Viadux 2B - Vibration and Movement Monitoring Summary

The site forms part of a larger development which comprises 2 Phases. The second phase, that this report relates to, includes the construction of a 22-storey residential tower, 74-storey mixed use tower, connecting podium structure, and conversion of the 1870's Grade II listed masonry Viaduct. The 74-storey tower and podium are to be constructed over the existing viaduct. The works also include partial removal of the existing cast-iron bridge deck to 'daylight' Great Bridgewater Street.

The site is currently developed with a Grade II viaduct structure that dates from circa 1876-1879 and is to be retained and incorporated into the scheme (Figure 2.2). The first floor of the viaduct structure is part of a larger structure constructed as part of the Manchester Central Station development.

The new construction involves various opening up works to the existing viaduct structure as well as installation of new CFA piles and pile-caps to support the new RC frame tower. Monitoring is to take place prior to construction at the site, and during the construction of the substructure and superstructure of the new development.

Vibration Monitoring

In order to control vibration emission during the works, continuous monitoring of vibration is to be undertaken. This will enable the contractor to monitor vibration and take suitable action if the agreed criteria are exceeded.

The following construction activities will pose risk to the Grade II listed viaduct due to vibration:

- Excavation
- Piling for foundations
- Piling for the core
- Breakthrough works, including:
    - Great Bridgewater Street Bridge Removal
    - Great Bridgewater Street Abutment Openings
    - Columns Openings
    - Core Opening
    - Core Entrance

To minimise the risk and ensure damage is not imparted on the structure, vibration monitoring is required throughout these works.

Vibration sensors should be monitored 24/7 to act as an early warning of excessive vibration.

Movement Monitoring;

Movement Monitoring instrumentation is to be used to measure the horizontal and vertical displacement of the viaduct structure. It is proposed that tiltmeters mounted to the arch barrels are used to understand arch deformation.
The differential settlement of the piers should be monitored monthly. Vertical Tiltmeters should be monitored 24/7 to act as an early warning of movement, supplemented by the monthly settlement readings.

- The tiltmeters (primary system) shall have an accuracy of +/- 0.1mm.
- The 3D survey targets (secondary system) shall have an accuracy of +/- 0.5mm in either horizontal or vertical direction.

Refer to Renaissance Vibration And Movement Monitoring Specification [V2B-REN-AA-XX-SP-S-00150, July 2025] for the full specification of movement and vibration monitoring extent and duration during the works.

VIADUX 2B, MANCHESTER

GREAT BRIDGEWATER STREET ABUTMENT OPENINGS - STRUCTURAL CALCULATIONS

Project No: 2209-01
June 2025
Rev - P01
V2B-REN-AA-XX-SC-03000

A-5 Great Bridgewater Street Bridge Abutment Openings

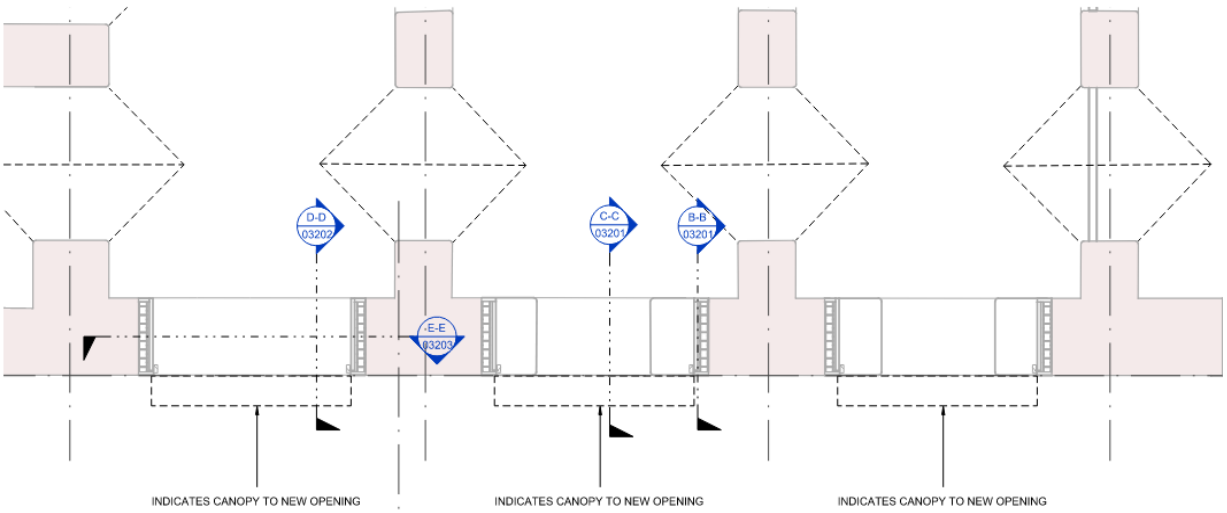
A-5.0	**Introduction**				
A-5.0.0	Introduction	A- 5- 0-	1	to	2
A-5.0.1	Structural Design Philosophy	A- 5- 0-	3	to	4
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A-5.3	**Substructure**				
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A-5.4	**Superstructure**				
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A-5.4.2	Opening 1 - Steel Frame Analysis	A- 5- 4-	1	to	3
A-5.4.3	Opening 1 - Steel Frame Design	A- 5- 4-	4	to	8
A-5.4.5	Opening 2 - Steel Frame Analysis	A- 5- 4-	10	to	12
A-5.4.6	Opening 2 - Steel Frame Design	A- 5- 4-	13	to	16
A-5.5	**Additional Masonry Checks**				
A-5.5.1	Opening Type 1 Masonry Stress Check Under Spreader Beam	A- 5- 5-	1	to	2
A-5.5.2	Opening Type 1 Abutment Overturning Check For Reduced Size	A- 5- 5-	2	to	7
A-5.5.3	Opening Type 1 Masonry Crushing Check For Reduced Pier Size	A- 5- 5-	8	to	9
A-5.5.4	Opening Type 2 Masonry Stress Check Under Spreader Beam	A- 5- 5-	10	to	11
A-5.5.5	Opening Type 2 Abutment Overturning Check For Reduced Size	A- 5- 5-	11		
A-5.5.6	Opening Type 2 Masonry Crushing Check For Reduced Pier Size	A- 5- 5-	11	to	13

A-5.0 INTRODUCTION

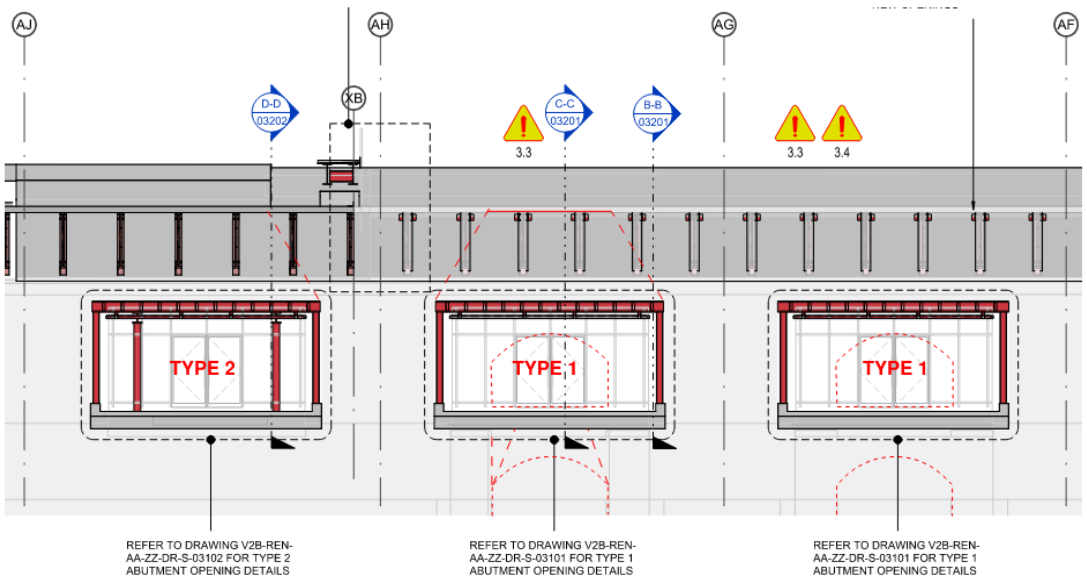
These calculations have been produced to demonstrate the adequacy of the proposed structural works associated with the new openings within the Great Bridgewater Street (GBS) abutment wall and that the existing viaduct stability is not adversely effected by them.

The structural proposals are to form three openings within the existing masonry abutment that runs along GBS as shown below. Reference should also be made to drawing series V2B-REN-AA-ZZ-DR-S-03100 for full details. The existing abutment wall is 2.45m thick and of solid masonry and lime mortar construction. Based on the site investigation and Phase 1 Viadux works the abutment is founded at circa 4.5m below ground level into the weathered sandstone. The abutment currently supports the existing cast iron arch bridge over GBS and two higher level precast concrete bridges installed in the 1980's. It also supports a steel and composite frame for a marquee enclosure which is to be removed.

The abutments form part of the lateral stability system to the viaduct structure. The segmental arches span parallel to the abutment with a visible joint between them and significant buttressing to the end spans. It is therefore assumed that no loads are transferred to the abutment from the segmental arches. The pointed masonry arches within the viaduct and the cast iron bridge beams over GBS will exert horizontal thrusts perpendicular to the GBS abutment wall. This is currently resisted by the return walls to which effectively form 'T' shaped buttressing. The openings have been sized to ensure that the T buttresses are maintained and calculations are included below to demonstrate this.



PLAN



ELEVATION

A-5.0 INTRODUCTION (CONT.)

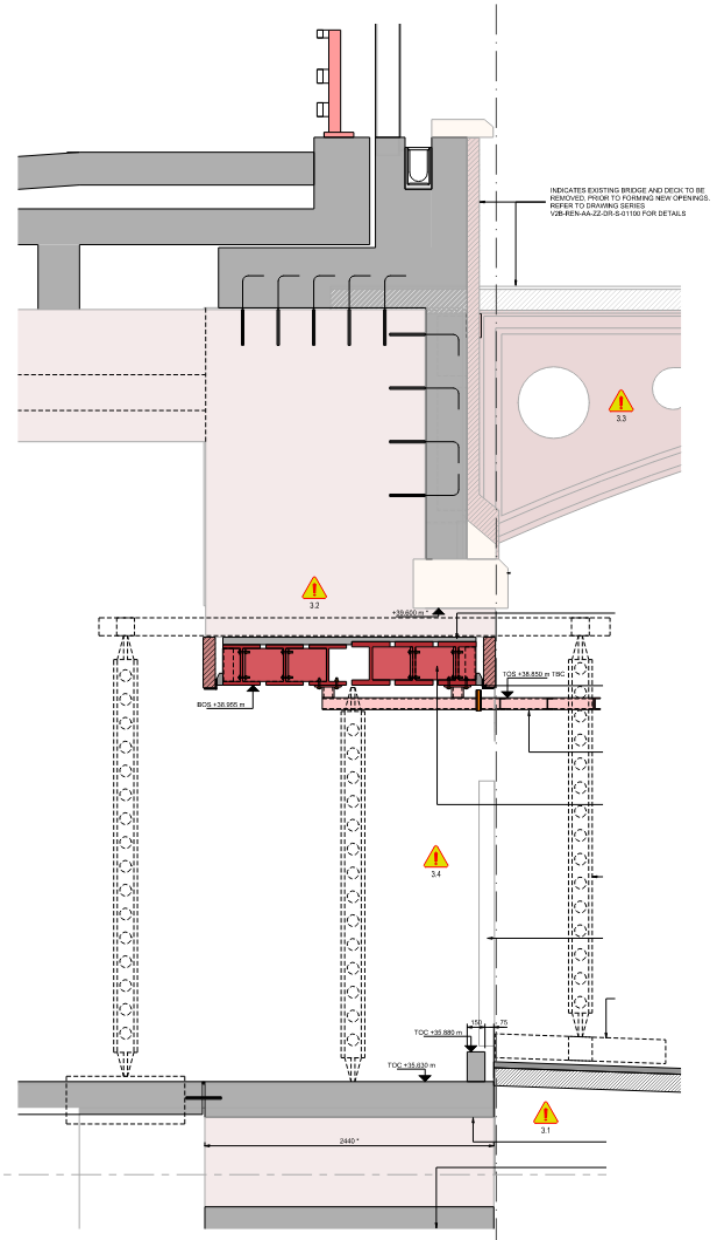
The proposed openings consist of two types:

OPENING TYPE 1

The middle and right hand side openings (Opening Type 1) enlarge existing arch openings. As indicated in the section below the existing 2.7m of masonry above the new opening is to be retained, together with the 15m clear span cast iron arches over GBS. A new RC liner wall and masonry facing along the GBS elevation, together with a new facade, footing and service yard slab are also to be installed above the opening. The existing bridge deck including all finishes, the sections of open mesh flooring, and marquee structure over GBS are all to be removed within the load influence zone of the new openings.

Each opening frame is to be 6.65m long x 3.65m high and formed with 6No. portal frames that are tied together to distribute the applied actions between them. These frame are to be supported on a reinforced concrete spreader base that is designed to distribute the point loads into the existing masonry below. The design philosophy and load paths are discussed further below.

Opening Type 1 is not to be installed until the bridge deck and finishes, and marquee have been removed to prevent overstressing of the frame.



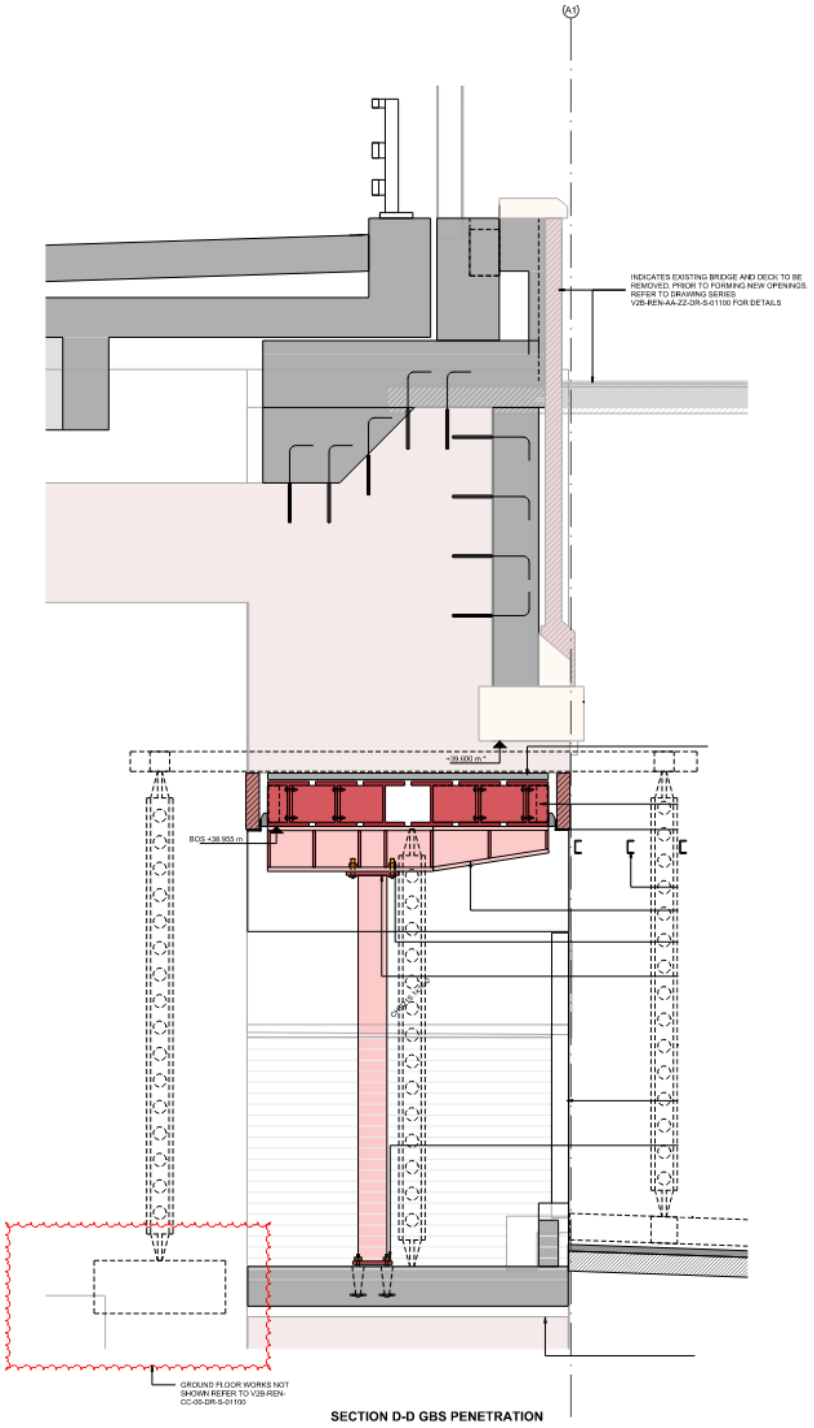
OPENING TYPE 1 SECTION

A-5.0 INTRODUCTION (CONT.)

OPENING TYPE 2

The left hand side opening (Opening Type 2) also enlarges an existing opening to support the existing masonry above, together with the 15m clear span cast iron arches over GBS. The existing bridge deck, included steel frame open mesh floor above is to be retained. A new steel frame footbridge is also to be installed.

The opening frame is to be 6.65m long x 3.65m high and formed with 6No. portal frames that are tied together to distribute the applied actions between them. Due to the higher loads intermediate steel beam and posts are provided to control deflections and more evenly distribute forces into the masonry below. The design philosophy and load paths are discussed further below.



OPENING TYPE 2 SECTION

A-5.0.1 STRUCTURAL DESIGN PHILOSOPHY

OPENING TYPE 1

The loads from the facade, footing, retaining wall, service yard slab and arches will all transfer into the mass of masonry above the new beams. Where loads are directly over the frames these will distribute evenly across the frames and be applied as UDL's. The mass of masonry above and around the new openings will result in loads within a 60 degree spread of the frame ends remaining within the adjacent masonry. For applied UDL's this will beneficial load distribution will be conservatively ignored and the loads applied over the full beam length. For incoming members, such as the cast iron arches, the applied loads will be assumed to distribute at a 45 degree spread in all directions and will be applied as a partial UDL along the frames which are within the influence zone. As the top of the beams are to be fully grouted the loads will distribute evenly out of the masonry and into the steelwork. For analysis purposes horizontal restraints will be modelled to the top frame members.

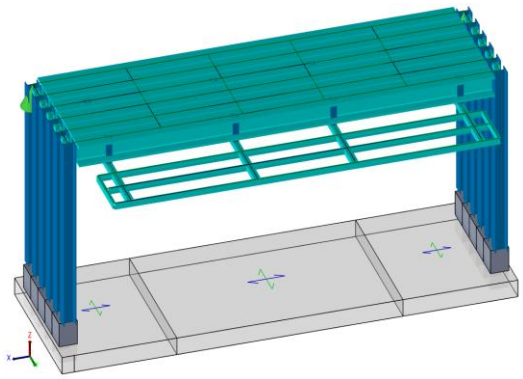
The structural frame will be analysed as 6No. steel portal frames with pinned bases. The applied loads will be transferred through the frame in combined bending and shear within the top members and bending, axial and shear within the posts. The steel beams forming the top of the frames are tied together at 5th points with fin plates to distribute the applied loads across the frames and to resist lateral torsional buckling of the beams. The back face of the posts are to be fully grouted following grouting of the top beams and removal of all propping. This provides enhances stiffness under imposed loads, whilst avoiding overstressing the column to beam connections under dead loads. For analysis this additional restraint will be conservatively ignored. The frames are laterally restrained in both horizontal planes by the existing mass masonry abutments so by inspection global buckling of the system is therefore not considered critical.

The forces from the steel frame will transfer into the RC upstand and into the RC spreader base, before transferring into the masonry in bearing. The spreader will act in direct bearing and a combination of bending and shear. A tension load will also occur and the RC spreader will tie the bases together. To realsitically assess the bearing of the spreader on the masonry, they will be modelled as a patch spring support based on a maximum 0.5mm settlement under the maximum applied loadcase. A relieving arch is present below the existing opening so the area of support will be reduced as shown below. In practice the bearing area will extend over a 60 degree spread from the edge of relieving arch, however, this will be conservatively ignored.

By inspection the removal of the bridge deck, combined with removal of the masonry within the enlarged opening will offset the new loadings from the facade, footings, retaining wall and service yard slab above a a global reduction in foundation stresses will occur. No checks on the foundations bearing pressures is therefore required.

The canopy frame is a lightweight steel frame that is fixed to the underside of the main frame beams. The purlins span between the SHS members which cantilever and are fixed back to two of the frames.

To demonstrate the adequacy of the frame the following checks will be undertaken:
- Derivation of applied actions
- Local and global checks on the proposed steel frame elements, including canopy
- Design of the RC upstands and spreader base
- Bearing Checks beneath the spreader



OPENING 1 ANALYSIS MODEL

A-5.0.1 STRUCTURAL DESIGN PHILOSOPHY (CONT.)

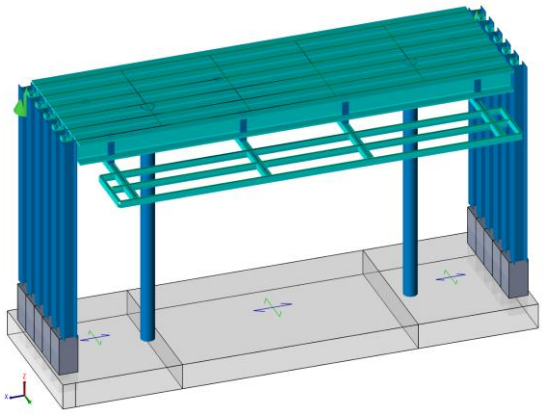
OPENING TYPE 2

The structural philosophy will be the same as Opening Type 1.

The loads distribution will be the same as Opening Type 1. As the lightweight steel infill bridge and facade are outside the 60 degree influence zone from the end of the frames and the forces from these will remain within the masonry as the frame deflects, these are safely ignored.

Due to the higher loadings from the retained bridge deck and mesh, 2No. intermediate tapered steel T beams supported on posts are provided to reduce deflections of the frame and forces within the top members. The steel T beams and post are modelled with fixed head and pinned base.

To demonstrate the adequacy of the frame the following checks will be undertaken:
- Derivation of applied actions
- Local and global checks on the proposed steel frame elements, including canopy
- Design of the RC upstands and spreader base
- Bearing Checks beneath the spreader



OPENING 2 ANALYSIS MODEL

A-5.0.2 SURVEYS & ARCHIVE INFORMATION

In assessing the existing structure reference has been made to the following survey information:

- Drawing SSL:1468Z:250:2:2:Lower, Survey Systems April 2020
- Drawing SSL:1468Z:100:1:3:SEC, Survey Systems April 2020
- System Systems Revit Model
- 1875 Archive drawings of original cast iron arch beams

Spot dimension checks have also been undertaken on site to verify the survey and archieve infromation adopted within the design.

A-5.0.3 DESIGN CODES

The structural design for the building and associated structural elements will be undertaken to the Eurocodes and where required 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 1996: (Eurocode 6) Design of masonry structures

For derivation of existing loads and material reference will also be made to:

- CS 454 'Assessment of highway bridges and structures', March 2020

A-5.0.4 DESIGN WORKING LIFE

In accordance with BS EN1990:2002 the structure is a building with a design working life category of 4. The design working life is therefore 50 years.

A-5.0.5 MATERIAL PROPERTIES

MATERIAL DENSITIES

For assessment the following unit weights will be assumed. For assessment of the masonry the characteristic weight obtained through testing will be used:
Source: Viadux intrusive works and material testing data report, 2020, CML (CML,2020)

Material		Unit Weight (kg/m^3)	Unit Weight (kN/m^3)
Metals	Cast Iron	7200	70.6
	Wrought Iron	7700	75.5
	Steel	7850	77.0
Concrete	Reinforced Concrete	2400	23.5
Masonry	Viadux Material Testing Overall Mean	1849	18.1
	Sandstone	2400	23.5
Surfacing	Bituminous Macadam (Tar)	2400	23.5
Fill	Gravel Ballast Fill*	2000	19.5
	Miscellaneous Fill	2200	21.6

* Density testing calculated a fill density of 19.5kN/m^3 which is to be used in the calculations going forward. The fill was originally identified in archive drawings to be clinker fill and ballast, however during other periods of works site-won materials have replaced/added to the existing fill. The fill is now made ground, comprised of general fill material including gravel and sand.

STRUCTURAL STEELWORK

The following steelwork properties are adopted within the design. Based on Table 3.1 of Eurocode 3, part 1-1

	t (mm)	fy N/mm^2	fu N/mm^2	E kN/mm^2	G kN/mm^2	α 1/K	γ_C
S275	≤40	275	430	210	81	1.20E-05	1.00
S275	>80	255	410	210	81	1.20E-05	1.00
S355	≤40	355	490	210	81	1.20E-05	1.00
S355	>80	335	470	210	81	1.20E-05	1.00

CONCRETE

The following concrete properties are adopted within the design. Based on Table 3.1 of Eurocode 2, part 1-1

Structural Member	Grade	Cylinder Strength fck	Dry Density	Wet Density	Partial Material Factor	Modulus of Elasticity[1]	Poisson's Ratio	Shear Modulus G[1]	Coefficient of Linear Thermal Expansion
		N/mm^2	kg/m^3	kg/m^3	γ_C	kN/mm^2	v	kN/mm^2	1/K
Foundations	RC35/45	35	2500	2600	1.5	30.06	0.20	14	0.000010

REINFORCEMENT

The following reinforcement properties are adopted within the design. Reinforcement class to BS4449 and designation to BS 8666. As the building is Class 3 the reinforcement is to be a minium class 500B to provide enhanced ductility.

	Type	Rib Type	Detailing Prefix	Class N/mm^2	Ductillity Grade	Strain at maximum force ε_{uk} (%)	Modulus of Elasticity N/mm^2	Partial Material Factor γ_C
Rebar	Loose bar	Type 2	B	500	B	5	205000	1.15

CONCRETE ANALYSIS MODIFICATION FACTORS

The reinforced concrete structure will generally be designed using elastic analysis for the ultimate limit and serviceability limit states. To account for non-linear behaviour (cracking of the elements etc) a 0.5 stiffness modification factor will be adopted as per Eurocode 2 for the RC upstands and spreader slab. Given the relatively low magnitude of forces and relative thickness of elements this is considered acceptable.

A-5.0.5 MATERIAL PROPERTIES (CONT.)

MASONRY

Material	Unit Compressive Strength (N/mm^2)
Viadux piers/abutments brickwork mean**	14

* For the Viadux material testing values refer to data in CML report 'Viadux intrusive works and material testing,' Apr-Aug 2020.

MORTAR CLASSIFICATION

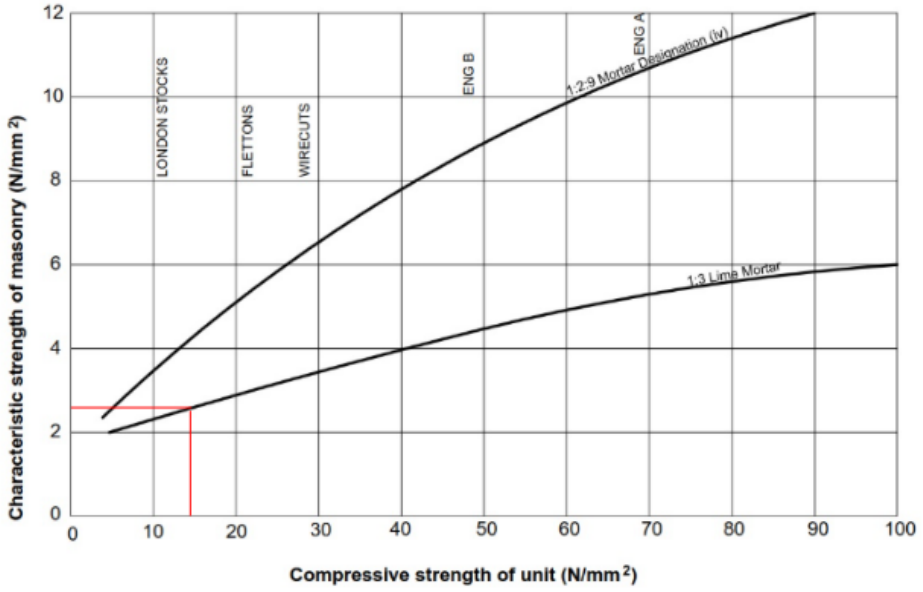
X-ray diffraction (XRD) testing carried out on the mortar samples from Viadux determined the usage of two types of mortar, defined in the table below. The mortar classification is based upon Ciria C656 table 2 (as below). Sample 18520/2 was found to have indicative mix proportions of 1:2-3 lime hydrate to sand.

REF	BINDER TYPE	MORTAR DESIGNATION	MORTAR STRENGTH (N/mm^2)
18520/2	NON-HYDRAULIC PURE AIR LIME	vii	0.5

*XRD gave mortar designation i to ii, conservatively assumed ii.
Refer to data in CML report 'Viadux intrusive works and material testing,' Apr-Aug 2020.

MASONRY

Figure 4.2.7a Characteristic strength of brick masonry



The low unit strength results obtained through the 24hr surface dry compression testing (Viadux intrusive works and material testing data report, Apr-Aug 2020, CML) give the lowest characteristic strength of masonry according to 4.2.7a, above:

Characteristic strength of masonry = 2.5 N/mm^2

A-5.0.5 MATERIAL PROPERTIES (CONT.)

MASONRY

MATERIAL FACTOR OF SAFETY

To account for the condition of the masonry a partial material factor γ_m will be used based upon the condition and joint factor in CS454.

Condition Factor $F_c = F_{cm} F_j$

- Masonry Condition Factor (CS454 Tb 7.5.1a) Only minor defects — F_{cm} = 0.9
- Joint Factor $F_j = F_w F_d F_{mo}$
  - Joint Width Factor (CS454 Tb 7.5.1b) Typically 10-12mm — F_w = 0.9
  - Joint Depth Factor (CS454 Tb 7.5.1c) Generally in good condition, unpointed up to 12. — F_d = 0.9
  - Joint Mortar Factor (CS454 Tb 7.5.1d) Conservatively assume loose and friable mortar — F_{mo} = 0.9

  F_j = 0.73
- Condition Factor — F_C = 0.66

Material Factor of Safety, $\gamma_m = 1/F_C$ — **γ_m = 1.52**

As the viaduct abutment is a mass masonry structure this factor of safety is considered more appropriate than those present in EC6 which are based on thin leaf masonry with allowance for chases etc. and would therefore be overly conservative.

A-5.1 DESIGN ACTIONS

A-5.1.1 INTRODUCTION

This calculation package includes a summary and associated derivation for the permanent (Gk), variable (Qk), wind (Wk) and accidential (Ad) actions that the two types of openings in the abutments will be designed for. It also includes the design combinations to be considered within deriving the design actions for the element design.

A-5.1.2 DESIGN LOAD COMBINATIONS

1 STR - $1.35G_k + 1.5Q_k$
2 STR - $1.35G_k + 1.5Q_k + 1.5\Psi_0 W_{ky}$
3 STR - $1.35G_k + 1.5W_{ky} + 1.5\Psi_0 Q_k$
4 STR - $1.0G_k + 0.0Q_k - 1.5W_{ky}$

Note: The frames are fully restrained by the existing mass masonry structure which by inspection are substantial enough to resist any equivalent horizontal forces resulting from the frames. EHF has therefore been ignored within the frame analysis.

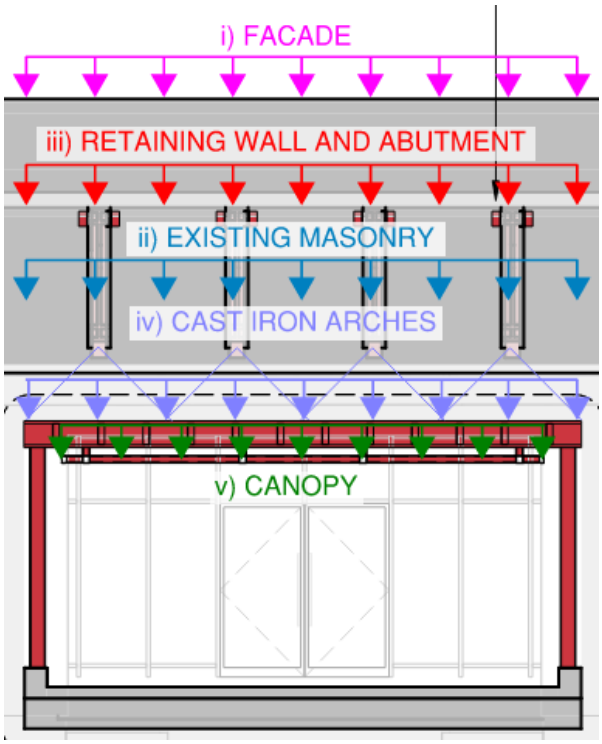
A-5.1.3 PERMANENT ACTIONS

INTRODUCTION

This section provides an overview of the permanent design loads for which each of the two frame types will be designed under normal conditions.

G_k - OPENING TYPE 1

The dead loads to be applied to the frames are summarised below with supporting calculations on the following pages. All loads will conservatively be applied along the full length of the frame, ignoring the 60 degree 'corbelling' of the masonry at the supports.



2.2 kN/m (All 6 Frames)

16.1 kN/m (All 6 Frames)

17.5 kN/m (All 6 Frames)

10.7 kN/m (Outer 3 Frames)

FOR INNER 3 FRAMES w_{Gk} 35.8 kN/m

FOR OUTER 3 FRAMES w_{Gl} 46.5 kN/m

NOTE: All the above dead loads exclude the self-weight of the structure which is automatically calculated in the analysis model

Figure 5.2 - Gk Loadings to Opening 1

A-5.1.3 PERMANENT ACTIONS (CONT.)

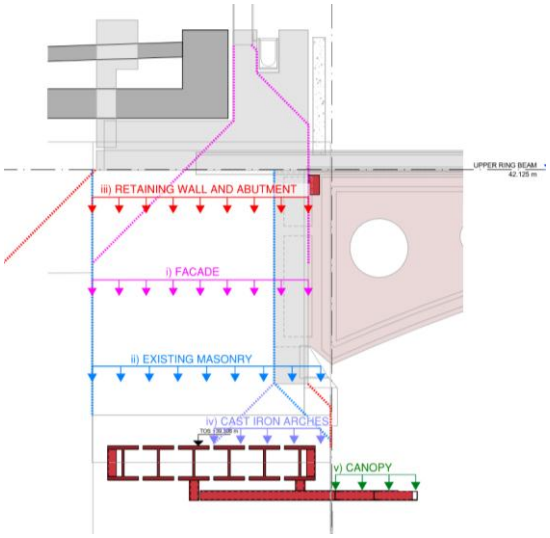


Figure 5.7 Application of G_k Loads to Opening Type 1

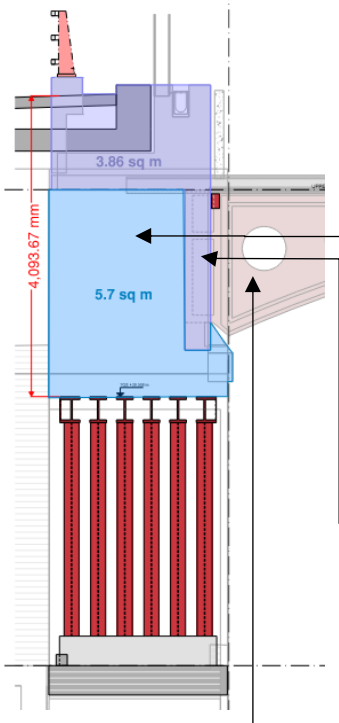
i) A new façade is proposed to the service yard above, as well as a vehicle restraint barrier alongside it.

ii) Existing masonry will be retained above the opening.

iii) A new retaining wall and abutment is proposed to form the edge of the new opening in the Great Bridgewater Street Bridge. Reference should be made to the Great Bridgewater Street Bridge Deck Removal Calculation pack for details.

iv) Although the bridge deck is to be removed, in the permanent case the cast iron arches and masonry facing will be reinstated

v) A new canopy will extend over the pavement of Great Bridgewater Street. The steelwork for this will be suspended from the main opening steelwork.



i) *FAÇADE LOAD:*

Tower Façade Height	=	9.6	m
Perforated aluminium panels	=	1.4	kN/m^2
(refer to Viadux Phase 1 Façade Steelwork Calculations)			
Line load on service yard edge	=	13.4	kN/m

Line load on each frame = 2.2 kN/m

(applied over full length of all 6 frames)

ii) *EXISTING MASONRY LOAD*

Cross-sectional area	=	5.70	m^2
Width of masonry	=	2.45	m
Density of masonry	=	18.1	kN/m^3
Line load on opening	=	105.0	kN/m

Line load on each frame = 17.5 kN/m

(applied over full length of all 6 frames)

iii) *LOAD FROM NEW RETAINING WALL, FAÇADE FOOTING AND SERVICE YARD BUILD-UP*

Cross-sectional area	=	3.86	m^2
Density of reinforced concrete	=	25	kN/m^3
Line load on opening	=	96.5	kN/m

Line load on each frame = 16.1 kN/m

(applied over full length of all 6 frames)

iv) *CAST IRON ARCHES AND MASONRY FACING (SEE FOLLOWING PAGE)*

A-5.1.3 PERMANENT ACTIONS (CONT.)

CAST IRON ARCHES AND MASONRY FACING

The Cast iron arches consist of a top chord and bottom chord, which are shown on archive drawing Figure 5.6. A 32mm thick flange runs between the chords (area taken from measured survey (Figure 5.5)

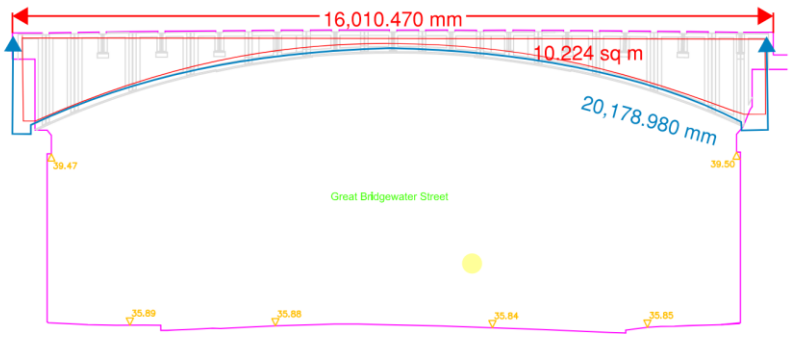


Figure 5.5 - Measured Survey Mark Up

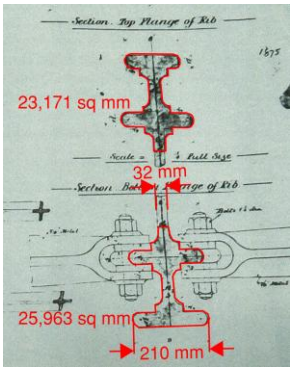


Figure 5.6 - Archive Detail

Top flange Cross-sectional area	=	23,180	mm^2
Top flange Length	=	16	m
Top Flange Volume	=	0.37	m^3
Bottom flange Cross-section	=	26,000	mm^2
Bottom flange Length	=	21	m
Bottom Flange Volume	=	0.5	m^3
Web Area at 32mm THK	=	10	m^2
Web Volume	=	0.35	m^3
Volume Cast Iron Arch	=	1.27	m^3
Material Density	=	70.6	kN/m^3
Load of 1 arch	=	89.5	kN

For overall cast iron arch weight, conservatively take **90.0 kN**

Arch Spacing	=	1.85	m
Arch Weight	=	90.0	kN
Reaction onto Abutment	=	45.0	kN
Equivalent UDL on support frame	=	24.3	kN/m
Single Skin Masonry facing	=	7.8	kN/m

Conservatively assume this distributes over minimum 3 frames

Line load on three outer frames **= *10.7 kN/m***

v) *CANOPY LOADS*

Assume cladding similar to Viadux 1 Main entrance:

3mm Aluminium Cladding (Top)	=	0.2	kN/m^2
Services	=	0.1	kN/m^2
3mm Aluminium Clad (Bottom)	=	0.2	kN/m^2
Total Area Load from Canopy	=	0.5	kN/m^2

For worst case gravity loading take ***0.8 kN/m²***
For worst case wind uplift take ***0.5 kN/m²***

FRAME 1 - GK DESIGN LOADS

Total Gk applied to all 6 frames of Opening 1 **= *35.8 kN/m***
Additional Load applied to outer three frames **= *10.7 kN/m***
Canopy Loading **= *0.8 kN/m²***

A-5.1.3 PERMANENT ACTIONS (CONT.)

G_k - OPENING TYPE 2

The dead loads to be applied to the frames are summarised below with support calculations on the following pages. All loads will conservatively be applied along the full length of the frame, ignoring the 60 degree 'corbelling' of the masonry at the supports. As the steel bridge load is relatively low this willl conservatively be applied full length

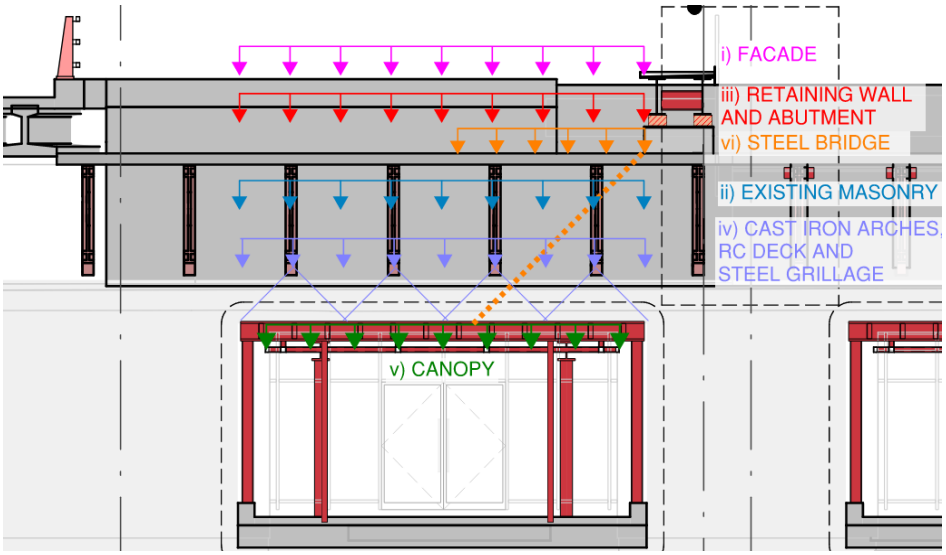


Figure 5.8 - Gk Loadings to Opening 2

i) 2.2 kN/m (All 6 Frames)

iii) 16.1 kN/m (All 6 Frames)

vi) 2.0 kN/m (All 6 frames half length)

ii) 17.5 kN/m (All 6 Frames)

iv) 40.1 kN/m (Outer 3 frames)

FOR INNER 3 FRAMES:

W_{Gk} = 38 kN/m

FOR OUTER 3 FRAMES:

W_{Gk} = 78 kN/m

CANOPY LOADING

W_{Gk} = 0.5 kN/m^2 MIN

W_{Gk} = 0.8 kN/m^2 MAX

NOTE: All the above dead loads exclude the self-weight of the structure which is automatically calculated in the analysis model

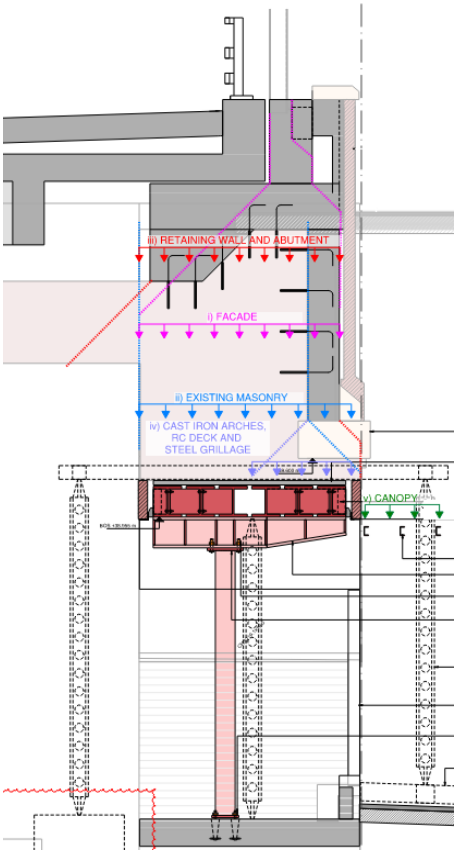


Figure 5.10 Application of G_k Loads to Opening Type 2

A-5.1.3 PERMANENT ACTIONS (CONT.)

As per Opening Type 1, loading is applied from:
(i) the façade above
(ii) the existing retained masonry
(iii) the new retaining wall and abutment
(iv) the cast iron arches including buckle plate, RC Slab, Paving, open mesh steel frame
(v) the canopy
(vi) the new steel bridge

i)	*FAÇADE LOAD:*	Refer to G_k - Opening 1 Calculation	=	**2.2**	**kN/m**
ii)	*EXISTING MASONRY LOAD*	Refer to G_k - Opening 1 Calculation	=	**17.5**	**kN/m**
iii)	*LOAD FROM NEW RETAINING WALL AND ABUTMENT*	Refer to G_k - Opening 1 Calculation	=	**16.1**	**kN/m**
v)	*CANOPY LOADS*				
	For worst case gravity loading take	Refer to G_k - Opening 1 Calculation	=	**0.8**	**kN/m²**
	For worst case wind uplift take	Refer to G_k - Opening 1 Calculation	=	**0.5**	**kN/m²**

iv) *CAST IRON BRIDGE SECTION*

In addition to the cast iron bridge beams opening 2 will also support the weight of the existing bridge deck and all associated build-ups above will be supported on the outer three frames.

Steel Open Mesh Floor	=	1.0	kN/m²		
Support Steelwork on plan	=	0.75	kN/m²		
65mm Concrete paviors	=	1.43	kN/m²		
50mm Sand	=	0.95	kN/m²		
225 THK RC Slab	=	5.6	kN/m²		
Buckle Plate 0.5 Inch with stiffeners	=	1.25	kN/m²		
Cast Iron Arch Weight		Refer to G_k - Opening 1 Calculation		**90.0**	**kN**
Cast Iron Arches (1.85c/c)	=	3.04	kN/m²		
Total UDL of bridge deck				**14.0**	**kN/m²**
Total line load from bridge	=	112.4	kN/m		
Single Skin Masonry facing	=	7.8	kN/m		
Line load on each frame		***(applied full length of outer 3 frames)***		***40.1***	***kN/m***

vi) *STEEL BRIDGE*

Bridge length = 15 m

BRIDGE DECK

10mm Weathered Steel Deck	=	10	mm	x	1.35	m	x	77	x	15 m length
	=	15.6	kN							
Conservatively allow for deck		20.0	kN							
Main Beams	=	2	x	155	kg	x		15		m length
(2No. UB 610x324x155)	=	45.6	kN							
Conservatively allow for beams		50.0	kN							
Ladders	=	14	x	28	kg	x		0.825		m width
(14No. PFC 260x75x28 over full width of bridge)										
	=	3.2	kN							
Conservatively allow for ladders		5.0	kN							

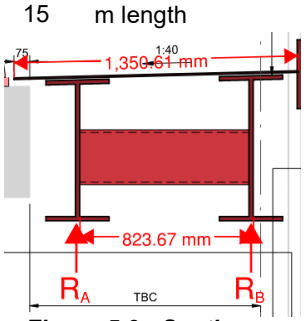


Figure 5.9 - Section

Total Bridge Load excluding parapet	=	*75.0*	*kN*

A-5.1.3 PERMANENT ACTIONS (CONT.)

PARAPET

Steel Parapet Web (10mm THK)	=	10	mm	x	3.25	m	x	77	x	15 m
	=	37.5	kN							
Steel Parapet Stiffeners (60mm x 15mm THK)	=	0.0009	mm^2	x	160	m	x	77		
	=	11.1	kN							
Total Parapet Load	=	48.6								
Conservatively allow for parapet		50.0	kN							

Reactions Due to Parapet

	P_A =	-5.3	kN
	P_B =	30.3	kN
Bridge end reactions	=	Total bridge load/4 + parapet reaction	
	R_A =	13.4	kN
	R_B =	49.1	kN

Conservatively take	R_A =	50.0	kN
	R_B =	50.0	kN
Load on Plan	=	100.0	kN

Load distributed over 8.23m = 12.2 kN/m (conservatively applied over half length of all 6 frames)

Line load on all 6 frames **= 2.0 kN/m**

FRAME 2 - GK DESIGN LOADS

Total Gk applied to all 6 frames of Opening 2	=	***35.8***	***kN/m***
Gk from steel bridge applied to half length of all 6 frames	=	***2.0***	***kN/m***
Additional Load applied to outer three frames	=	***40.1***	***kN/m***
Canopy Loading	=	***0.8***	***kN/m^2***

A-5.1.4 VARIABLE ACTIONS

Q_k - APPLIED LOADINGS TO OPENING 1

For derivation of service yard loading, refer to Section A-0.2.1 .

Area Load on Plan = 15 kN/m^2 (from service yard)

Load acting on opening = 15 x 2.5 m width

= 37.5 kN/m

Line load on each frame (applied over full length of all 6 frames) **= 6.25 kN/m**

Q_k - APPLIED LOADINGS TO OPENING 2

SERVICE YARD

Line load on each frame (from service yard, applied over full length of all 6 frame **= 6.25 kN/m**

MESH GRILLAGE AREA

Area Load on Plan = 5 kN/m^2 (from service yard)

5.0 kN/m^2 x 7.5 m bridge width

Line load on each frame (applied full length of outer 3 frames) **= 12.5 kN/m**

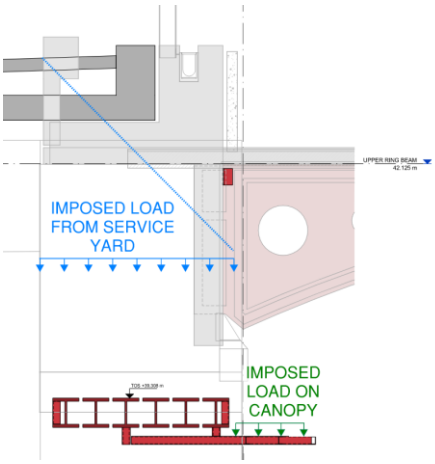
STEEL BRIDGE AREA

Area Load on Plan = 5.0 kN/m^2

Load from half of bridge = 50.6 kN

Load distributed over 8.23m = 6.2 kN/m

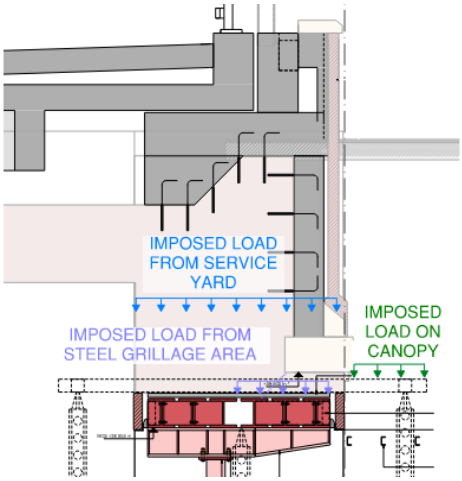
Line load on each frame (applied over full length of all 6 frames) **= 1.0 kN/m**



OPENING TYPE 1

FOR ALL 6 FRAMES w_{Qk} = 6.5kN/m

CANOPY LOADING w_{Qk} = 0.60kN/m^2



OPENING TYPE 2

FOR INNER 3 FRAMES w_{Qk} = 6.5kN/m

FOR OUTER 3 FRAMES w_{Qk} = 19.0kN/m

CANOPY LOADING w_{Qk} = 0.60kN/m^2

A-5.1.5 WIND ACTIONS (W_k)

W_k - OPENINGS 1 AND 2 - CANOPIES

As wind tunnel testing for the proposed scheme is not yet available, reference has been made to the wind tunnel testing which was completed as part of the original office block proposals (Figure 5.11). The wind tunnel test gives design pressures of up to 3kN/m^2 on the 3m high parapet to the podium level which is circa 14m above the entrance canopies . The facade pressure at entrance area will be in the region of 1.5kN/m^2. For assessment of the canopy conservatively take a design wind load of 2kN/m^2 and apply the pressure cefficient pressures for a monopitch canopy.

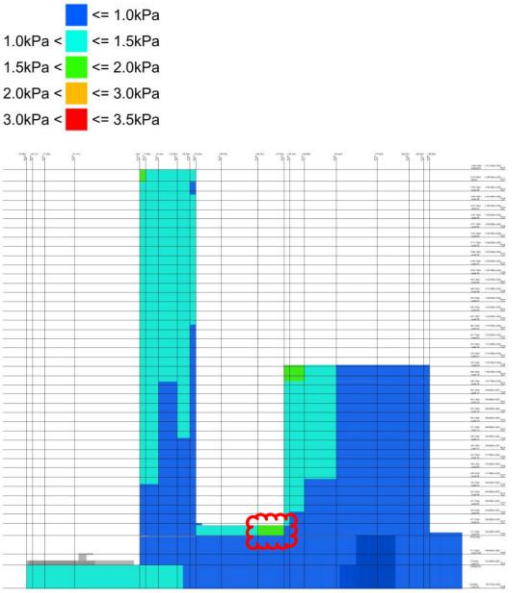
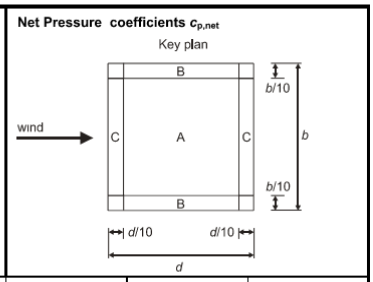


Figure 5.11 - W_k Loadings to Canopies

Table 7.6 — $c_{p,net}$ and c_f values for monopitch canopies



Roof angle α	Blockage φ	Overall Force Coefficients c_f	Zone A	Zone B	Zone C
0°	Maximum all φ	+ 0,2	+ 0,5	+ 1,8	+ 1,1
	Minimum $\varphi = 0$	- 0,5	- 0,6	- 1,3	- 1,4
	Minimum $\varphi = 1$	- 1,3	- 1,5	- 1,8	- 2,2

Figure 5.12 - $C_{P,net}$ values from BS-EN 1991-1-4

This conservatively gives the most onerous cases shown below

Zone A Pressure	(over middle 5670x765mm area)	=	2	x	1.5	=	+/-	**3.0**	**kN/m^2**
Zone B Pressure	(over b/10 = 85mm)	=	2	x	1.8	=	+/-	**3.6**	**kN/m^2**
Zone C Pressure	(over d/10 = 630mm)	=	2	x	2.2	=	+/-	**4.4**	**kN/m^2**

W_k - OPENINGS 1 AND 2 - BRIDGE DECK

The bridge is adjacent to the proposed tower, however the tower is set back from Great Bridgewater Street with a podium and circa 3m high parapet in front of it. This willl disrupt downdrafts from the new tower and avoid the avoid very high loads. Conservatively adopt wind pressures from the podium and apply these on the bridge. In practice, the wind loads will be significantly smaller than the applied imposed loads and by inspection will not be the critical design case.

Area Load on Plan = 2.0 kN/m^2

Line load on each frame = 2.0 kN/m^2 x 7.5 m bridge width **= *5.0* *kN/m***

(applied full length of outer 3 frames)

A-5.1.6 SNOW LOAD (Q_k)

INTRODUCTION

The following calculations determine the design snow load for the canopy to BS EN 1991-1-3.

SITE SNOW LOAD

Snow Load - Zone 3 — $S_{k,,map}$ = 0.5 kN/m² (NA Fig NA.1)
Site Altitude — A = 35.9 m
Site Snow Load — $S_{k,,site}$ = 0.38 kN/m²

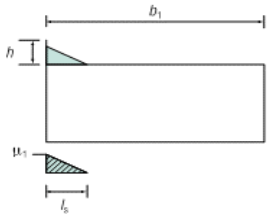
ROOF SNOW LOAD

$S_k = \mu_1 \ S_{k,site}$ (EC eq 5.2)

For flat roof — μ_1 = 0.8
Design snow load — ***S_k = 0.30 kN/m²*** ***Therefore not critical to design***

SNOW DRIFT AT BUILDING INTERFACES

Snow drift is checked against the viaduct:



Parapet height — h = 12.5 m
Building Length — b_1 = 1.2 m
Drift length (lesser of 5h, b1 or 15m) — l_s = 1.2 m

Direction 1 (NNE/SSW)

Shape coefficient is lesser of:

$\mu_1 = 2h/S_{k,site}$ $\mu_1 = 2b/l_s$ $\mu_1 = 8$
$\mu_1 = 2$

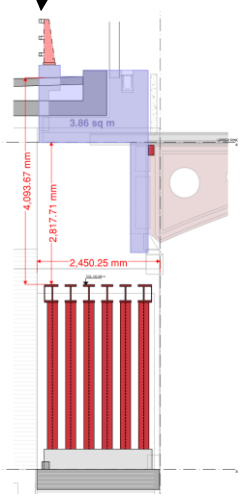
Accidential drift load — ***$S_{k,drift}$ = 0.76 kN/m²***
Drift Length — ***l_s = 1.20 m***

$$0.76 \times \frac{1.20}{2} = 0.45 < 0.6\text{kN/m}^2$$ ***imposed load. Conservatively adopt design imposed load.***

A-5.1.7 ACCIDENTAL ACTIONS (A_d)

VEHICLE IMPACT TO SERVICE YARD RESTRAINT BARRIER (OPENING 1)

Horizontal impact load	=	825	kN (Refer to Section A-1.3 vehicle barrier loading)
Resulting moment	=	742.5	kNm at 0.9m impact height
Push/pull force from moment	=	371.25	kN assuming 2m lever arm
Load distributed over 10.4m	=	35.7	kN/m
Line load on each frame	=	5.9	kN/m (applied over full length of 6 frames)



The line loads under normal use are a minimum of 6.25kN/m on each frame with the design undertaken with full factors of safety. By inspection the accidental load with reduced factors of safety is therefore not critical and can be ignored for the design of the openings

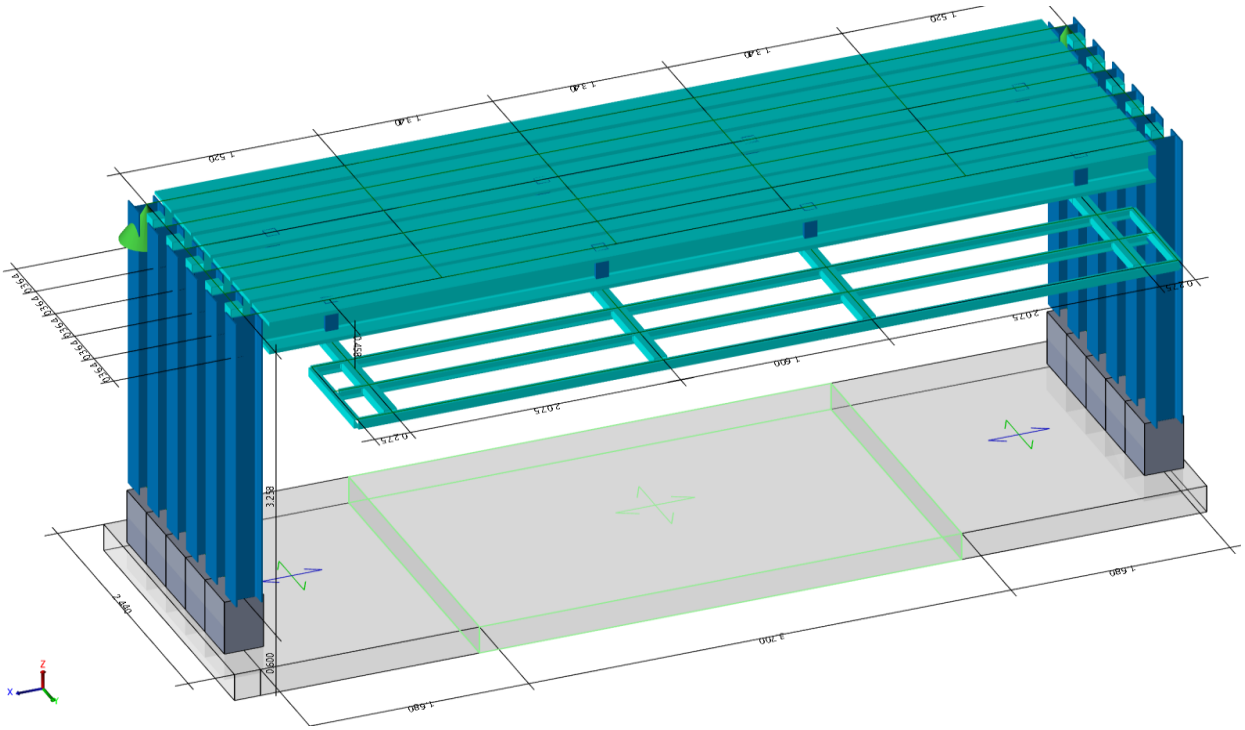
A-5.2 MODEL SUMMARY

A-5.2.1 INTRODUCTION

Opening Types 1 and 2 are modelled using Tekla Structural Designer. The following section details the parameters used in these models in order to demonstrate that they accurately represent the geometry and loadings presented in Sections A-5.0 and A-5.1.

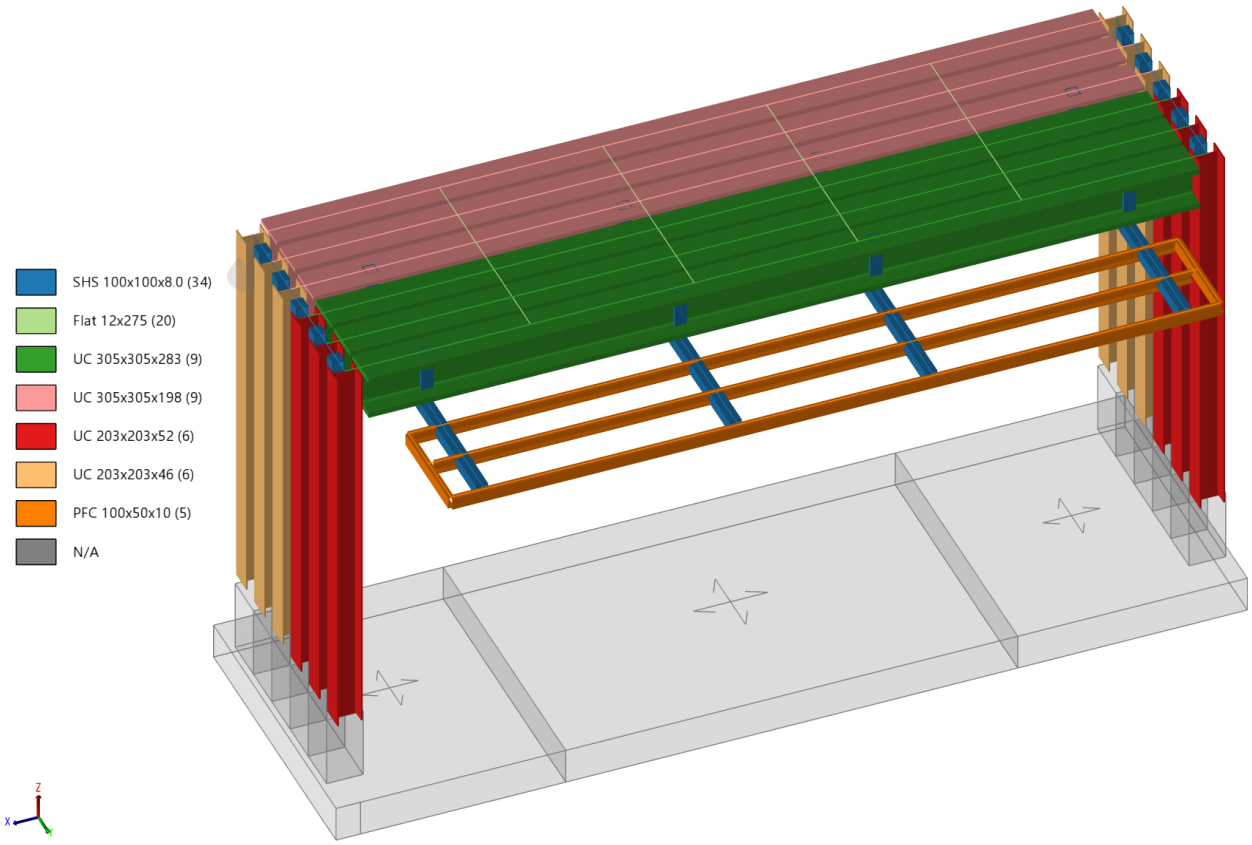
A-5.2.2 OPENING TYPE 1

GEOMETRY

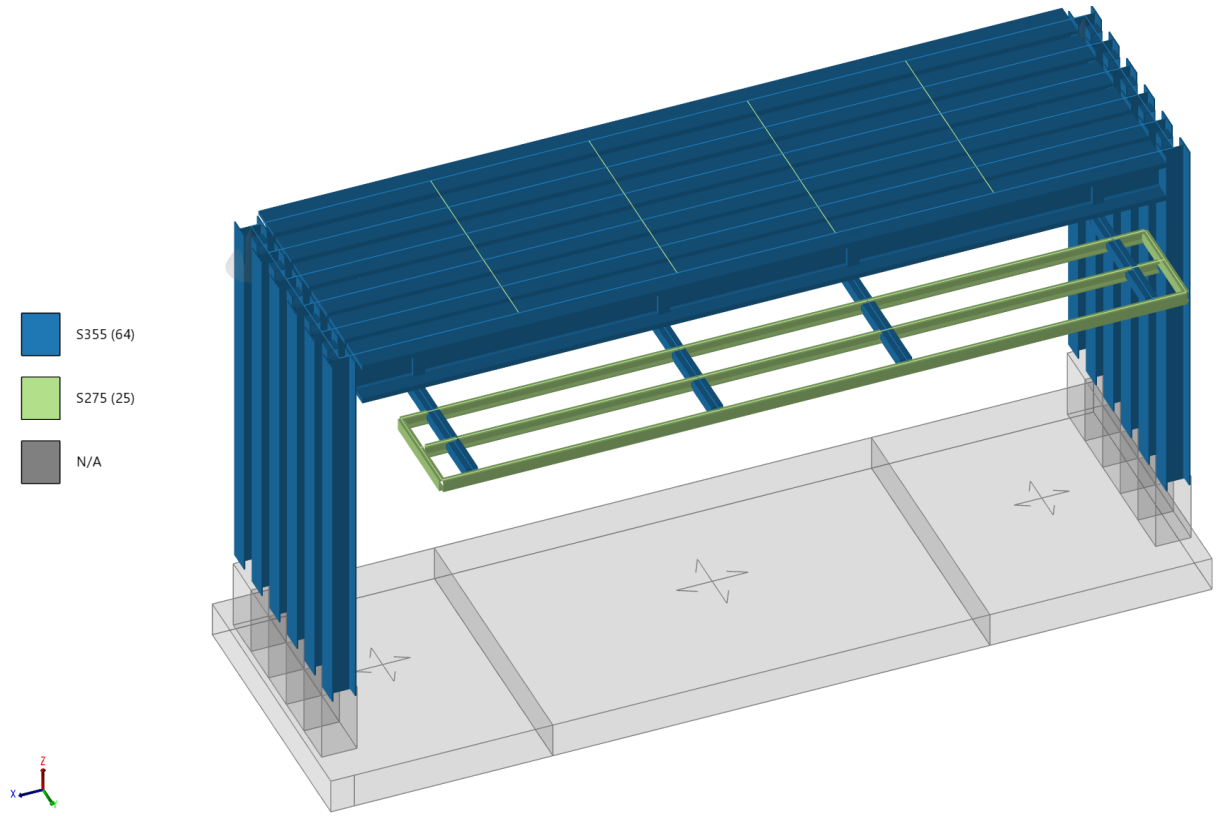


A-5.2.2 OPENING TYPE 1 (CONT.)

SECTIONS

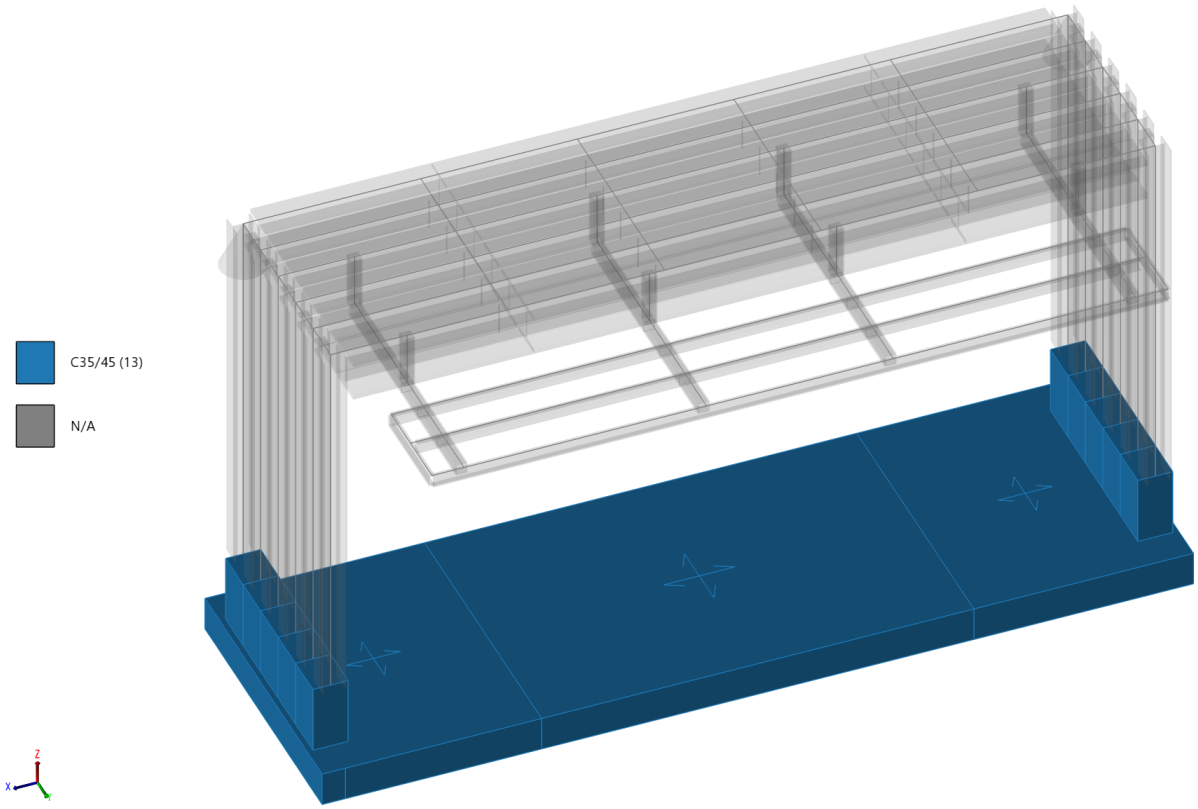


MATERIALS

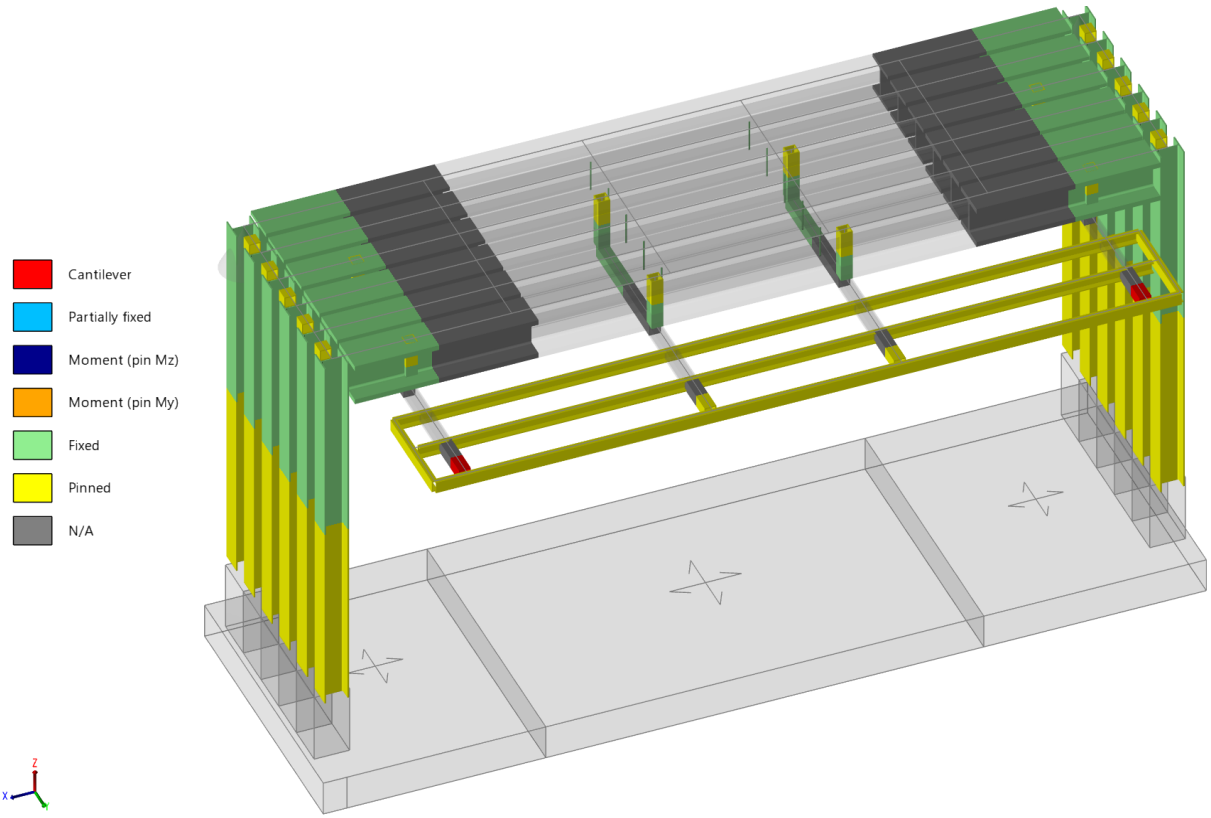


A-5.2.2 OPENING TYPE 1 (CONT.)

MATERIALS (CONT.)



FIXITY



A-5.2.2 OPENING TYPE 1 (CONT.)

STIFFNESS MODIFIERS

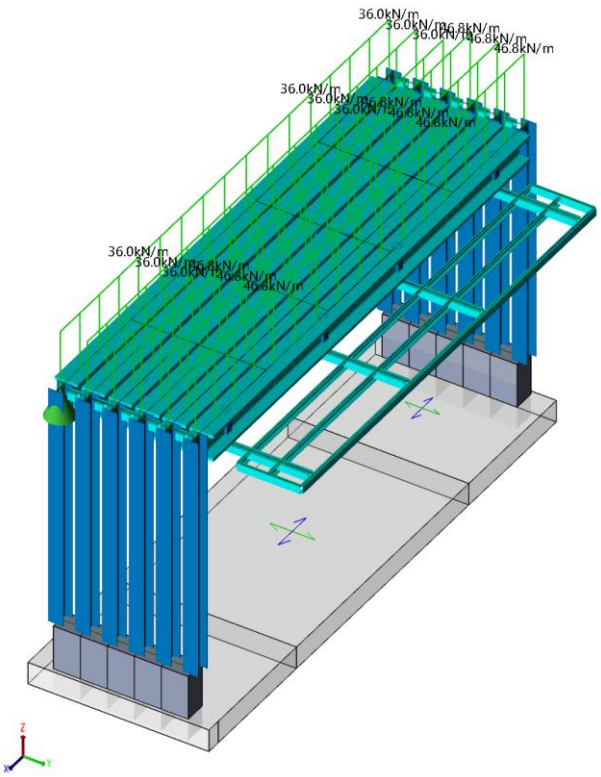
Analysis Settings

- 1st Order Non-Linear
- 2nd Order Non-Linear
- 1st Order Modal
- 2nd Order Buckling
- 1st Order Seismic
- Iterative Cracked Section Analysis
- Modification Factors
  - Concrete
    - Building Analysis
    - Grillage chase-down
    - FE chase-down
    - Modal Analysis (and RSA)
    - Slab Deflection
    - Staged Construction

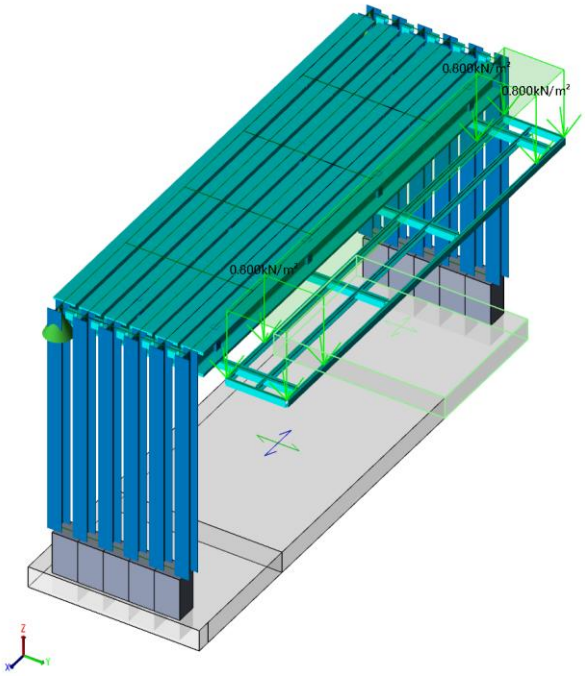
Element Type	E	G	I torsion	I major	I minor	Area	A || minor	A || major	t
Mid Pier Wall Cracked	0.20	0.20	1.00	1.00	1.00	1.00	1.00	1.00	
Mid Pier Wall Uncracked	0.40	0.40	1.00	1.00	1.00	1.00	1.00	1.00	
Meshed Wall Cracked	0.20	0.20							1.00
Meshed Wall Uncracked	0.40	0.40							1.00
Column Cracked	1.00	1.00	0.20	0.20	0.20	1.00	1.00	1.00	
Column Uncracked	1.00	1.00	0.40	0.40	0.40	1.00	1.00	1.00	
Beam Cracked	1.00	1.00	0.01	0.20	0.20	1.00	1.00	1.00	
Beam Uncracked	1.00	1.00	0.01	0.40	0.40	1.00	1.00	1.00	
Flat Slab	0.50	0.50							1.00
Foundation Mat	0.50	0.50							1.00
Slab on Beams	0.05	0.05							1.00
Precast Slab	1.00	1.00							1.00
Composite Slab	1.00	1.00							1.00

LOADS

G_K - FRAMES



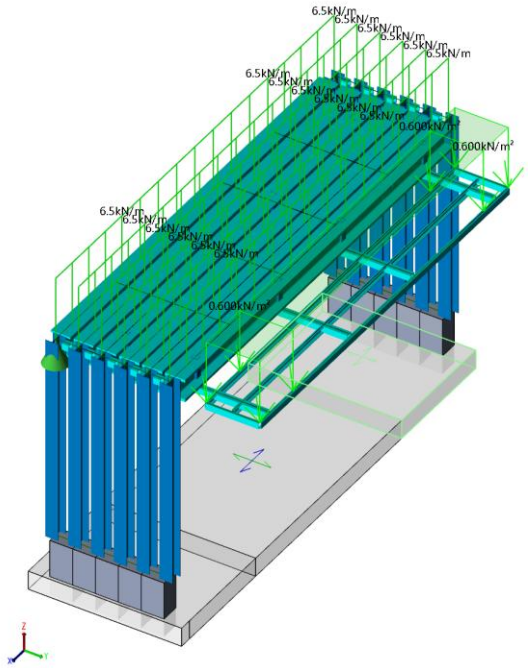
G_K - CANOPY



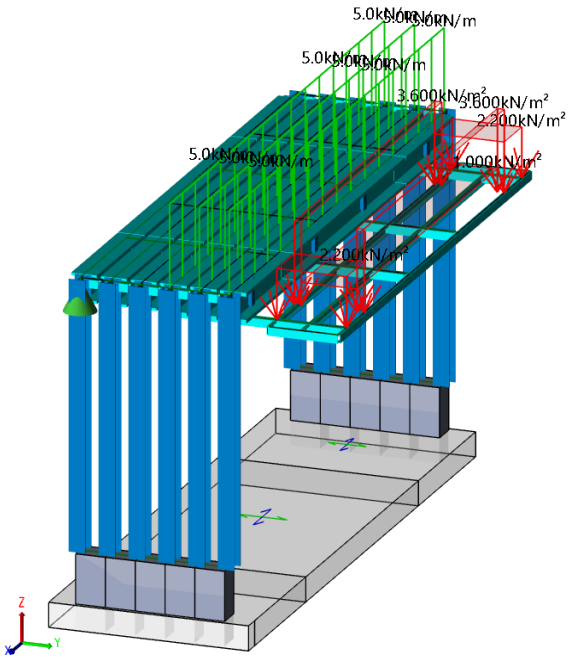
A-5.2.2 OPENING TYPE 1 (CONT.)

LOADS (CONT.)

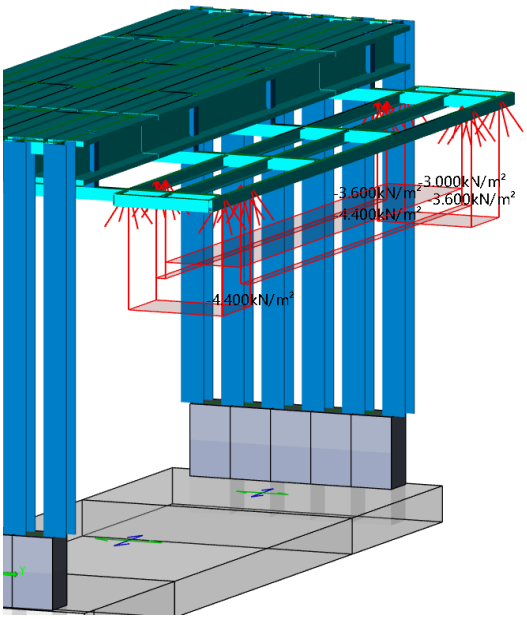
Q_k - IMPOSED LOADS



W_k - WIND

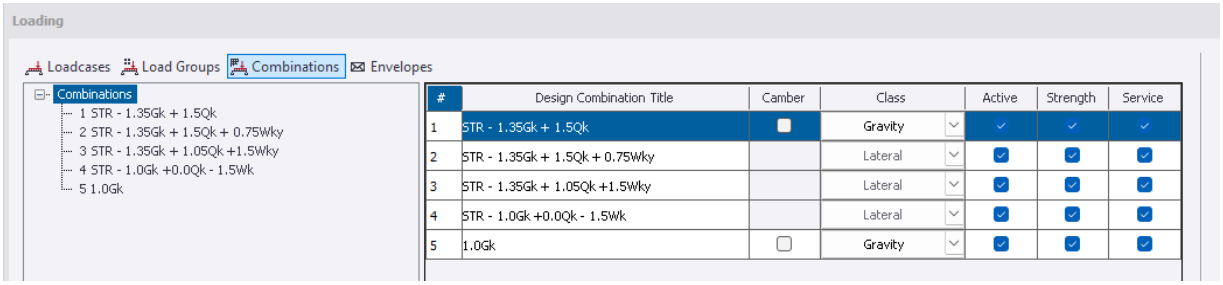


W_k - WIND UPLIFT



A-5.2.2 OPENING TYPE 1 (CONT.)

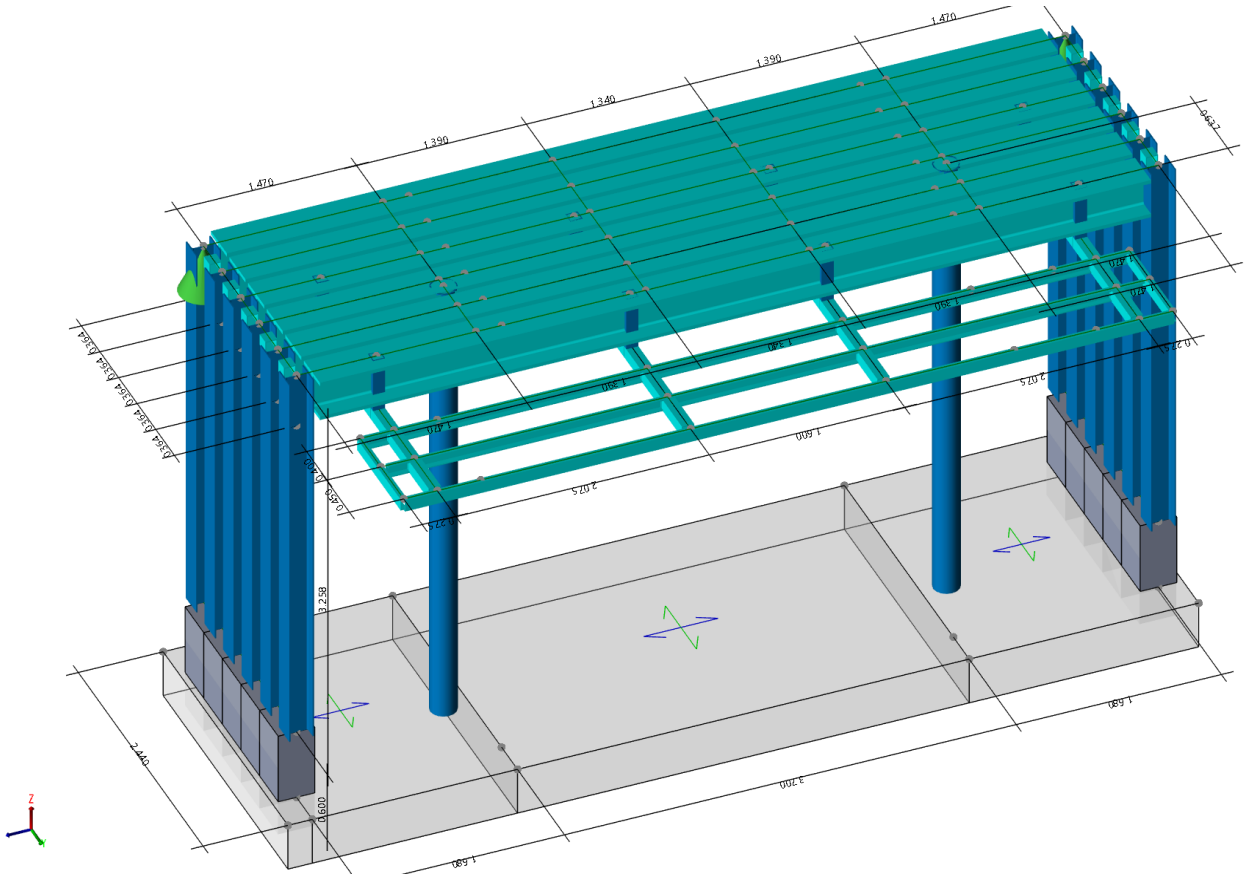
LOAD COMBINATIONS



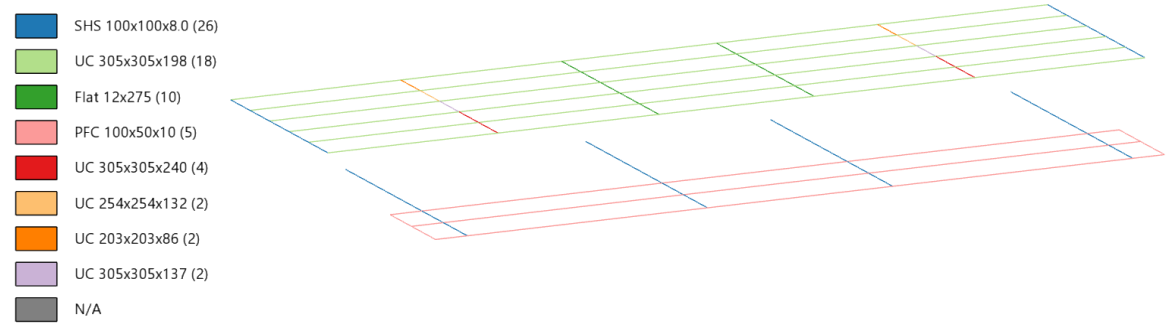
#	Design Combination Title	Camber	Class	Active	Strength	Service
1	STR - 1.35Gk + 1.5Qk	☐	Gravity	☑	☑	☑
2	STR - 1.35Gk + 1.5Qk + 0.75Wky		Lateral	☑	☑	☑
3	STR - 1.35Gk + 1.05Qk +1.5Wky		Lateral	☑	☑	☑
4	STR - 1.0Gk +0.0Qk - 1.5Wk		Lateral	☑	☑	☑
5	1.0Gk	☐	Gravity	☑	☑	☑

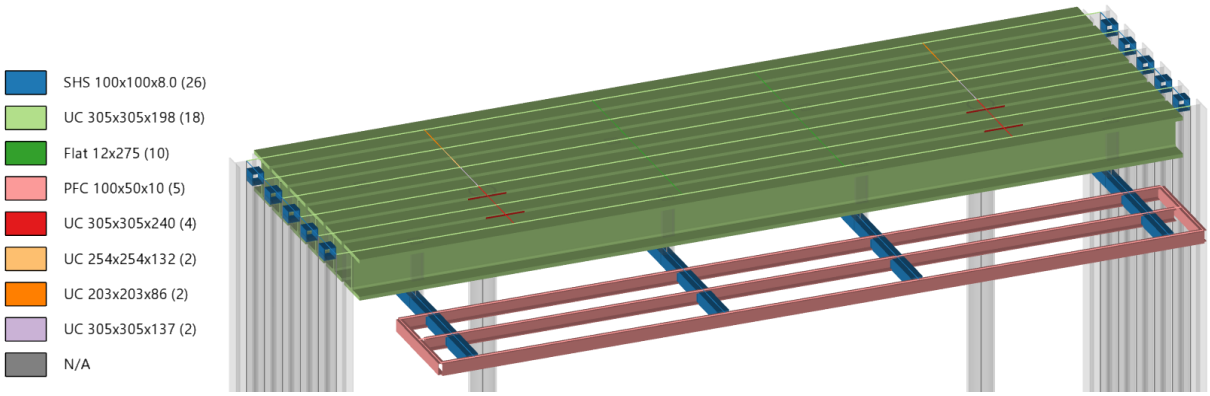
A-5.2.3 OPENING TYPE 2

GEOMETRY



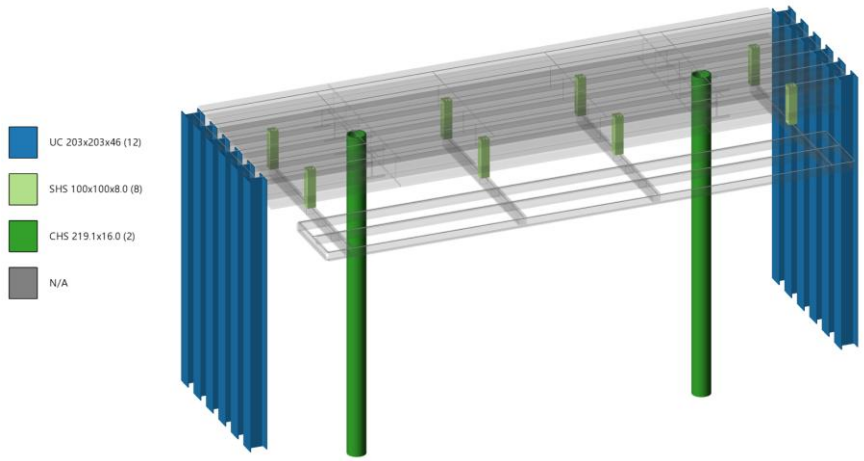
SECTIONS - BEAMS



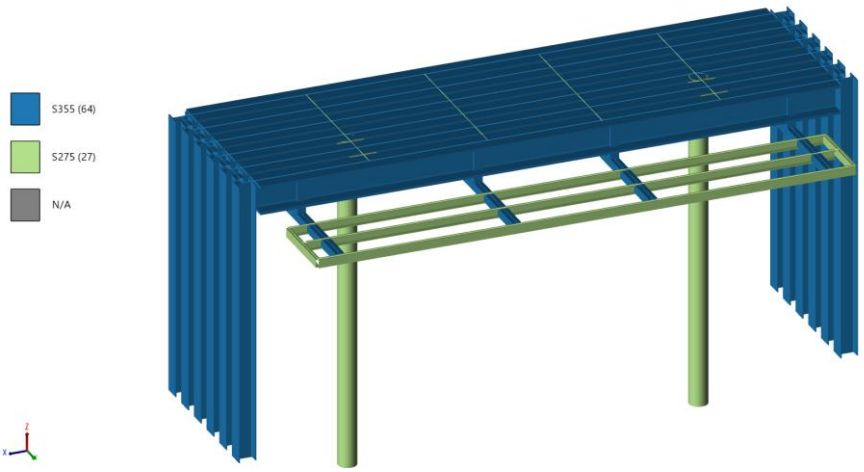


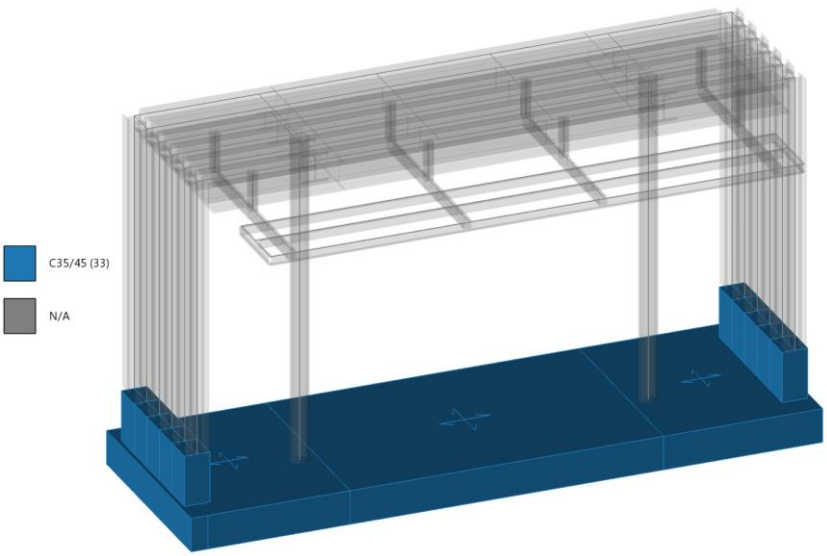
A-5.2.3 OPENING TYPE 2 (CONT.)

SECTIONS - COLUMNS



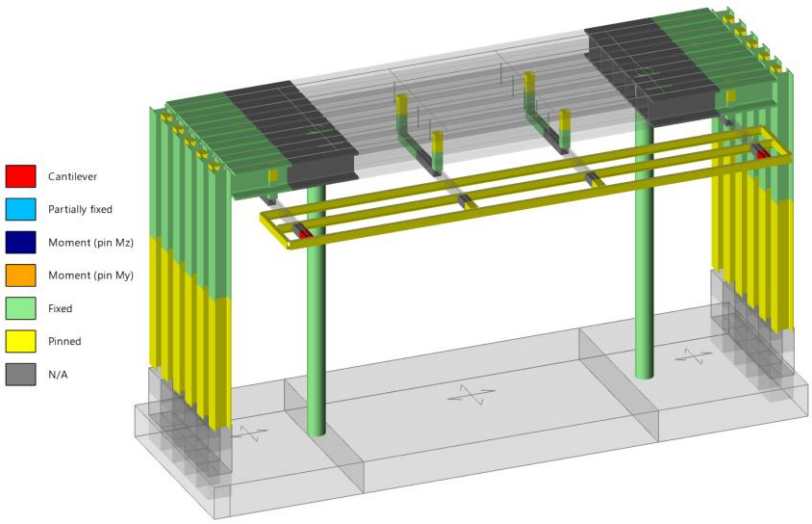
MATERIALS





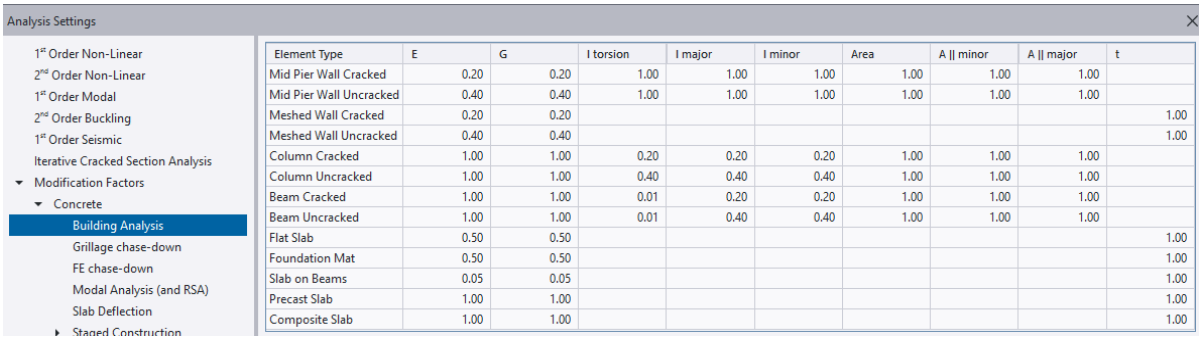
A-5.2.3 OPENING TYPE 2 (CONT.)

FIXITY



A-5.2.3 OPENING TYPE 2 (CONT.)

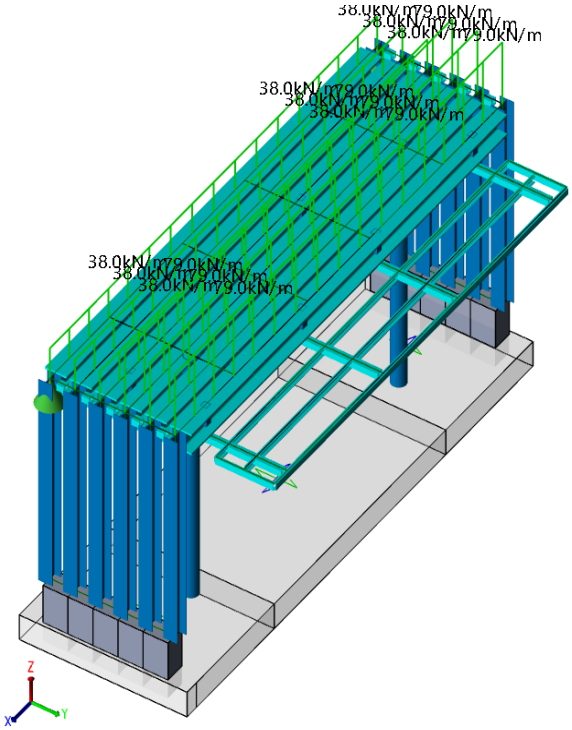
STIFFNESS MODIFIERS



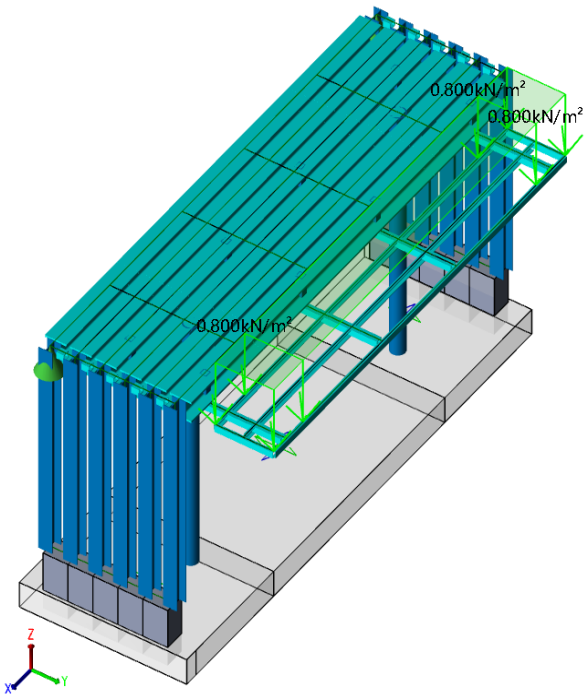
Element Type	E	G	I torsion	I major	I minor	Area	A || minor	A || major	t
Mid Pier Wall Cracked	0.20	0.20	1.00	1.00	1.00	1.00	1.00	1.00	
Mid Pier Wall Uncracked	0.40	0.40	1.00	1.00	1.00	1.00	1.00	1.00	
Meshed Wall Cracked	0.20	0.20							1.00
Meshed Wall Uncracked	0.40	0.40							1.00
Column Cracked	1.00	1.00	0.20	0.20	0.20	1.00	1.00	1.00	
Column Uncracked	1.00	1.00	0.40	0.40	0.40	1.00	1.00	1.00	
Beam Cracked	1.00	1.00	0.01	0.20	0.20	1.00	1.00	1.00	
Beam Uncracked	1.00	1.00	0.01	0.40	0.40	1.00	1.00	1.00	
Flat Slab	0.50	0.50							1.00
Foundation Mat	0.50	0.50							1.00
Slab on Beams	0.05	0.05							1.00
Precast Slab	1.00	1.00							1.00
Composite Slab	1.00	1.00							1.00

LOADS

G_K - FRAMES



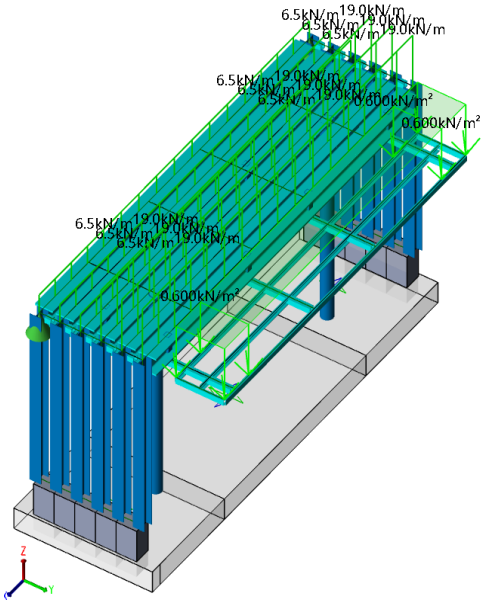
G_K - CANOPY



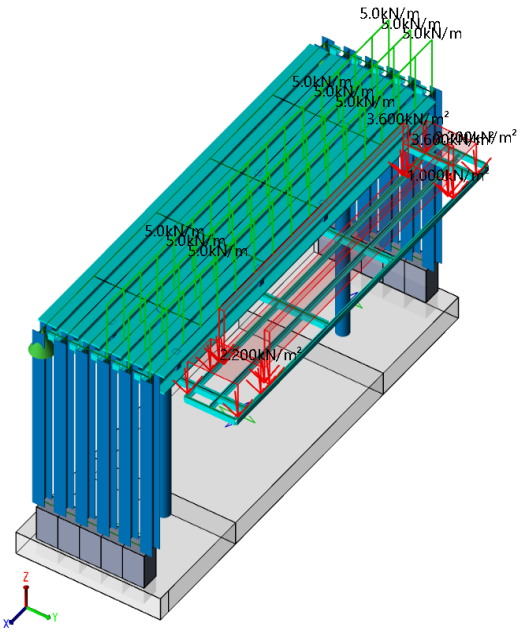
A-5.2.3 OPENING TYPE 2 (CONT.)

LOADS (CONT.)

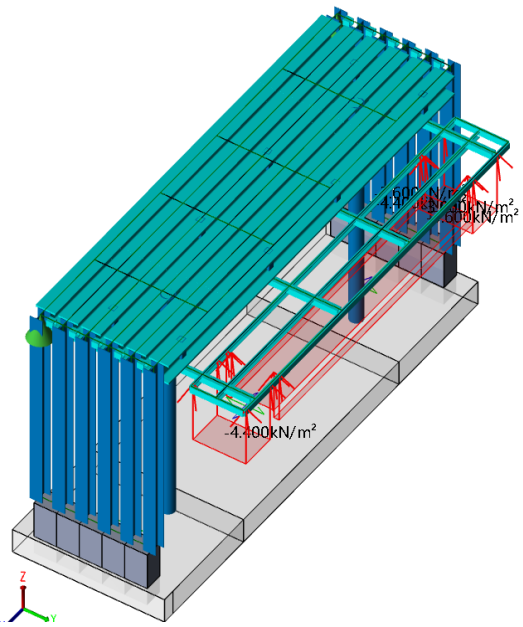
Q_k - IMPOSED LOADS



W_k - WIND

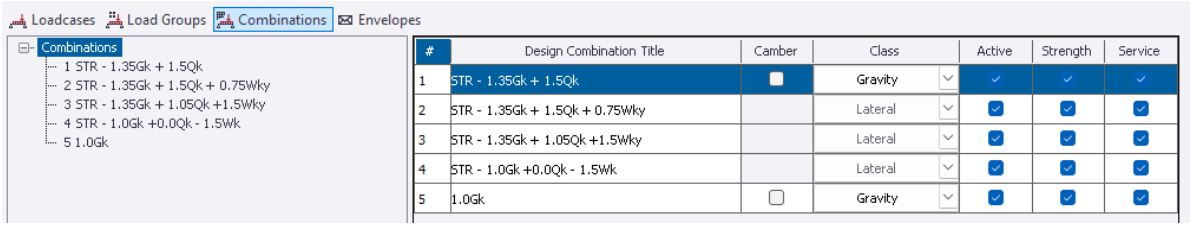


W_k - WIND UPLIFT



A-5.2.3 OPENING TYPE 2 (CONT.)

LOAD COMBINATIONS



#	Design Combination Title	Camber	Class	Active	Strength	Service
1	STR - 1.35Gk + 1.5Qk	☐	Gravity	☑	☑	☑
2	STR - 1.35Gk + 1.5Qk + 0.75Wky		Lateral	☑	☑	☑
3	STR - 1.35Gk + 1.05Qk +1.5Wky		Lateral	☑	☑	☑
4	STR - 1.0Gk +0.0Qk - 1.5Wk		Lateral	☑	☑	☑
5	1.0Gk	☐	Gravity	☑	☑	☑

A-5.3 SUBSTRUCTURE

A-5.3.0 INTRODUCTION

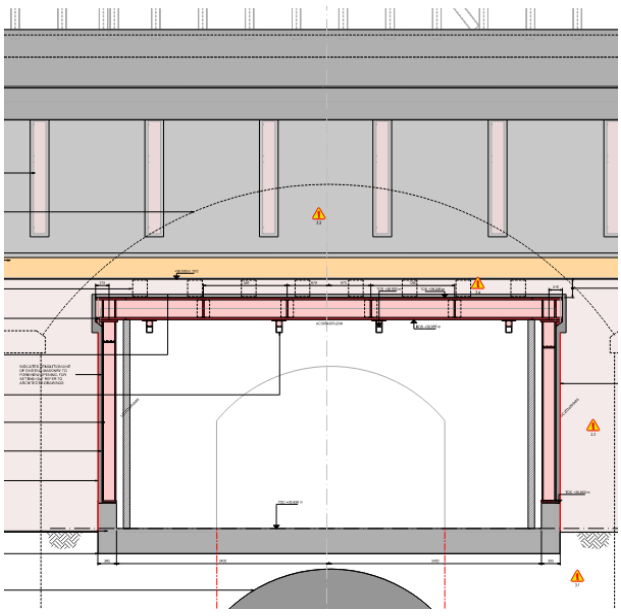
The steel frames will create an alternate load path to transfer loads from the structure above the new openings down into upstands and subsequently into an RC spreader beam foundation below. This calculation pack will demonstrate that these elements are adequate to transfer the required loads to the ground for both Opening Type 1 and Opening Type 2.

A-5.3.1 OPENING TYPE 1 DESIGN PHILOSOPHY

The steel frame bears onto a 300mm thick upstand wall, which distributes the loads into the spreader beam below. The spreader beam is a 300mm thick RC slab.

The upstand wall is designed using a mixture of Tekla Structural Designer and a hand calculation. The walls are modelled as 2D meshed plate elements which span between the base of each column and distribute the loads from the steel columns above into the spreader beam in bending.

The spreader beam is designed as three separate 2D meshed plate elements. This enables the outer two panels to be set with a non-linear spring stiffness of 4,000,000 kN/m^2/m to reflect the spreader beam bearing onto a solid masonry pier below. The middle section is is set with a spring stiffness of zero, since it sits above the relieving arch as shown below.



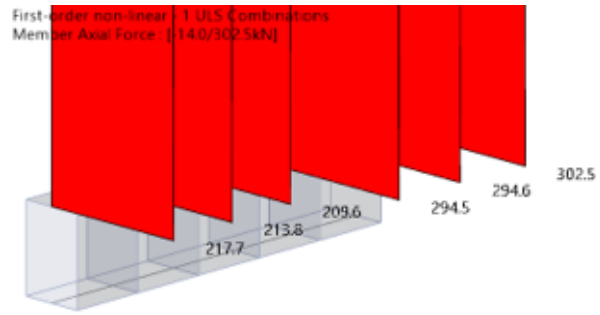
Upstand
2195 x 420 x 300THK
Reinforcement Summary:
B12 Bars:175c/c
B12 Lacers

Spreader Beam
300 THK RC Slab
Reinforcement Summary:
B12 Main Bars @175c/c B2/T2
B12 U-Bars @ 175 B2/T2
B12 U-Bars @ 200 B1/T1

A-5.3.2 OPENING TYPE 1 - UPSTAND ANALYSIS AND DESIGN

The upstand will conservatively be design based on an equivalent 300mm deep x 364mm long section under the most highly loaded column. This section is treated as a cantilever beam with fixed base for moment and shear checks.

From the analysis model, the following worst case loadings are applied:



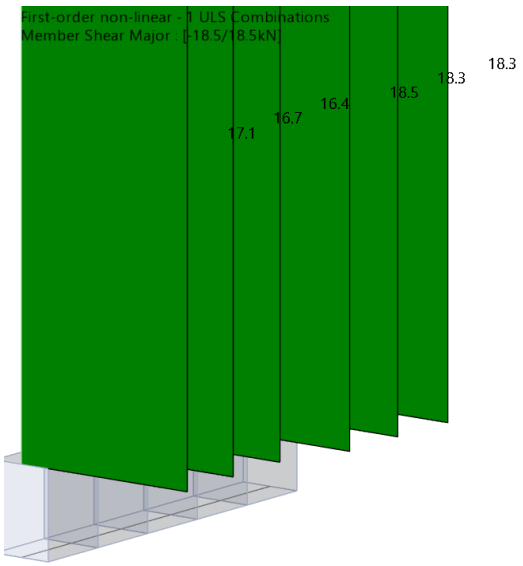
Axial

$$\theta_{Ed} = \frac{305000}{300 \times 364}$$

N_{Ed} = **305 kN**

θ_{Ed} = **2.8 N/mm²**

As the axial stress is beneficial but less than $0.1f_{ck}$, conservatively ignore in design of upstand.



Shear

V_{Ed} = **20 kN**

d = **240 mm**

Effective depth,
d = 300 - 40 - 12 - 16/2

$$v_{Ed} = \frac{20 \times 10^3}{364 \times 0.9 \times 240}$$

v_{Ed} = **0.25 N/mm²**

For RC35/45 concrete and 0.25% reinforcement, with no additional shear reinforcement:

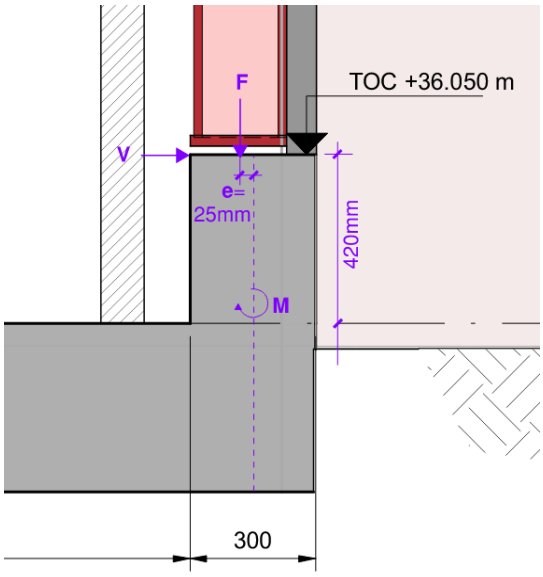
$v_{Rd,c}$ = **0.53 N/mm²**

U.R. = **0.48**

0.25 % x 364 x 240 / 0.36 = 607 mm²/m

Adopting B12 - 175 c/c, no shear reinforcement required

$A_{s,prov}$ = **646 mm²/m**



Bending

Conservatively assume eccentricity of 50mm

M_{Ed} = 20 kN x 0.42 m

\- 305 kN x 0.05 m = **-6.9 kNm**

Conservatively assume z = 0.9d

$$A_s = \frac{M}{0.95 f_y\, 0.9d}$$

$$= \frac{7.0}{0.95 \times 500 \times 0.9 \times 0.25}$$

= **65 mm²**

$A_{s,min}$ = 0.17%

= 0.0017 x 300 x 420 = **214 mm²**

$A_{s,req}$ = 214 / 0.36 = **588 mm²/m**

ADOPT B12 - 175c/c

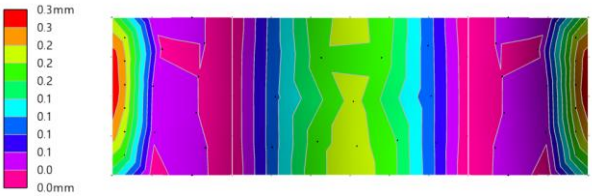
$A_{s,prov}$ = **646 mm²/m**

A-5.3.3 OPENING TYPE 1 - SPREADER BEAM ANALYSIS AND DESIGN

The following worst case forces are extracted from the Tekla Structural Designer model:

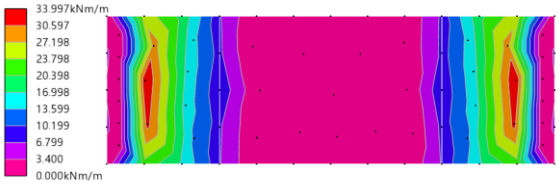
DEFLECTION

First-order non-linear - 1 STR - 1.35Gk + 1.5Qk
Member Moment Major : No results
Panel Deflection Total : [0.0/0.3mm]

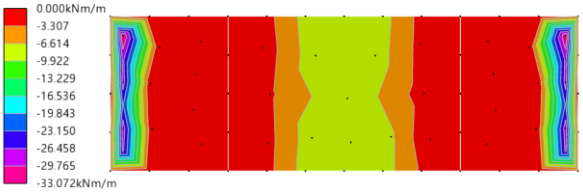


MOMENTS

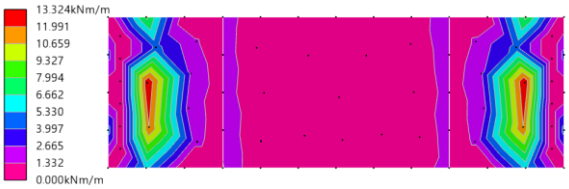
First-order non-linear - 1 STR - 1.35Gk + 1.5Qk
Panel Design Mdx top (Wood Armer) : [0.000/33.997kNm/m]



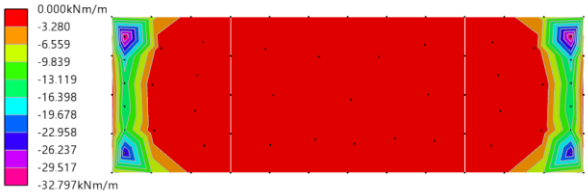
First-order non-linear - 1 STR - 1.35Gk + 1.5Qk
Panel Design Mdx bottom (Wood Armer) : [-33.072/0.000kNm/m]



First-order non-linear - 1 STR - 1.35Gk + 1.5Qk
Panel Design Mdy top (Wood Armer) : [0.000/13.324kNm/m]

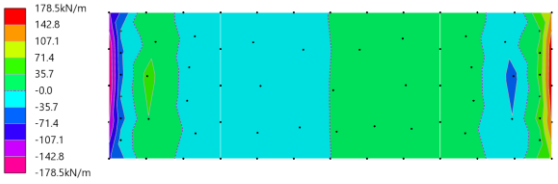


First-order non-linear - 1 STR - 1.35Gk + 1.5Qk
Panel Design Mdy bottom (Wood Armer) : [-32.797/0.000kNm/m]



SHEAR

First-order non-linear - 2 STR - 1.35Gk + 1.5Qk + 0.75Wky
Panel Fxz (z axis) : [-178.5/178.5kN/m]



A-5.3.3 OPENING TYPE 1 - SPREADER BEAM ANALYSIS AND DESIGN (CONT.)

Properties

f_{ck} = **35** **N/mm²**
Depth = **300** **mm**
top cover = **40** **mm**
side cover = **40** **mm**
bottom cover = **40** **mm**
C_{dev} = **10**

Flexural Check

(6.1, How To Design Concrete Structures Using Eurocode 2 Figure 2)

A consistent mat of reinforcement is provided top and bottom based on the worst case moment determined from the strip results in the X direction, with additional rebar provided where the moment capacity exceeds the resistance of the provided reinforcement. The required reinforcement is as follows

Consistent Mat

Design Moment, M_{Ed} (from Tekla) = 35 kNm/m

Depth to bottom reinforcement in B2 layer, d = 300 - 50 - 12 - 12 /2 → **d = 232 mm**

$K = M/(bd^2 f_{ck})$
$= 35 \times 10^6 / (1000 \times 232^2 \times 35)$ → **K = 0.019**

K< K' → 0.019 < 0.168 **Compression reinforcement not required.**

Lever arm, $l_a = 0.5(1+\sqrt{(1-3.53k)})$
$= 0.5 (1+\sqrt{(1-3.53 \times 0.019)})$
$= 0.98 < 0.95\ d$
$z = 0.95\ d$ → **z = 220 mm**

Required Reinforcement, $A_{s,req} = M/(f_{yd} z)$
$= 35 \times 1000 / (220 \times 0.435)$ → **$A_{s,req}$ = 365 mm²/m**

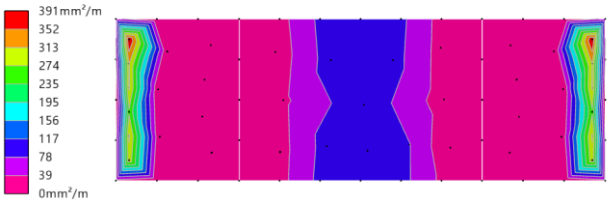
$A_{s,min} = 0.0017 \times b_t \times d$ (eq. 9.1N)
$= 0.0017 \times 1000 \times 232$ → **$A_{s,min}$ = 394 mm²/m**

Provide B 12 at 175 centres = 646 mm²/m
U.R. = 0.56

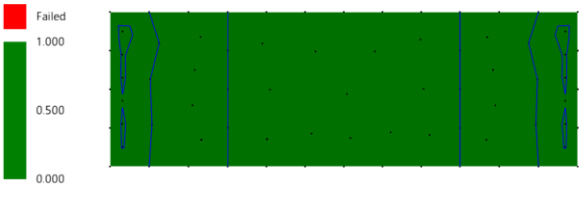
The above reinforcement is provided as a consistent mat to the top and the bottom of the spreader beam.

TEKLA VERIFICATION

First-order non-linear - 1 ULS Combinations
Panel As (Required) X Bottom : [0/391mm²/m]



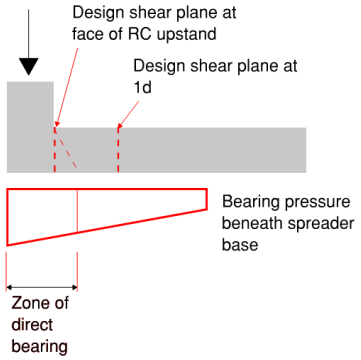
First-order non-linear - 1 ULS Combinations
Panel As (Required / Provided) X Bottom : [0.000/0.691]

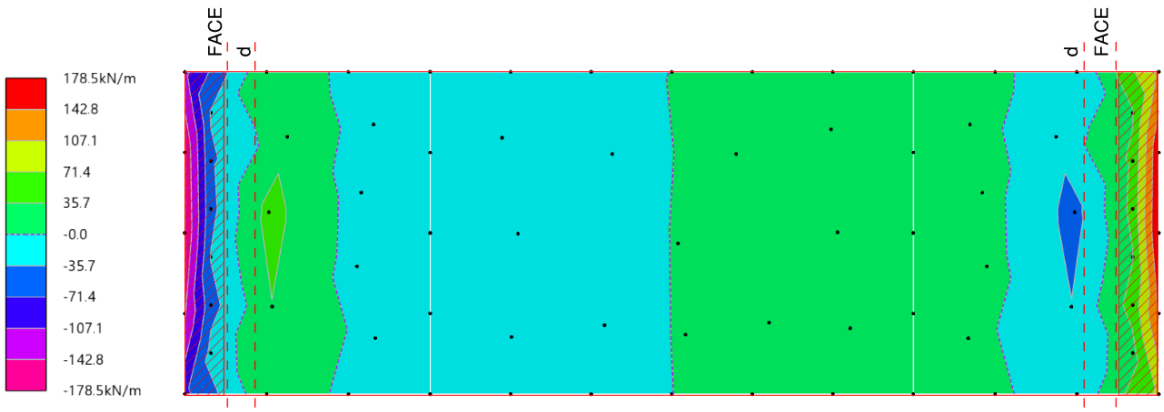


A-5.3.3 OPENING TYPE 1 - SPREADER BEAM ANALYSIS AND DESIGN (CONT.)

SHEAR CHECK

A large proportion of the loads from the columns are transfered through the upstand and spreader footing in direct bearing with a circa 60 degree spread. To ensure that the base has sufficient shear capacity checks will be undertaken at the face of the upstand and at d from support





UPSTAND FACE CHECK

The maximum shear force at the face is — **V_{ed} = 110 kN/m**

The maximum shear stress is therefore $v_{ED} = V_{ED} / (0.9 b_w z)$

$$= \frac{110}{0.9 \times 1000 \times 232}$$

v_{ed} = 0.53 N/mm²

For RC35/45 concrete with a cot θ = 2.5 the maximum shear resistance is — **$v_{Rd,max\ cot\ \theta = 2.5}$ = 4.15 N/mm²**

(U.R. = 0.13)

Therefore face shear acceptable

SHEAR CHECK AT d

The maximum shear force at d from the face of the upstand is — **V_{ed} = 75 kN**

The maximum shear stress is therefore $v_{ED} = V_{ED} / (0.9 b_w z)$

$$= \frac{75}{0.9 \times 1000 \times 240}$$

v_{ed} = 0.35 N/mm²

For members without shear reinforcement, the maximum shear resistance is — **v_{Rd} = 0.51 N/mm²**

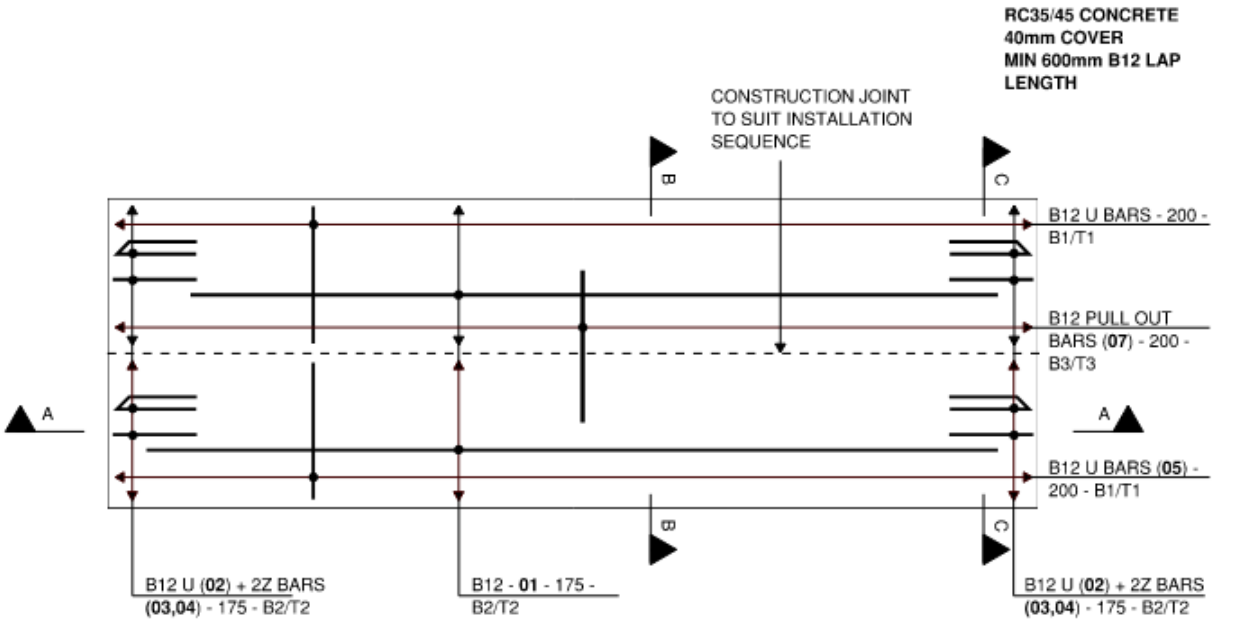
$d = 240$ mm

$\rho_1 = As/(bd) = 646/(1000 \times 240) = 0.27\%$

(U.R. = 0.68)

Therefore face shear acceptable

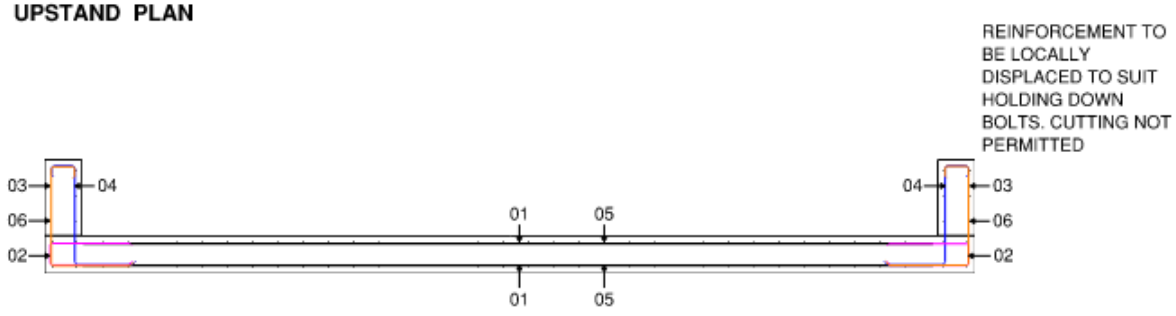
A-5.3.4 OPENING TYPE 1- REINFORCEMENT INTENT



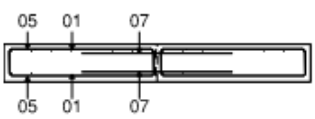
BASE PLAN



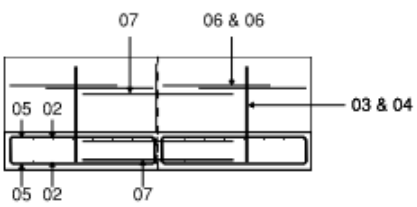
UPSTAND PLAN



SECTION A-A



SECTION B-B



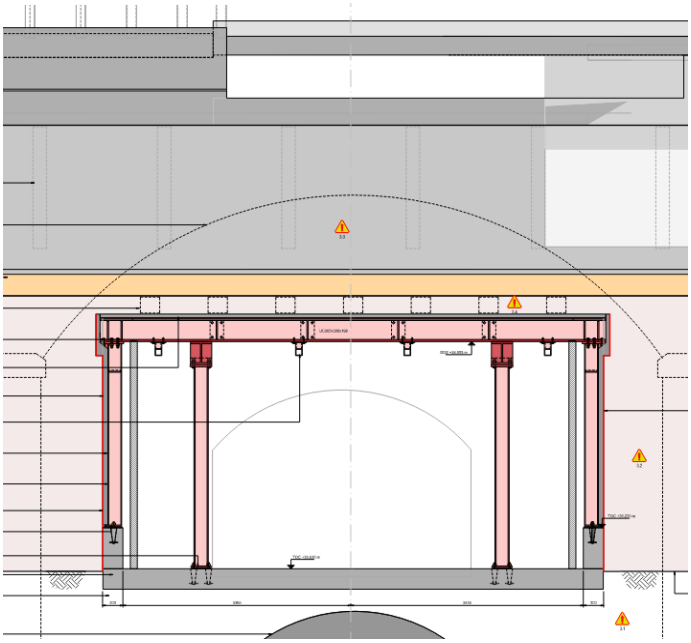
SECTION C-C

A-5.3.5 OPENING TYPE 2 DESIGN PHILOSOPHY

As with Opening Type 1, the steel frame bears onto a 300mm thick upstand wall, which distributes the loads into the spreader beam below. Opening Type 2 also supports loads from the existing RC deck and steel grillage area above. These loads are transferred into a 400mm thick RC spreader beam by intermediate steel posts.

The upstand wall is designed using a mixture of Tekla Structural Designer and a hand calculation. The walls are modelled as 2D meshed plate elements which span between the base of each column and distribute the loads from the steel columns above into the spreader beam in bending.

The spreader beam is designed as three separate 2D meshed plate elements. This enables the outer two panels to be set with a non-linear spring stiffness of 4,000,000 kN/m^2/m to reflect the spreader beam bearing onto a solid masonry pier below. The middle section is is set with a spring stiffness of zero, since it sits above the relieving arch as shown below.



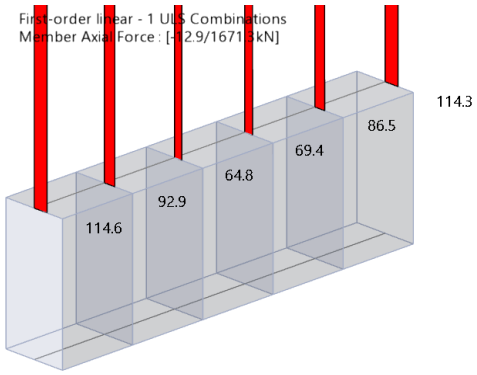
Upstand
2195 x 420 x 300THK
Reinforcement Summary:
B12 Bars:175c/c
B12 Lacers

Spreader Beam
400 THK RC Slab
Reinforcement Summary:
B12 Main Bars @175c/c - T2
B20 Main Bars @175c/c -B2
B12 U-Bars @175c/c - T1
B16 U-Bars @175c/c - B1

A-5.3.6 OPENING TYPE 2 - UPSTAND ANALYSIS AND DESIGN

The upstand will conservatively be design based on an equivalent 300mm deep x 364mm long section under the most highly loaded column. This section is treated as a cantilever beam with fixed base for moment and shear checks.

From the analysis model, the following worst case loadings are applied:



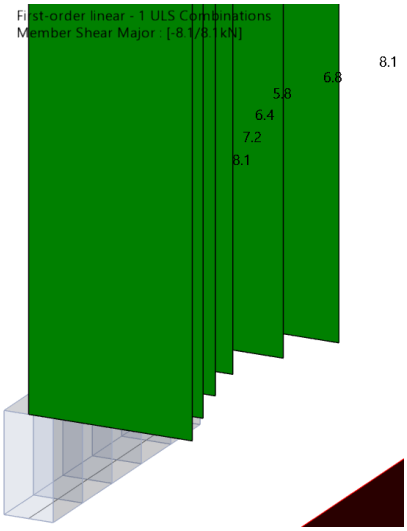
Axial

$$\theta_{Ed} = \frac{120000}{300 \times 364}$$

N_{Ed} = **120 kN**

θ_{Ed} = **1.1 N/mm²**

As the axial stress is beneficial but less than $0.1f_{ck}$, conservatively ignore in design of upstand.



Shear

V_{Ed} = **10 kN**

d = **240 mm**

Effective depth,
d = 300 - 40 - 12 - 16/2

$$v_{Ed} = \frac{10 \times 10^{3}}{364 \times 0.9 \times 240}$$

v_{Ed} = **0.13 N/mm²**

For RC35/45 concrete and 0.25% reinforcement, with no additional shear reinforcement:

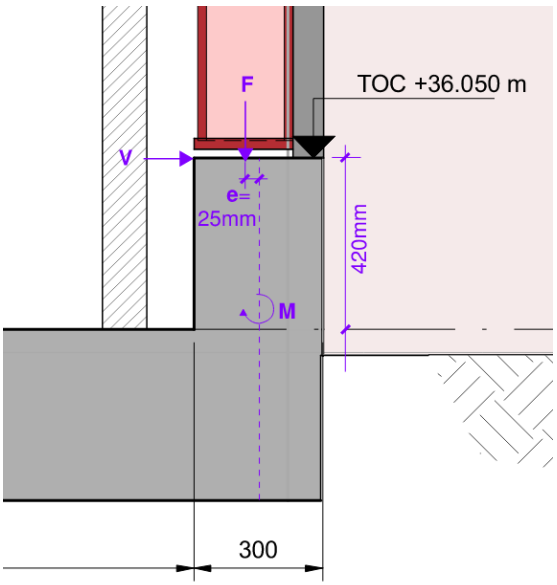
$v_{Rd,c}$ = **0.53 N/mm²**

U.R. = **0.24**

0.25 % x 364 x 240 / 0.36 = 607 mm²/m

Adopting B12 - 175 c/c, no shear reinforcement required

$A_{s,prov}$ = **646 mm²/m**



Bending

Conservatively assume eccentricity of 50mm

M_{Ed} = 10 kN x 0.42 m

\- 120 kN x 0.05 m = **-1.8 kNm**

Conservatively assume z = 0.9d

$$A_s = \frac{M}{0.95 f_y 0.9d}$$

$$= \frac{2.0}{0.95 \times 500 \times 0.9 \times 0.25}$$

= **19 mm²**

$A_{s,min}$ = 0.17%

= 0.0017 x 300 x 420 = **214 mm²**

$A_{s,req}$ = 214 / 0.36 = **588 mm²/m**

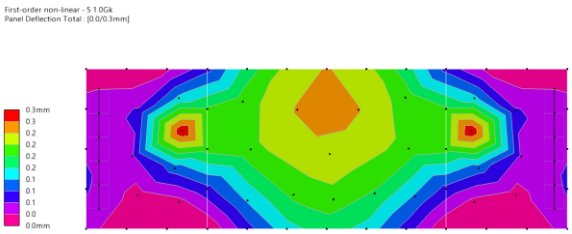
ADOPT B12 - 175c/c

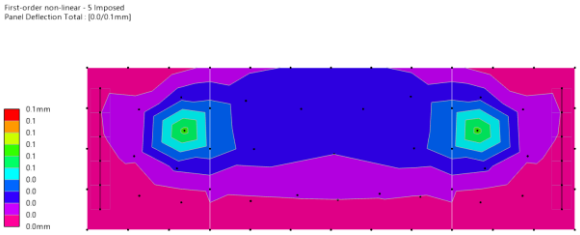
$A_{s,prov}$ = **646 mm²/m**

A-5.3.7 OPENING TYPE 2 - SPREADER BEAM ANALYSIS AND DESIGN

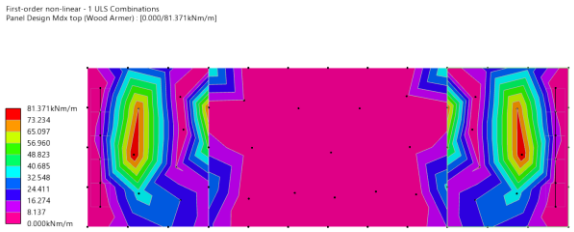
The following worst case forces are extracted from the Tekla Structural Designer model:

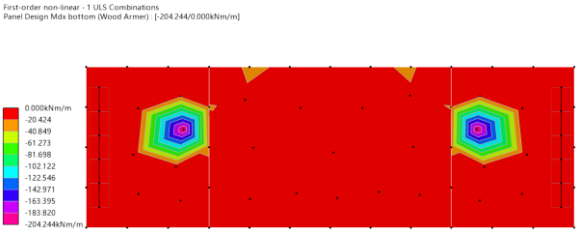
DEFLECTION

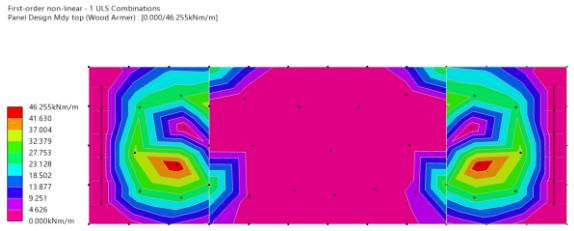


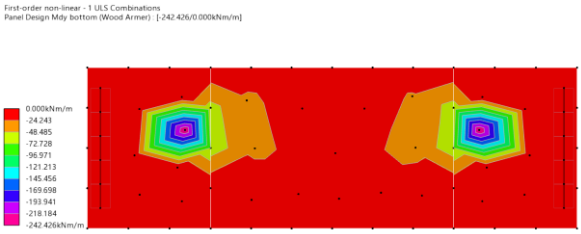


MOMENTS

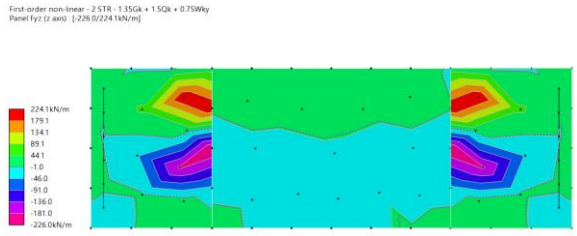








SHEAR



A-5.3.8 OPENING TYPE 2 - SPREADER BEAM ANALYSIS AND DESIGN (CONT.)

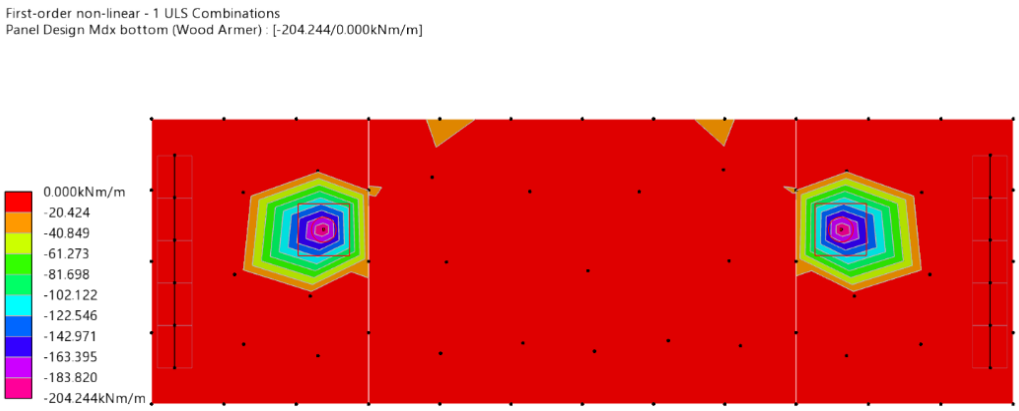
Properties

f_{ck} = **35** **N/mm²**
Depth = **400** **mm**
top cover = **40** **mm**
side cover = **40** **mm**
bottom cover = **40** **mm**
C_{dev} = **10**

Flexural Check

(6.1, How To Design Concrete Structures Using Eurocode 2 Figure 2)

A consistent mat of reinforcement is provided top and bottom based on the worst case moment determined from the strip results in the X direction. The moment plot in the y-direction for the bottom of the slab is the worst case, showing a peak moment of 205kNm/m. The peak moment occurs where the column is modelled, due to Tekla modelling the column as a stick element. In reality, the 450x450 base plate would provide additional stiffness and the peak moment would also be redistributed over a distance of L/4 between columns. Conservatively take moment at face of baseplates as shown below.



Consistent Mat

Design Moment, M_{Ed} (from Tekla) = 150 kNm/m

Depth to bottom reinforcement in B2 layer, d = 400 - 50 - 20 - 20 /2 **d = 320 mm**

$K = M/(bd^2f_{ck})$
$= 150 \times 10^6 / (1000 \times 320^2 \times 35)$ **K = 0.042**

K< K' → 0.042 < 0.168 **Compression reinforcement not required.**

Lever arm, $l_a = 0.5(1+\sqrt{(1-3.53k)})$
$= 0.5 (1+\sqrt{(1-3.53 \times 0.042)})$
$= 0.96 < 0.95\ d$
$z = 0.95\ d$ **z = 304 mm**

Required Reinforcement, $A_{s,req} = M/(f_{yd}z)$
$= 150 \times 1000 / (304 \times 0.435)$ **$A_{s,req}$ = 1134 mm²/m**

$A_{s,min} = 0.0017 \times b_t \times d$ (eq. 9.1N)
$= 0.0017 \times 1000 \times 320$ **$A_{s,min}$ = 544 mm²/m**

Provide B 20 at 175 centres = 1795 mm²/m
(U.R. = 0.63)

The above reinforcement is provided as a consistent mat to the top and the bottom of the spreader beam.

A-5.3.8 OPENING TYPE 2 - SPREADER BEAM ANALYSIS AND DESIGN (CONT.)

Shear Check

The axial loads from the upstands above are significantly lower than for Opening Type 1. Therefore, by inspection, no additional shear reinforcement is required in at the upstand interface.

The two circular column sections bear onto the spreader beam using 450 x 450mm steel base plates.



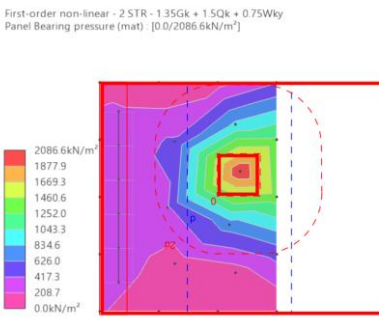
Effective column size $c_{1,2}$ = **450** mm

Effective depth, **d** = **320** mm

Shear checks at:

1. punching shear check at face of column base plate
2. punching shear check at 2d from face of column base plate (edge)
3. Beam shear check at 1d from face of column base plate

1. Face Shear



Based on Tekla output, for bearing pressure under the baseplate, left

Bearing pressure based on bearing area of 450 x 450mm baseplate.

Effective depth, d_x = 400 - 50 - 16 - 16/2

Effective depth, d_y = 400 - 50 - 16/2

For a rectangular column:

$$W_1 = \frac{c_1^2}{2} + c_1 c_2 + 4 c_2 d + 16 d^2 + 2\pi d c_1 \qquad (6.41)$$

where:

c_1 is the column dimension parallel to the eccentricity of the load

c_2 is the column dimension perpendicular to the eccentricity of the load

V_{Ed} = 1,750 kN

σ_{ave} = 1,700 kN/m^2

= 344 kN

$V_{ed,0}$ = 1,406 kN

M_{Ed} = 15 kNm

d_x = 320 mm

d_y = 342 mm

d = 331 mm

u_0 = 1,800 mm

k = 0.6

W_1 = 3.6

β = 1.150

$$v_{Ed} = \beta \frac{V_{Ed}}{u_0 d}$$

v_{Ed} = 2.71 N/mm^2

$f_{cd} = \alpha_{cc} f_{ck} / \gamma_c$, where $\alpha_{cc} = 1$

$u = 0.6(1-f_{ck}/250))$

$v_{Rd,max} = 0.5 u f_{cd}$

f_{cd} = 23.3 N/mm^2

u = 0.5

$v_{Rd,max}$ = 6.02

U.R. = 0.45

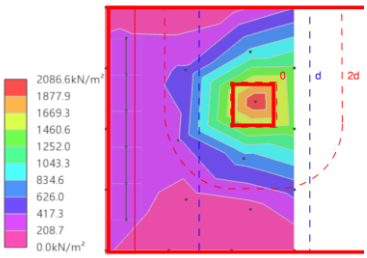
Therefore spreader beam depth ok

A-5.3.8 OPENING TYPE 2 - SPREADER BEAM ANALYSIS AND DESIGN (CONT.)

2. Shear Check at 2d



Applied Shear stress



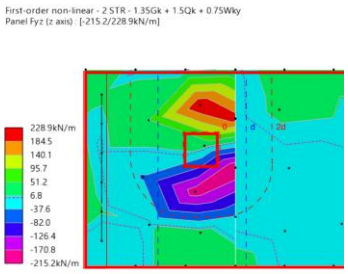
Based on Tekla output, average bearing pressure to 2d, left

Bearing pressure based on bearing area of 450 x 450mm baseplate.

Conservatively adopt 1.4

$$v_{Ed} = \beta \frac{V_{Ed}}{u_i d}$$

N_{Ed}	=	1750	kN
σ_{ave}	=	550	kN/m²
		2.57	m²
	=	1413	kN
V_{Ed}	=	337	kN
β	=	1.400	
$v_{Ed\ 1}$	**=**	**0.42**	**N/mm²**



Because the bearing area is eccentric and doesn't cover the full area, conservatively take the peak shear at the 2d perimeter based on the analysis

Worst case take linear shear of 100kNm/m

V_{Ed}	=	100	kN/m
β	=	1.400	
$v_{Ed\ 1}$	**=**	**0.42**	**N/mm²**

Conservatively adopt $v_{Ed\ 1}$ = 0.42 for design purposes

Shear Resistance

Effective depth, d_x = 400 - 50 - 16 - 16/2

Effective depth, d_y = 400 - 50 - 16/2

$$v_{Rd,c} = C_{Rd,c}k(100\rho f_{ck})^{1/3}$$

d_x	=	320	mm
d_y	=	342	mm
d	=	331	mm
u_2	=	3,430	mm
$C_{Rd,c}$	=	0.12	
k	=	1.777	

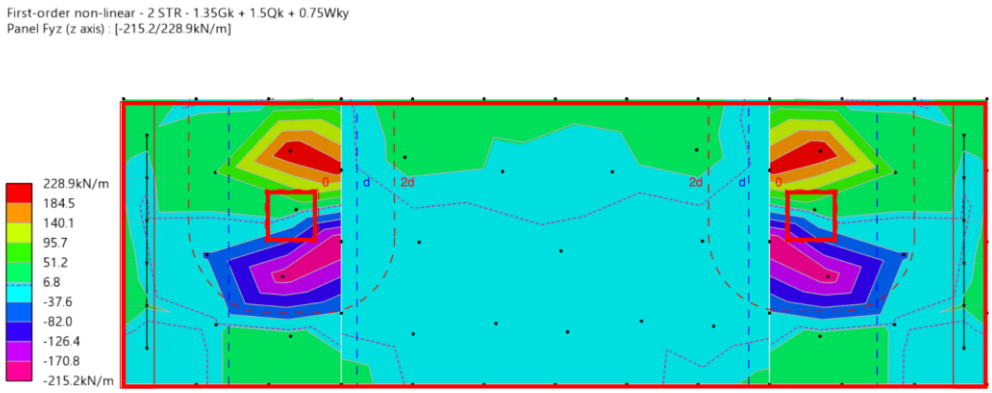
Adopting B 16 at 175 centres = 1149 mm²/m

Adopting B 20 at 175 centres = 1795 mm²/m

$\rho_{l,y}$	=	0.35	%
$\rho_{l,z}$	=	0.54	%
ρ_l	=	0.0043	
$v_{Rd,c}$	**=**	**0.53**	**N/mm²**
U.R.	**=**	**0.80**	

A-5.3.8 OPENING TYPE 2 - SPREADER BEAM ANALYSIS AND DESIGN (CONT.)

Beam Shear Check at 1d



The maximum shear force at d from the edge of the baseplate is **V_{ed} = 130 kN/m**

The maximum shear stress is therefore $v_{ED} = V_{ED} / (0.9 b_w z)$

$$= \frac{130}{0.9 \times 1000 \times 314}$$

v_{ed} = 0.46 N/mm²

For members without shear reinforcement, the maximum shear resistance is

$$v_{Rd,c} = C_{Rd,c} k (100 \rho f_{ck})^{1/3}$$

$C_{Rd,c}$ = 0.12

k = 1.798

$\rho_{l,y}$ = 0.37 %

$\rho_{l,z}$ = 0.57 %

ρ_l = 0.0046

$v_{Rd,c}$ = 0.54 N/mm²

Adopting B 16 at 175 centres = 1149 mm²/m

Adopting B 20 at 175 centres = 1795 mm²/m

(U.R. = 0.85)

Therefore beam shear acceptable

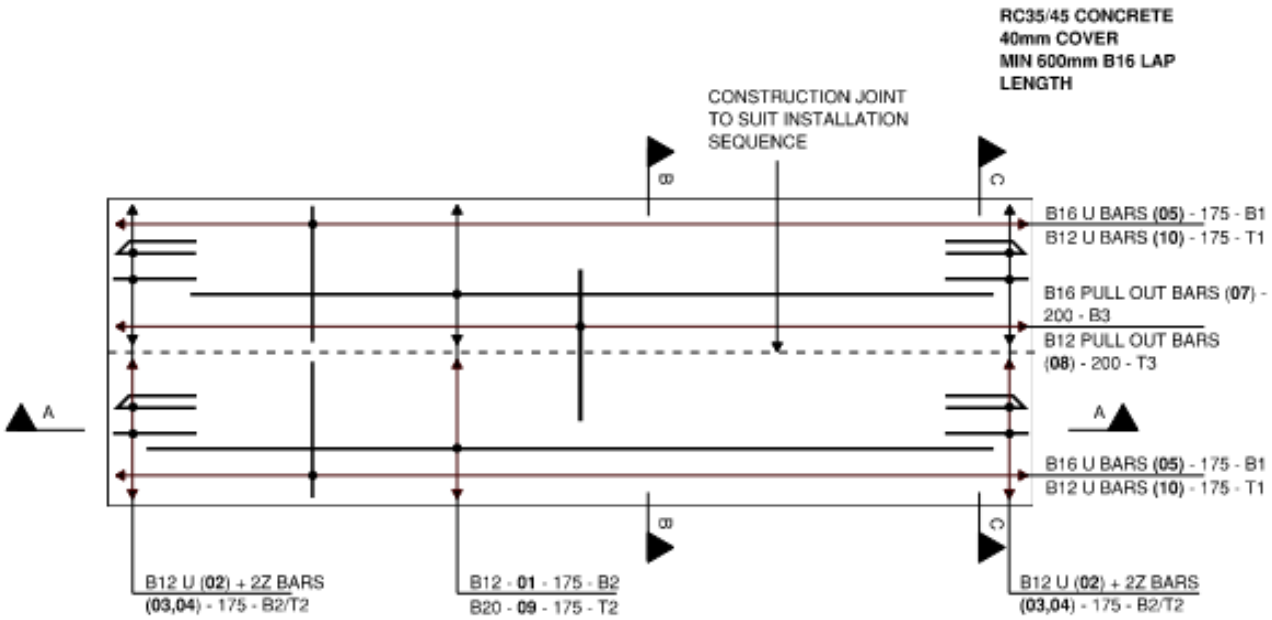
project **VIADUX PHASE 2B**

zone **A**

package **GBS ABUTMENT OPENINGS**

job no. 2209-01			
pg no. A- 5- 3- 14			
	PE	PL	CE
eng	RLC	KAG	
date	May-21	May-21	

A-5.3.8 OPENING TYPE 2- REINFORCEMENT INTENT

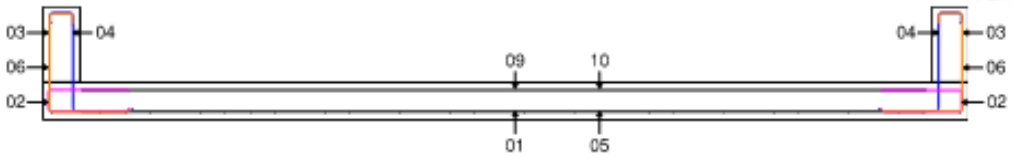


BASE PLAN

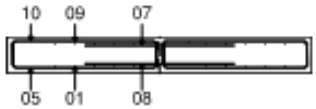


UPSTAND PLAN

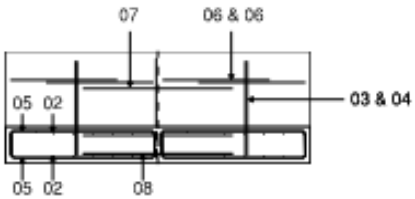
REINFORCEMENT TO BE LOCALLY DISPLACED TO SUIT HOLDING DOWN BOLTS. CUTTING NOT PERMITTED



SECTION A-A



SECTION B-B



SECTION C-C

A-5.4 SUPERSTRUCTURE

A-5.4.1 INTRODUCTION

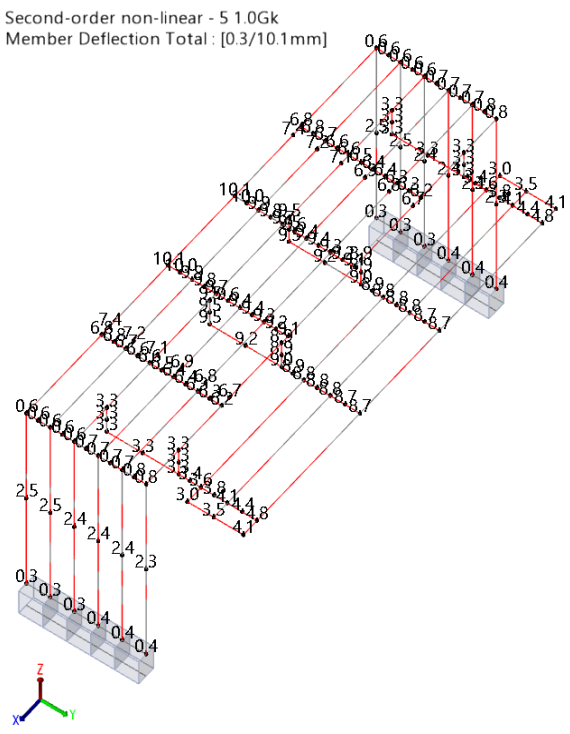
The following calculation pack details the modelling, analysis and design of the superstructure elements of the Great Bridgewater Street Abutment Openings. The frames have been modelled using Tekla Structural Designer and loads applied as per Section A-5.1 and A-5.2 of this calculation pack.

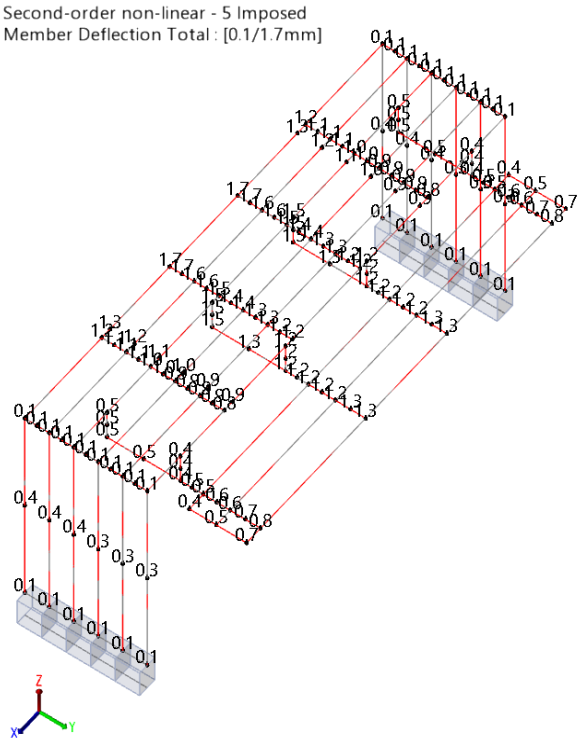
Opening 1 has frames of two sizes to account for the bigger loads which are applied on the canopy side.
The heavier frames consist of 203UC52 columns and 305UC283 beams.
The ligher frames consist of 203UC46 columns and 305UC198 beams.
The two different weights of frames are analysed and designed in the following section.

A-5.4.2 OPENING 1 - STEEL FRAME ANALYSIS

The following screenshots from the Tekla Structural Designer model are provided as a summary of the forces present in the steel frame.

DEFLECTION - WORST CASE SLS





OVERALL FRAME DEFLECTION

L = 7.2 m

δ_{Gk} = 10.1 mm (L/ 713)

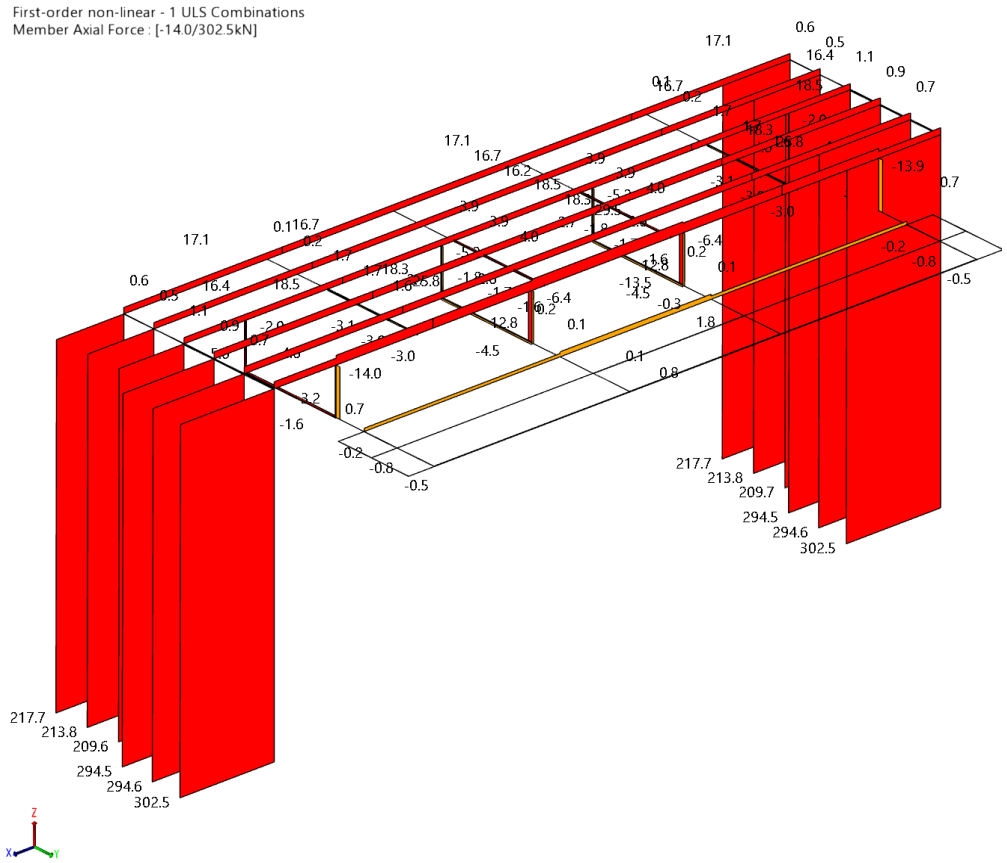
δ_{Qk} = 1.7 mm (L/ 4235)

δ_{total} = 11.8 mm (L/ 610) <L/500

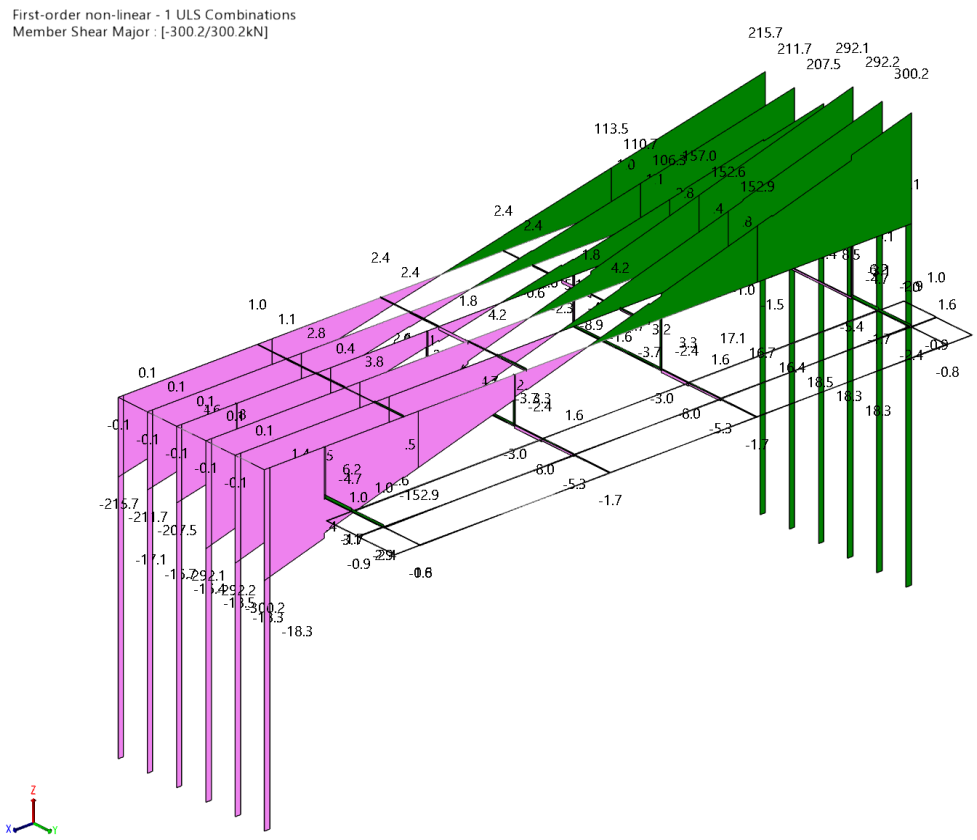
The above is conservative as it ignores the 60% direct spread of load throught the masonry at the ends and any arching action from the 2.7m of masonry above. In reality anticipated deflections will be 50-70% of the above. By inpsection deflections acceptable

A-5.4.2 OPENING 1 - STEEL FRAME ANALYSIS (CONT.)

AXIAL FORCE



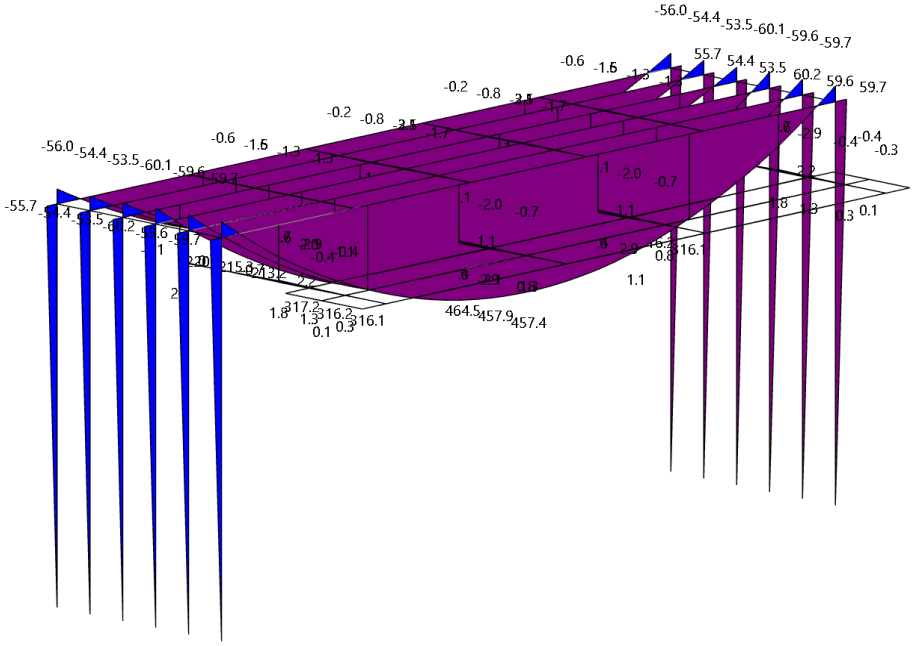
SHEAR FORCE



A-5.4.2 OPENING 1 - STEEL FRAME ANALYSIS (CONT.)

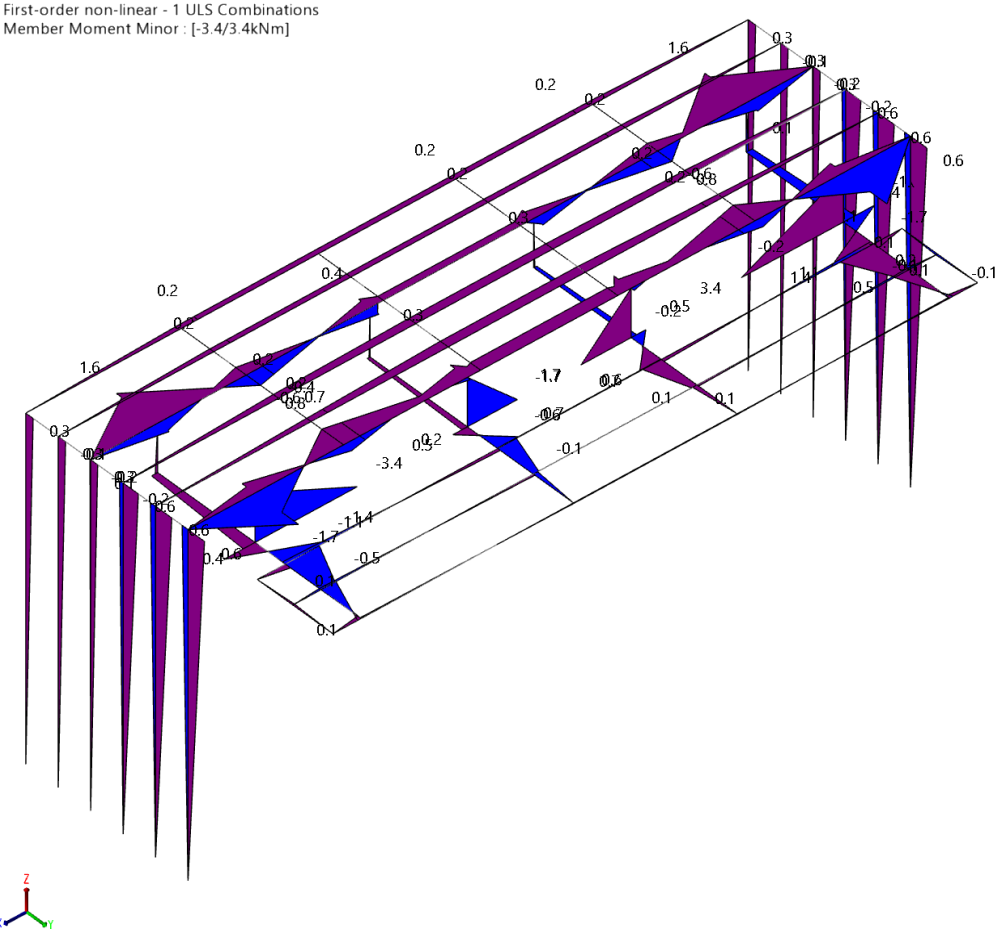
MOMENT MAJOR

First-order non-linear - 1 ULS Combinations
Member Moment Major : [-60.2/464.5kNm]



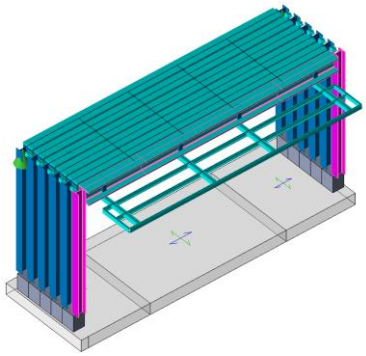
MOMENT MINOR

First-order non-linear - 1 ULS Combinations
Member Moment Minor : [-3.4/3.4kNm]



A-5.4.3 OPENING 1 - STEEL FRAME DESIGN

DESIGN - HEAVIER FRAME



COLUMN - UC 203x203x52

SC A/6 results (EN 1993-1-1, 2005)

- Summary UC 203x203x52(S355)
- Classification
- Shear Major
- Shear Minor
- Buckling Shear Web
- Moment Major
- Moment Minor
- Axial
- Axial Bending Combined
- Buckling Lateral Torsional
- Buckling Compression
- Buckling Combined

Summary UC 203x203x52(S355)

Design Condition	Combination Name	Design Value	Design Capacity	Units	U.R.	Status
Classification	1	Class 1	-	-	-	Pass
Shear Major	3	18.3	407.1	kN	0.045	Pass
Shear Minor	No	Significant	Forces	kN	-	Not required
Buckling Shear Web	-	22.94	48.82	-	-	Pass
Moment Major	3	59.7	202.9	kNm	0.294	Pass
Moment Minor	4	0.6	93.9	kNm	0.007	Pass
Axial	3	302.5	2370.5	kN	0.128	Pass
Axial Bending Combined	3	-	-	-	0.096	Pass
Buckling Lateral Torsional	1	0.0	0.0	kNm	0.000	Warning
Buckling Compression	1	0.0	0.0	kN	0.000	Warning
Column stack is not effectively restrained for major\minor axis buckling Warning						
Buckling Combined	1	-	-	-	0.000	Warning

To avoid excessive moments in the connections, grouting behind the columns will not occur until the frame has deflected under dead load. The columns are therefore modelled as unrestrained and Tekla will not perform a buckling check. Since no buckling check has occurred in the Tekla model, the following hand check is included:

	N_{ed}	**=**	**310** **kN**
	$N_{pl,Rd}$	=	2,350 kN
Limiting value of buckling lengths = N_{ed} / $N_{pl,Rd}$	n	=	0.13
Buckling length = 3.258m, conservatively take	L	=	4.0 m
Buckling resistance moment	$M_{b,Rd}$	=	167 kNm
Compression resistance in the major axis	$N_{b,y,R}$	=	1980 kN
Compression resistance in the minor axis	**$N_{b,z,R}$**	**=**	**1260** **kN**
	(U.R.	**=**	**0.25** **)**

A-5.4.3 OPENING 1 - STEEL FRAME DESIGN (CONT)

BEAM - UC305x305x283

SB 1/A/6-1/D/6 results (EN 1993-1-1, 2005)

Summary UC 305x305x283(S355)
- Classification
- Shear Major
- Shear Minor
- Buckling Shear Web
- Moment Major
- Moment Minor
- Axial
- Axial Bending Combined
- Buckling Lateral Torsional
- Buckling Compression
- Buckling Combined
- Torsion
- Deflection

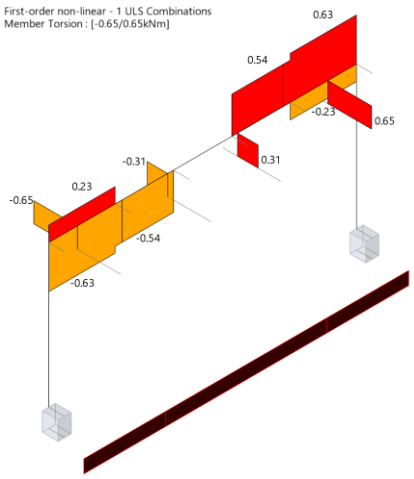
Summary UC 305x305x283(S355)

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Classification	1	Class 1	-	-	-	Pass
Shear Major	3	-300.2	2072.5	kN	0.145	Pass
Shear Minor	4	-4.0	4926.6	kN	0.001	Pass
Buckling Shear Web	-	10.340	50.253	-	-	Pass
Moment Major	3	457.4	1716.7	kNm	0.266	Pass
Moment Minor	3	-1.7	785.6	kNm	0.002	Pass
Axial	3	29.5	12122.9	kN	0.002	Pass
Axial Bending Combined	3	-	-	-	0.245	Pass
Buckling Lateral Torsional	3	316.1	1716.7	kNm	0.184	Pass
Buckling Compression	3	29.5	11116.4	kN	0.003	Pass
Buckling Combined	3	-	-	-	0.268	Pass
Torsion	-	-	-	-	0.000	Beyond Scope
Member not pinned at both ends Beyond Scope						
Deflection Self weight	1	0.2	-	mm	-	-
Deflection Slab	1	0.0	14.8	mm	0.000	Pass
Deflection Dead	1	2.6	7.4	mm	0.349	Pass
Deflection Imposed	1	0.4	10.3	mm	0.036	Pass
Deflection Wind	3	0.4	18.5	mm	0.020	Pass
Deflection Total	3	3.4	18.5	mm	0.182	Pass

Beam size is deflection governed. Utilisation = 0.27

Tekla does not check for member torsion in members with fixed connections, see separate check below

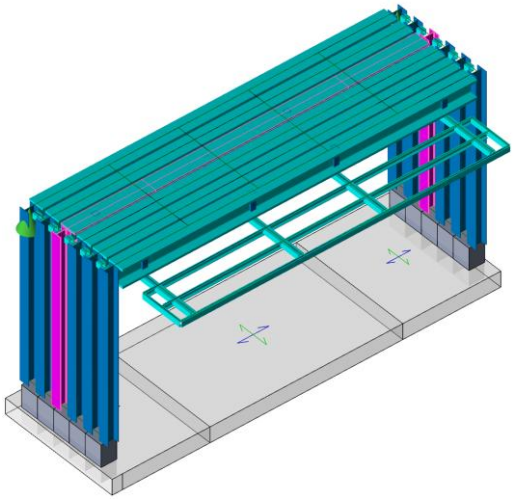
BEAM TORSION CHECK



Max Torsion = 0.65kNm, therefore by inspection OK

A-5.4.3 OPENING 1 - STEEL FRAME DESIGN (CONT)

DESIGN - LIGHTER FRAME



COLUMN - UC 203x203x46

SC D/3 results (EN 1993-1-1, 2005)

- Summary UC 203x203x46(S355)
- Classification
- Shear Major
- Shear Minor
- Buckling Shear Web
- Moment Major
- Moment Minor
- Axial
- Axial Bending Combined
- Buckling Lateral Torsional
- Buckling Compression
- Buckling Combined

Summary UC 203x203x46(S355)

Design Condition	Combination Name	Design Value	Design Capacity	Units	U.R.	Status
Classification	1	Class 2	-	-	-	Pass
Shear Major	2	-16.4	369.3	kN	0.044	Pass
Shear Minor	No	Significant	Forces	kN	-	Not required
Buckling Shear Web	-	25.17	48.82	-	-	Pass
Moment Major	2	-53.5	178.1	kNm	0.300	Pass
Moment Minor	1	0.3	82.1	kNm	0.003	Pass
Axial	2	209.6	2102.4	kN	0.100	Pass
Axial Bending Combined	2	-	-	-	0.096	Pass
Buckling Lateral Torsional	1	0.0	0.0	kNm	0.000	Warning
Buckling Compression	1	0.0	0.0	kN	0.000	Warning
Column stack is not effectively restrained for major\minor axis buckling Warning						
Buckling Combined	1	-	-	-	0.000	Warning

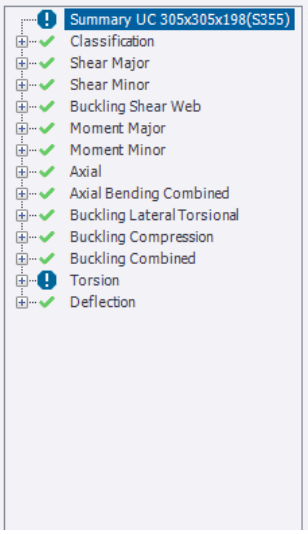
To avoid excessive moments in the connections, grouting behind the columns will not occur until the frame has deflected under dead load. The columns are therefore modelled as unrestrained and Tekla will not perform a buckling check. Since no buckling check has occurred in the Tekla model, the following hand check is included:

	N_{ed} =	**220**	**kN**
	$N_{pl,Rd}$ =	2,080	kN
Limiting value of buckling lengths = N_{ed} / $N_{pl,Rd}$	n =	0.11	
Buckling length = 3.258m, conservatively take	L =	4.0	m
Buckling resistance moment	$M_{b,Rd}$ =	143	kNm
Compression resistance in the major axis	$N_{b,y,R}$ =	1750	kN
Compression resistance in the minor axis	**$N_{b,z,R}$ =**	**1100**	**kN**
	(U.R. =	**0.2**	**)**

A-5.4.3 OPENING 1 - STEEL FRAME DESIGN (CONT)

BEAM

SB 1/A/3-1/D/3 results (EN 1993-1-1, 2005)

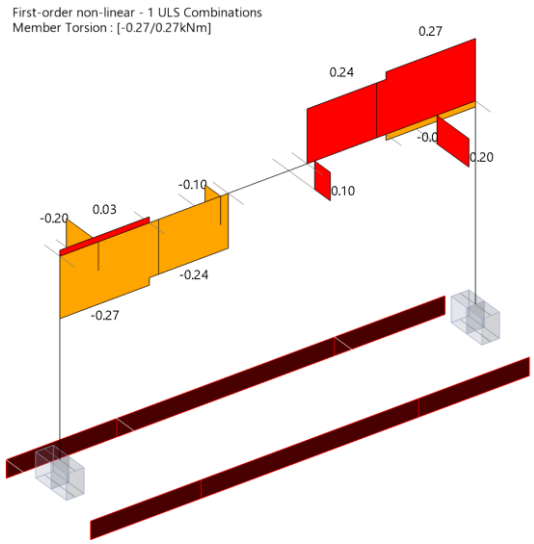


Summary UC 305x305x198(S355)

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Classification	1	Class 1	-	-	-	Pass
Shear Major	2	207.5	1492.2	kN	0.139	Pass
Shear Minor	4	-3.9	3564.4	kN	0.001	Pass
Buckling Shear Web	-	14.508	49.519	-	-	Pass
Moment Major	2	310.1	1193.4	kNm	0.260	Pass
Moment Minor	3	1.6	546.1	kNm	0.003	Pass
Axial	2	16.4	8758.4	kN	0.002	Pass
Axial Bending Combined	2	-	-	-	0.221	Pass
Buckling Lateral Torsional	2	213.2	1193.4	kNm	0.179	Pass
Buckling Compression	2	16.2	7669.7	kN	0.002	Pass
Buckling Combined	2	-	-	-	0.261	Pass
Torsion	-	-	-	-	0.000	Beyond Scope
Deflection Self weight	1	0.2	-	mm	-	-
Deflection Slab	1	0.0	14.8	mm	0.000	Pass
Deflection Dead	3	2.9	7.4	mm	0.386	Pass
Deflection Imposed	1	0.5	10.3	mm	0.045	Pass
Deflection Wind	3	0.1	18.5	mm	0.006	Pass
Deflection Total	2	3.5	18.5	mm	0.192	Pass

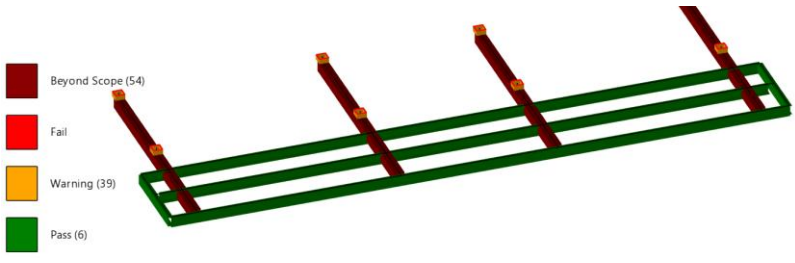
Tekla does not check for member torsion in members with fixed connections, see separate check, below

BEAM TORSION CHECK



Max Torsion = 0.27kNm, therefore by inspection OK

CANOPY SECONDARY BEAMS - PFC 100x50x10



All canopy PFC sections pass Tekla Design. Therefore OK.

A-5.4.3 OPENING 1 - STEEL FRAME DESIGN (CONT)

CANOPY PRIMARY BEAMS

SB C1/H/3-C1/H/13 results (EN 1993-1-1, 2005)

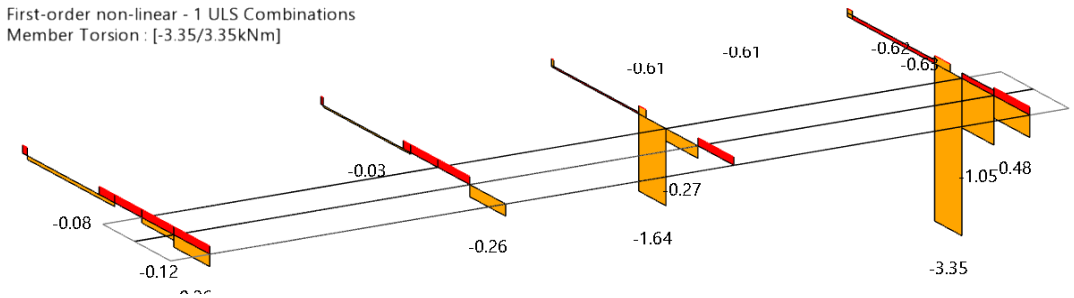
Summary SHS 100x100x8.0(S355)
- Classification
- Shear Major
- Shear Minor
- Buckling Shear Web
- Moment Major
- Moment Minor
- Axial
- Axial Bending Combined
- Buckling Lateral Torsional
- Buckling Compression
- Buckling Combined
- Torsion
- Deflection

Summary SHS 100x100x8.0(S355)

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Classification	1	Class 1	-	-	-	Pass
Shear Major	3	8.5	294.7	kN	0.029	Pass
Shear Minor	3	-7.3	294.7	kN	0.025	Pass
Buckling Shear Web	-	-	-	-	-	Not applicable
Moment Major	3	-5.7	34.9	kNm	0.164	Pass
Moment Minor	3	-1.4	34.9	kNm	0.041	Pass
Axial	3	4.6	1020.7	kN	0.005	Pass
Axial Bending Combined	3	-	-	-	0.053	Pass
Buckling Lateral Torsional	1	-	-	-	-	Not applicable
Buckling Compression	3	4.6	977.0	kN	0.005	Pass
Buckling Combined	3	-	-	-	0.141	Pass
Torsion	-	-	-	-	0.000	Beyond Scope
Member not pinned at both ends Beyond Scope						
Deflection Self weight	1	0.0	-	mm	-	-
Deflection Slab	-	No loads		mm	-	Not required
Deflection Dead	1	-0.0	0.7	mm	0.032	Pass
Deflection Imposed	1	-0.0	0.9	mm	0.009	Pass
Deflection Wind	4	-0.1	2.5	mm	0.040	Pass
Deflection Total	3	-0.1	1.7	mm	0.031	Pass

Tekla does not check for member torsion in members with fixed connections, see separate check, below

Torsion Check



T_{Ed} = 3.35 kNm

$T_{Rd} = W_T \cdot (f_y / \sqrt{3}) / \gamma_{M0}$

W_T = 116 cm^3

f_y = 500 N/mm^2

γ_{M0} = 1

T_{Rd} = 33.5 kNm

$T_{Ed} < T_{Rd}$

Therefore OK

CANOPY HANGERS

SC H/6 results (EN 1993-1-1, 2005)

Summary SHS 100x100x8.0(S355)
- Classification
- Shear Major
- Shear Minor
- Buckling Shear Web
- Moment Major
- Moment Minor
- Axial
- Axial Bending Combined
- Buckling Lateral Torsional
- Buckling Compression
- Buckling Combined

Summary SHS 100x100x8.0(S355)

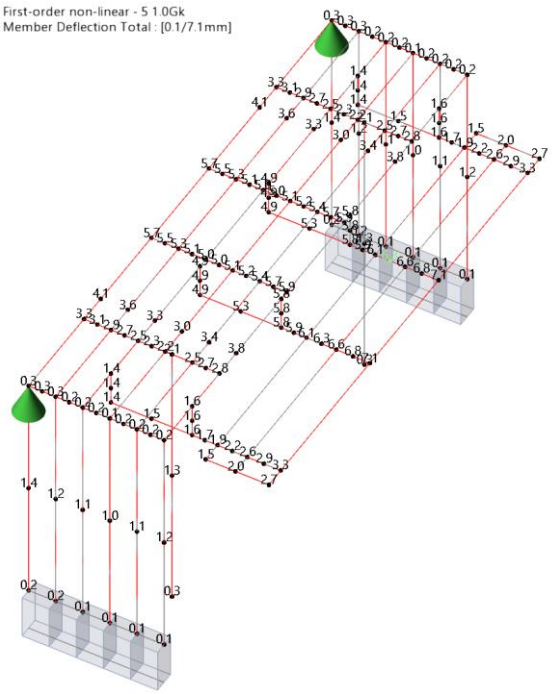
Design Condition	Combination Name	Design Value	Design Capacity	Units	U.R.	Status
Classification	1	Class 1	-	-	-	Pass
Shear Major	3	-4.7	294.7	kN	0.016	Pass
Shear Minor	3	7.5	294.7	kN	0.025	Pass
Buckling Shear Web	No Significant Forces			kN	-	Not applicable
Moment Major	3	2.2	34.9	kNm	0.062	Pass
Moment Minor	3	-3.4	34.9	kNm	0.098	Pass
Axial	3	-14.0	1020.7	kN	0.014	Pass
Axial Bending Combined	3	-	-	-	0.031	Pass
Buckling Lateral Torsional	1	-	-	-	-	Not applicable
Buckling Compression	1	0.0	0.0	kN	0.000	Warning
Column stack is not effectively restrained for major\minor axis buckling Warning						
Buckling Combined	1	-	-	-	0.000	Warning

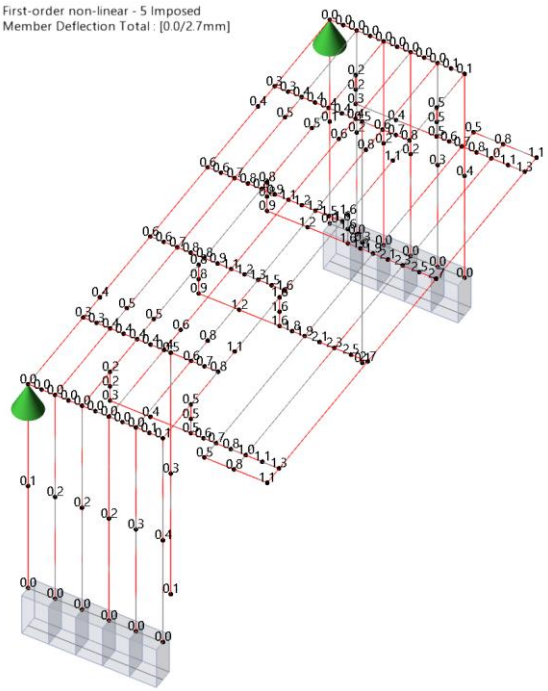
The canopy hangers are stub elements with combined Axial and Bending utilisation < 1. Therefore, by inspection, OK

A-5.4.5 OPENING TYPE 2 - STEEL FRAME ANALYSIS

The following screenshots from the Tekla Structural Designer model are provided as a summary of the forces present in the steel frame.

DEFLECTION - WORST CASE SLS





OVERALL FRAME DEFLECTION

L = 7.2 m

δ_{Gk} = 6.0 mm (L/ 1,200)

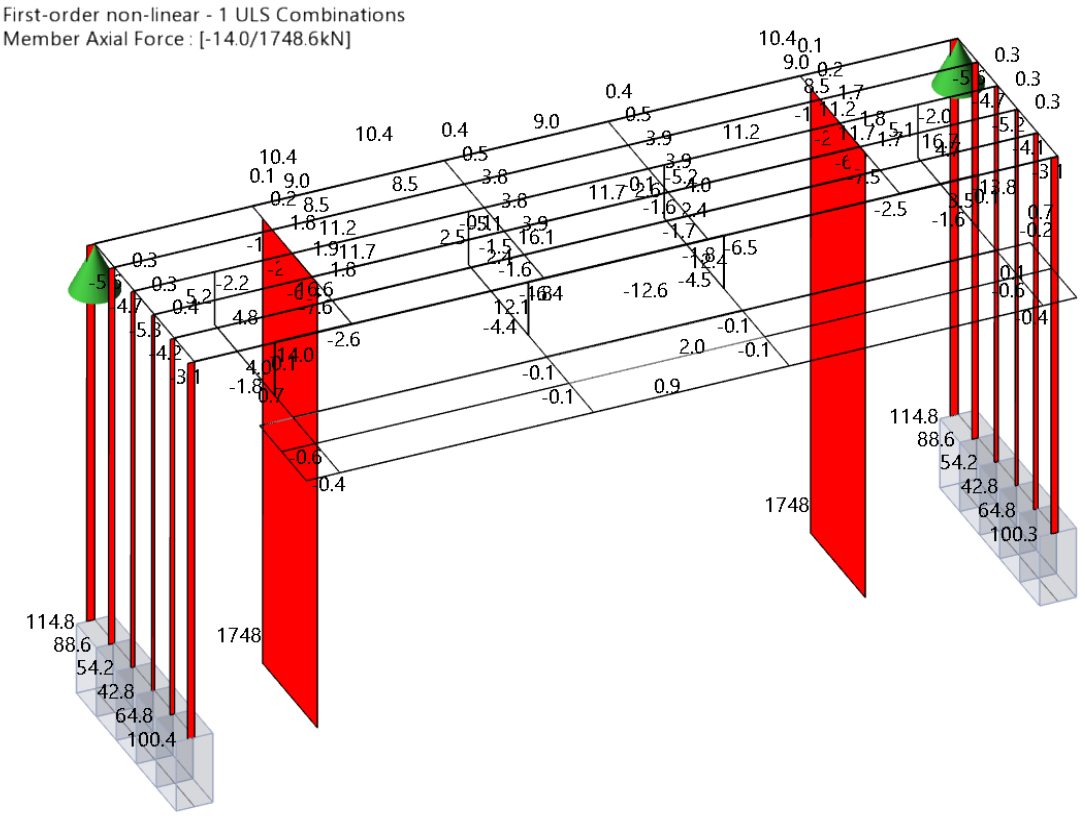
δ_{Qk} = 1.7 mm (L/ 4,235)

δ_{total} = 7.7 mm (L/ 935) <L/500

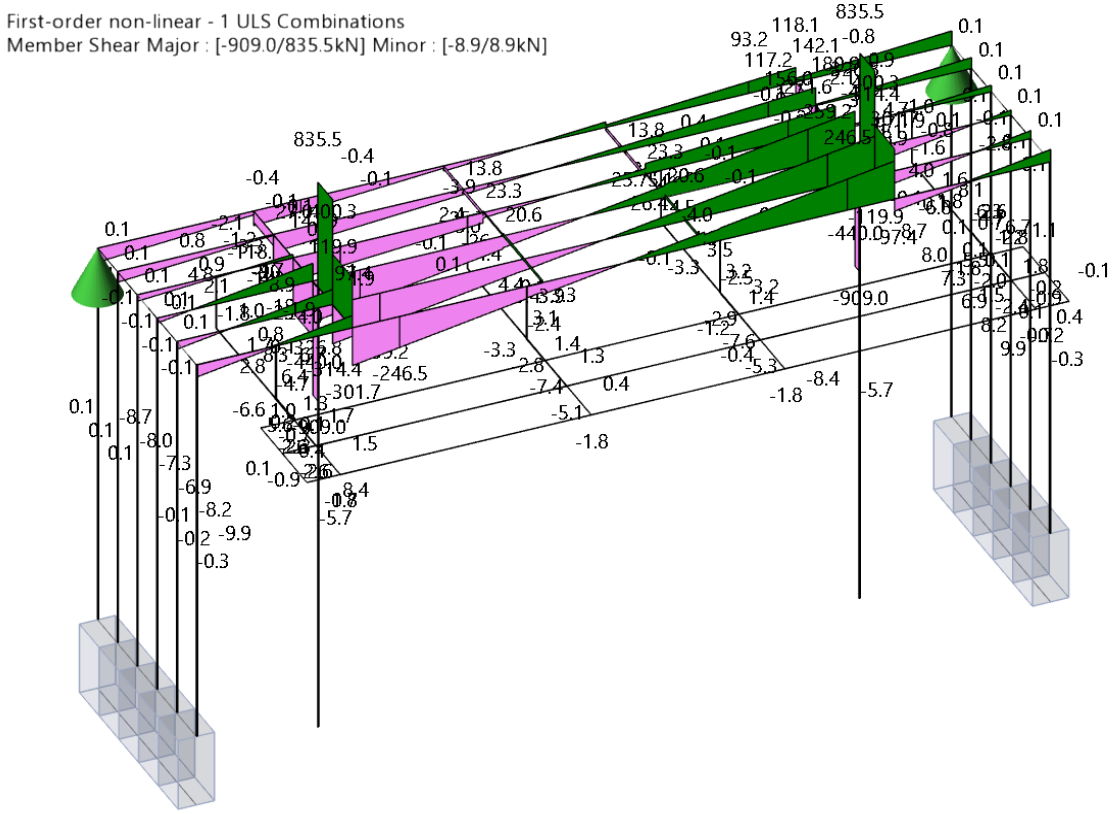
The above is conservative as it ignores the 60% direct spread of load throught the masonry at the ends and any arching action from the 2.7m of masonry above. In reality anticipated deflections will be 50-70% of the above. By inpsection deflections acceptable

A-5.4.5 OPENING TYPE 2 - STEEL FRAME ANALYSIS (CONT.)

AXIAL FORCE

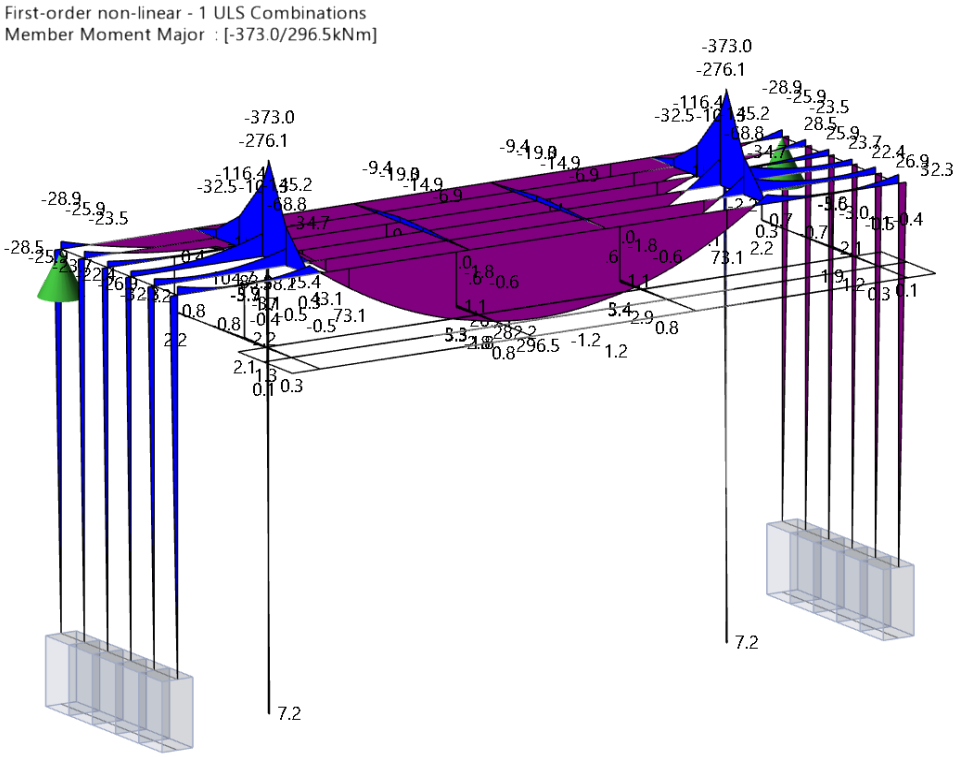


SHEAR FORCE

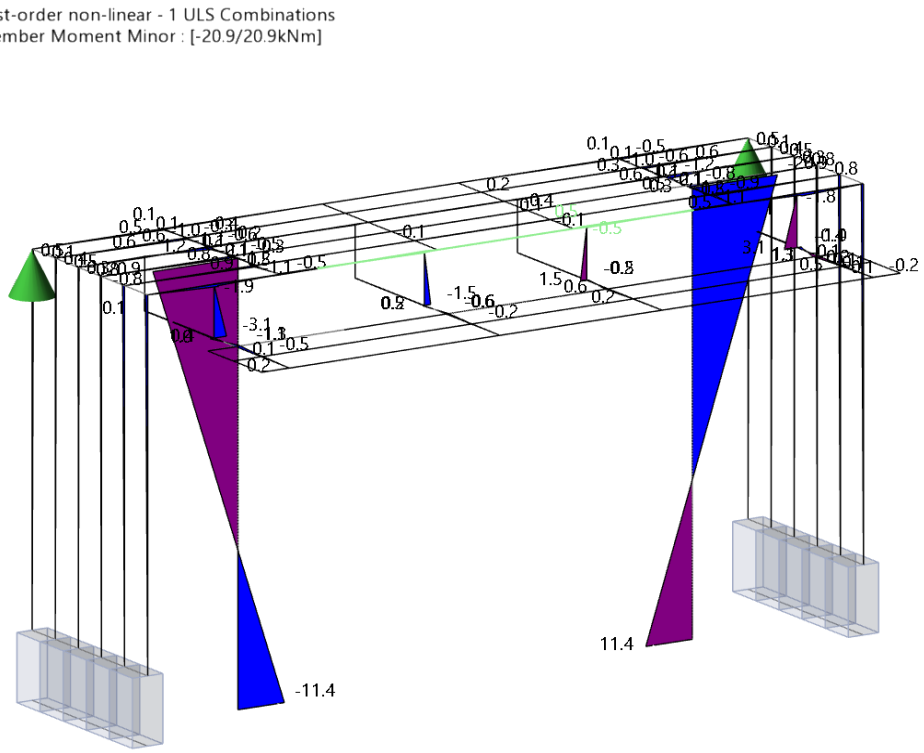


A-5.4.5 OPENING TYPE 2 - STEEL FRAME ANALYSIS (CONT.)

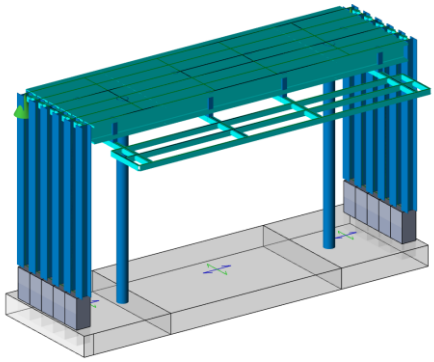
MOMENT MAJOR



MOMENT MINOR



A-5.4.6 OPENING TYPE 2 - STEEL FRAME DESIGN



COLUMN - UC 203x203x46

SC D/1 results (EN 1993-1-1, 2005)

- Summary UC 203x203x46(S355)
- Classification
- Shear Major
- Shear Minor
- Buckling Shear Web
- Moment Major
- Moment Minor
- Axial
- Axial Bending Combined
- Buckling Lateral Torsional
- Buckling Compression
- Buckling Combined

Summary UC 203x203x46(S355)

Design Condition	Combination Name	Design Value	Design Capacity	Units	U.R.	Status
Classification	1	Class 2	-	-	-	Pass
Shear Major	1	-8.7	369.3	kN	0.024	Pass
Shear Minor	No	Significant	Forces	kN	-	Not required
Buckling Shear Web	-	25.17	48.82	-	-	Pass
Moment Major	1	-28.5	178.1	kNm	0.160	Pass
Moment Minor	4	0.5	82.1	kNm	0.006	Pass
Axial	1	114.8	2102.4	kN	0.055	Pass
Axial Bending Combined	1	-	-	-	0.031	Pass
Buckling Lateral Torsional	1	0.0	0.0	kNm	0.000	Warning
Buckling Compression	1	0.0	0.0	kN	0.000	Warning
Buckling Combined	1	-	-	-	0.000	Warning

To avoid excessive moments in the connections, grouting behind the columns will not occur until the frame has deflected under dead load. The columns are therefore modelled as unrestrained and Tekla will not perform a buckling check. Since no buckling check has occurred in the Tekla model, the following hand check is included:

	N_{ed} =	**120**	**kN**
	$N_{pl,Rd}$ =	2,080	kN
Limiting value of buckling lengths = N_{ed} / $N_{pl,Rd}$	n =	0.06	
Buckling length = 3.258m, conservatively take	L =	4.0	m
Buckling resistance moment	$M_{b,Rd}$ =	143	kNm
Compression resistance in the major axis	$N_{b,y,R}$ =	1,750	kN
Compression resistance in the minor axis	**$N_{b,z,R}$ =**	**1,100**	**kN**
	(U.R. =	**0.11**	**)**

A-5.4.6 OPENING 2 - STEEL FRAME DESIGN (CONT)

BEAM - UC305x305x198

SB 1/A/6-1/D/6 results (EN 1993-1-1, 2005)

- Summary UC 305x305x198(S355)
- Classification
- Shear Major
- Shear Minor
- Buckling Shear Web
- Moment Major
- Moment Minor
- Axial
- Axial Bending Combined
- Buckling Lateral Torsional
- Buckling Compression
- Buckling Combined
- Torsion
- Deflection

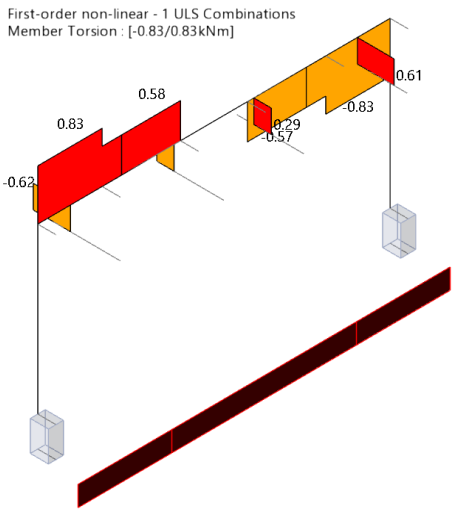
Summary UC 305x305x198(S355)

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Classification	1	Class 1	-	-	-	Pass
Shear Major	2	301.7	1492.2	kN	0.202	Pass
Shear Minor	4	-4.0	3564.4	kN	0.001	Pass
Buckling Shear Web	-	14.508	49.519	-	-	Pass
Moment Major	2	296.5	1193.4	kNm	0.248	Pass
Moment Minor	3	-1.9	546.1	kNm	0.003	Pass
Axial	2	16.7	8758.4	kN	0.002	Pass
Axial Bending Combined	2	-	-	-	0.248	Pass
Buckling Lateral Torsional	2	73.1	1193.4	kNm	0.061	Pass
Buckling Compression	2	16.1	7669.7	kN	0.002	Pass
Buckling Combined	2	-	-	-	0.249	Pass
Torsion	-	-	-	-	0.000	Beyond Scope
Deflection Self weight	1	0.1	-	mm	-	-
Deflection Slab	1	0.0	14.8	mm	0.000	Pass
Deflection Dead	1	2.4	7.4	mm	0.328	Pass
Deflection Imposed	1	0.7	10.3	mm	0.064	Pass
Deflection Wind	3	0.3	18.5	mm	0.014	Pass
Deflection Total	2	3.3	18.5	mm	0.177	Pass

Beam size is deflection governed. Utilisation = 0.33

Tekla does not check for member torsion in members with fixed connections, see separate check below

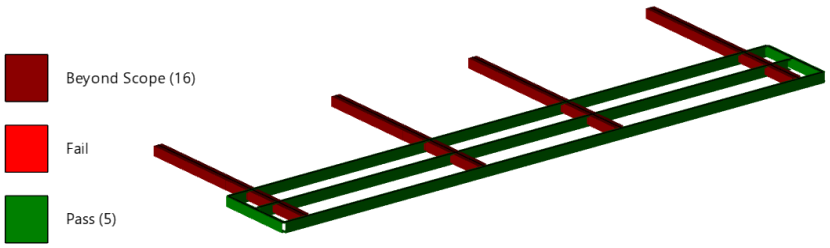
BEAM TORSION CHECK



Max Torsion = 0.83kNm, therefore by inspection OK

A-5.4.6 OPENING 2 - STEEL FRAME DESIGN (CONT)

CANOPY SECONDARY BEAMS - PFC 100x50x10



All canopy PFC sections pass Tekla Design. Therefore OK.

CANOPY PRIMARY BEAMS

SB C1/H/3-C1/H/13 results (EN 1993-1-1, 2005)

- Summary SHS 100x100x8.0(S355)
- Classification
- Shear Major
- Shear Minor
- Buckling Shear Web
- Moment Major
- Moment Minor
- Axial
- Axial Bending Combined
- Buckling Lateral Torsional
- Buckling Compression
- Buckling Combined
- Torsion
- Deflection

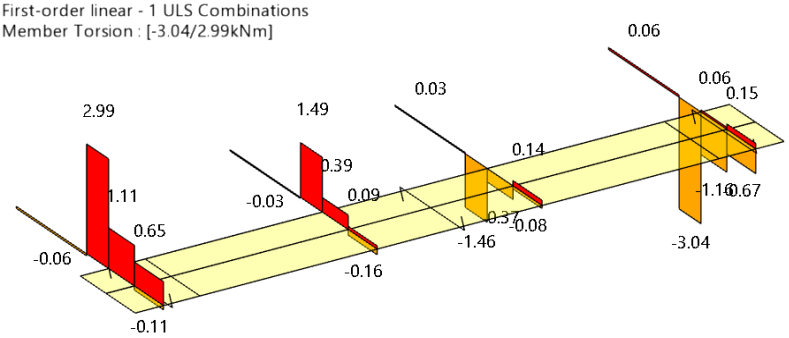
Summary SHS 100x100x8.0(S355)

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Classification	1	Class 1	-	-	-	Pass
Shear Major	3	8.3	294.7	kN	0.028	Pass
Shear Minor	2	-6.5	294.7	kN	0.022	Pass
Buckling Shear Web	-	-	-	-	-	Not applicable
Moment Major	3	-5.8	34.9	kNm	0.167	Pass
Moment Minor	2	-1.2	34.9	kNm	0.036	Pass
Axial	3	4.8	1020.7	kN	0.005	Pass
Axial Bending Combined	3	-	-	-	0.054	Pass
Buckling Lateral Torsional	1	-	-	-	-	Not applicable
Buckling Compression	3	4.8	977.0	kN	0.005	Pass
Buckling Combined	3	-	-	-	0.144	Pass
Torsion	-	-	-	-	0.000	Beyond Scope
Deflection Self weight	1	0.0	-	mm	-	-
Deflection Slab	1	0.0	1.8	mm	0.000	Pass
Deflection Dead	1	-0.0	0.7	mm	0.033	Pass
Deflection Imposed	1	-0.0	0.9	mm	0.012	Pass
Deflection Wind	4	-0.1	2.5	mm	0.042	Pass
Deflection Total	3	-0.1	1.7	mm	0.032	Pass

Tekla does not check for member torsion in members with fixed connections, see separate check, below

Torsion Check

First-order linear - 1 ULS Combinations
Member Torsion : [-3.04/2.99kNm]



$T_{Rd} = W_T \cdot (fy / \sqrt{3}) / \gamma_{M0}$

$W_T = 116 \text{ cm}^3$

$fy = 500 \text{ N/mm}^2$

$\gamma_{M0} = 1$

T_{Ed} = 3.1 kNm

T_{Rd} = 33.5 kNm

T_{Ed} < T_{Rd}

Therefore OK

A-5.4.6 OPENING 2 - STEEL FRAME DESIGN (CONT)

CANOPY HANGERS

SC G/6 results (EN 1993-1-1, 2005)

- Summary SHS 100x100x8.0(S355)
- Classification
- Shear Major
- Shear Minor
- Buckling Shear Web
- Moment Major
- Moment Minor
- Axial
- Axial Bending Combined
- Buckling Lateral Torsional
- Buckling Compression
- Buckling Combined

Summary SHS 100x100x8.0(S355)

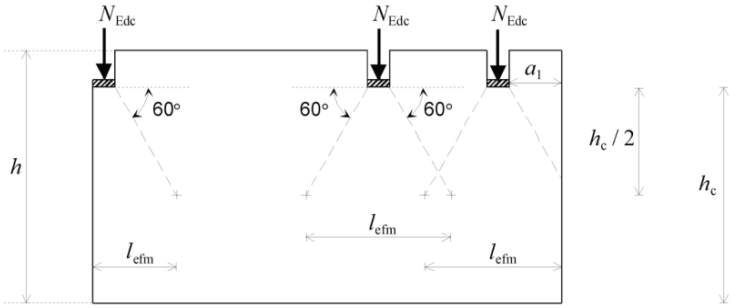
Design Condition	Combination Name	Design Value	Design Capacity	Units	U.R.	Status
Classification	1	Class 1	-	-	-	Pass
Shear Major	3	-4.6	294.7	kN	0.016	Pass
Shear Minor	3	-6.8	294.7	kN	0.023	Pass
Buckling Shear Web	No Significant Forces			kN	-	Not applicable
Moment Major	3	2.1	34.9	kNm	0.061	Pass
Moment Minor	3	3.1	34.9	kNm	0.089	Pass
Axial	3	-13.7	1020.7	kN	0.013	Pass
Axial Bending Combined	3	-	-	-	0.028	Pass
Buckling Lateral Torsional	1	-	-	-	-	Not applicable
Buckling Compression	1	0.0	0.0	kN	0.000	Warning
Buckling Combined	1	-	-	-	0.000	Warning

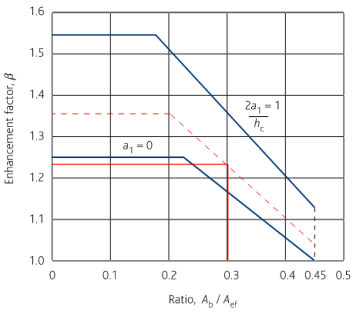
The canopy hangers are stub elements with combined Axial and Bending utilisation < 1. Therefore, by inspection, OK

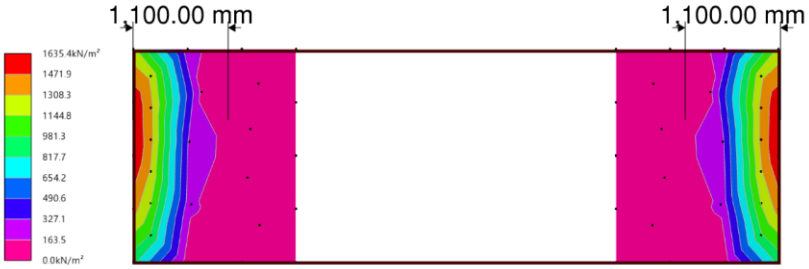
A-5.5 ADDITIONAL MASONRY CHECKS

A-5.5.1 OPENING TYPE 1 MASONRY STRESS CHECK UNDER SPREADER BEAM

By inspection of the bearing pressure plots the spreader is effectively acting as a padstone inset a_1 from the edge of wall.







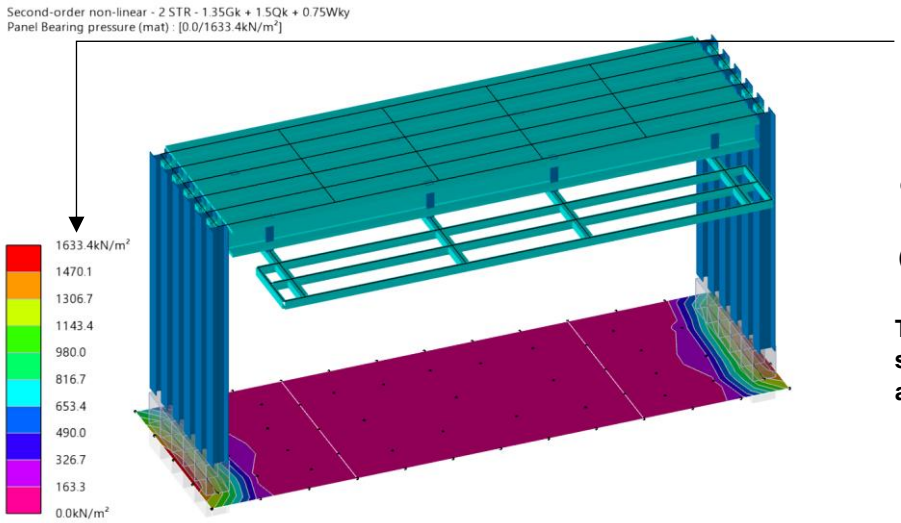
Based on the bearing plots the 'effective' padstone is circa 2.45m x 1.1m	A_b =	2.7	m^2
Based on the bearing plots the edge of 'effective' padstone is circa	a_1 =	0.7	m
The wall is founded at circa 4.5m below ground. Conservatively take	h_c =	3.8	m
Effective length at $h_c/2 = h_c/2+1.1+a_1$	l_{efm} =	3.7	m
Effective bearing area of wall at $h_c/2$	A_{ef} =	9.1	m
Bearing Ratio	$\frac{A_b}{A_{ef}}$ =	0.30	
	$\frac{2a_1}{h_c}$ =	0.37	

This calculation refers to a mass masonry structure, not at risk of bursting. The recommended value of β=1.0 for Group 2,3 and 4 masonry units is based on slender walls and not considered applicable in this situation.

Therefore bearing stress enhancement factor from graph	β =	1.23	
Stress due to design load	$\sigma_{b,Rd}$ =	$\frac{\beta f_k}{\gamma_m}$	
Refer to Section 5.0 for characteristic strength	f_k =	2.5	N/mm^2
Refer to Section 5.0 for material factors	γ_m =	1.52	
	$\sigma_{b,Rd}$ =	**2.0**	**N/mm^2**

Refer to extract for structural designer model for bearing pressure under the required load combination (next page).

A-5.5.1 OPENING TYPE 1 MASONRY STRESS CHECK UNDER SPREADER BEAM (CONT.)



σ_{Ed} = **1.64** **N/mm²**

$\sigma_{b,Rd}$ = **2.0** **N/mm²**

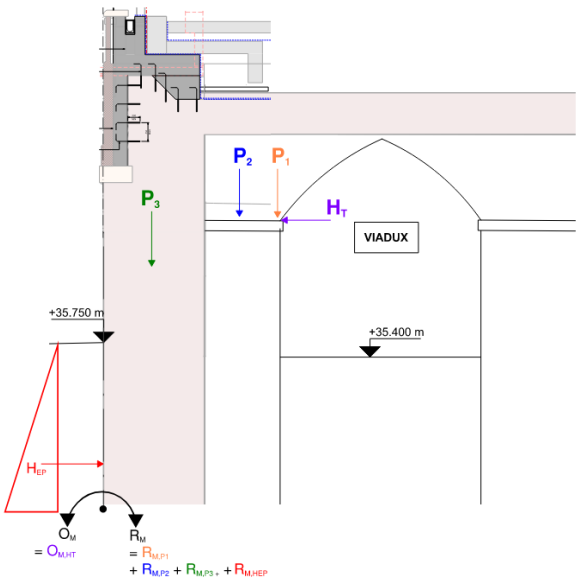
(U.R. = 0.81 < 1)

Therefore bearing beneath spreader foundation adequate

Note: the above assessment is very conservative as the peak applied stress is being compared against the general bearing capacity. In reality the spreader and upstand are very stiff and the bearing pressure will be more evenly distributed and will be circa 1-1.2N/mm^2.

A-5.5.2 OPENING TYPE 1 ABUTMENT OVERTURNING CHECK FOR REDUCED SIZE

The check shown in this section proves that the overturning moments resulting from the thrust force from the pointed arches can be resisted by the new, reduced abutment size.



From the detailed assessment below the overturing and restoring moments are summarised below:

Total Overturning Moment = 10427 kNm

Total Restoring Moment = 16639 kNm

FOS = 1.6

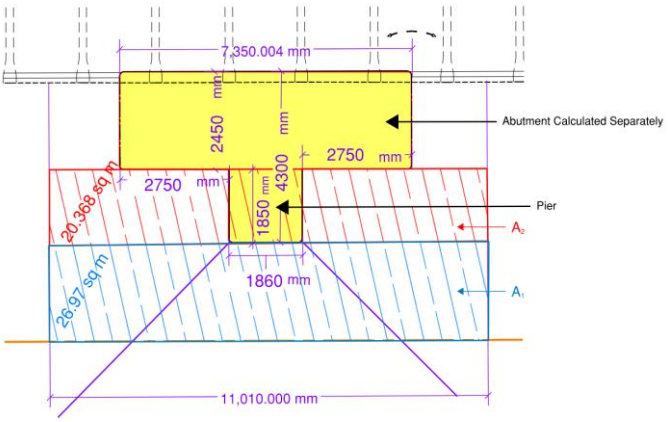
As the FOS against overturing is > 1.5 reduce pier width due to enlarged openings are acceptable.

Note assessment is conservative as moments taken at foundation level, with reduce pier width taken full height and restoring benefit of earth pressures and cast iron bridge ignored

Local stress checks on the masonry also indicate that the masonry is acceptable

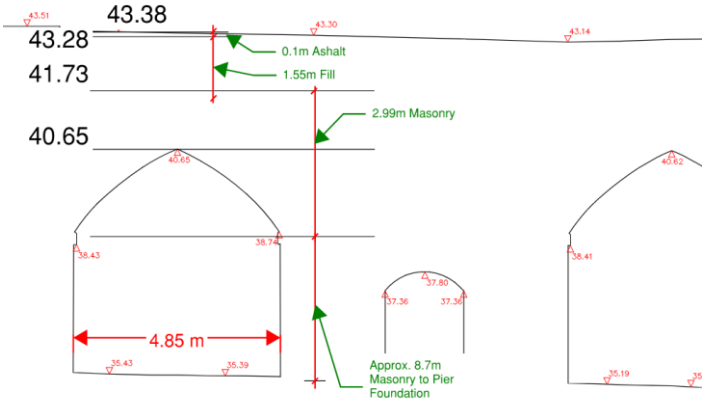
A-5.5.2 OPENING TYPE 1 ABUTMENT OVERTURNING CHECK FOR REDUCED SIZE (CONT.)

DIMENSIONS - EXISTING

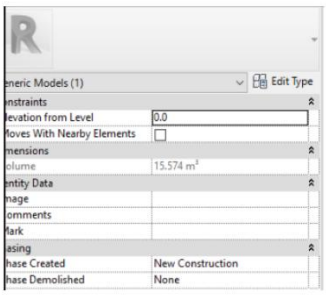


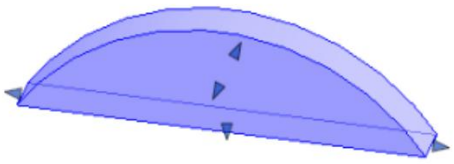
Area A_1 = 26.97 m^2
Area A_2 = 20.37 m^2

PLAN

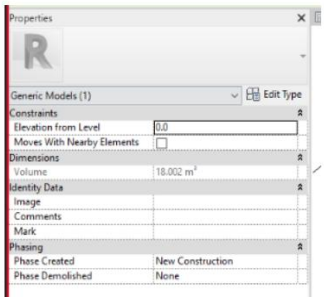


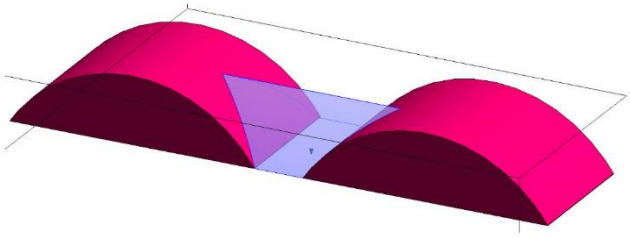
SECTION





Segmental Arch Void
= 15.57 m^3





Pointed Arch Void
= 18.0 m^3

ARCH VOID VOLUMES

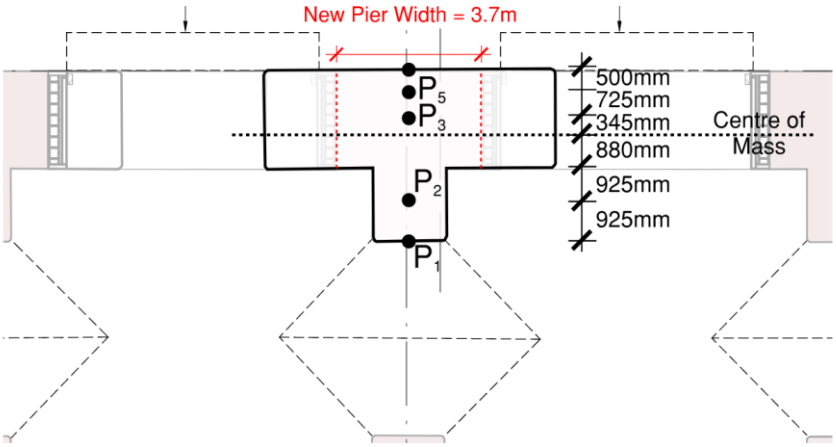
A-5.5.2 OPENING TYPE 1 ABUTMENT OVERTURNING CHECK FOR REDUCED SIZE (CONT.)

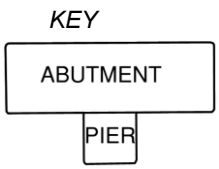
LOADING

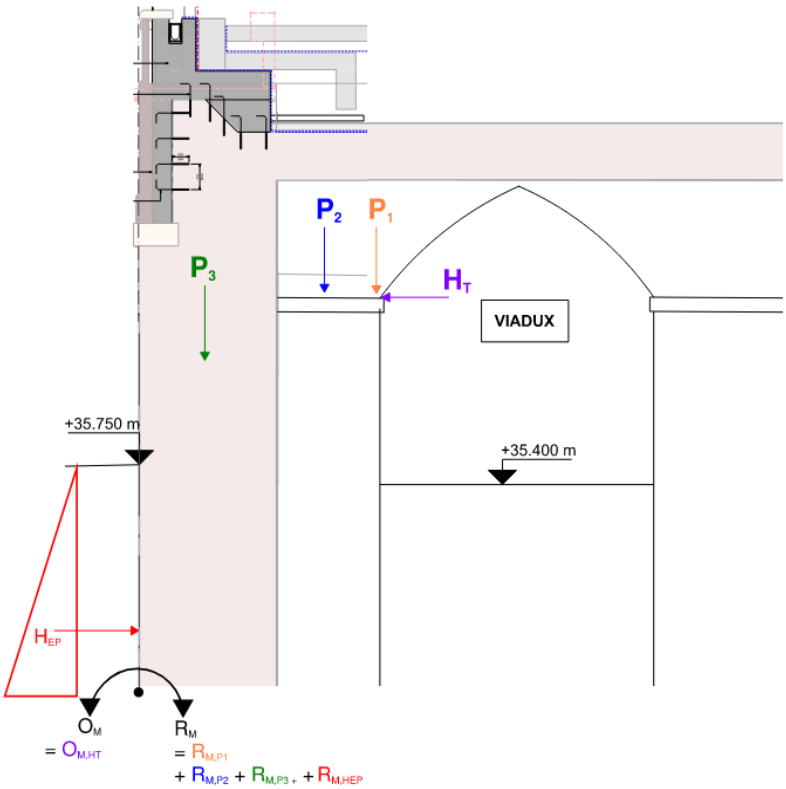
Loading is applied from:
- The masonry arches
- The selfweight of the pier
- The selfweight of the abutment (reduced size)

Conservatively horizontal earth pressure from the Great Bridgewater Street side of the abutment is ignored.
Conservatively weight of cast iron arches in permanent case is ignored

The loads applied to the abutment are shown on the following page, but are applied to the abutment and pier at the locations shown below.







A-5.5.2 OPENING TYPE 1 ABUTMENT OVERTURNING CHECK FOR REDUCED SIZE (CONT.)

P_1 - Loads from Area A_1

G_k - Dead Loads

Asphalt	=	26.97	x	0.1	x	22.6			=	61	kN
Fill	=	26.97	x	1.55	x	20.6			=	861	kN
Arches	=	(26.97	x	2.99							
		- 2.45	x	15.57							
		- 18.0	/	2)	x	18.6			=	623	kN

Total G_k = 1,545 kN

Q_k - Imposed Loads

Q_k = 15 kN/m^2

15 x 26.97 **Total Q_k = 405 kN**

P_2 - Loads from Area A_2 and Pier

G_k - Dead Loads

Asphalt	=	20.37	x	0.1	x	22.6			=	61	kN
Fill	=	20.37	x	1.55	x	20.6			=	861	kN
Arches	=	(20.37	x	2.99							
		- 1.85	x	15.57	)	x	18.6		=	597	kN
Pier Selfweight	=	(1.85	x	1.86	x	8.7	)	x 18.6	=	557	kN

Total G_k = 2,076 kN

Q_k - Imposed Loads

Q_k = 15 kN/m^2

15 x 20.37 **Total Q_k = 306 kN**

P_3 - Loading on Abutment (conservatively taking reduced reduced size for all)

G_k - Dead Loads

Asphalt	=	2.45	x	3.7	x	0.1	x	18.6	=	17	kN
Fill	=	2.45	x	3.7	x	1.55	x	18.6	=	261	kN
Masonry above	=	2.45	x	3.7	x	2.99	x	18.6	=	504	kN
Selfweight	=	2.45	x	3.7	x	8.7	x	18.6	=	1,467	kN

Total G_k = 2,249 kN

Q_k - Imposed Loads

Q_k = 15 kN/m^2

15 x 2.45 x 3.7 **Total Q_k = 136 kN**

H_T - Horizontal Thrust Force

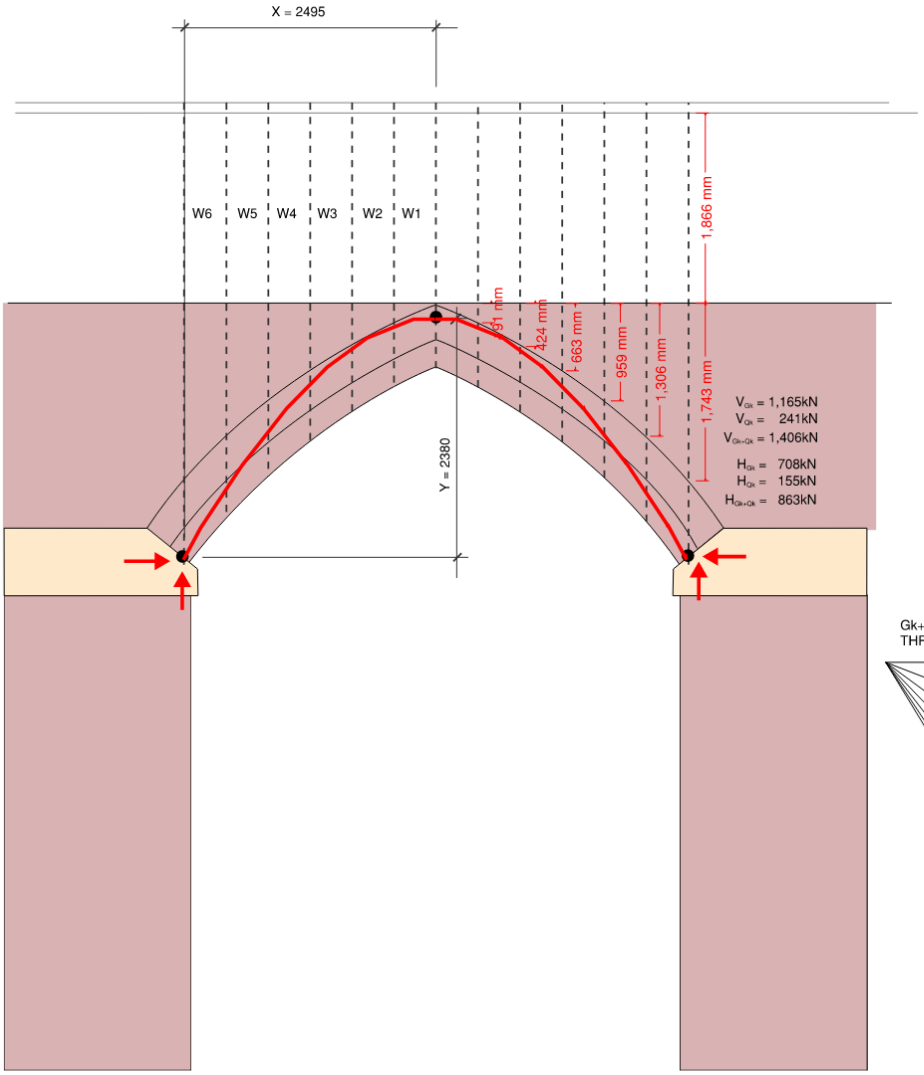
For horizontal thrust force, refer to Funicular Polygon Diagram on following page

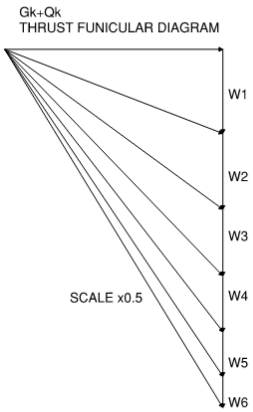
Total H_{Qk} = 710 kN

Total H_{Gk} = 160 kN

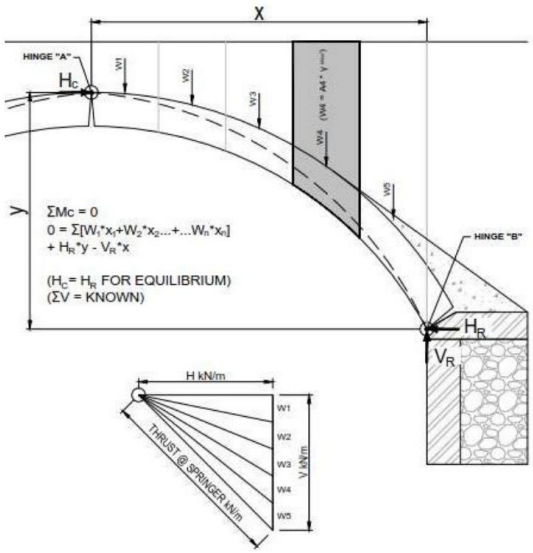
A-5.5.2 OPENING TYPE 1 ABUTMENT OVERTURNING CHECK FOR REDUCED SIZE (CONT.)

Funicular Polygon Diagram - Pointed Arch



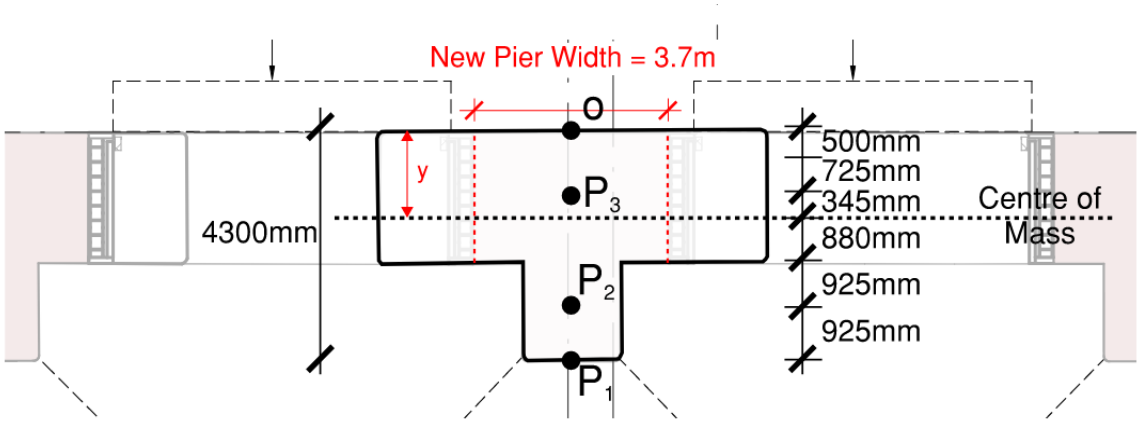


TYPICAL POINTED ARCH
1:50



A-5.5.2 OPENING TYPE 1 ABUTMENT OVERTURNING CHECK FOR REDUCED SIZE (CONT.)

MOMENT DERIVATION



To determine the overturning and restoring moments, take moments around point O and at bottom of foundation leve

OVERTURNING MOMENT DUE TO HORIZONTAL THRUST

$O_{M,H,Gk}$	=	710	x	8.7	**Total $M_{O,Gk}$ =**	**6,177**	**kNm**
$O_{M,H,Qk}$	=	160	x	8.7	**Total $M_{O,Qk}$ =**	**1,392**	**kNm**

RESTORING MOMENT DUE TO VERTICAL FORCES

$R_{M,P1,Gk}$	=	1,545	x	4.3	=	6,644	kNm
$R_{M,P2,Gk}$	=	2,076	x	3.375	=	7,007	kNm
$R_{M,P3,Gk}$	=	2,249	x	2.15	=	4,836	kNm
	$N_{M,Gk}$ =	**5,871**			**Total $M_{R,Gk}$ =**	**18,487**	**kNm**
$R_{M,P1,Qk}$	=	405	x	4.3	=	1,740	kNm
$R_{M,P2,Qk}$	=	306	x	3.375	=	1,031	kNm
$R_{M,P3,Qk}$	=	136	x	2.15	=	292	kNm
	$N_{M,Qk}$ =	**846**			**Total $M_{R,Gk}$ =**	**3,063**	**kNm**

ULS OVERTURNING MOMENT = $1.35M_{Gk} + 1.5M_{Qk}$ = **10,427 kNm**

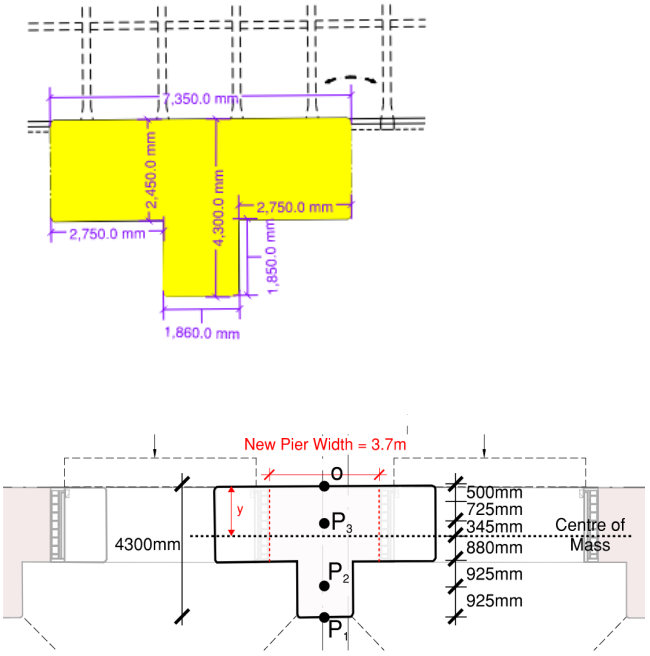
CONSERVATIVELY TAKING $0.9M_{Gk}$ RESTORING MOMENT = **16,639 kNm**

FOS = 1.6

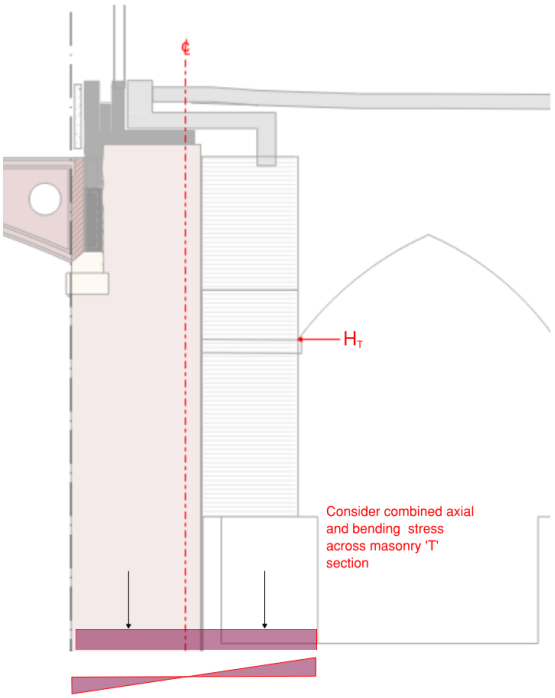
THEREFORE OK

A-5.5.3 OPENING TYPE 1 MASONRY CRUSHING CHECK FOR REDUCED PIER SIZE

To assess the reduced pier size consider the resulting compressive stresses across the section, based on the 'T' section.



PLAN



SECTION

Pier Properties

$A_1 = 1.85 \times 1.86 = 3.44\ m^2$

$A_2 = 3.70 \times 2.45 = 9.07\ m^2$

$A_{pier} = 3.44 + 9.07 = 12.5\ m^2$ → **$A_{pier} = 12.51\ m^2$**

$$y = \frac{3.44 \times 3.38 + 9.07 \times 1.225}{3.44 + 9.07}$$ → **$y = 1.8\ m$**

$I_{zz} = \Sigma(A_i d_i^2 + I_i)$

$= 3.44 \times 2.43 + 0.95$

$+$

$9.07 \times 0.35 + 4.53$ → **$I_{zz} = 17.0\ m^4$**

$$W_{el,z,top} = \frac{I}{y} = \frac{17.0}{1.8}$$ → **$W_{el,z,top} = 9.4\ m^3$**

$$W_{el,z,bottom} = \frac{I}{y} = \frac{17.0}{2.5}$$ → **$W_{el,z,bottom} = 6.9\ m^3$**

A-5.5.3 OPENING TYPE 1 MASONRY CRUSHING CHECK FOR REDUCED PIER SIZE (CONT.)

ANALYSIS

Factored vertical loads N_{ed} = 1.35 x 5,871 + 1.5 x 846 **N_{ed} = 9,194 kN**

Component vertical loads from Section 5.3.2

$P_{1,Gk}$ =	1,545 kN	$H_{T,Gk}$ =	710 kN
$P_{2,Gk}$ =	2,076 kN	$H_{T,Qk}$ =	160 kN
$P_{3,Gk}$ =	2,249 kN		
$P_{1,Qk}$ =	405 kN		
$P_{2,Qk}$ =	306 kN		
$P_{3,Qk}$ =	136 kN		

Factored restoring moment

Taking moments around centroid of 'T' section

M_{ed} = 1.35 x (1,545 x 2.5 + 2,076 x 1.58 - 2,249 x 0.58 - 710 x 8.7)
+
1.5 x (405 x 2.5 + 306 x 1.58 - 136 x 0.58 - 160 x 8.7)

= -455 + 33.6 **M_{ed} = -421.7 kNm**

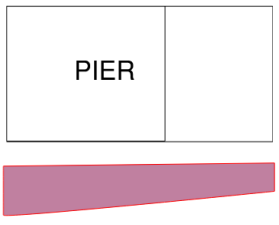
$$\text{Applied stress} = \frac{N_{ed}}{A} +/- \frac{M_{ed}}{W_{el,z}}$$

$$\text{Axial Stress} = \frac{N_{ed}}{A} = \frac{9{,}194}{12.51} = 0.74 \text{ N/mm}^2$$

$$\text{Moment stress in flange (top)} = \frac{M_{ed}}{W_{el,z,top}} = \frac{422}{9.37} = 0.05 \text{ N/mm}^2$$

$$\text{Moment stress in leg (bottom)} = \frac{M_{ed}}{W_{el,z,top}} = \frac{422}{6.85} = 0.06 \text{ N/mm}^3$$

APPLIED STRESS SUMMARY



σ_{Ed} = 0.67 N/mm^2

σ_{Ed} = 0.80 N/mm^2

By inspection masonry remains in compression and has significant capacity for additional overturning moments

MASONRY CAPACITY ASSESSMENT

Due to the scale of the masonry walls the applied peak stresses will be compared to the masonry capacity. As the moments has been accounted for in the stress block, and given the scale of the structure, the simplified approach allow for eccentircities in EC6 will not be adopted.

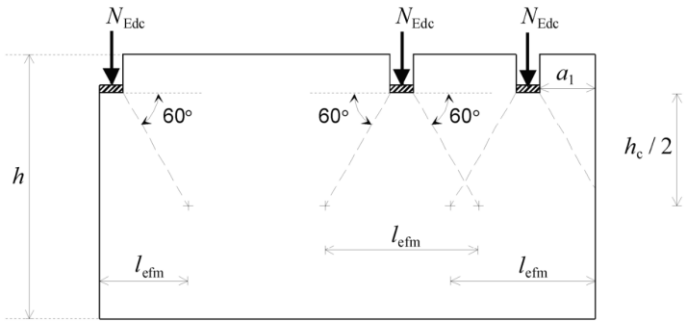
σ_{Rd} = 1.64 N/mm^2

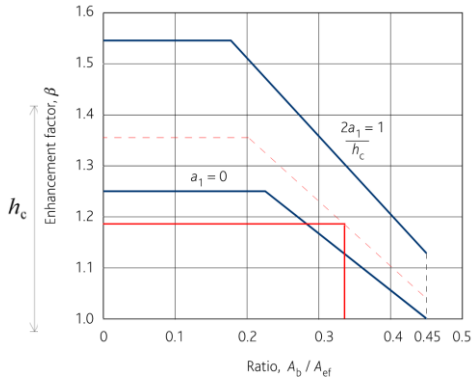
(U.R. = 0.48 < 1)

THEREFORE EXISTING PIERS ADEQUATE FOR REDUCTION DUE TO OPENINGS

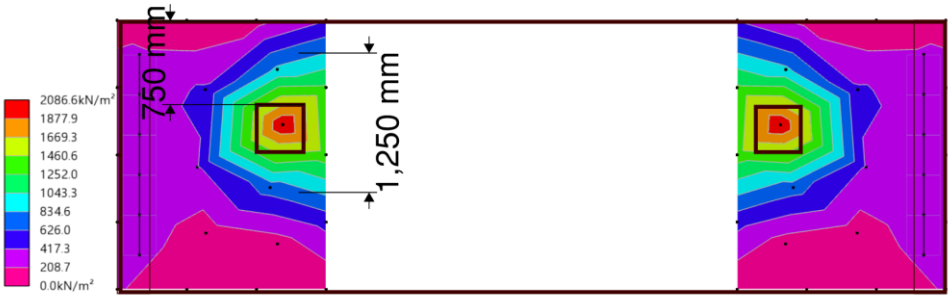
A-5.5.4 OPENING TYPE 2 MASONRY STRESS CHECK UNDER SPREADER BEAM

By inspection of the bearing pressure plots the spreader is effectively acting as a padstone inset a_1 from the edge of wall.





First-order non-linear - 2 STR - 1.35Gk + 1.5Qk + 0.75Wky
Panel Bearing pressure (mat) : [0.0/2086.6kN/m²]



Based on the bearing plots the 'effective' padstone is circa 2.45m x1.25m	A_b =	3.1	m^2
Based on the bearing plots the edge of 'effective' padstone is circa	a_1 =	0.75	m
The wall is founded at circa 4.5m below ground. Conservatively take	h_c =	3.8	m
Effective length at $h_c/2 = h_c/2+1.1+a_1$	l_{efm} =	3.8	m
Effective bearing area of wall at $h_c/2$	A_{ef} =	9.2	m
Bearing Ratio	$\frac{A_b}{A_{ef}}$ =	0.33	
	$\frac{2a_1}{h_c}$ =	0.39	

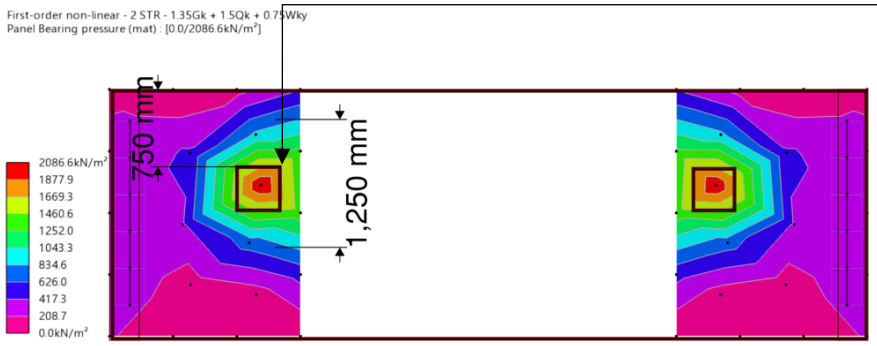
This calculation refers to a mass masonry structure, not at risk of bursting. The recommended value of β=1.0 for Group 2,3 and 4 masonry units is based on slender walls and not considered applicable in this situation.

Therefore bearing stress enhancement factor from graph	β =	1.19	
Stress due to design load	$\sigma_{b,Rd}$ =	$\frac{\beta f_k}{\gamma_m}$	
Refer to Section 5.0 for characteristic strength	f_k =	2.5	N/mm^2
Refer to Section 5.0 for material factors	γ_m =	1.52	
	$\sigma_{b,Rd}$ =	**2.0**	**N/mm^2**

Refer to extract for structural designer model for bearing pressure under the required load combination (next page).

A-5.5.4 OPENING TYPE 2 MASONRY STRESS CHECK UNDER SPREADER BEAM (CONT.)

Note: The peak applied stress given in the Tekla analysis is overly conservative, as it disregards the beneficial spreading effect of the column baseplate. Therefore peak stress is taken at the edge of the baseplate as shown below.



σ_{Ed} = 1.9 N/mm^2

$\sigma_{b,Rd}$ = 2.0 N/mm^2

(U.R. = 0.97 < 1)

Therefore bearing beneath spreader foundation adequate

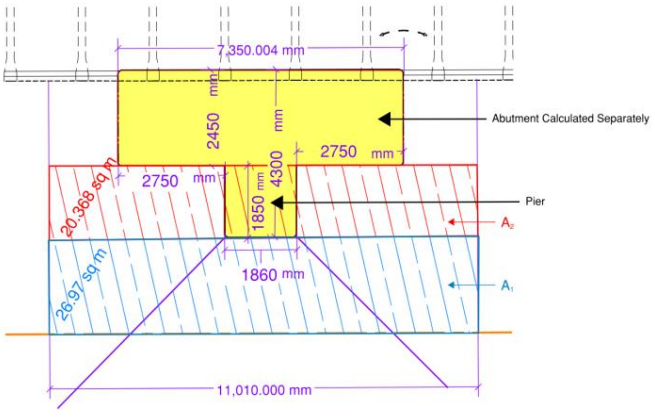
Note: the above assessment is very conservative as the peak applied stress is being compared against the general bearing capacity. In reality the spreader and upstand are very stiff and the bearing pressure will be more evenly distributed and will be circa 1-1.2N/mm^2.

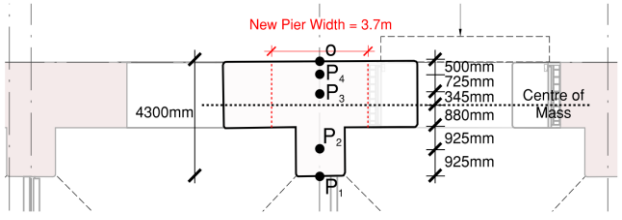
A-5.5.5 OPENING TYPE 1 ABUTMENT OVERTURNING CHECK FOR REDUCED SIZE

Abutment Opening Type 2 supports a greater vertical load from the existing mesh grillage area above. Therefore by inspection, the overturning force on this abutment is less critical than that acting on Opening Type 1. Therefore refer to Section A-5.5.2 for overturning check.

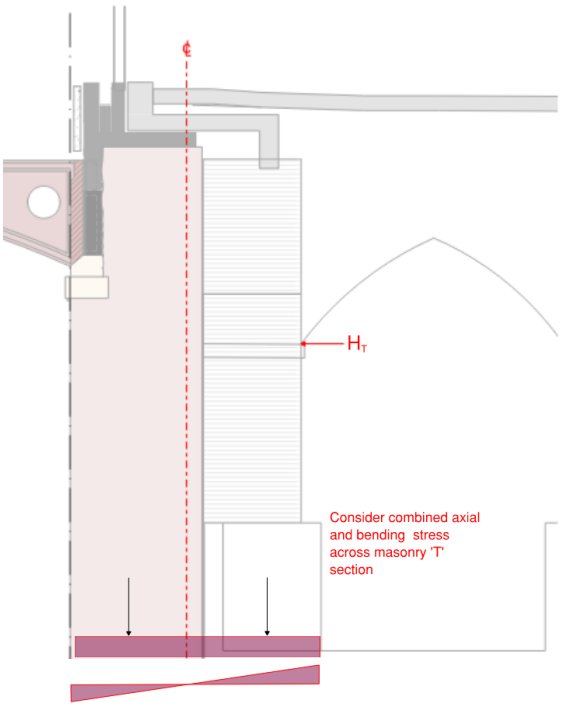
A-5.5.6 OPENING TYPE 2 MASONRY CRUSHING CHECK FOR REDUCED PIER SIZE

To assess the reduced pier size consider the resulting compressive stresses across the section, based on the 'T' section.





PLAN



SECTION

A-5.5.6 OPENING TYPE 2 MASONRY CRUSHING CHECK FOR REDUCED PIER SIZE (CONT.)

PIER PROPERTIES (Refer to section A-5.3.3)

A_{pier} = 12.51 m^2
y = 1.82 m
I_{zz} = 17.0 m^4
$W_{el,z,top}$ = 9.4 m^3
$W_{el,z,bottom}$ = 6.9 m^3

ANALYSIS

Component vertical loads from Section 5.3.2

$P_{1,Gk}$ =	1,545	kN	$H_{T,Gk}$ =	710	kN
$P_{2,Gk}$ =	2,076	kN	$H_{T,Qk}$ =	160	kN
$P_{3,Gk}$ =	2,249	kN			
$P_{1,Qk}$ =	405	kN			
$P_{2,Qk}$ =	306	kN			
$P_{3,Qk}$ =	136	kN			

In addition to the above loads, the abutments bordering Opening Type 1 support an additional load from the existing bridge (P_4), which is applied at 500mm from the point of overturning.

From Section A-5.1.3:

Total line load from bridge = 112.4 kN/m

$P_{4,Gk}$ = 112.4 kN/m x 11.01 m = 1,237 kN

From Section A-5.1.4:

$P_{4,Qk}$ = 12.5 kN/m x 11.01 m = 138 kN

$P_{Total,Qk}$ = 7,108
$P_{Total,Gk}$ = 984

Factored vertical loads N_{ed} = 1.35 x 7,108 + 1.5 x 984 **N_{ed} = 11,071 kN**

Factored restoring moment

Taking moments around centroid of 'T' section

M_{ed} = 1.35 x (1,545 x 2.5 + 2,076 x 1.58 - 2,249 x 0.58 - 1,237 x 0.5 - 710 x 8.7)
+
1.5 x (405 x 2.5 + 306 x 1.58 - 136 x 0.58 - 138 x 0.5 - 160 x 8.7)

= -1,290 + -70 **M_{ed} = -1,360 kNm**

$$\text{Applied stress} = \frac{N_{ed}}{A} \pm \frac{M_{ed}}{W_{el,z}}$$

$$\text{Axial Stress} = \frac{N_{ed}}{A} = \frac{11,071}{12.51} = 0.89 \text{ N/mm}^2$$

$$\text{Moment stress in flange (top)} = \frac{M_{ed}}{W_{el,z,top}} = \frac{1,360}{9.37} = 0.15 \text{ N/mm}^2$$

$$\text{Moment stress in leg (bottom)} = \frac{M_{ed}}{W_{el,z,top}} = \frac{1,360}{6.85} = 0.20 \text{ N/mm}^3$$

A-5.5.6 OPENING TYPE 2 MASONRY CRUSHING CHECK FOR REDUCED PIER SIZE (CONT.)

APPLIED STRESS SUMMARY



σ_{Ed} = **1.08** N/mm^2

σ_{Ed} = **0.69** N/mm^2

By inspection masonry remains in compression and has significant capacity for additional overturning moments

MASONRY CAPACITY ASSESSMENT

Due to the scale of the masonry walls the applied peak stresses will be compared to the masonry capacity. As the moments has been accounted for in the stress block, and given the scale of the structure, the simplified approach allow for eccentircities in EC6 will not be adopted.

σ_{Rd} = **1.64** N/mm^2

(U.R. = 0.66 < 1)

THEREFORE EXISTING PIERS ADEQUATE FOR REDUCTION DUE TO OPENINGS