



BURY INTERCHANGE PHASE 1

STRUCTURAL DESIGN CRITERIA

Project No 2401-09

December 2025

Rev: P03

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1. INTRODUCTION

The following document has been developed to inform wider stakeholder of the design principles considered in the development of the RIBA stage 4 superstructure and substructure package for Bury Interchange development (Phase 1).

The proposals should be reviewed by the client, consultant team, and potential construction partner, with any comments provided to be incorporated within the next stage of the design development. Changes to the spatial use and occupancy, impacting defined criteria, may affect the basis of design and subsequent design response.

This document should be read in conjunction with the latest issue of structural drawings, specifications and reports along with the latest site investigation report. This report is provided as a general introduction only, and if any of the information contained in the report conflicts with the latest structural information, the structural drawings, calculations, and specifications take precedence.

This Structural Design Criteria should be read in conjunction with Employers Requirements, any supply chain information along with the following Renaissance documents:

CDM Designer's Hazard Register:	16012-REN-ZZZ-XXX-T-SE-00001
Site Constraints:	16012-REN-BUO-XXX-D-SE-00020
	16012-REN-BRY-XXX-D-SE-00021
	16012-REN-BRY-XXX-D-SE-00022

1.2 Design Codes

The structural design for the building and associated structural elements will be undertaken to the Eurocodes and, if necessary, additional design guides. The main European Standard are listed below. Reference will also be made to various parts of each code and associated UK National Annexes.

- BS EN 1990: (Eurocode 0) Basis of structural design
- BS EN 1991: (Eurocode 1) Actions on structures
- BS EN 1992: (Eurocode 2) Design of concrete structures
- BS EN 1993: (Eurocode 3) Design of steel structures
- BS EN 1994: (Eurocode 4) Design of composite steel & concrete structures
- BS EN 1995: (Eurocode 5) Design of timber structures
- BS EN 1996: (Eurocode 6) Design of masonry structures
- BS EN 1997: (Eurocode 7) Geotechnical design

1.3 Design Working Life

In accordance with BS EN 1990:2002, the structures, with the exception of the footbridge, are generally a building with a design working life category of 4 with a corresponding design life of 50 years.

For the Union Square Footbridge, the structure design life is in accordance with CD 350 The Design of Highway Structure Table 7.1 & the UK National Annex to BD EN 1990;2002 Table NA2.1.

- The structural elements will be design working life category 5 (120 years).
- Replaceable elements such as bearings, expansion Joints, waterproofing system, safety barriers, parapet will be working category 2 (25 years).



- **Site Boundary**
- **Phase 1:**
Metrolink access and platform improvement works including northern temporary stair
- **Phase 1: Supplementary Application**
Temporary access/egress from Angouleme Way to North temporary stair
- **Phase 2:**
Main interchange and residential development

2. PLATFORM TEMPORARY ACCESS STAIRCASE & PEDESTRIAN CANOPY

2.1 Design loads

The structural design will be based upon the following permanent (G_k) and variable (Q_k) actions.

2.1.1 Permanent Actions (G_k)

Permanent actions comprise the self-weight of the structural frame and associated fixed items such as cladding, ceiling, services, finishes etc. For residential developments with fixed wall arrangement, it also includes partitions.

The self-weight of the structural frame will be calculated from the unit weights given in BS EN 1991-1-1 and from known weights of the material used where information on build-ups etc is provided.

The permanent actions associated finishes applied to the frame are presented in Tables 2.1 and 2.2. The Architect and Main Contractor are to review the assumed build-ups and ensure they work within the design allowances. Where this is not feasible the Engineer should be notified to agree revised loading and implication on the completed design.

Ref	Area	Description	Uniform Load (kN/m ²)	Total
Canopy				
SDL01a	Canopy (General)	Single skin roof sheet	0.12	0.75
		Roof deck (46mm)	0.13	
		Ceiling	0.15	
		Services	0.35	
SDL01b	Canopy (Maintenance Walkway)	Open grate floor system	1.00	1.75
		Single skin roof sheet	0.12	
		Roof deck (46mm)	1.13	
		Ceiling	0.15	
		Services	0.35	
Temporary Access Stairs				
SDL02	Temporary Access Stairs	Finishes (allowance) 8-10 mm Stiffened plate	0.20 0.65	0.85

Table 2.1 – Permanent Actions – Floor Dead Loads

REF	Area	Description	Uniform Load (kN/m ²)	Line Load (kN/m)
LL01	Metal Balustrade	Horizontal Rails Infill Rails SFS System	-	0.50
LL02	Glazed wind Barrier ¹	15mm laminate 15mm toughened HST Frame, fins & brackets	0.40 0.40 0.15 0.95	2.85
LL03	Fascia cladding and SFS ²	Single sheet panel (70mm) SFS	0.12 0.15 0.27	0.32

¹ Loads are based on 3 m cladding height to canopy Glazed wind barrier.

² Loads are based on 1.2m fascia height

Table 2.2 – Permanent Actions – Cladding Dead Loads

2.1.2 Variable Actions (Q_k)

The imposed loads associated with the occupancy of the building are summarised in Table 3.3. These are based on the UK National Annex to BS EN 1991-1-1:2002.

Ref	Area	Description	Uniform Load (kN/m ²)	Point Load (kN)
Canopy				
IL01a	Canopy (General)	H	0.75	0.9
IL01b	Canopy (Maintenance Walkway)	-	0.75	2.0
Temporary Access Stairs				
IL02	Temporary Access Stairs	C37	5.0	3.6

Table 2.3 – Variable Actions – Floor Live Loads

2.1.3 Wind Loading (Q_k , wind)

The wind loading for the overall structure and stability design will be calculated in accordance with BS EN 1991-1-4 and the associated UK Nation Annex base on the site parameters below.

The site is in the center of Bury. The following basic parameters will be adopted when determining the overall building wind loads. The building wind loads will also be refined to allow for surrounding buildings, distance to shore, direction method etc to provide an efficient design.

Parameters	A
Site basic velocity (fig NA.1) $v_{b, map}$ (m/s)	22.5
Site altitude, A (m)	87.5
Building height, h (m)	6.40
Altitude factor, $(1+0.001A(10/h)^{0.2})$	1.085
Fundamental basic velocity $v_{b, o}$ (m/s)	24.4
Mean velocity pressure $q_b = 0.63v_b^2$ (kN/m ²)	0.37
Distance to shoreline (m)	50
Exposure factor (fig NA.7), $c_e(z)$	1.7
Distance in town (m)	<5km
Correction factor (fig NA.8), $c_{e, T}(z)$	1.00
Peak Velocity Pressure q_p (kN/m ²)	0.67

Table 2.4 – Wind Load

Table 2.4 calculates the site wind pressure and is to be adjusted to allow for pressure coefficient in accordance with BS EN 1991-1-4 and the UK National Annex. For all cladding elements, support systems and roofing systems the specialist suppliers are to undertake detailed wind loading calculations to determine the local design wind pressures for the elements and components being considered.

2.1.4 Snow Loading (Q_k , snow)

The snow loading for the structural frame design will be calculated in accordance with BS EN 1991-1-3 and the associated UK National

Parameters	A
Snow Zone Z	3
Site altitude, A (m)	87.5
Ground Snow Load s_k (kN/m ²) $s_k = [0.15 + (0.1Z + 0.05)] + (A - 100)/525$	0.48

Annex base on the site parameters below.

Table 2.5 – Snow Load

Where required allowances are to be made for snow drifting.

For all cladding elements, support systems and roofing systems the specialist roofing supplier is to undertake detailed snow load calculations.

2.1.5 Horizontal Loads on Parapets & Partition Walls (Q_k)

The horizontal imposed loading for parapets, balustrades and walls is to be calculated in accordance with BS EN 1991 as per the tables below.

Area		Point Load (kN)	Uniform Load (kN/m ²)	Line Load ¹ (kN/m)
Footways or pavements greater than 3 m	C5(X)	1.5	1.5	1.5

¹ The line load is to be taken at the height of the parapet but not greater than 1.2m above finished floor level.

Table 2.6 – Variable Actions – Horizontal Balustrade & Parapet Loads

Balustrades and handrails are to be designed to limit horizontal displacement to acceptable limits to prevent alarm for excessive movement. The final design is to be to the balustrade suppliers design and detail and in accordance with the relevant Eurocodes. As a guide the following deflection limits to cantilever balustrades should not be exceeded:

Steel, Concrete & Timber	– Height/180	≤ 25mm
Glass	– Height/65	≤ 25mm

2.2 Serviceability Performance Criteria

The structural design will be designed on the following serviceability criteria with primarily concern the comfort of building occupants and the appearance of the works.

2.2.1 Movements

The maximum overall platform Canopy plan is circa 81m x 6m. An assessment of the structure has been carried out to ensure that a movement joint to allow for differential movement along the length of the canopy is not required.

2.2.2 Vertical Deflection Limits

The vertical deflection of slabs, relative to the supports, will be limited to the criteria in Table 4.1. Differential deflection limits between supports is also indicated.

Deflection Criteria	Internal	Slab Edge-Cladding Line
Total Deflection	$L/250 \leq 30\text{mm}$	$L/250 \leq 25\text{mm}$
Cladding	-	$L/500 \leq 12\text{mm}$
Normal Finishes(imposed)	$L/360 \leq 25\text{mm}$	
Brittle Finishes(imposed)	$L/500 \leq 20\text{mm}$	
Total deflections(imposed)	$L/500 \leq 25\text{mm}$	$L/500 \leq 15\text{mm}$

Table 2.7 – Floor Vertical Deflection Limits

For cantilever elements the equivalent span $L = 2 \times L_{\text{cantilever}}$. As an example, the long-term total vertical deflection limit for a cantilever is $L_{\text{cantilever}}/125$.

2.2.3 Horizontal Deflection Limits

For the serviceability limit state, the horizontal deflections from wind loading (50-year return period) and 'equivalent horizontal loads' from the frame will be limited to the following:

- Overall drift $\delta_{\text{total}} = H/500$
 - Interstorey drift $\delta_{\text{storey}} = H/300$
- H = storey height or height above ground

2.3 Fire Resistance

All aspects of fire protection for this project are the responsibility of the Fire Consultant and Architect including the fire rating and necessary measure for the provision of fire protection and associated detailing. Renaissance will adopt the fire rating as specified by others in the provision of cover to reinforcement when designing reinforced concrete frames. All other aspects of fire protection measures are the

responsibility of the Architect and Fire Consultant and not Renaissance.

2.4 Durability

BS ISO 15686 classifies components as permanent, replaceable, and maintainable construction. Table 6.1 gives the anticipates design life for the structural components forming the frame together with the anticipated maintenance times. First maintenance is defined as the first major maintenance required and assumes that regular inspections and minor maintenance is carried out at regular intervals.

The existing RC foundations are to be unchanged as part of the Canopy refurbishment and therefore have been omitted from this section. Renaissance will undertake a comparison study to determine that the foundations loadings will not be made worse.

Location	Type	Design Life	Anticipated time to first maintenance	Maintenance/ Inspections
RC Superstructure	Accessible & Inaccessible	50 years	N/A	N/A
Steel Superstructure	Inaccessible	50 Years	N/A	N/A
Steel Superstructure	Accessible	50 Years	15 years	Periodic inspections, minor maintenance

Table 2.8 – Design Life and Time to First Maintenance

The long-term durability of the concrete frame elements to achieve the design working life will be achieved by providing adequate cover to the reinforcement and appropriate concrete mix specification as recommended in the relevant design codes.

For steel frame elements, corrosion protection and/or galvanising will be required to provide long term durability. These systems typically have a life to first major maintenance of 10-15 years depending on the exposure category, location, and environment. The choice of protection system will be influenced by the location and aesthetic requirements.

2.5 Robustness & Resistance to Disproportionate Collapse

The structural design for the buildings is to be robust and reduce the sensitivity of the building to disproportionate collapse in the event of an accident. The assessment will be in accordance with requirements of Building Regulations Part A3 and BS EN 1991-1-7.

As the structure is a public building not exceeding two-storeys, the canopy is consequence class 2A. For Class 2 buildings a systematic risk assessment of the building is not required, in accordance with BS EN 1991-1-7.

For Class 2A structures, horizontal ties need to be provided, and therefore will be taken as:

Internal Ties $T_i = 0.8 (G_k + \psi Q_k) s L \geq 75\text{kN}$

Perimeter Ties $T_p = 0.4 (G_k + \psi Q_k) s L \geq 75\text{kN}$

Horizontal Ties $T_p = A (G_k + \psi Q_k) s L \geq 75\text{kN}$

Where:

s = spacing of the ties

L = span of the ties

A = Areas of floor supported by column

In addition to the above, the minimum tying requirements provided in section 9.10 of BS EN 1992-1-1 is met.

3. UNION SQUARE FOOTBRIDGE

3.1 Design loads

The structural design will be based upon the following permanent (G_k) and variable (Q_k) actions.

3.1.1 Permanent Actions (G_k)

Permanent actions comprise the self-weight of the structural frame and associated fixed items such as cladding, ceiling, services, finishes etc. For residential developments with fixed wall arrangement, it also includes partitions.

The self-weight of the structural frame will be calculated from the unit weights given in BS EN 1991-1-1 and from known weights of the material used where information on build-ups etc is provided.

The permanent actions associated finishes applied to the frame are presented in Tables 2.1 and 2.2. The Architect and Main Contractor are to review the assumed build-ups and ensure they work within the design allowances. Where this is not feasible the Engineer should be notified to agree revised loading and implication on the completed design.

Ref	Area	Description	Uniform Load (kN/m ²)	Total
SDL01	Footbridge path (General)	Stiffened deck plate (8/10 mm) U-frame deck (Floor) Waterproofing Drainage Layer Services	0.65 0.65 0.05 0.10 0.25	1.70
SDL02	Abutment Base	Stiffened deck plate (8/10 mm) Services	0.65 0.25	0.90
SDL03	Articulate Deck	Stiffened deck plate (8/10 mm) Waterproofing Drainage Layer Service	0.65 0.05 0.10 0.35	1.15
SDL04	Bridge Access Stairs	Finishes (allowance) 8-10 mm Stiffened plate	0.20 0.65	0.85

Table 3.1 – Permanent Actions – Floor Dead Loads

Ref.	Area	Description	Uniform Load (kN/m ²)	Line Load (kN/m)
IL01	Footbridge Path ¹	S.W of stop chord (allowance) Perforated Steel Plate 8mm Web Panel U-frame (Cladding)	1.00 0.40 <u>1.30</u> 2.70	-
IL02	Stairs Balustrade	Horizontal Rails Infill Rails SFS System	-	0.50
IL03	Abutment Transition ²	Steel Flat Bar (allowance) Frame, fins & brackets	1.00 <u>0.15</u> 1.15	1.50

¹ Footbridge loads are given as elevational area loads in this report, and have been resolved as line loads for various heights of cladding. The arrangement of the cladding *line* loads will be assessed separately to reflect the ratio of Perforated steel to solid façade as the design progresses

² Abutment transition cladding is based on 1.3m height (scaled from architectural elevations).

Table 3.2 – Permanent Actions – Cladding Dead Loads

3.1.2 Variable Actions (Q_k)

The imposed loads associated with the occupancy of the building are summarised in Table 3.3. These are based on the UK National Annex to BS EN 1991-1-1:2002.

Ref	Area	Description	Uniform Load (kN/m ²)	Point Load (kN)
IL01	Pedestrian Path	LM4	5.0	10
IL02	Abutment Base	C38	7.5	4.5
IL03	Articulate Deck	C38	7.5	4.5
IL04	Bridge Access Stairs	C35	4.0	4.0

Table 3.3 – Variable Actions – Floor Live Loads

3.1.3 Wind Loading (Q_k , wind)

The wind loading for the overall building and stability design will be calculated in accordance with BS EN 1991-1-4 and the associated UK Nation Annex base on the site parameters below.

The site is in the center of Bury. The following basic parameters will be adopted when determining the overall building wind loads. The building wind loads will also be refined to allow for surrounding buildings, distance to shore, direction method etc to provide an efficient design.

Parameters	A
Site basic velocity (fig NA.1) $v_{b, map}$ (m/s)	22.5
Site altitude, A (m)	92.8
Building height, h (m)	6.40
Altitude factor, $(1+0.001A(10/h)^{0.2})$	1.085
Fundamental basic velocity $v_{b, o}$ (m/s)	24.4
Mean velocity pressure $q_b = 0.63v_b^2$ (kN/m ²)	0.37
Distance to shoreline (m)	50
Exposure factor (fig NA.7), $c_e(z)$	1.7
Distance in town (m)	<5km
Correction factor (fig NA.8), $c_{e, T}(z)$	1.00
Peak Velocity Pressure q_p (kN/m²)	0.68

Table 3.4 - Wind Load (worst case direction-N)

Table 2.4 calculates the site wind pressure and is to be adjusted to allow for pressure coefficient in accordance with BS EN 1991-1-4 and the UK National Annex. For all cladding elements, support systems and roofing systems the specialist suppliers are to undertake detailed wind loading calculations to determine the local wind pressures for the elements and components being considered.

3.1.4 Snow Loading (Q_k , snow)

The snow loading for the structural frame design will be calculated in accordance with BS EN 1991-1-3 and the associated UK National Annex base on the site parameters below.

Parameters	A
Snow Zone Z	3
Site altitude, A (m)	92.8
Ground Snow Load s_k (kN/m ²) $s_k = [0.15 + (0.1Z + 0.05)] + (A - 100)/525$	0.47

Table 3.5 – Snow Load

Where required allowances are to be made for snow drifting.

For all cladding elements, support systems and roofing systems the specialist roofing supplier is to undertake detailed snow load calculations.

3.1.5 Horizontal Loads on Parapets & Partition Walls (Q_k)

The horizontal imposed loading for parapets, balustrades and walls is to be calculated in accordance with BS EN 1991 as per the tables below.

Area		Point Load (kN)	Uniform Load (kN/m ²)	Line Load ¹ (kN/m)
Top of bridge parapet	-	-	-	3.0 ²
Bridge Access Stairs Balustrade	C2, C3, C4, C5 (Vi)	1.5	1.0	0.74

¹ The line load is to be taken at the height of the parapet but not greater than 1.2m above finished floor level.

³ Line loads refer to EN 1991-2, N.A 2.32

Table 3.6 – Variable Actions – Horizontal Balustrade & Parapet Loads

Balustrades and handrails are to be designed to limit horizontal displacement to acceptable limits to prevent alarm for excessive movement. The final design is to be to the balustrade suppliers design and detail and in accordance with the relevant Eurocodes. As a guide the following deflection limits to cantilever balustrades should not be exceeded:

Steel, Concrete & Timber	– Height/180	≤ 25mm
Glass	– Height/65	≤ 25mm

3.1.6 Temperature Loads

The thermal loading for structure is to be calculated in accordance with BS EN 1991-1-5:2003 as per the tables below.

Area	Min. Shade Air Temp. ¹ (°C)	Max. Shade Air Temp. ¹ (°C)	Min. Construction Temp. (°C)	Max. Construction Temp. (°C)
Steel Footbridge ²	-15	35	0	20
External Concrete Element ³	-16	33	10	10

¹ Adjusted for 120 years

2. Coefficient of thermal expansion taken as 0.00012 in line with BS EN 1993-1-1, clause 3.2.6 Bridge thermal load to be provided by bridge designer (OPS).
3. Coefficient of Thermal Expansion taken as 0.000001°C in line with BS EN 1992-1-1, Clause 3.1.3.5.

3.2 Serviceability Performance Criteria

The structural design will be designed on the following serviceability criteria with primarily concern the comfort of building occupants and the appearance of the works.

3.2.1 Movements

At this stage it has been assumed that the bridge deck is discontinuous over the pier supports and at abutment location. An articulated link to be is also to be provided between the lift and stair structure, to allow for differential movement between the bridge and lift/stair structure.

3.2.2 Vertical Deflection Limits

The vertical deflection of slabs, relative to the supports, will be limited to the criteria in Table 4.1. Differential deflection limits between supports is also indicated.

Deflection Criteria	Internal	Slab Edge-Cladding Line
Total Deflection	$L/250 \leq 30\text{mm}$	$L/250 \leq 25\text{mm}$
Cladding	-	$L/500 \leq 12\text{mm}$
Normal Finishes(imposed)	$L/360 \leq 25\text{mm}$	
Brittle Finishes(imposed)	$L/500 \leq 20\text{mm}$	
Total deflections(imposed)	$L/500 \leq 25\text{mm}$	$L/500 \leq 15\text{mm}$

Table 3.7 – Floor Vertical Deflection Limits

For cantilever elements the equivalent span $L = 2 \times L_{\text{cantilever}}$. As an example, the long-term total vertical deflection limit for a cantilever is $L_{\text{cantilever}}/125$.

Slabs will be fixed to the core to offset dead-load deflections where total deflection exceeds the above limit.

Deflection limits for long-span elements (greater than 9m) will be assessed on a case by case basis and may require relaxation from the above to provide an efficient design.

3.2.3 Horizontal Deflection Limits

For the serviceability limit state, the horizontal deflections from wind loading (50-year return period) and 'equivalent horizontal loads' from the frame will be limited to the following:

- Overall drift $\delta_{\text{total}} = H/500$
H = storey height or height above ground

3.2.4 Floor Vibrations

To reduce the susceptibility of occupied floors to unacceptable floor vibrations, the minimum beam and system frequencies in Table 6.09 will be adopted, unless justified by more detailed analysis.

Location	Element	Floor System
Bridge Footpath	5Hz	4Hz

Table 3.8 - Minimum Vibration Criteria

In addition, detailed footfall vibration assessments will be undertaken in accordance with the following:

CCIP-016 'A Design Guide for Footfall Induced Vibration of Structure,' Concrete Society, 2006

PD 6688-2 'Background to the National Annex to BS EN 1991-2 Traffic Loads on Bridges', Annex A, 2011

For initial assessment the following multiplying response factors, based on continuous vibration as presented in Table 6.10 will be adopted in line with SCI guide P345.

Location	Response Factor
Bridge Footpath	24

Table 3.9 – Recommend Response Factors

The footbridge is a bolted steel structure, a suggested damping ratio in line with CCIP-016 is considered for the footfall dynamics analysis.

Location	Damping ratio
Bridge Footpath	0.5t o 1.0%

Table 3.10 – Damping Ratio

3.3 Fire Resistance

For steel framed structures the fire protection to the elements and connections is to the Architects specification and details. This may take the form of spray applied systems, intumescent coatings, or fire boarding or other means.

Where proprietary thermal breaks are provided these are to be reviewed by the architect and fire consultant to ensure that the fire protection requirements are achieved.

The above is for guidance only. For details refer to the information provided by the Fire Consultant and Architect. All aspects of fire protection for this project are the responsibility of the Fire Consultant and Architect including the fire rating and necessary measure for the provision of fire protection and associated detailing. Renaissance will adopt the fire rating as specified by others in the provision of cover to reinforcement when designing reinforced concrete frames. All other aspects of fire protection measures are the responsibility of the Architect and Fire Consultant and not Renaissance.

3.4 Durability

BS ISO 15686 classifies components as permanent, replaceable, and maintainable construction. Table 6.1 gives the anticipated design life for the structural components forming the frame together with the anticipated maintenance times. First maintenance is defined as the first major maintenance required and assumes that regular inspections and minor maintenance is carried out at regular intervals

Location	Type	Design Life	Anticipated time to first maintenance	Maintenance/ Inspections
RC Foundations	Inaccessible	120 years	N/A	N/A
RC Superstructure	Accessible & Inaccessible	120 years	N/A	N/A
Steel Superstructure	Inaccessible	120 Years	N/A	N/A
Steel Superstructure	Accessible	120 Years	15 years	Periodic inspections, minor maintenance

Table 3.11 – Design Life and Time to First Maintenance

For steel frame elements, corrosion protection and/or galvanising will be required to provide long term durability. These systems typically have a life to first major maintenance of 10-15 years depending on the exposure category, location, and environment. The choice of protection system will be influenced by the location and aesthetic requirements.

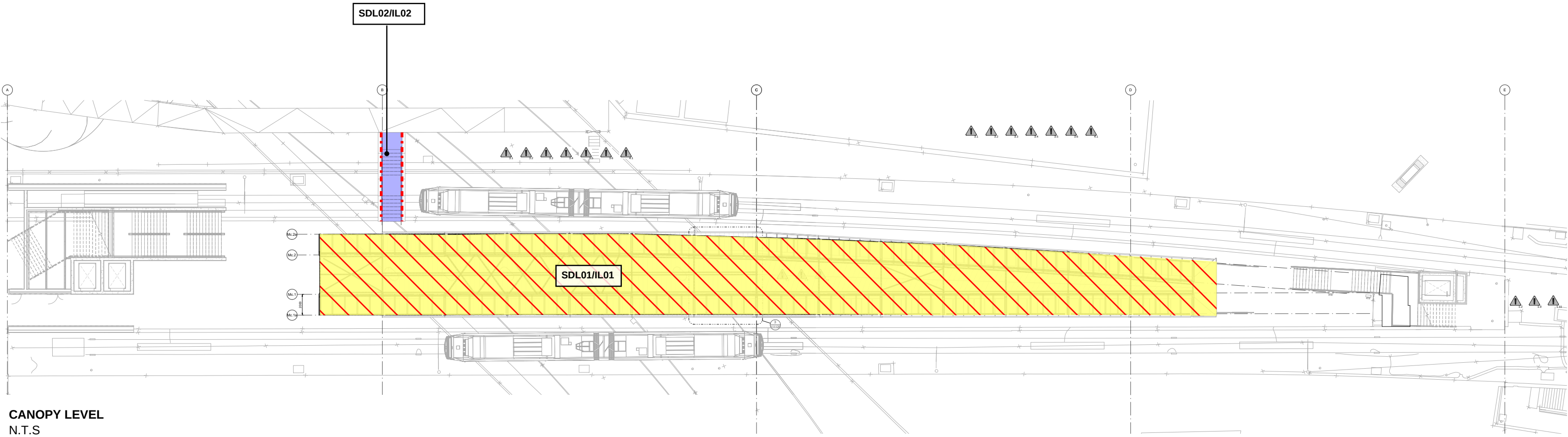
3.5 Robustness & Disproportionate Collapse

The structural design for the structure is to be robust and reduce the sensitivity of the structure to disproportionate collapse in the event of an accident. The assessment will be in accordance with requirements of BS EN 1991-2 and BS EN 1991-1-7.

The steel bridge is designed in line with the UK National Annex to BS EN 1991-2 NA2.36, for load model 4 (Crowd loads), a characteristic footway surcharge of 5 kN/m². This is also in accordance with BS EN 1991-2 clause 5.3.2.1. The "service vehicle" loading given in BS EN 1991-2 has not been designed for given the intended usage of the bridge. It is assumed vehicle barriers will be implemented to prevent any vehicular access in the final case.

Bridge supports are not subject to vehicle impact, as there is no highway located within proximity to the bridge. To account for accidental impact from trams, a concrete deflection barrier is to be installed during construction of third track to protect the bridge support in the occasion of derailment of tram services. Barrier requirement is subject to a risk assessment in accordance with UIC 772-2.

APPENDIX A – PLATFORM TEMPORARY ACCESS STAIRS AND PEDESTRIAN CANOPY LOADING PLAN



GENERAL NOTES:

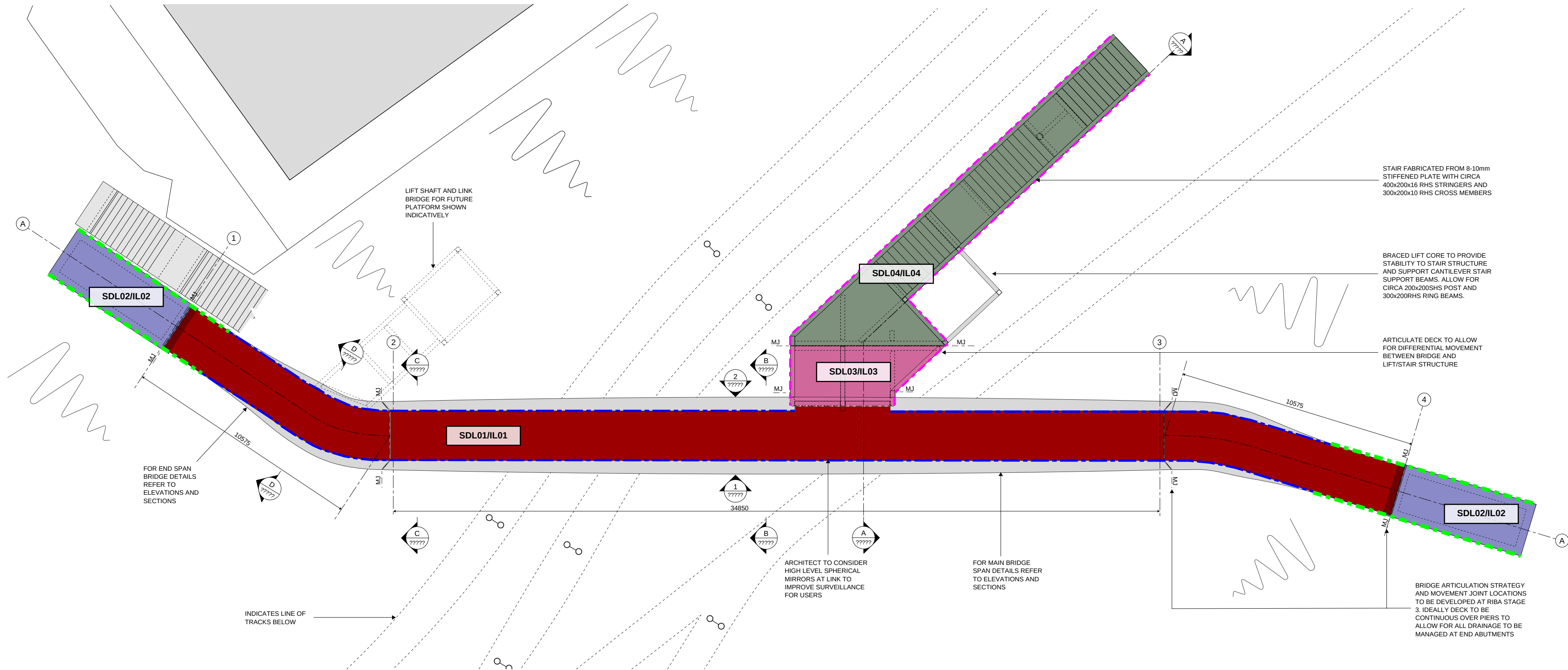
1. THIS DRAWING TO BE READ IN CONJUNCTION WITH ALL RELEVANT ARCHITECT'S DRAWINGS AND ENGINEER'S DRAWINGS AND SPECIFICATIONS
2. DO NOT SCALE FROM THIS DRAWING. ANY AMBIGUITIES, OMISSION AND ERRORS SHALL BE BROUGHT TO THE ENGINEER'S ATTENTION IMMEDIATELY. ALL DIMENSIONS TO BE CHECKED AND VERIFIED ON SITE.

P02	UPDATED TO SUIT S4 DESIGN CHANGES	15.12.25	VN	PW
P01	ISSUED FOR INFORMATION	20.01.25	NA	BH
Rev:	Description:	Date:	By:	Chkd:
renaissance Renaissance Associates Ltd Carvers Warehouse 77 Dale Street Manchester M1 2HG				
Project: BURY INTERCHANGE				
Drawing Title: APPENDIX A - PLATFORM TEMPORARY ACCESS STAIRCASE & PEDESTRIAN CANOPY LOADING PLANS				
Status: RIBA STAGE 3				
Size:	Date:	Drawn by:	Designed by:	Checked by:
A1	20.01.25	NA	-	BH
Scale:	NTS	Project No:	2401-09	
Project:	Originator:	Volume:	Level:	Type:
10612-REN-ZZZ-XXX-T-SE-00041				
Role: Category/Number: Rev:				

APPENDIX B – UNION SQUARE BRIDGE LOADING PLAN

- GENERAL NOTES:
- THIS DRAWING TO BE READ IN CONJUNCTION WITH ALL RELEVANT ARCHITECT'S DRAWINGS AND ENGINEER'S DRAWINGS AND SPECIFICATIONS
 - DO NOT SCALE FROM THIS DRAWING. ANY AMBIGUITIES, OMISSION AND ERRORS SHALL BE BROUGHT TO THE ENGINEER'S ATTENTION IMMEDIATELY. ALL DIMENSIONS TO BE CHECKED AND VERIFIED ON SITE.

REF.	AREA
SDL01 / IL01	Footbridge Path
SDL02 / IL02	Abutment Base
SDL03 / IL03	Articulate Deck
SDL04 / IL04	Bridge Access Stairs
IL01	Footbridge Path
IL02	Stairs Balustrade
IL03	Abutment Transition



FOOTBRIDGE PLAN
N.T.S

P01	ISSUED FOR INFORMATION	20.01.25	NA	BH
Rev:	Description:	Date:	By:	Chkd:
renaissance Renaissance Associates Ltd Carvers Warehouse 77 Dale Street Manchester M1 2HG				
Project: BURY INTERCHANGE				
Drawing Title: APPENDIX A - UNION SQUARE FOOTBRIDGE LOADING PLAN				
Status: RIBA STAGE 3				
Size: A1	Date: 20.01.25	Drawn by: NA	Designed by: -	Checked by: BH
Scale: NTS	Project No: 2401-09			
Project:	Originator:	Volume:	Level:	Type: Role: Category/Number: Rev:
10612-REN-ZZZ-XXX-T-SE-00041				