
	$\dot{V}_{uc} = \frac{0.24}{\gamma_{mv}} \xi_s \rho_s^{\frac{1}{3}} f_{cu}^{\frac{1}{3}}$	$b_w d$										
	where:											
	γ_{mv} is the part	tial factor for shea	defined in Section 2.									
	ξ_s is the dep $\xi_s = \left(\frac{50}{d}\right)$	oth factor, taken as 2) ^{0.25} but not less t	han 0.7									
	ρ_s is the rat	o of longitudinal re	inforcement	17								
	$\rho_8 = \frac{100}{b_0}$	$\frac{A_a}{d}$ but not less that	0.15, nor greater than	1 3.0	Contraction in the second s							
	A _s is the are beyond t	he section being co	insidered, and, for she	at continues at least a d ar at supports, continue	s at least to the							
	Support. Where to	op and bottom rein	forcement are provided	I, the area in tension un	der the loading							
	which pro	oduces the shear fo	orce is to be used.									
	C. 500	0.25	4.05									
	$\zeta s = \frac{500}{409.58}$	_ =	1.05									
	Ps <u>100</u> 457.2	x x	<u>3206.51</u> = 409.58	1.71 =	limited to 3.00							
	Shear resistance Vuc is e	nahanced by carryin	g out Equation 7.10.1									
	$1 + \frac{0.15N}{A_c}$											
	where:											
	N is the ulti	mate axial load in	Newtons, but not grea	ater than $0.11 f_{cu} A_c$								
	A _c is the are	ea of the entire co	ncrete section, in units	s of mm ²								



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Hince 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 walls restaining forces Hinge location Section on Fender											
the fender walls						orcement					
Bar Dia (")	Bar Dia (mm)	Area of Bar (mm ²)	Spacing (mm)		No. Bars	(mm²	2)				
5/8	15.88	197.93	457.2		3.00	593.8	30				
5/8	15.88	197.93	152.4		6.56	1298.	77				
=	2 =	50.8 mm									
=	914.4 -	50.8 -	15.88	=	847.73	mm					
• <u>−</u>	60.65 KN 60.65										
	<pre>v</pre>	e of the column being restricted transversely Hinge loc on Fender Bar Dia (') Bar Dia (mm) 5/8 15.88 = 2 = = 914.4 - = 60.65 KN = 60.65 KN	e of the column being restricted transversely and longitudinally, it is as walls restaining forces Hinge location 914.4 914.4 914.4 914.4 914.4 914.4 914.4 915 /8 197.93 197.93 197.93 197.93 197.93 197.93 197.93 197.93 197.93 197.93 197.93 197.93 197.93 197.93 197.93 197.93 197.93 197.93 197.93 197.93 197.93 197.93 197.93 197.93 197.93 197.93 197.93 197.93 197.93 197.93 197.93 197.93 197.93 197.93 197.93 197.93 197.93 197.93 197.93 197.93 197.93 197.93 197.93 197.93 197.93 197.93 197.93 197.93 197.93 197.93 197.93 197.93 197.93 197.93 197.93 197.93 197.93 197.93 197.93 197.93 197.93 197.93 197.93 197.93 197.93 197.93 197.93 197.93 197.93 197.93 197.93 197.93 197.93 197.93 197.93 197.93 197.93 197.93 197.93 197.93 197.93 197.93 197.93 197.93 197.93 197.93 197.93 197.93 197.93 197.93 197.93 197.93 197.93 197.93 197.93 197.93 197.93 197.93 197.93 197.93 197.93 197.93 197.93 197.93 197.93 197.93 197.93 197.93 197.93 197.93 197.93 197.93 197.93 197.93 197.93 197.93 197.93 197.93 197.93 197.93 197.93 197.93 197.93 197.93 197.93 197.93 197.93 197.93 197.94 197.95 197.95 197.95 197.95 197.95 197.95 197.95 197.95 197.95 197.95 197.95 197.95 197.95 197.95 197.95 197.95 197.95 197.95 197.95 197.95 197.95 197.95 197.95 197.95 197.95 197.95 197.95 197.95 197.95 197.95 197.95 197.95 197.95 197.95 197.95 197.95 197.95 197.95 197.95 197.95 197.95 197.95 197.95 197.95 197.95 197.95 197.95 197.95 197.95 197.95 197.95 197.95 197.95 197.95 197.95 197.95 197.9	s of the column being restricted transversely and longitudinally, it is assumed that the outer walls of the column being restricted transversely and longitudinally, it is assumed that the outer walls of the column being restricted transversely and longitudinally, it is assumed that the outer walls of the column being restricted transversely and longitudinally, it is assumed that the outer walls of the column being restricted transversely and longitudinally, it is assumed that the outer walls of the column being restricted transversely and longitudinally, it is assumed that the outer walls of the column being restricted transversely and longitudinally, it is assumed that the outer walls of the column being restricted transversely and longitudinally, it is assumed that the outer walls of the column being restricted transversely and longitudinally, it is assumed that the outer walls of the column being restricted transversely and longitudinally, it is assumed that the outer walls of the column being restricted transversely and longitudinally, it is assumed that the outer walls of the column being restricted transversely and longitudinally, it is assumed that the outer walls of the column being restricted transversely and longitudinally, it is assumed that the outer walls of the column being restricted transversely and longitudinally, it is assumed to the column being restricted transversely and longitudinally, it is assumed to the column being restricted transversely and longitudinally, it is assumed to the column being restricted transversely and longitudinally and		Image: second data seco	c c				

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CS 455, 7.21	Due to the articulation of the intermediate pier, there is no bending moment with the pier, therefore the base will not be assessed for ben Shear resistance without shear reinforcement The shear resistance of the base will be assessed under the more severe of the following conditions	ding									
	 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 										
	 Direct along a venue section exerciting across the run would of the base, at a distance equal to the enective depth from the rail Durching chear around the loaded area 										
	2) I uncling shear around the loaded area.										
	An the above will be calculated using the requirements of the shear resistance of stabs according to section of 0.03 455										
	Equation 6.5 Shear resistance of concrete slabs more than 3d from a support										
	$V_{uc} = rac{0.27}{\gamma_{mv}} \xi_s ho_s^{rac{1}{3}} f_{cu}^{rac{1}{3}} b_w d$										
	$\xi s = \frac{0.25}{847.73} = 0.88$										
	$Ps = \frac{100 \times 593.80}{914.4 \times 847.73} = 0.08 = limited to = 0.15$										
	$Vuc = \underbrace{0.27}_{1.15} x \qquad 0.88 \qquad x \qquad 0.15 \qquad x \qquad 1/3 \qquad x \qquad 914.4$	x 847.73	= #	kN							
	Applied shear force per m										
	if the applied shear force is less than the resistance then the element passes = Applied force = 60.65 Resistance = 208.99 Pass / Fail = PASS	kN kN									
	Hinge Assessment										
		\bigcirc	101.6								
	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 rectan	gular throat as the hinge	e is not a true f	reussient hing	je.						
	In accordance with CS 468 CI 3.2 the hinge will only be assessd at serviceabilty limit state										
	Equation 3.14 compressive resistance of hinge throat										
	$N \in \frac{2a_1b_1t_{ou}}{c}$										
	ີງທ where:										
	N is the axial force (N) on the throat:										
	a1 is the effective width of the throat (mm); b1 is the effective length of the throat (mm);										
	feu 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										
	<u>2a1 x b1 x fcu</u> = <u>2 x 101.6 x 381 x 15.00</u> ym 1.5	= 774192 774.192	N kN								

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	Verification				
	Is N less than the compressive resistance = PASS				
	Utilisation = 67.83 %				







on Way				CALCO	LATION SP	1661				
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e to Eccentric load	ing from Carriageway S	lab								
	698.5 e	= 0.152	m (assumed a	acting around centre line of o	ind centre line of column)					
	57.0									
	- 63	75 kNm (E	actored)							
↓	Woment			_ 00.1		lotorcuj				
	Bending	due to deadload:								
	Force	= 33	kNm							
	e	= 0.152	m							
	Moment	= 33	x 0.152	= 5.02	kNm (fa	ctored from M	odel)			
.152	I otal Mo	ment =	63.75 +	5.02 = 68.76	kNm					
te pier being pinned	at the top of bottom, the	re is no bending in the co	olumn, therefore the column v	vill be assessed for shear or	nly.					
the Intermediate C	olumn									
ent					1					
Bar Dia (")	Bar Dia (mm)	Area of Bar (mm ²)	Spacing (mm)	No. Bars	ea of Reinforce	ement (mn				
3/8	9.525	71.26	101.6	1.00	71.20	6				
3/4	19.05	285.02	101.6	1.00	285.0	2				
		500.71								
1	25.4	506.71	N/A	4	2026.8	33				
3/4	19.05	285.02	N/A	4	1140.0	09				
In the set of longituding the set of longitu	1 1/2 = 261.94 mm 155746.52 mm ² n Concrete Crushing tance based on concr mud 0.7 - nan 3d from a support ear defined in Section 2 as ss than 0.7 I reinforcement han 0.15, nor greater th	38.1 mm rete crushing 15.00 x 250 x	<u>15.00</u> x 1.5	304.8 x 261.94	= ##	kN				
	$\frac{1}{1} \frac{1}{1} \frac{1}$	Bending Force 698.5 e Moment Moment Moment Moment Moment Bending Force e Moment M	Berding due to Eccentric load: Force = 381.26 698.5 e = 0.152 Moment = 381.26 Moment Factored for SV loading Bending due to deadload: Force = 33 e = 0.152 Moment = 33 152 Total Moment = 38 152 Moment = 38.1 mm = 261.94 mm = 155746.52 mm ² Total Sear Based on Concrete crushing Inum Shear Based on Concrete Crushing Inum Shear resistance based on concrete crushing $= \frac{f_{x0}}{250} (\frac{f_{res}}{T_{res}}) b_{w}d$ the column diameter 0.36 x 0.7 - <u>15.00</u> x resistance more than 3d from a support $f_{x}^{2}b_{w}d$	$\frac{P + Q}{Q} = \frac{Q}{Q} + $	Benching the universal and the second of th	Ber Claim to transmission of the set of control of the set of the	The control control in the control into the set of the control is a set of the control into into into the control into into			

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	$\xi_{5} = \underbrace{\begin{array}{c} 0.25 \\ 500 \\ 261.94 \end{array}}_{261.94} = 1.18$ Ps $\underbrace{\begin{array}{c} 100 \\ 304.8 \end{array}}_{304.8} \times \underbrace{\begin{array}{c} 1013.41 \\ 261.94 \end{array}}_{261.94} = 1.27 = 1 \text{ limited to } 3.00$				
	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 = 555.18 x 1.65 = 916.047 kN or	= 916047	N		
		N			
	Utimate axial load = min value of = 91604/ or 256981./5 = 256981.8	N			
	0.45 v 050004.7545 0.0475				
	Shear stress enhancement Factor = $1 + \frac{0.15 \times 250981.7545}{155746.52} = 0.2475$				
	10 10				
	Vuc = <u>0.24</u> x 1.18 x 1.27 x 15.00 x 304.8 1.15	x 261.94	= ##	kN	
	Enchancement Factor = 52.30 x 0.2475 = 12.94 kN				
	$N_u = \left(\frac{0.6f_{cu}}{\gamma_{mc}}\right) bd_c + \left(\frac{f_y}{\gamma_{ms}}\right) A'_{sl} + \sigma_{s2} A_{s2}$ $d' = 9.525 \text{ mm}$				
	d_c = taken as h = 304.8 mm d2	= 261.94	mm		
	$A'_{sl} = \frac{3166.92}{2} = 3166.92 \text{ mm}^2$ b	= 304.8			
	σ_{s2} = from figure 3.13.1 = $\frac{230}{1.15}$ = 7.83 n/mm ²				
	$A_{s_2} = 3166.92 \text{ mm}^2$				
	b _ 204.9				
	II = 30%.0				
	Nu = <u>0.6 x 15.00</u> x 304.8 x 304.8 <u>1.5</u>	= 557.42	kN		
	= <u>230</u> x 3166.92	= 633.38	kN		
	1.15 - 7.83 y 3166.92	- 24.78	kN		
	Nu = 1215.59 kN				
	Ultimate bending resistance moment in columns				
	Equation 7.4.1b Ultimate resistance moment in columns $M_u = \left(\frac{0.3f_{cu}}{\gamma_{mc}}\right) b d_c (h - d_c) + \left(\frac{f_y}{\gamma_{ms}}\right) A'_{sl} \left(\frac{h}{2} - d'\right) - \sigma_{s2} A_{s2} \left(\frac{h}{2} - d_2\right)$				
	dc = 2 x d' due to looking at tension = 2 x 9.525 = 19.05 mm				



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	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 = <u>1053230</u> = <u>3455.48</u> = 11.34 <u>304.8</u> 304.8	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				

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	Assessment of in service parapets							
	The parapets over the structure are of precast con	crete construction. The parapets connecte	d to the parapet beam by 1 1/2" do	wels.				
	The parapet assessment will be conducted in acco	ordance with CS 461 Section 3, Risk asses	sment of exisiting parapet site and	prioritation of parap	et upgradin	IQ.		
	Due to Station Way bridge spanning over LUL infa	structure, the parapet will be assessed usi	ng figure 3.2b of CS 461.	F		.9-		
		Figure 3.2b Assessment flowchart , bridge	s over railways					
		Very los	risk Low risk Medium risk	High risk				
	Compile data							
	-							
	Rac < 100		1	H4a upgrade				
	Yas							
	R _{CON1} < C _{MM}			N2 upgrade				
	No							
	ROUT CALL NO	Monitor	only					
	Yes							
	Research 10	No.						
	Yes		N1/N2 upgrade					
		Now	wick Undertake as part of	Stand-alone				
		requir	ed maintenance or major works	schemes				
			1					
	Table 3.5 Definition	Tor parameters used in risk proces	Notos	1				
	R _{ALARP}	ALARP-based risk ranking score	Refer to A2 and A4 of Append	tix A.				
	R _{INC}	incursion risk ranking score for the highest scoring corner	Refer to Appendix B (also see sub-clauses 3.5.1 a	nd 3.5.2).				
	RCONT	remnant resistance of the parapet	expressed as a proportion of containment resistance C_{REQ} . Section 4 for guidance.	the required . Refer to				
	CALL	allowable resistance of the parapet	Refer to Equation 3.6.					
	C _{MIN}	minimum resistance of the parapet	Refer to Equation 3.7 and Tab	ole 3.8.				
	C _{REQ} required containment resistance of the parapet Refer to Equation 3.6 and Table 3.6.							

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																•	•
Ē	Equatio	n A.1	ALARP	P - based risk	ranking	g score											
F	R _{ALARP}	=		AADT	х	F ₁	x 1000	F ₂	х		F ₃						
	AADT		_	8033	(taker	from FCC	Traffic Count	t dated 31	01 22 - 06	02 22)							
ſ	=.		_	5	(tartor	104-3304)				.02.22)							
	-		-	4.40	(- 0 C)										
ľ	2		=	1.12	()	ootway wiu	un = 2.5)										
ľ	3		=	1.00	(r	new drilled a	ancnors)										
F	R _{ALARP}	=		8033	х	5	x	1.12	х		1.00	_=	4.50				
							1000	0									
1	ncursic	on Ris	sk Rank	<u>ting</u>													
	The ove	erall sco	ore for a b	oridge is obtained	d by adding	g all 14 factors	s together.										
	As a gui doubling range of	ide, an g of the f risk va	increase risk, so 6 alues. A s	of two in a score is twice as bad score of 90 implie	e for any of as 4, and es that the	f the factors or 12 is eight tim risk is approx	for an overall r les worse than 6 imately a millior	isk score im 6. This give 1 times bigg	nplies a s a wide ler than a								
	score of The sco	1 50. oring rea	gime assu	imes that no fac	tor needs	a score of zer	o, as even the b	pest protecti	on still								
	allows a	a slim ci ors are	hance of a to rank br	a vehicle or debr ridges according	to score,	ng the line. assessing the	highest scoring	j bridges in i	more detail								
	to see h more we improve	ow the ould su ments.	y can be i ggest that This doe	Improved. As a g t highway author is not rule out sir	guide, scor rities shou mple and c	res of 100 or n Id at least con cost-effective i	nore are signific sider the practic mprovements a	cant and sco cability of t bridges tha	ores of 70 or at score								
	less tha 1) for bi	n 70. M	Aitigation a	action is not stric	tly require	ed when: at either score	e one for factor :	1 (road app	roach								
	2) for br	ainmen ridges o	t) or score carrying m	e of 1 for factor 5 notorway and du	al carriage	graphy); and, way roads the	at score one for 5 (site topograp)	factor 1 (ros	ad								
l	or les	ss for fa	actor 8 (ve	ehicle parapet re	silience).	ar i tor factor :	s faire rohoðrahi	.y, anu a S	Sole of two								
F	For Mas	s Cor	ncrete Pa	arapet	rail												
ľ	- 1 IGUIO	actor	S			lue	tification										
f	1 =	40101	- 12 1	Medium / lig	htly woo	ded approa	ches / imperf	fect fencin	ng								
- 11	∠ =			omayni													
f	3 =		2	Slight hump	back					Tet-	J	57		100			
f f f	3 = 4 = 5 =		2 3 1 5	Slight hump 30mph Vehicle / De	back bris unlil	kley to foul t	rack			Tota	I	57	<	100			
f f f f	3 = 4 = 5 = 6 = 7 =		2 3 1 5 1	Slight hump 30mph Vehicle / De Residential No obvious	back bris unlil parking o hazards	kley to foul t on approach	track n to structure			Tota	I	57	<	100			
f f f f f f	3 = 4 = 5 = 6 = 7 = 8 = 9 =		2 3 1 5 1 5 1 1	Slight hump 30mph Vehicle / De Residential No obvious Assumed as Footpath at	back bris unlil parking o hazards s masonr least 2m	kley to foul t on approach ry parapet i on both sic	track to structure	d viciblo		Tota	I	57	<	100			
f f f f f f f f f	3 = 3 4 = 3 5 = 3 6 = 3 7 = 3 8 = 3 10 = 3 11 = 3 12		2 3 1 5 1 5 1 1 1 1	Slight hump 30mph Vehicle / De Residential No obvious Assumed as Footpath at signage / ma Local B clas	back bris unlil parking o hazards s masonr least 2m arkings f	kley to foul t on approach ry parapet o on both sic it for purpos	track h to structure des ses, clear and	d visible		Tota	l	57	<	100			
f f f f f f f f f f	3 = 44 = 55 = 76 = 77 = 88 = 99 = 101 = 112 = 113 = 114		2 3 5 1 5 1 1 1 1 1 11	Slight hump 30mph Vehicle / De Residential No obvious Assumed as Footpath at signage / ma Local B clas Straight trac Light rail	back bris unlil parking o hazards s masonr least 2m arkings f s road sk up to 4	kley to foul t on approach y parapet on both sic it for purpos 15mph	track h to structure des ses, clear and	d visible		Tota	ı	57	<	100			
f f f f f f f f f f	3 = 4 = 5 = 6 = 7 = 8 = 9 = 10 = 11 = 12 = 114 = 144 = 114 =		2 3 5 1 5 1 1 1 1 1 11 12	Slight hump 30mph Vehicle / De Residential No obvious Assumed as Footpath at signage / ma Local B clas Straight trac Light rail Very heavily	back bris unlil parking o hazards s masonr least 2m arkings f as road k up to 4 v used ro	kley to foul t on approach ry parapet o on both sic it for purpos 45mph pute	track n to structure des ses, clear and	d visible		Tota	l	57	<	100			
f f f f f f f f f f f f f f f f f f f	3 = 4 = 5 = 6 = 7 = 8 = 9 = 10 = 11 = 12 = 13 = 14 = 7 CONT		2 3 1 5 1 1 1 1 11 12 =	Slight hump 30mph Vehicle / De Residential No obvious Assumed as Footpath at signage / ms Local B clas Straight trac Light rail Very heavily 34%-66%	back bris unlil parking o hazards s masonri least 2m arkings f s road ck up to 4 v used ro =	kley to foul t on approach o on both sic it for purpos 45mph pute 1.00	track h to structure des ses, clear and	d visible		Tota	1	57	K	100			
f f f f f f f f f f f f f f f f f f f	3 = 3 4 = 3 5 = 3 6 = 3 7 = 3 8 = 3 9 = 3 10 = 3 11 = 3 12 = 3 112		2 3 1 5 1 5 1 1 1 1 1 1 1 2 =	Slight hump 30mph Vehicle / De Residential No obvious Assumed as Footpath at signage / m Local B clas Straight trac Light rail Very heavily 34%-66%	back bris unlill parking o hazards s masonr least 2m arkings f is road k up to 4 v used ro = N2	kley to foul t on approach y parapet o n both sic it for purpos 15mph uute 1.00 Therefore	rrack h to structure des ses, clear and	d visible 1	N2	Tota	d	57	<	100			
f f f f f f f f f f f f f f f f f f f	3 = 3 4 = 3 5 = 3 6 = 3 7 = 3 8 = 3 10 = 3 11 = 3 11 = 3 12 = 3 11 = 3 12 = 3		2 3 1 5 1 1 1 1 1 1 1 1 2 = =	Slight hump 30mph Vehicle / De Residential No obvious Assumed as Footpath at Local B clas Straight trac Light rail Very heavily 34%-66% 1 0.67C _{RFO}	back bris unlill parking o hazards s masonri least 2m arkings f is road k up to 4 v used ro = N2 =	kley to foul t on approach y parapet on both sic it for purpos 45mph uute 1.00 Therefore 0.67	rrack h to structure des ses, clear and = x	d visible 1	N2 =	Tota	0.67	57 N2	<	100			
f f f f f f f f f f f f f f f f f f f	3 = 3 4 = 5 5 = 6 7 = 3 8 = 3 10 = 3 111 = 3 112 = 3 111 = 3 112 = 3 111 = 3		2 3 5 1 5 1 1 1 1 1 1 1 1 2 = =	Slight hump 30mph Vehicle / De Residential No obvious Assumed as Footpath at signage / m Local B clas Straight trac Light rail Very heavily 34%-66% 1 0.67C _{REQ}	back bris unlii parking o hazards s masonri least 2m arkings f is road k up to 4 v used ro = N2 =	kley to foul t on approach on both sic it for purpos I5mph uute 1.00 Therefore 0.67	rrack h to structure des ses, clear and = x	d visible 1 1	N2 =	Tota	0.67	57 N2	<	100			
	3 = 3 4 = 3 5 = 3 7 = 3 9 = 3 110 = 3 111 = 3 111 = 3 112 = 3 114 = 3 C_{REQ} C_{ALL}		2 3 5 1 5 1 1 1 1 1 1 2 = =	Slight hump 30mph Vehicle / De Residential No obvious Assumed as Footpath at signage / m Local B clas Straight trac Light rail Very heavily 34%-66% 1 0.67C _{REQ} 0.50	back bris unlii parking o hazards s masonn least 2m arkings f s road ck up to 4 r used ro = N2 = N2	kley to foul t on approach y parapet on both sic it for purpos 45mph uute 1.00 Therefore 0.67	rrack h to structure des ses, clear and = x	d visible 1 1	N2 =	Tota	0.67	57 N2	<	100			
ffffffffffffffffffffffffffffffffffffff	3 = 4 5 = 5 7 = 8 8 = 9 10 = 112 = 112 = 112 = 112 = 112 = 112 = 112 = 112 = 114	ry	2 3 1 5 1 5 1 5 1 1 1 1 1 1 1 1 1 2 = = =	Slight hump 30mph Vehicle / De Residential No obvious Assumed as Footpath at Local B clas Straight trac Light rail Very heavily 34%-66% 1 0.67C _{REQ} 0.50	back bris unliil parking of hazards s masonni least 2m arkings f s road k up to 4 v used ro = N2 = N2	kley to foul t on approach y parapet on both sic it for purpos 45mph uute 1.00 Therefore 0.67	rrack h to structure des ses, clear and = x	d visible 1 1	N2 =	Tota	0.67	57 N2	<	100			
f f f f f f f f f f f f f f f f f f f	3 = 4 = 5 = 5 = 6 = 7 = 8 = 10 = 11 = 12 = 11 = 12 = 13 = 14 = Call Call Call Call Call Call Call Call	ry	2 3 1 5 1 5 1 5 1 1 1 1 1 1 1 1 1 1 2 = = = = = = = = =	Slight hump 30mph Vehicle / De Residential No obvious Assumed as Footpath at signage / m Local B clas Straight traci Light rail Very heavily 34%-66% 1 0.67C _{REQ} 0.50	back bris unlill parking masonn masonn least 2m markings f k up to 4 v used ro = N2 = N2 <	kley to foul t on approach y parapet o on both sig it for purpos 15mph uute 1.00 Therefore 0.67	rrack h to structure des ses, clear and = x Yes	d visible 1 1	N2 =	Tota	0.67	57 N2	<	100			

JACOBS								CALCUI	ATION SHEET
OFFICE	MANCHESTER			PAGE	No.	A 1	CONT'N PAGE No.		A 2
JOB No. & TITLE	ESSEX COUNTY COUNC	IL - STATION WAY	BRIDGE ASSESSME	NT ORIGI	NATOR	MA	DATE		24/02/2025
SECTION	ASSESSMENT OF ORIGIN	NAL FOOTWAY SL	ABS (SPAN 1 & 5)	CHEC	KER	CATII	DATE		06/03/2025
REF			CALCULATION						
	APPROACH SPAN (SPAN 1)	- ORIGINAL FOOT	WAY SLAB						
	Note: Accidental vehicle loc combined with pedestrian RESULTS SUMMARY Longitudinal Bending	GE MODEL USED ading has not been loading. By inspection	considered as slab has sion, slab not adequate f	210 210 210 210 200 200 200 200	to be only of accider	/ just adequate ntal vehicle load	for permanen ling.	t loading	
	Main Slab								
	ULS Bending								
	Loading / Effect Sagging (Pedestrian Loading)	*Capacity (kN.m) 3.6	*DL+SDL (kN.m) 2.6	* LL (kN.m) 0.8	*Tota	al BM (kN.m) 3.5	Pass / Fail PASS	Utilisation (%) 97	
	ULS Shear								
	Loading / Effect	*Capacity (kN)	*DL+SDL (kN)	*LL (kN)	*To	otal 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 Downstand ULS Bending Loading / Effect	compared with app **Capacity (kN.m)	lied shear force at supp	oort, therefo	ore conser	vative. al BM (kN.m)	Pass / Fail	Utilisation (%)	
	Loading)	27.0	7.8	2.4		10.2	PASS	38	
	ULS Shear								
	Loading / Effect	**Capacity (kN)	**DL+SDL (kN)	**LL (kN)	**T(otal 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 (a Transverse Bending - Main	t bottom of section I <u>Slab</u>)						
		Lon	ngitudinal Member (Ma	in Slab)	Transv	erse Member (I	Vlain Slab)		
	Width of member		365mm			390mm			
	Sagging (bottom) reinforce	ement	3/8" dia at 6" centre	s	3/8" dia at 3"cent		ntres		
	Max ULS applied sagging b moment (for width above)	ending)	3.5 kN.m		2.4 kN.m				
	Capacity		3.6 kN.m		> 3.6 (By inspect		pection)		
	Conclusion		ОК			OK (By Inspecti	on)		

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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	CALCULATIC	N			OUTPUT
	MATERIAL PROPERTIES AND PARTIAL FACTORS				
	Reinforced Concrete				
AIP, CI 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, CI 3.10	Mild steel reinforcement characteristic yield strength	= 230	N/mm ²		
	Design Young's Modulus of steel reinforcement	= 2000	000 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	$\gamma_{\rm ms}$ = 1.15			
CS 455, Tbl 2.13a	Partial factor for concrete	$\gamma_{mc} = 1.5$			
CS 455, Tbl 2.13a	Partial factor for shear in concrete	$\gamma_{mv} = 1.25$			
CS 455, Tbl 2.13a	Partial factor for bond	$\gamma_{mb} = 1.4$			
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	$\gamma_{f3} = 1.1$			
CS 454 Tbl 3.4	Partial factor for live loading	$\gamma_{fL} = 1.5$			



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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	CATU	DATE	06/02/2025
		<u> </u>	CATI		06/03/2025
REF	CALCULATIO	DN			OUTPUT
		390 mm			
		2			
	y"	~ •	150	mm	
	Deinforcement (only used for BC design checks)				
	None ~				
	ez origin Centroid				
	Cross sectional area (A) 0	Value .0585	I		
	Second moment of area about y axis (lyy) 0.10 Second moment of area about z axis (lzz) 0.74	9687E-3 1488E-3			
	Product moment of area (lyz) Torsional constant (J) 0.33	0.0 2485E-3			
	Effective shear area in z direction (Asy) 0.0 Effective shear area in z direction (Asz) 0.0 Eccentricity in x direction (ev)	48763 0.0			
	Eccentricity in z direction (ez)	0.0			
	Visualise Tapering >>	Section details			
	Transverse Members (A.C	• (3)			
	Name (Main siab)				
			(alua	_	
	Thermal expansion Toisson's ratio	24	.0E6).2	kN/m2	
	Mass density	2	2.4	t/m3	
	Name Longitudinal members	~	▲ (4)		
			• • •		
	Elastic				
	Dynamic properties		Value	kN/m2	
	Thermal expansion Young's modulus Poisson's ratio	2	4.0E6 0.2	+/m2	
			.UE-6	1/113	
	Name Transverse members	~	▲ (3)		
	L				

JACOBS							CALCULATION SHEET
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JOB No. & TITLE	ESSEX COUNTY COU	JNCIL - STATION V	WAY BRIDGE ASSESSMENT	ORIGINATOR	MA	DATE	24/02/2025
SECTION	ASSESSMENT OF OF	RIGINAL FOOTWA	Y SLABS (SPAN 1 & 5)	CHECKER	CATII	DATE	06/03/2025
REF			CALCULATIO	DN .			OUTPUT
	LOADING						
	Self-weight						
	Width = 2	00 + 1790 + 2	200 = 2190				
	depth = 2	50 mm 150 m	nm 250 mm				
	Length = 31	120 mm					
	Density = 24	4.0 kN/m3					
L	Infactored selfweight = 27	.59 kN					
	Factored selfweight = 27	γ _{fL} .59 x 1.15 x	γ _{f3} 1.1 = 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 =	γ _{fL} 82.7 x 1.2 x	γ_{f_3} 1.1 = 109.1 kN				
	Concrete pavings						
	Width =	2190 mm					
	depth =	50.8 mm					
	Length =	3120 mm					
	Density =	21.6 kN/m3					
Unfact	ored Con Paving Loading =	7.5 kN					
Fact	ored Con paving Loading =	γ _{fL} 7.5 x 1.2 x	γ _{f3} 1.1 = 9.9 kN				
	Surfacing						
	Width =	2190 mm					
	depth =	100 mm					
	Length =	3120 mm					
	Density =	24.0 kN/m3					
Unfa	ctored Surfacing Loading =	16.4 kN					
Fa	ctored Surfacing Loading =	γ _{fL} 16.4 x 1.75 x	γ _{f3} 1.1 = 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 -	γ _{fL} 34.2 x 15 v	γ _{f3} 1.1 = 56.4 kN				











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JOB No. 8. 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	CALCULATIO	DN			OUTPUT
	SHEAR CAPACITY - MAIN SLAB				
	For 365mm wide member:				
Page ref. A 7	$As = 196.0 \text{ mm}^2$				
Page ref. A 7	d = 100.1 mm				
	Maximum shear Resistance based on concrete crushing		1	W # MAN & CP	-
Eqn. 5.6a	$V_{\text{max}} = 0.36 \left(0.7 - \frac{f_{cu}}{270} \right) \left(\frac{f_{cu}}{2} \right) b_w d$		14/	And and a state of the state of	2
	$(250)(\gamma_{mc})^{-1}$		mer leter /	- the 's have B J'p	
	$V_{max} = 0.36 \times \begin{bmatrix} 0.7 & -\frac{15.0}{250} \end{bmatrix} \times \frac{15}{1.50} \times 365 \times 100 = 8$	34.2 kN		SECTION CC	
	Shear Resistance more than 3d from a support	Į.			
Eqn. 6.5	$V_{uc} = rac{0.27}{c} \xi_s ho_s^{rac{1}{2}} f_{cu}^{rac{1}{2}} b_w d$				
	1000	H.			37
	$\xi s = \frac{500}{100}^{0.25} = 1.49 \qquad \qquad \xi_s \text{ is the depth factor, taken} \\ \xi_s = \left(\frac{500}{d}\right)^{0.25} \text{ but not less}$	as ss than 0.7		* * * * * * * * * * * * * * * *	
	$\rho_s = \frac{100 \times 196.0}{365 \times 100} = 0.54$				
	$V_{uc} = 0.27 \times 1.49 \times 0.54^{-1/3} \times 15.0^{-1/3} \times 365.0 \times 100^{-1/3}$				
	V _{uc} = 23.6 kN Capacity at 3d (365mm wide member)				

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JOB No.	ESSEX COUNTY COUNCIL - STATION WAY BRIDGE ASSESSMENT	ORIGINATOR		DATE	04/00/2005
SECTION	ASSESSMENT OF ORIGINAL FOOTWAY SLABS (SPAN 1 & 5)	CHECKER	CATII	DATE	06/03/2025
	SHEAR RESISTANCE OF THE DOWNSTAND (CONSERVATIVELY IGNORE T	HE SHEAR REINFO	DRCEMENT)	•	
5.6	The assessment shear force shall not exceed:				
	1) V_{max} as defined in Equation 5.6a, anywhere;	and and			
	2) V_{nc} as defined in Equation 5.6c, within 3 <i>d</i> of a support.	pport, and, 19	4	A the stierus	05 th 6 p
Egg E 6o		d		1 0	
Eqn. 5.6a	$v_{\text{max}} = 0.50$ X $(0.70 - \frac{1}{2})$ X $(\frac{1}{2})$ X D_w X 250 γ_{mc}	. u	14		
			Li	,	
	$f_{cu} = \frac{15}{N/mm^2}$		Section	. WW	
	$Y_{mc} = \frac{1.50}{2.03}$ mm = 8" (at bottom of section)		MERIAN	vnn	
Page ref. A 8	d = 197 mm				
	∴ V _{max} = 92.2 kN				
Eqn. 5.6b	V_{uc} = 0.24 x ξ_s x $\rho_s^{1/3}$ x $f_{cu}^{1/3}$ x b_w x d				
	Υ _{mv} where:				
	γ_{mv} is the partial factor for shear ξ_s is the depth factor				
	$\rho_s \text{is the ratio of longitudinal reinforcement}$				
	γ _{mν} = 1.25				
	$\xi_{\rm s} = (500)^{1/4} > 0.70$				
	$\xi_s = 1.26$				
	$\rho_{s} = \frac{100 \ x \ A_{s}}{b_{w} \ x \ d} > 0.15 \ \& < 3.00$				
	$A_s = 855 \text{ mm}^2$				
	ρ _s = 2.14				
	$\therefore V_{uc} = \frac{30.8}{kN} $ Capacity at 3d				
Eqn. 5.6c	$V_u = \max \left\{ egin{array}{c} rac{3d}{a_c} \Gamma V_{uc} \ rac{0.24}{\gamma_{mv}} arsigma_s (0.15 f_{cu})^rac{1}{3} b_w d \end{array} ight.$				
Eqn. 9.1b	$f_{ub} = \frac{\kappa \kappa_{cov} \beta \sqrt{f_{cu}}}{\gamma_{mb}}$				
	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/mm ² $K_{cov} = 1.0$ (taken as 1)				
	f _{ub} = 1.08				
	$F_{ub} = f_{ub}pL_a$				
	P = 179.5 mm (For 3 no. bars)				
	$L_a = 197 \text{ mm}$ (conservative taken as d)				
	F _{ub} = 38.2 kN				

JACOBS				CALCU	LATION SHEET					
OFFICE	MANCHESTER	PAGE No.	4.40	CONT'N						
JOB No.	ESSEX COUNTY COUNCIL - STATION WAY BRIDGE ASSESSMENT	ORIGINATOR	A 13	PAGE No. DATE						
& TITLE			MA		24/02/2025					
SECTION	ASSESSMENT OF ORIGINAL FOOTWAY SLABS (SPAN 1 & 5)	CHECKER	CATII	DATE	06/03/2025					
Eqn. 5.6d	$\Gamma = \min \left\{ egin{array}{c} \sqrt{rac{z \ F_{ub}}{3d} V_{uc}} \ 1.0 \end{array} ight.$									
Page ref. A 8	z = 157.6									
	$\Gamma = \left(\frac{157.6}{590.9} \times \frac{38.2}{30.8}\right)^{1/2}$									
	$\Gamma = (0.27 \times 1.24)^{1/2}$									
	Γ = 0.57									
	$V_u = \frac{3d}{a_v} \Gamma V_{uc} = \frac{3 \times 197}{197} \times 0.575 \times 30.8 = 53.1$ (a)	t d)								
	$V_{v} = \frac{0.24}{\gamma_{mv}} \xi_{s} (0.15 f_{cu})^{\frac{1}{3}} b_{w} d = \frac{0.24}{1.25} \times 1.26 \times (0.15 \times 10^{-3})^{\frac{1}{3}} k_{w} d = \frac{0.24}{1.25} \times 1.26 \times (0.15 \times 10^{-3})^{\frac{1}{3}} k_{w} d = \frac{0.24}{1.25} \times 1.26 \times (0.15 \times 10^{-3})^{\frac{1}{3}} k_{w} d = \frac{0.24}{1.25} \times 1.26 \times (0.15 \times 10^{-3})^{\frac{1}{3}} k_{w} d = \frac{0.24}{1.25} \times 1.26 \times (0.15 \times 10^{-3})^{\frac{1}{3}} k_{w} d = \frac{0.24}{1.25} \times 1.26 \times (0.15 \times 10^{-3})^{\frac{1}{3}} k_{w} d = \frac{0.24}{1.25} \times 1.26 \times (0.15 \times 10^{-3})^{\frac{1}{3}} k_{w} d = \frac{0.24}{1.25} \times 1.26 \times (0.15 \times 10^{-3})^{\frac{1}{3}} k_{w} d = \frac{0.24}{1.25} \times 1.26 \times (0.15 \times 10^{-3})^{\frac{1}{3}} k_{w} d = \frac{0.24}{1.25} \times 1.26 \times (0.15 \times 10^{-3})^{\frac{1}{3}} k_{w} d = \frac{0.24}{1.25} \times 1.26 \times (0.15 \times 10^{-3})^{\frac{1}{3}} k_{w} d = \frac{0.24}{1.25} \times 10^{-3} \times 10^{$	5.0) ^{1/3} x 203	x 197 = 12	2.7 kN						
	$\therefore V_u = 53.1 \text{ kN} $ Capacity at d									

JAC	OBS								CALCULAT	ION S	HEET	
Office	Manchester								Page No.	B 1	Calc No.	
Job No. & Title	ECC - Assessment o	f Station Way Bridge							Calcs by	MA	Date	Mar-25
Section	Main Deck (Span 2 to	o 4) - Carriageway slab							Checker	CAT II	Date	Mar-25
		S FOR ASSESSMENT		AB (SPAN 2 -			AR					
		EMENT	OT MAIN DECK SE									
Record dras:								Main Bean	m			
LC5/2 LC5/6		Main Beam	T	ransverse direc	tion	A		I I				
				<		▼ 8"			>			
		1' 0"						1'0" i				
				6' 0" 1.83	m							
	Idealised diagra	am										
		1						F				
	For Hogging:	Fixed end						F	Fixed end			
		4		1.83	m			Ē				
				1.05				1				
	For Sagging:	Free ends (Simply										
		supported)										
		-		1.83	m			1				
	MATERIAL PRO	OPERTIES AND PARTIA	AL FACTORS									
	Reinforced Cor	ncrete										
AIP, CI 3.10	Characteristic St	trength of Concrete				=	15.0	N/mm ²				
CS 454	Density of Reinf	orced Concrete				=	24	kN/m ³				
table 4.1.1a	Steel Reinforce	ement										
AIP, CI 3.10	Mild steel reinfor	rcement characteristic y	ield strength			=	230	N/mm ²				
	Design Young's	Modulus of steel reinfor	rcement			=	200000	N/mm ²				
	Carriageway &	Footway Surfacing										
CS 454 Table 4.1.1a	Unit weight of Bi	ituminous Macadam (tai	r)			=	24	kN/m ³				
	Partial Factors											
CS 455 Table	Partial Factor fo	r reinforcement			γ _{ms}	=	1.15					
2.13a												
CS 455 Table 2.13a	Partial Factor to	r Concrete			Ϋ́mc	=	1.5					
CS 455 Table	Partial Factor fo	r shear in concrete			γ _{mv}	=	1.25					
2.13a												
CS 455 Table 2.13a	Partial Factor fo	r Bond			Ϋ́mb	=	1.4					
CS 454,	Partial Factor fo	r Concrete Deadload				=	1.15					
Tbl 3.4												
CS 454, Tbl 3.4	Partial Factor fo	r Surfacing				=	1.75					
CS 454,	Partial Factor fo	r nomral traffic, restricte	d traffic and footwa	y loading	ΥfL	=	1.5					
1013.4	lag	comont affects and a										
CS 454 CI 3.9	inaccurate asse	ssment effects at ULS			Ύf3	=	1.1					

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b No. & Title	ECC - Assessment of Station Way Br	idge			Calcs by	MA	Date Mar-25						
ection	Main Deck (Span 2 to 4) - Carriagewa	ay Slab - Results Summary			Checker	CAT II	Date Mar-25						
	<u>Results Summary</u> <u>Normal Traffic Loading</u> Bending	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 / Fa	Utilisation (%)						
	44 Tonnes Sagging	45.1	34.4	38.8	PASS	86							
	44 Tonnes Hogging	-50.1	-38.7	PASS	77								
	Shear												
	Loading / Effect	Capacity (kN/m)	DL+SDL (kN/m)	LL (kN/m)	Total SF (kN/m)	Pass / Fa	I 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						
	<u>SV80 Loading</u> Bending												
	Loading / Effect	Capacity (kN.m/m)	DL+SDL (kN.m/m)	LL (kN.m/m)	Total BM (kN.m/m)	Pass / Fa	Utilisation (%)						
	SV80 Sagging	45.1	34.0	38.5	PASS	85							
	SV80 Hogging	-50.1	-3.0	-27.4	-30.4	PASS	61						
	Shear		<u>.</u>										

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:

The calculations are based on Pucher Charts and a line beam model assuming 1.0m wide carriageway slab.
 For hogging moment, full fixity along the supported edges of each deck slab bay is assumed (conservative).
 For sagging moment, no moment fixity along the supported edges is assumed (conservative).
 Capacities are based on the defects identified in Appendix E of the AIP.
 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).

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Section	Main Deck (Span 2 to 4) - Carriagew	ay Slab - Capacity					Checker	CAT II Date	Mar-25		
	CARRIAGEWAY TRANSVERSE	SLAB - BENDING C	APACITY (FULL SEC	TION WITHOUT DEFE	CTS)	6.10		11 Juli			
	eser	1:0- 4/10' \$ bars \$ 5	**	- 440 ' 6 bare D 344' 6		5-5 8. 4 para D	0.5 'A				
	*******				1		10' \$ Lore \$ 8'p				
	Main longitudinal Transve beam for slab and s	erse direction reinforcement lab bending	Main longitudinal beam Section of carriagew	vay slab from archive [Vlain longitu beam Drg LC 5/6	ıdinal	y (44 y	Main longitudinal beam			
LC 5/6	Reinforcement										
	Bar Type	Bar Dia (")	Bar Dia (mm)	Area of Bar (mm ²)	Spa Inches	cing mm	No. Bars (per m width)	Area of Reinforcem (mm ² /m)	ent		
	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			
	Slab Depth Effective Depth z $M_u = \frac{230}{1.15}$ $M_u = 0.225$	= = _ x 2019.8 x 15.00 1.5	203.2 mm 203.2 - 124.02 mm 5 x	38.1 - 124.02 = 1000 x	7.144 50.1 158	= kNm 2	158 mm = 56.14	kNm			
	M _u = 50.1 <u>Hogging (Transverse)</u>	kN.m/m	(without defects)								
	Cover to Reinforcement	=	1 1/2" =	38 mm							
	Slab Depth Effective Depth	=	203.2 mm 203.2 -	38 -	7.1	=	158.1 mm				
	Z	=	124.02 mm								
	M _u = <u>230</u> 1.15	x 2019.8	5 x	124.02 =	50.10	kNm					
	M _u = <u>0.225</u>	x 15.00 1.5	x	1000 x	158.1	2 =	56.21 kNm				
	M _u = 50.1	kN.m/m	(without defects)								

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Section	Main	Deck (S	Span 2 to 4)) - Carriagewa	ay Slab - Capacity				Checker	CAT II	Date	Mar-25	
		CARRIA	AGEWAY TR	RANSVERSE	SLAB - BENDING (CAPACITY (ASSUMING	3 1MM SECTION LOS	S TO TRANSVERSE B	OTTOM BARS*)				
			Devition		Bar Dia	Bar Dia (corroded)	2	On a since (sum)	No. Dour	Area of	Reinforceme	ent	
			Bar Typ	be	(uncorroded)	(mm)	Area of Bar (mm ⁻)	Spacing (mm)	No. Bars (corroded) (mm ²)				
		Tra	insverse Bot	ttom Bar	14.3	13.3	138.7	79.4	12.6		1747.0		
		Т	ransverse T	op Bar				No defects	•	•			
	L			-									
	1	Sagging	g (Transver	rse)									
		Momen	t Resistanc	e for beams	without compression								
		Mu	= N	Vinimum value	e dervied from the fo								
		Cover to	o Reinforcer	ment	=								
		Slab De	pth		=	203.2 mm							
		Effective	e Depth		=	203.2 -	38 -	6.64 =	158.5 mm				
		Z			=	129.11 mm							
		Mu	=	230	x 1747.0	00 x	129.11 =	45.11 kNm					
		-	_	1.15									
		Mu	= _	0.225	x 15.00) x	1000 x	158 ²	= 56.49	kNm			
					1.5								
	Ιſ	Mu	=	45.1	kN.m/m	(Allowing for corrosion)						
	'												
		* Refer	to photo no.	. 3 in Appendi	x E of AIP - Allowan	ce for corroded transve	erse reinforcement in se	offit of the deck.					
		Defect is	s localised ((5 no. transve	rse bars exposed). 1	m width of slab include	es approx. 12.5 no. trar	sverse bars. Allow for	corrosion of these 5 no.	bars by as	suming a 1mr	m	
	1	loss of c	diameter to a	all bars within	1m strip.					,	5		

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Section	Main Deck (Span 2 to 4) - Carriage	eway Sla	b - DL& SDL							CI	hecker	CAT II	Date	Mar-25
		<u>G</u>									I		I		I
	DL & SID loading														
				Depth (m)	v	Per metre width	×	Density	×	Υ _{fl}	v	γ _{f3}	- 18	kN/m	\$1.5
	Self-weight			0.2	^	1.0	^	24	~	1.15	~	1.0	- 4.0	khl/m	111.0
									x	1.15	x	1.1	= 0.07		OLS OLS
	Surfacing			0.1	х	1.0	x	24	x	1.0	x	1.0	= 2.4	KIN/M	515
									х	1.75	х	1.1	= 4.62	kN/m	ULS
	Total DL & SID (Un	nfactored)											7.2	kN/m (SLS)	
	Total DL & SID (factored)											10.7	kN/m (ULS)	I	
															l
	Bending Moment a	and Shear	Force di	ue to ULS DI	L & SDL										
	M _{Sagging} =	wL ² /8	=	4.	.5 kN.m	(Assuming simp	oly supp	oorted)							
	М		_	3	0 kN m	(From line bear	n mode	Lassuming	-	1		10.7	kN/m		E
	Hogging -		-	5.	.0 KN.III	fixed end suppo	orts)					1828.8	mm		F
	Shear at d =		=	7.	.7 kN	(From line beam model assuming fixed end supports)							2.95	2E6	
	Shear at 3d = = 5.5 kN				(From line beam model assuming fixed end supports)					E3		111111	LINE L	e.	
							,		150mm	T	-927 1	E3 -1.493E6	-1.077E6		
									d from	supp				1.057F3 9	705.E.B
									-			7455	2,734E3		-
										705E3 +7.699E3	-6.452E3				
										450mm 3d from sup	D				
											F				

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Section	Main Deck (Span 2 to 4)	- Carriage	eway Slab - Live Load	ding			Checker	CAT II	Date	Mar-25
	Live Loading (Nor	mal Traffic	- 44T)							
CS 454 Table B.1	Axle load =	113	kN	(Primary wheel load)		Contact are 300	ea for 44T) mm	Wheel loa the top of	d dispersed (2 the slab.	2V:1H) to
	Wheel load =	56.5	kN Unfactored	Surfacin	a			100 mm		
	Traffic flow factor	0.9		Slab	J			150 mm		
	Impact factor =	1.8			•	400) mm			
	Lane factor =	1.0								
	$\gamma_{\rm fl} =$	1.5								
	$\gamma_{f3} =$	1.1								
	Wheel Load =	151.0	kN Factored							
	Wheel Load -	943.9	kN/m2	Factored (For 0.4m x 0.4m cont	act area)					
	Wheel Load -	343.3	KIN/1112		actarea					
CS 454	Axle load =	74	kN	(Secondary wheel load)		0	(/ / T	14/h1 l	d diana and (
Table B.1	Wheel load =	37	kN Unfactored			300) mm	the top of	the slab.	20:1H) to
	Traffic flow factor	0.9		Surfacin	g/	/		100 mm		
	Impact factor =	1.0	Impact factor	only applied to primary Slab				150 mm		
	Lane factor =	1.0	wheel load		•	400) mm			
	$\gamma_{\rm fl} =$	1.5								
	V _{f2} =	1.1								
	Wheel Load -	54.9	kN Eactored							
	Wheel Load -	54.5	KIN TACIOTEU							
	Wheel Load =	343.4	kN/m2	Factored (For 0.4m x 0.4m cont	act area)					
	Transverse Saggin	ng Moment	due to Normal Traf	fic						
	Pucher charts are u	ised to wor	k out the sagging mo	ment by assuming zero moment fix	kity along the supporte	d edges of de	ck slab.			
				CALCULATION SHEE	T Net: Hary:					
	Project Station V	Vay Bridg	e	113 11 3 A 1 1 1 A	By Bank. MA 03/03/202	25				
	Subject: Assessme	nt of car	riageway slab - S	5 imply supported case - 44T	By CATU					
	Calculation of P	late Mom	ents According to	Pucher	Instructions					
	Chan	Span =	1.83 m	a plate sinp with two supported edges	Redraw					
	Contour re	solution =	10	Area resolution = 10						
	11				111					
	6 (()		100		-111					
			(
			Primary \	Wheel load Secondary	Wheel load					
						er -				
	Define rectangular pa	tch load area		Accidental Redraw Calculate	Scales 0.0					
	Patch	Centre x-m	y-m m	eight Pressure Volume Moment m KN/m ² (8π.β) KNm/m	0.6					
	2	1.300	0.000 0.400 0	400 944 0.2490 31.32 400 343.5 0.0682 3.03	2.0					
	4				4.0					
	7				7.0					
	10			Total: 0.3152 34.35						
	Note: Pr	ositions of load	ds are defined from the ce	ntre of the plate with x-negative being up in mass that are off the edge of the plate are d	the above 16 iscounted 18					
	in Ti	the calculatione volume calculation	n. culated is the dimensionles	ss volume, β.	20					
	U									
C	OBS	CALCULA	TION SH	IEET						
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е	Manchester	Page No.	B 7	Calc No.						
). &	ECC - Assessment of Station Way Bridge	Calcs by	MA	Date	Mar-2					
n	Main Deck (Span 2 to 4) - Carriageway Slab - Live Loading	Checker	CAT II	Date	Mar-2					
	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 work out the worst wheel position for the applied load effects for hogging moment. The worst wheels position are shown be Image: Station Way Bridge Image: Station Way Bridge Subject Assessment of carriageway slab - Fixed end case - 44T Image: Station Way Bridge Subject Assessment of carriageway slab - Fixed end case - 44T Image: Station Way Bridge Calculation of Plate Moments According to Pucher Image: Carriageway slab - Fixed end case - 44T First 16 ms-support moment infuence surface for the edge of a plate-strip with two restanded edge Image: Redraw Image: Span =	slab. Sensitivity ana	Ilysis have be	en carried ou	t to					
	Primary Wheel load Secondary Wheel load Define rectangular patch load areas Accidental Pedraw Calculate -7.0 Image: Carbon of the second area Accidental Pedraw Calculate -7.0 Image: Carbon of the second area Accidental Pedraw Calculate -7.0 Image: Carbon of the second area Accidental Pedraw Calculate -7.0 Image: Carbon of the second area Accidental Pedraw Calculate -7.0 Image: Carbon of the second area Accidental Pedraw Calculate -7.0 Image: Carbon of the second area Accidental Pedraw Calculate -7.0 Image: Carbon of the second area Accidental Pedraw Calculate -7.0 Image: Carbon of the second area Accidental Pedraw Calculate -7.0 Image: Carbon of the second area Accidental Pedraw Calculate -7.0 Image: Carbon of the second area Accidental Pedraw Calculate -7.0 Image: Carbon of the second area Acciden									

 9
 70al

 10
 Total;
 -0.2909
 -35.70

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,



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ECC - Assessment of Station Way Bridge Main Deck (Span 2 to 4) - Carriageway Slab - Live Loading CALCULATION SHEET Project. Station Way Bridge Subject Assessment of carriageway slab - Fixed end case - 44T Charts Subject Assessment of carriageway slab - Fixed end case - 44T Charts Calculation of Plate Moments According to Pucher Image: Station Way Bridge Subject Assessment of carriageway slab - Fixed end case - 44T Charts Contour resolution = 10 Contour resolution = 10 Contour resolution = 10 Primary Wheel load Secondary Wheel load Define rectangular patch load areas Accidental Retinw Calculate Patch Terming with the image three of the state with x-negative being on the above into the state of the state with x-negative being on the above into the state with x-negative being on the above into the state with x-negative being on the above into the state with x-negative being on the above into the state with x-negative being on the above into the state with x-negative being on the above into the state with x-negative being on the above into the state with x-negative being on the above into the state with x-negative being on the above into the state with x-negative being on the above into the state with x-negative being on the above into the state with x-negative being on the above into the state with x-negative being on the above into the state with x-negative being on t	Calcs by Checker	MA CAT II	Date	Mar-25 Mar-25
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Section	Main Deck (Span 2 to 4)	- Carriage	way Slab - Live Load	ding	Checker	CATI	Date	Mar-
	Live Loading (SV8	D)						
S 458	Axle load =	130	kN	(Primary wheel load) Contact are	ea for SV80	Wheel load	d dispersed (2	2V:1H)
. 3.8	Wheel load =	65	kN Unfactored	350) mm	the top of	the slab.	
	Overload factor =	1.2		Surfacing		100 mm		
	DAF =	1.16		Stab 450) mm	150 mm		
	V. =	11						
	r ii							
	$\gamma_{f3} =$	1.1						
	Wheel Load =	109.2	kN Factored					
	Wheel Load =	539.3	kN/m2	Factored - (For 0.45m x 0.45m contact area)				
S 458	Axle load =	130	kN	(Secondary wheel load)				
. 3.8	Wheel load =	65	kN Unfactored					
	Overload factor =	1.1						
	DAF =	1.16						
	V _{fi} =	1.1						
	· · ·							
	$\gamma_{f3} =$	1.1						
	Wheel Load =	100.1	kN Factored	_				
	Wheel Load = Wheel Load = <u>Transverse Saggir</u> Pucher charts are u Project: Station N Subject: Assessm	100.1 494.3 ag moment sed to work Way Brin ent of c	kN Factored kN/m2 due to SV80 c out the sagging mo dge arriageway - Si	Factored - (For 0.45m x 0.45m contact area) ment by assuming zero moment fixity along the supported edges of de CALCULATION SHEET No. By: Date: 03032025 Check By: Date: By: Date:	ck slab.			
	Wheel Load = Wheel Load = <u>Transverse Saggin</u> Pucher charts are u Project: Station N Subject: Assessmu Calculation of <u>Chart</u> Contour r	100.1 494.3 ag moment sed to work way Brin ent of c Plate Mo to span =	kN Factored kN/m2 due to SV80 cout the sagging mo dge arriageway - Si ments Accordin ence surface for the cer 1.83]m 10	Factored - (For 0.45m x 0.45m contact area) ment by assuming zero moment fixity along the supported edges of de CALCULATION SHEET No. By: Date: 03032025 Check Date: 03032025 Check Date: 03032025 Check Date: By: Date: 03032025 Check Date: By: Date: Date: 03032025 Check Date: By: Date: Date: Date: Date: <th>ck slab.</th> <th></th> <th></th> <th></th>	ck slab.			
	Wheel Load = Wheel Load = Transverse Saggin Pucher charts are u Project: Station V Subject: Assessme Calculation of Ochart Contour n Second Define rectangular p Patch 1 2 3	100.1 494.3 ag moment sed to worl way Brin ent of c Plate Mo 1-mx - Influ span = esolution = control of c span = esolution = control of c the second second second second second data second se	kN Factored kN/m2 due to SV80 cout the sagging mo dge anriageway - Si ments Accordin ence surface for the cer 1.83 m 10 back of the ser y - m m 0.000 0.450 0	Factored - (For 0.45m x 0.45m contact area) ment by assuming zero moment fixity along the supported edges of de CALCULATION SHEET No. By: Date: Obset Date: <	ck slab.			

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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,



Hogging moment with the wheel position at d from support,



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Section	Main D	Deck (Span 2 to 4) - Carriagew	ay Slab - Shear Cap	acity			Checker	CAT II	Date	Mar-25			
	s	HEAR RESISTANCE OF THE	CARRIAGEWAY SL	<u>AB</u>			•			_			
		Bar Type	Bar Dia (")	Bar Dia (mm)	Area of Bar (mm ²)	Spacing (mm)	No. Bars	Area of Re					
		Transverse Bottom Bar	9/16	14.3	160.3	79.4	12.6	2019.8					
								2010.8					
		Transverse Top Bar	9/16	14.3	160.3	79.4	12.6	20)19.8				
	м	laximum shear Resistance ba	sed on concrete cr	ushing									
		Maximum shear Resistance based on concrete crushing											
		Equation 5.6a	a Maximum shear resist	tance based on concrete	crushing								
	V	max = $V_{max} = 0.36$	$\left(0.7 - \frac{f_{eu}}{250}\right) \left(\frac{f_{eu}}{\gamma_{me}}\right) b$, d									
	V	max = 0.36	x 0.7	- 15.00	x <u>15.00</u>	x 1000	x 158	= 36	3.9 kN				
		hear Desistance more than 2	d from a ournert	230	1.5			(Fei i	netre widtri)				
	5	near Resistance more than 30	u irom a support										
	,	V _{uc} = Equation 6	8.5 Shear resistanc	e of concrete slabs	more than 3d from a s	support							
		$V_{uv} = \frac{0.2}{\gamma_m}$	$\frac{7}{4} \xi_s \rho_s^{\frac{1}{3}} f_{cu}^{\frac{1}{3}} b_w d$										
	ξε	$s = \left(\frac{500}{158}\right)$	0.25 =	1.33									
	Р	s = 100	x 2019.8	35 =	1.28								
		1000	х	158	1/3	1/3							
	,	$V_{uc} = 0.27$	x 1.33	х	1.28 x	15.00 x	1000 x	158					
		1.25											
	V	uc = 121.8	KN (per metre wid	tn @ 3a from support)									
	s	hear Resistance within 3d of	a support										
		quation E &c Chear registance	e within 2d of a cun	nort									
		cquation 5.6c Snear resistance	e within 3d of a sup	port									
	1	$V_{u} = \max \left\{ \begin{array}{l} \frac{0.24}{\gamma_{max}} \xi_{s} (0.45 f_{cu})^{\frac{1}{3}} \end{array} \right.$	$b_w d$										
	0	where:				34							
	1.1	 a, is the distance of the a flexible bearing or 	e section measured fr the face of a support	om the edge of a rigid , where $d \leq a_v \leq 3d$	bearing, the centre-line o	pl							
		Γ is the factor to accou	int for short anchora	ge lengths, defined in E	quation 5.6d								
	E	Equation 5.6d Factor to accou	nt for the effect of s	hort anchorage lengt	hs								
	I	$T = \min \left\{ \sqrt{\frac{z}{3d} \frac{F_{ab}}{V_{ac}}} \right\}$											
		(1.0											
		F_{ab} is the total anchorag	e force that can be d	eveloped in the longitu	dinal tension reinforcing								
		bars at the front face	of the support accor	ding to Section 9, but r	not greater than $\frac{A_{A}f_{B}}{\gamma_{m_{A}}}$								
		 is the flexural lever a 5.2.2b 	irm at ULS at a positi	on 3d from the support	calculated from Equatio	n							
	E	quation 9.1a Anchorage resis	stance										
	1	$f_{ub} = f_{iub}pL_a$											
		fink is the average ancho	brage bond strength	over the effective anch	orage length, given by								
		Equation 9.1b; <i>n</i> is the effective perim	eter taken as										
		$p = \pi \phi$ for a single t $p = (1.2 - 0.2N) \Sigma$	par, or $(\pi \phi)$ for a bundled g	roup of N bars, valid up	p to $N = 4$.								
		is the nominal bar dia	ameter										
		L_n is the effective anchorador is the effective anchorador is the effective anchorador is a second seco	orage length at the p	osition where the resist	tance is being determine	ed.							
L	1												

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Job No. & Title	ECC - Ass	essment o	of Station Way	Bridge									Calcs by	MA	Date	Mar-25
Section	Main Deck	(Span 2 t	o 4) - Carriage	way Slab -	Shear Cap	pacity							Checker	CAT II	Date	Mar-25
	Equ <i>fub</i> k	$=\frac{kk_{con}}{2}$.1b Averag $v\beta\sqrt{f_{cu}}$ γ_{mb} 1.0	e ancho	rage boi	nd stren _{Ymb}	gth =	1.4								
CS 455, Tbl 9.1	в	=	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}	=	$F_{ub} = f_u$	$_{ab}pL_a$												
	Р	=	565.5	mm/m												
	La	=	158.0	mm (cor	nservative	taken as d)										
	F_{ub}	=	96.4	kN/m												
	Equa	tion 5.6d	Factor to acco	unt for the	effect of s	short anch	orage lengt	ths								
	$\Gamma =$	$\min \left\{ \ \lor \right.$	$ \frac{z}{3d} \frac{F_{nb}}{V_{uc}} $ 1.0													
	whe	re: F _{ab} is the	ne total anchora s at the front fac	ge force th	at can be d	leveloped ir rding to Sec	n the longitu	dinal tensior	n reinforcing							
		= is th 5.2.	ne flexural lever 2b	arm at ULS	S at a posit	ion 3d from	the support	t calculated I	from Equation							
pg ref. B 3	d =	=	158	mm												
	Z =	=	124.0)	1/2										
	Г	=	<u>124.0</u> 473.9	х	<u>96.4</u> 121.8											
	Г :	=	(0.26	x	0.79	1/2 9)	=	(0.207)	1/2 =	0.455						
	Vu =	=	$\frac{3d}{a_v}\Gamma V_{uc}$	=	3	x 158	158	x	0.455	x	121.8	=	166.3	kN/m at av	r = d	
	Vu =	$\frac{0.24}{2}\xi_{s}($	$(0.15f_{cu})^{\frac{1}{3}}b_{u}$	_v d =	0.24	x	1.33	x	(0.15	x	15.0)	1/3 x	1000	x 158	= 53.0	kN/m
		2me			1.25											
	Vu	=	166.3	kN (pe	r metre wid	Ith at d from	n support)									





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Section	Main Deck (Span 2 to 4) - Service Bay Slab	Checker	CAT II	Date	Mar-25
Record drgs: LC5/2 LC5/6	CALCULATIONS FOR ASSESSMENT OF SERVICE BAY SLAB (SPAN 2 – 4) SLAB ARRANGEMENT Kerb Beam Transverse direction for slab reinforcement and bending 10° 6' 3° 1.9 m	am			
	Idealised diagram				
	As shown on sheet C3, the top transverse reinforcement has half of the area of the bottom transverse reinforcement. Th	erefore, it appears that	the origina	design inten	t
	For Sagging: Free ends (simply supported) 1.9 m Material Properties and Partial Factors				
	Reinforced Concrete				
AIP, CI 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				
AIP, CI 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 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				
CS 455 Table	Partial Factors γ_{ms} = 1.15				
2.13a CS 455 Table 2.13a	Partial Factor for Concrete $\gamma_{mc} = 1.5$				
CS 455 Table 2.13a	Partial Factor for shear in concrete $\gamma_{mv} = 1.25$				
CS 455 Table 2.13a	Partial Factor for Bond Ymb = 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 $\gamma_{f3} = 1.1$				
CS 454, Tbl 3.4	Partial Factor for accidental vehicle loading and footway loading γ_{fL} = 1.5				

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Job No. & Title	ECC - Assessment of Station Way Bridge	Calcs by	М	1A	Date	Mar-25
Section	Main Deck (Span 2 to 4) - Service Bay Slab - Results Summary	Checker	CA	ТII	Date	Mar-25

RESULTS SUMMARY

AVL = Accidental vehicle loading (table B.1 of CS 454)

PLL = Pedestrian live loading (5 kN/m²)

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 E	Bay Slab - Bending Ca	apacity				Checker	CAT II	Date	Mar-2
	BENDING CAPACITY - SERVI	<u>CE BAY TRANSVER</u>	SE SLAB	nsverse direction	Parapet bea	ım 3				
G LC 5/6	Reinforcement									
	Bar Type	Bar Dia (")	Bar Dia (mm)	Area of Bar (mm ²)	Spacing		No. Bars (per m width)	Area of Reinforcement (mm		
	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			
	Transverse Top Bar	3/8	9.53	71.3	6 1/4"	158.8	6.3			
	Longitudinal Top Bar	3/8	9.53	71.3	12"	304.8	3.3		233.8	
	Slab Depth Effective Depth z M _u = 230 1.15	= 1 1/2" = 6" = 152.4 = 94.5 _ x 897.7	= 38.1 = 152.4 - 38.1 mm	mm - 4.76 94.5 =	= 17.0	109.5 kNm/m	mm			
	M _u = <u>0.225</u>	x 15.0	X	1000 x	110	2	= 27.0	kNm/m		
	M _u = 17.0	kN.m/m								

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Job No. & Title	ECC - Assessment of Station Way Brid	lge							Ca	lics by	MA	Date	Mar-25
Section	Main Deck (Span 2 to 4) - Service Bay	Slab - DL& SDL a	ind Pede	estrian Loading					CI	necker	CAT II	Date	Mar-25
	APPLIED LOADING (FOR 1.0M W	(IDTH OF SLAB)											
	DL & SDL												
	Self-weight	Depth (m) 0.152	x	Per metre width 1.0	x	24	x	γ _{fl} 1.0	x	Υ _{f3} 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
				4.0						4.0	10.0		01.0
	FIII	0.6	х	1.0	х	22	x	1.0	x	1.0	= 13.2	2 KN/m	SLS
							x	1.2	x	1.1	= 17.4	F KIN/M	ULS
	Total DL & SDL (Unfactored)										19.3	3 kN/m	SLS
	Total DL & SDL (factored)										26.7	/ kN/m	ULS
	Bending Moment and Shear Ford	ce due to DL & SI	DL (ULS)										
	Max M _{Saming} = $wL^2/8$	= 12.1	kN.m/m	(Assuming sim	oly sup	ported)							
				(3- 1		,							
	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 Ford	ce due to pedestr	ian load	ina (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 sim	oly sup	ported)							
	Shear at support	= 7.9	kN/m										
	Shear at d =	= 7.0	kN/m										
	Shear at 3d -	- 51	kN/m										
		- 0.1	KI WITI										

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Job No. & Title	ECC - Assessment of St	tation Way	Bridge						Calcs by	MA	Date	Mar-2
Section	Main Deck (Span 2 to 4)	- Service	Bay Slab	- AVL					Checker	CAT II	Date	Mar-2
	Accidental Vehicle	loading (44T & 26T)								
S 454 able B.1	Axle load =	113	kN (Pr	imary load)				Contact an 300	ea for wheel load 0 mm	Wheel loa the top of	ad dispersed (the slab.	2V:1H) t
	Wheel load =	56.5	kN Un	factored		Surfacing				100 mm		
P, CI 4.13	Traffic flow factor =	0.9				Fill				600 mm		
	Impact factor =	1.8				- Slab				150 mm		
	Lane factor =	1.0				Giab		100	10 mm			
	$\gamma_{fI} =$	1.5										
	$\gamma_{f3} =$	1.1										
	Wheel Load =	151.0	kN Fa	ctored	(Primary wheel load)							
	Wheel Load =	151.0	kN/m ²		Factored - (For 1000m	1 x 1000m co	ntact area)					
]							
454	Axle load =	74	kN (Se	econdary loa	ad)							
ие Б.1	Wheel load =	37	kN Un	factored								
, CI 4.13	Traffic flow factor =	0.9										
	Impact factor =	1.0	Im	bact factor o	only applied to primary							
	Lane factor =	1.0	WI	eerioau								
	$\gamma_{\rm fl} =$	1.5										
	$\gamma_{f3} =$	1.1										
	Wheel Load =	54.9	kN Fa	ctored	(Secondary wheel load	d)						
					_							
	Wheel Load =	54.9	kN/m ²		Factored - (For 1000m	1 x 1000m co	ntact area)	Axle spaci	ng = 1	.30 m		
	Project: Station V	Vay Brid	dge		CALCUL	ATION SH	By:	Rev: Date: 14/03/2025				
	Subject: Service b	ay slab	- Simpl	y supp -	44T		Check By:	Date				
	Calculation of F	Plate Mo	ments A	ccording	to Pucher		Instruc	tions				
	Chart 1	- mx - Influe	ence surface	for the centre	of a plate strip with two su	pported edges	Redra	BW				
	Contour re	span = esolution =	1.9	n	Area resolution =	10		-				
	1											
	6/1							1				
				16	6							
	11 -						131	1				
				Luque		Losonana		/				
				Primai	y Wheel load	Secondary	Wheel load					
	Define sector sector	tab land as					Scales:	0.0				
	Patch	Cen	tre	Width	Height Pressure V	olume Mon	nent	0.4 0.6				
	1 2	x-m 0.000 1.300	y - m 0.000 0.000	m 1.000 1.000	m KN/m ² 1 1.000 151 0 1.000 54.9 0	8π.β) KNr 9472 20 3797 2.1	n/m	0.8 1.0 2.0				
	3							3.0 4.0				
	6 7						= =	6.0 7.0				
	8 9 10							8.0	Max LILS	Ponding mom	ont (44T 8 26	τ)
	Note: D	locitions of k	anda ara dat	ined from the	Total: 1	.3269 23	54	14	=	23.5	kN.m/m	,
	d ir	rawing and in the calcula	x-positive be	ing down. An	y areas that are off the edg	e of the plate a	e discounted	18 20				
	T	ne volume o	calculated is	the dimension	liess volume, β.							



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Section	Main Deck (Span 2 to 4)	- Service	Bay Slab - AVL					Checker	CAT II	Date	Mar-25
		Looding	Destricted Troffic	7 57)							
CS 454	Accidental venicle	Eoading	KN (Primory lood			Co	ontact area	a for wheel load	Wheelloa	d disported (2)/-1U) to
Table B.1	Axie load =	59	KN (Primary load)		Cd	300 300	mm	the top of	the slab.	2V:1H) to
	vvneel load =	29.5	KIN Unfactored		Surfacing				100 mm		
AIP, CI 4.13	Impost factor	0.9			Fill				600 mm		
		1.0			Slab	/			150 mm		
	Lane factor =	1.0				•	1000	mm			
	$\gamma_{fI} =$	1.5									
	$\gamma_{f3} =$	1.1									
	Wheel Load =	78.9	kN Factored	(Primary wheel load)							
	Wheel Load =	78.9	kN/m ²	Factored - (For 1000n	n x 1000m conta	ct area)					
				_							
CS 454 Table B 1	Axle load =	15	kN (Secondary lo	ad)							
	Wheel load =	7.5	kN Unfactored								
AIP, CI 4.13	Traffic flow factor =	0.9									
	Impact factor =	1.0	Impact factor wheel load	only applied to primary							
	Lane factor =	1.0									
	$\gamma_{\rm fl} =$	1.5									
	$\gamma_{f3} =$	1.1									
	Wheel Load =	11.1	kN Factored	(Secondary wheel loa	d)						
	W/bool Lood	44.4	kh1/m²	Fastarad (Far 1000r	v 1000m conto	at area)	ula anagin	~			
	Project Station V	Vov Pri	los	CALCUL	ATION SHEE	T No: Rev: By: Date:					
	Subject: Service b	ay slab	- Simply supp -	7.5T		Check Date	03/2025				
	Calculation of F	Plate Mo	ments According	to Pucher		Instructions					
	Chart 1	- mx - Influe	ence surface for the cent	re of a plate strip with two su	pported edges	Redraw	ī				
	Contour re	Span =	1.9 m 10	Area resolution =	10						
							-				
							-				
	111										
	1 (()		661								
			111								
	11										
			Prin	ary Wheel load		Secondary Wheel lo	bad				
						Seales: 0.0					
	Define rectangular pa	tch load ar	eas	Accidental	edraw Calculate	e 0.2 0.4					
	Patch	x-m 0.000	y-m m 0.000 1.000	m KN/m ²	(8π.β) KNm/m 0.9472 10.73	0.8					
	2	2.000	0.000 1.000	1.000 11.1	0.1705 0.27	2.0	S.				
	5					5.0		Max ULS ben	ding mome	nt (7.5T AVL))
	7 8 9					- 7.0		=	11.0	KIN.III/III	
	10			Total	11177 11.01	3					
	Note: P d	ositions of la rawing and	oads are defined from th c-positive being down. A	e centre of the plate with x-n ny areas that are off the edg	egative being up in e of the plate are d	the above 16 liscounted 18	6				
	ir T	the calcula he volume of	tion. alculated is the dimension	nless volume, β.		20	10				
							-				

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Job No. & Title	ECC - Assessment of St	ation Way	Bridge				Calcs by	MA	Date	Mar-25
Section	Main Deck (Span 2 to 4)	- Service	Bay Slab - AVL				Checker	CAT II	Date	Mar-25
	Accidental Vehicle	Loading	(Restricted Traffic	- 3T)						
CS 454	Axle load =	21	kN (Primary lo	ad)		Contact ar	rea for wheel load	Wheel load	dispersed	
Table B.1	Wheel load =	10.5	kN Unfactored			30	0 mm	(2V:1H) to t	he top of the	е
AIP. CI 4.13	Traffic flow factor =	0.9		-	Surfacing			100 mm		
	Impact factor =	1.8			Fill			600 mm		
	Lane factor =	1.0			Slab			150 mm		
	V _{fl} =	1.5				۹ 100	00 mm 🕨			
		0.04	IN Francis							
	wheel Load =	28.1	KIN Factored	(Primary wheel load)						
	Wheel Load =	28.1	kN/m²	Factored - (For 1000m	n x 1000m cor	ntact area)				
CS 454 Table B.1	Axie load =	9	KN (Secondar	y load)						
	Traffic flow factor	4.5	KIN UNTACTOREC	1						
mr, 014.13	Impact factor -	1.9	Impact fac	tor only applied to primary						
	Lane factor =	1.0	wheel load	ephose to prinding						
	Va =	15								
		1.0								
	$\gamma_{f3} =$	1.1		(O	-0					
	wheel Load =	0.7	KIN FACIOIRU	(Secondary wheel load	u)					
	Wheel Load =	6.7	kN/m ²	Factored - (For 1000m	n x 1000m cor	ntact area) Axle spaci	ing = 2.0) m		
	Project: Station V Subject: Service b	Vay Bri	dge - Simply supr	CALCUL	ATION SH	By: Date: 14/03/2025 Check Date:				
	Calculation of F	Plate Mo	ments Accord	ing to Pucher		By:				
	Chart 1	- mx - Influ	ence surface for the c	entre of a plate strip with two su	pported edges	Redraw				
	Contour	Span =	1.9 m	Area resolution =	10					
	111									
			10	(a)						
						Annual and a second				
			Pi	rimary Wheel load		Secondary Wheel load				
	10 Car 200		inte			Scales:0.0				
	Define rectangular pa	tch load ar Cen	tre Width	Accidental Re Height Pressure	olume Mon	0.2 0.4 0.6				
	1	x-m 0.000 2.000	y-m m 0.000 1.000 0.000 1.000	m KN/m ² 1.000 28.1 0 1.000 6.7 0	(8π.β) KNn 0.9472 3.8 0.1705 0.1	n/m 0.8 12 1.0 16 2.0				
	3 4					3.0				
	5 6 7					5.0 6.0 7.0	Max ULS ben	dina moment	(3T AVL)	
	8					8.0	=	4.0 H	kN.m/m	
	10									
	Note: P	ositions of l	aske are defined from	Total:	1.1177 3.9	19 14 15				
	Note: P	ositions of I rawing and the calcula	oads are defined from x-positive being down tion.	Total: the centre of the plate with x-n . Any areas that are off the edg	egative being up the of the plate an	n the above 16 re discounted 18 20				
	Note: P d i T	ositions of I rawing and the calcula he volume o	oads are defined from x-positive being down tion. calculated is the dime	Total: the centre of the plate with x-n . Any areas that are off the edg nsionless volume, β.	egative being up the plate ar	19 14 19 in the above 16 re discounted 18 20				
	Note: P d T	ositions of I rawing and the calcula he volume o	oads are defined from x-positive being down tion. calculated is the dimen	Total: the centre of the plate with x-n _ Any areas that are off the edg nsionless volume, β.	1.1177 3.9 egative being up se of the plate an	19 14 in the above 16 re diacounted 18 20				
	Note P d ir T	ositions of I rawing and the calcula he volume o	oads are defined from x-positive being down tion. calculated is the dimen	Total: the centre of the plate with x-n . Any areas that are off the edg nsionless volume, β.	I.1177 3.9 egative being up ge of the plate ar	19 14 a in the above 16 re discounted 18 20				

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Job No. & Title	ECC - Ass	sessment of S	Station Way E	Bridge							Calc	s by	MA	Date	Mar-25			
Section	Main Deck	(Span 2 to 4	4) - Service E	Bay Slab - Shear (Capacity						Che	cker	CAT	ll Date	Mar-25			
	SHE4	AR RESISTA	NCE OF THE	SERVICE BAY	SLAB													
		Bar Ty	ре	Bar Dia (")	Bar Dia	(mm)	Area of B	ar (mm²)	Spacing	(mm)	No.	Bars width)	Area	of Reinforce	ement			
	т	ransverse Bo	ottom Bar	3/8	9.5	3	71	.3	79.4	Ļ	12	2.6		897.7				
		Transverse	Top Bar	3/8	9.5	3	71	.3	158.	8	6	.3		448.9				
										-		-						
	Maxir Equatio $V_{ m max} =$	mum shear F n 5.6a Maximu $0.30\left(0.7 - \frac{f}{2}\right)$	Resistance b m shear resista $\left(\frac{1}{2}\right)\left(\frac{f_{cu}}{\gamma_{uv}}\right)b_{u'}$	ased on concrete Ince based on concr d	e crushing ete crushing													
			- X A marx	C		~)											
	V _{max}	=	0.36	x 0.7		15.0	x	15.0 1.5	x	1000	x	110	= (252.4 kN per metre wid	th)			
	Shea	r Resistance	more than 3	3d from a suppor	t													
	Equation $V_{ m uc}$:	ation 6.5 Sh = $\frac{0.27}{\gamma_{mv}} \xi_s \rho_s^{\frac{1}{2}}$	ear resistand $f_{cu}^{rac{1}{2}}b_w d$	ce of concrete si	abs more than	3d from a	a support											
	ξs	=	$\left(\frac{500}{110}\right)$	0.25 =	1.46													
	ρ	=	100 1000	x 89	97.7 110	=	0.82	(Based or	n bottom reinf	orcement)							
	V _{uc}	=	<u>0.27</u> 1.25	x 1	.46	x	(0.82)	1/3	х ((15.0)	1/3	x	100	0 x 11	10			
	V _{uc}	=	79.8	kN (per m wid	th - at 3d)													
	Shea	r Resistance	within 3d of	f a support														
	Equa	tion 5.6c She	ear resistance	e within 3d of a s	upport													
	$V_{u} =$	$\max\left\{ \frac{0.24}{2m\nu} \right\}$	$\frac{3d}{a_{e}}\Gamma V_{uc}$ $\xi_{s}(0.15f_{co})^{\frac{1}{3}}$	$b_{w}d$														
	Equa	ation 5.6d Fa	ctor to accou	int for the effect o	of short anchore	ge length	s											
	$\Gamma =$	$\min \left\{ \begin{array}{c} \sqrt{\frac{2}{10}} \\ 1 \end{array} \right\}$	(Fab Vac 0															
	Equa	ation 9.1a An	chorage resi	istance														
	Lub	- Juhl/La																
	Eq	uation 9.1b A	Average anch	orage bond stre	ngth													
	Int	Tent	Teq															
	k	=	1.0		γ _{mb}	=	1.4											
CS 455, Tbl 9.1	В	=	0.39	(Plain bars in tension	i) ģ	=	9.53	mm										
	f _{cu}	=	15.0	N/mm ²	K _{cov}	=	1.0	(taken as	1)									
	f _{ub}	=	1.08				Luh	is the ave Equation	erage anchora 9.1b;	age bond :	strength ove	er the effect	ive anch	orage length,	given by			
	F_{ub}	=	$F_{ub} = f_{ub}$	pL_a			P	is the effinition $p = \pi \phi$ f	ective perime for a single ba	ter, taken i r, or	as							
	Р	=	377.0	mm/m	(Bottom reir	forcement	t)	p = (1.2)	$(-0.2N)\sum (\pi$	 (a) for a bit a bit	undled grou	p of N bars	s, valid u	p to $N = 4$.				
	La	=	110	mm (conservati	vely taken as d)		L_a	is the eff	ective anchor	age length	at the posi	tion where	the resis	tance is being	determined.			
	F_{ub}	=	44.6	kN/m			Fut	is the tol	tal anchorage	force that	can be dev	eloped in th	e longitu	idinal tension	reinforcing			
Pg ref. C 3	d	=	109.5	mm				bais at t	ne nonciace (or the supp	Join accordin	ig to secili	ALS, DUI	nor greater as	mar Trade			
Pg ref. C 3	z	=	94.5				2	5.2.2b	exural lever an	m at ULS i	s at a position 3d from the support calculated from Equation							
	г	=	94.5	x <u>44.6</u>	1/2													
	г	=	0.29	x 0) .56	=	(0.160)	1/2 =	0.401									
	Vu	=	$\frac{3d}{a_v}\Gamma V_{uc}$	=3	x 110	110	x	0.401	x	79.8	=	95.9	kN/m	(at av = d)				
	V _u :	$=\frac{0.24}{\gamma_{mv}}\xi_s(0.1)$	$(15f_{cu})^{\frac{1}{3}}b_w$	<i>d</i> = 0.24 1.25	x	1.46	x	(0.15	x	15)	^{1/3} X	1000	x	109.5 =	= 40 kN/m			
	Vu	=	95.9	kN (per metre	width at d)													

Office	OR2					CALCULA	TION SF	IEET	
	Manchester					Page No.	C 10	Calc No.	
) No. & Title	ECC - Assessment of Station Way Bridge					Calcs by	MA	Date	Mar-2
ection	Main Deck (Span 2 to 4) – Service Bay Slab – Shear (A	VL)				Checker	CAT II	Date	Mar-2
						•			
	ULS SHEAR DUE TO ACCIDENTAL VEHICLE LC	DADING							
	All wheel loads applied to 1.0m x 1.0m area (allow	ing for dispersal through	surfacing and	fill).					
	Conservatively assume that shear due to primary v	wheel load is resisted by	1.0m width of	slab.					
					w =	Primary wheel load	ULS)		
		v	v						
	Max Shear at d from support = d	= 0.11	1.0		0.79	1			
		< ►		1.9 m		`			
	Shear at d = <u>(1.9 - 0.11</u> 1.9	- 0.5)	w	=	0.68 w				
			W						
	Max Shear at 3d from support =	3d = 0.33	1.0		0.57	f			
		•		1.9 m					
	Shear at 3d = (1.9 - 0.33	- 0.5)	w	=	0.56 w				
	1.9								
	Accidental Vehicle Load (kN)	At d	m width of sla At 3d	ab)					
	44T, 26T & 18T 151	102.6	85.0						
	7.5T 79	53.7	44.5						
	27 28	10.0	45.0						

Appendix C. Record Drawings











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