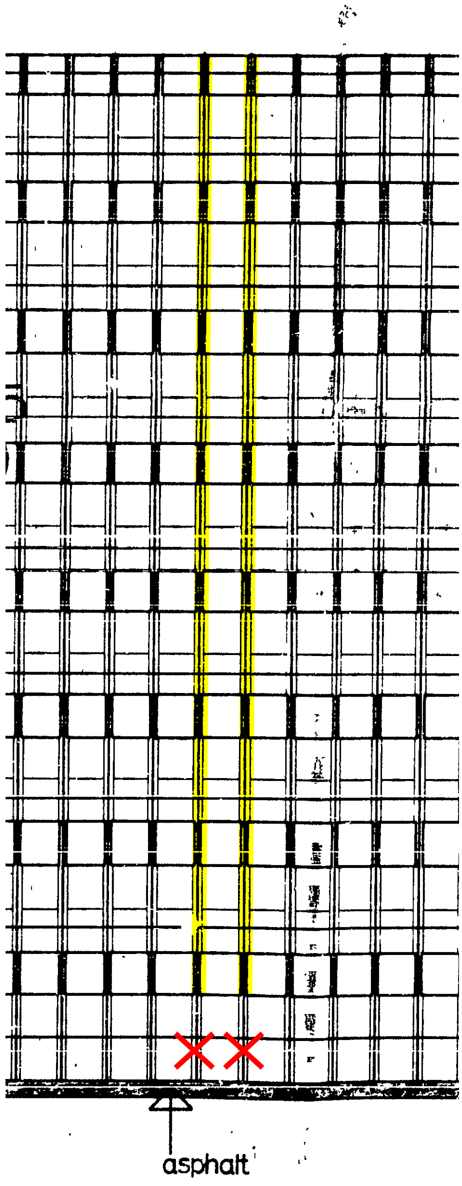
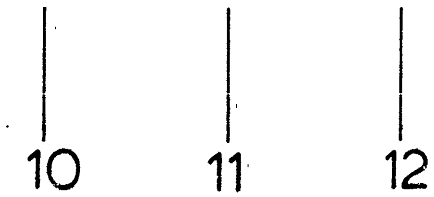
4 DELIVERY OPENING STEELWORK

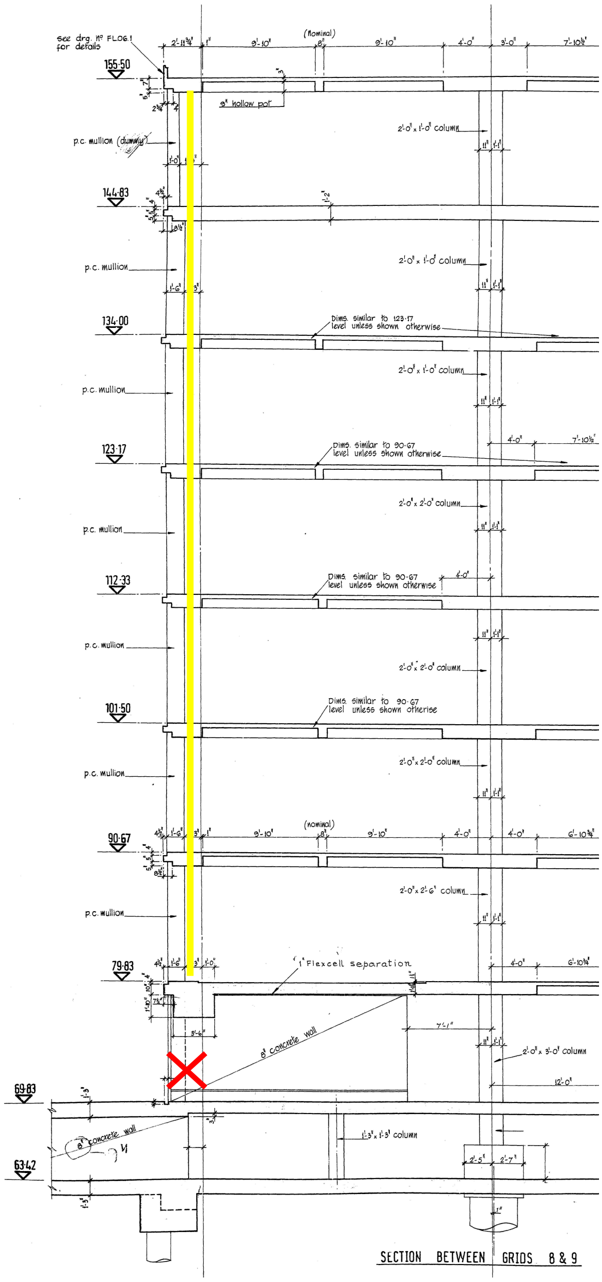
4.1 LOAD TAKEDOWN OF EXISTING COLUMNS

A full load takedown check has been completed in order to determine the axial load in the existing columns which are to be removed to allow for the MRI scanner delivery entrance, this has been based on existing structural information and drawings.



ELEVATION





SECTION BETWEEN GRIDLINE 8&9

Project: Birkbeck University - MRI			Status: INFORMATION	
Date 09/01/18 By PG Checked NP	Job no. 70038590	Section 01	Sheet no. 01	Rev 01

Rev	Date	Details	Tel / Fax
01	09/01/18	Load Takedown Analysis	

Part: Removal of 2 Columns for MRI installation

REF | OUTPUT

7 Floors above, plus roof level

(Plant) Roof DL: 7.0 kN/m² SDL = 0.85 kN/m² LL = 5.0 kN/m²

(Office) Lv5 DL = 8.4 kN/m² SDL = 0.85 kN/m² LL = 3.5 kN/m²

(Office) Lv4 DL = 7.0 kN/m² SDL = 0.85 kN/m² LL = 3.5 kN/m²

(Office) Lv3 DL = 7.0 kN/m² SDL = 0.85 kN/m² LL = 3.5 kN/m²

(Office) Lv2 DL = 7.0 kN/m² SDL = 0.85 kN/m² LL = 3.5 kN/m²

(Office) Lv1 DL = 7.0 kN/m² SDL = 0.85 kN/m² LL = 3.5 kN/m²

(Office) GF DL = 7.0 kN/m² SDL = 0.85 kN/m² LL = 3.5 kN/m²

(Library) LG DL = 8.4 kN/m² SDL = 0.85 kN/m² LL = 5.0 kN/m²

Office Imposed Load = 2.5 kN/m²
Library Imposed Load = 4.0 kN/m²
Lightweight Plant Load = 5.0 kN/m²
Services & Finishes = 0.85 kN/m²
Demountable Partitions = 1.0 kN/m² (All except Roof)
0.35m thk Slab = 0.35 x 24 = 8.4 kN/m²
Hollow Pot Floor Construction = 0.35 x 20kN/m³ = 7 kN/m²

Tributary Area = 4.0 m x 1.19 m = 4.76 m² (Considered Area per Column)
≈ 4.8 m²

Dead Load = [2 x (8.4 + 0.85) x 4.8] + [6 x (7.0 + 0.85) x 4.8]
= 314.8 kN (unfactored)
x1.35 = 425.1 kN (factored)

Live Load = 2(5.0 x 4.8) + 6(3.5 x 4.8)
= 148.8 kN (unfactored)
x1.5 = 223.2 kN (factored)

S/W of Column = 0.457 x 0.230 x 24m High x 25kN/m² = 65 kN (Unfactored)
x 1.35 = 90kN (Factored)

∴ Total (Per Column) = 542 kN (unfactored) ⇒ 550 kN
= 739 kN (factored) ⇒ 750 kN

Project UCLH, Birbeck University - MRI			Status Information	
Date 09/01/2018 By NP Checked MB	Job no. 70038590	Section	Sheet no. 1 of 1	Rev 01

Rev	Date	Details
01	09/01/2018	

Part

Delivery Opening Concrete to Steel Through Bolts

REF OUTPUT

4No. Through Bolts are required for connection of the cut concrete columns to the steel PFC's. A total design load of 750kN per column is required to be resisted by the through bolts. As a conservative approach, it is considered that 2 x 4No. anchor bolts in each side of the column are taking 375kN each in single shear.

From the Hilti load table below, for a HIT-V-5.8 with HIT-RE 500-SD in C20/25 Concrete the design shear resistance is 112 kN per anchor.

Hilti HIT-RE 500-SD with HIT-V rod

Basic loading data (for a single anchor)

All data in this section applies to For details see Simplified design method

- Correct setting (See setting instruction)
- No edge distance and spacing influence
- *Steel* failure
- Base material thickness, as specified in the table
- One typical embedment depth, as specified in the table
- One anchor material, as specified in the tables
- Concrete C 20/25, $f_{ck,cube}$ = 25 N/mm²
- Temperate range I (min. base material temperature -40℃, max. long term/short term base material temperature: +24℃/40℃)
- Installation temperature range +5℃ to +40℃

Embedment depth [a)] and base material thickness for the basic loading data. Mean ultimate resistance, characteristic resistance, design resistance, recommended loads.

Anchor size	M8	M10	M12	M16	M20	M24	M27	M30
Typical embedment depth [mm]	80	90	110	125	170	210	240	270
Base material thickness [mm]	110	120	140	165	220	270	300	340

a) The allowed range of embedment depth is shown in the setting details. The corresponding load values can be calculated according to the simplified design method.

Mean ultimate resistance: concrete C 20/25 – $f_{ck,cube}$ = 25 N/mm², anchor HIT-V 5.8

			Data according ETA-07/0260, issue 2009-01-12							
Anchor size			M8	M10	M12	M16	M20	M24	M27	M30
Non cracked concrete										
Tensile $N_{Ru,m}$	HIT-V 5.8	[kN]	*18,9*	*30,5*	*44,1*	*83,0*	*129,2*	*185,9*	*241,5*	*295,1*
Shear $V_{Ru,m}$	HIT-V 5.8	[kN]	*9,5*	*15,8*	*22,1*	*41,0*	*64,1*	*92,4*	*120,8*	*147,0*
Cracked concrete										
Tensile $N_{Ru,m}$	HIT-V 5.8	[kN]	*18,9*	*30,5*	*44,1*	65,2	110,8	146,1	196,0	226,2
Shear $V_{Ru,m}$	HIT-V 5.8	[kN]	*9,5*	*15,8*	*22,1*	*41,0*	*64,1*	*92,4*	*120,8*	*147,0*

Characteristic resistance: concrete C 20/25 – $f_{ck,cube}$ = 25 N/mm², anchor HIT-V 5.8

			Data according ETA-07/0260, issue 2009-01-12							
Anchor size			M8	M10	M12	M16	M20	M24	M27	M30
Non cracked concrete										
Tensile N_{Rk}	HIT-V 5.8	[kN]	*18,0*	*29,0*	*42,0*	70,6	111,9	153,7	187,8	224,0
Shear V_{Rk}	HIT-V 5.8	[kN]	*9,0*	*15,0*	*21,0*	*39,0*	*61,0*	*88,0*	*115,0*	*140,0*
Cracked concrete										
Tensile N_{Rk}	HIT-V 5.8	[kN]	*16,1*	*22,6*	*31,1*	44,0	74,8	109,6	132,3	152,7
Shear V_{Rk}	HIT-V 5.8	[kN]	*9,0*	*15,0*	*21,0*	*39,0*	*61,0*	*88,0*	*115,0*	*140,0*

Design resistance: concrete C 20/25 – $f_{ck,cube}$ = 25 N/mm², anchor HIT-V 5.8

			Data according ETA-07/0260, issue 2009-01-12							
Anchor size			M8	M10	M12	M16	M20	M24	M27	M30
Non cracked concrete										
Tensile N_{Rd}	HIT-V 5.8	[kN]	12,0	19,3	28,0	33,6	53,3	73,2	89,4	106,7
Shear V_{Rd}	HIT-V 5.8	[kN]	7,2	12,0	16,8	31,2	48,8	70,4	92,0	112,0
Cracked concrete										
Tensile N_{Rd}	HIT-V 5.8	[kN]	8,9	12,6	17,3	20,9	35,6	52,2	63,0	72,7
Shear V_{Rd}	HIT-V 5.8	[kN]	7,2	12,0	16,8	31,2	48,8	70,4	92,0	112,0

As 8 x 112 = 896 kN > 750 kN therefore OKAY
Provide M30 (8.8) HIT-V Threaded rod with HIT-RE 500-SD

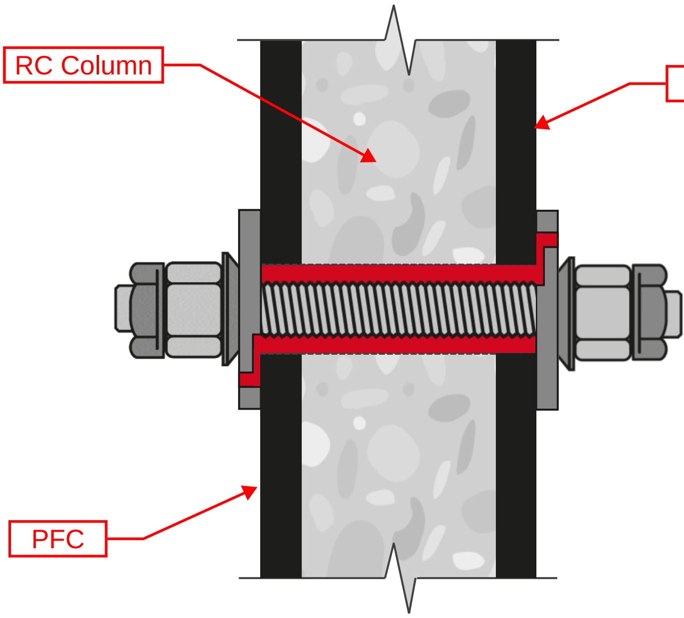
Project UCLH, Birbeck University - MRI			Status Information	
Date 09/01/2018 By NP Checked MB	**Job no.** 70038590	**Section**	**Sheet no.** 1 of 1	**Rev** 01

Rev	Date	Details
01	09/01/2018	

Part
Delivery Opening Concrete to Steel Through Bolts

REF OUTPUT

Installation of the through bolts and resin should be as follows:



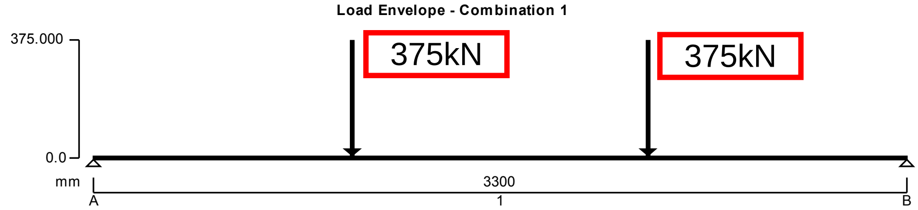
- Drill hole through RC Column.
- Clean Bore hole as per manufactures requirements
- Install road with Hilti filling washer set on each side.
- Dispense HIT-RE 500 Resin through hole until entire cavity is filled.
- Tighten nut to required torque setting.

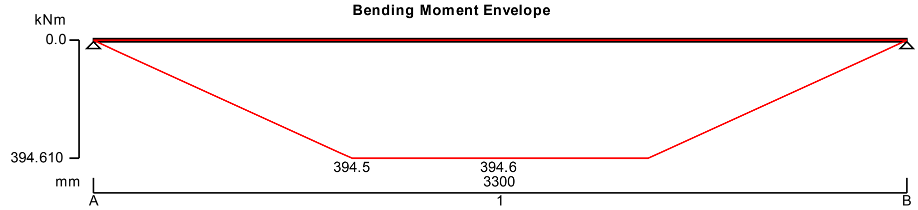
STEEL BEAM ANALYSIS & DESIGN (EN1993-1-1:2005)

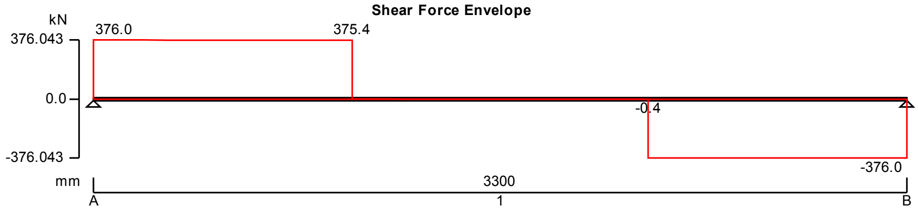
In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009 and the UK national annex

TEDDS calculation version 3.0.13

NB: Two PFC sections to be used (one either side of the column ot be removed columns). Therefore half the total load per column (750kN factored) will be used in the design of the PFC.







Support conditions

Support A	Vertically restrained Rotationally free
Support B	Vertically restrained Rotationally free

Applied loading

Beam loads	Permanent point load 375 kN at 1050 mm Permanent point load 375 kN at 2250 mm Permanent self weight of beam × 1	Factored Loads

Load combinations

Load combination 1	Support A	Permanent × 1.00 Variable × 1.00
	Span 1	Permanent × 1.00 Variable × 1.00
	Support B	Permanent × 1.00 Variable × 1.00

Analysis results

Maximum moment M_{max} = **394.6** kNm M_{min} = **0** kNm

Maximum shear V_{max} = **376** kN V_{min} = **-376** kN

Deflection δ_{max} = **10.1** mm δ_{min} = **0** mm

Maximum reaction at support A R_{A_max} = **376** kN R_{A_min} = **376** kN

Unfactored permanent load reaction at support A $R_{A\_Permanent}$ = **376** kN

Maximum reaction at support B R_{B_max} = **376** kN R_{B_min} = **376** kN

Unfactored permanent load reaction at support B $R_{B\_Permanent}$ = **376** kN

Section details

Section type **UKPFC 430x100x64 (Tata Steel Advance)**

Steel grade **S355**

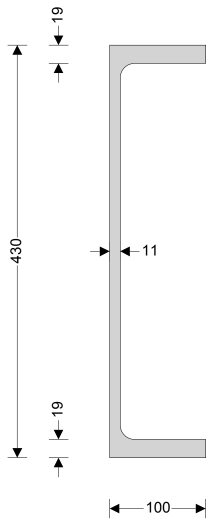
EN 10025-2:2004 - Hot rolled products of structural steels

Nominal thickness of element $t = \max(t_f, t_w)$ = **19.0** mm

Nominal yield strength f_y = **345** N/mm^2

Nominal ultimate tensile strength f_u = **470** N/mm^2

Modulus of elasticity E = **210000** N/mm^2



Partial factors - Section 6.1

Resistance of cross-sections γ_{M0} = **1.00**

Resistance of members to instability γ_{M1} = **1.00**

Resistance of tensile members to fracture γ_{M2} = **1.10**

Lateral restraint

Span 1 has full lateral restraint

Effective length factors

Effective length factor in major axis K_y = **1.000**

Effective length factor in minor axis K_z = **1.000**

Effective length factor for torsion $K_{LT.A}$ = **1.000**

$K_{LT.B}$ = **1.000**

Classification of cross sections - Section 5.5

$\varepsilon = \sqrt{[235 \text{ N/mm}^2 / f_y]}$ = **0.83**

Internal compression parts subject to bending - Table 5.2 (sheet 1 of 3)

Width of section c = d = **362** mm

$c / t_w = 39.9 \times \varepsilon <= 72 \times \varepsilon$ Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section $c = b - t_w - r_1 = \mathbf{74}$ mm

$c / t_f = 4.7 \times \varepsilon <= 9 \times \varepsilon$ Class 1

Section is class 1

Check shear - Section 6.2.6

Height of web $h_w = h - 2 \times t_f = \mathbf{392}$ mm

Shear area factor $\eta = \mathbf{1.000}$

$h_w / t_w < 72 \times \varepsilon / \eta$

Shear buckling resistance can be ignored

Design shear force $V_{Ed} = max(abs(V_{max}), abs(V_{min})) = \mathbf{376}$ kN

Shear area - cl 6.2.6(3) $A_v = A - 2 \times b \times t_f + (t_w + r_1) \times t_f = \mathbf{4903}$ mm^2

Design shear resistance - cl 6.2.6(2) $V_{c,Rd} = V_{pl,Rd} = A_v \times (f_y / \sqrt{[3]}) / \gamma_{M0} = \mathbf{976.5}$ kN

PASS - Design shear resistance exceeds design shear force

Check bending moment major (y-y) axis - Section 6.2.5

Design bending moment $M_{Ed} = max(abs(M_{s1\_max}), abs(M_{s1\_min})) = \mathbf{394.6}$ kNm

Design bending resistance moment - eq 6.13 $M_{c,Rd} = M_{pl,Rd} = W_{pl.y} \times f_y / \gamma_{M0} = \mathbf{421.6}$ kNm

PASS - Design bending resistance moment exceeds design bending moment

Check vertical deflection - Section 7.2.1

Consider deflection due to permanent and variable loads

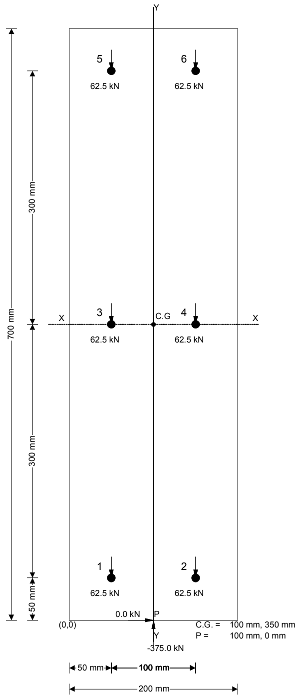
Limiting deflection $\delta_{lim} = L_{s1} / 250 = \mathbf{13.2}$ mm

Maximum deflection span 1 $\delta = max(abs(\delta_{max}), abs(\delta_{min})) = \mathbf{10.085}$ mm

PASS - Maximum deflection does not exceed deflection limit

BOLT GROUP ANALYSIS

Tedds calculation version 1.0.02



Geometry of bolt group

Number of rows	R = **3**
Number of columns	C = **2**
Pitch distance	S_x = **100** mm
Gauge distance	S_y = **300** mm
Edge distance in vertical direction	d_y = **50** mm
Edge distance in horizontal direction	d_x = **50** mm

Load data

Vertical load applied on bolt group	P_y = **-375.000** kN
Horizontal load applied on bolt group	P_x = **0.000** kN
X coordinate of vertical force	X = **100** mm
Y coordinate of horizontal force	Y = **0** mm

Center of gravity of bolt group

X distance of center of bolt group	$X_c = ((C - 1) \times S_x) / 2 + d_x$ = **100** mm
Y distance of center of bolt group	$Y_c = ((R - 1) \times S_y) / 2 + d_y$ = **350** mm

Load eccentricity from center of gravity of bolt group

Eccentricity of vertical load from C.G.	$e_x = abs(X - X_c)$ = **0** mm
Eccentricity of horizontal load from C.G.	$e_y = abs(Y - Y_c)$ = **350** mm
Moment about center of gravity	$M = P_x \times (Y - Y_c) - P_y \times (X - X_c)$ = **0.000** kNm

Bolt number	Bolt distance from centre of gravity		Direct shear		Torsional shear		Total force
	X_i (mm)	Y_i (mm)	P_{dx} (kN)	P_{dy} (kN)	P_{tx} (kN)	P_{ty} (kN)	(kN)
1	-50	-300	0.0	62.5	0.0	0.0	62.5
2	50	-300	0.0	62.5	0.0	0.0	62.5
3	-50	0	0.0	62.5	0.0	0.0	62.5
4	50	0	0.0	62.5	0.0	0.0	62.5
5	-50	300	0.0	62.5	0.0	0.0	62.5
6	50	300	0.0	62.5	0.0	0.0	62.5

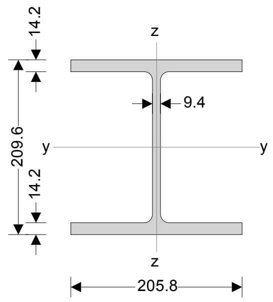
Diameter of bolt	Tensile stress area	Tension resistance	Shear resistance	
			Single shear	Double shear
d mm	A_s mm²	$F_{t,Rd}$ kN	$F_{v,Rd}$ kN	$2 \times F_{v,Rd}$ kN
12	84.3	48.6	32.4	64.7
16	157	90.4	60.3	121
20	245	141	94.1	188
24	353	203	136	271
30	561	323	215	431

Extract from TATA Steel Blue Book

STEEL COLUMN DESIGN (EN 1993-1-1)

In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009 and the UK national annex

TEDDS calculation version 1.0.12



NB. In order to accommodate the width of the removed columns, two UC sections will be provided See UCL-WSP-00-GF-DR-S-200104.

Column and loading details

Column details

Column section	**UKC 203x203x60**
System length for buckling about y axis	L_y = **3000** mm
System length for buckling about z axis	L_z = **3000** mm

Sway

The column is not part of a sway frame in the direction of the z axis

The column is not part of a sway frame in the direction of the y axis

Column loading

Axial load	N_{Ed} = **375** kN (Compression)
Moment about y axis at end 1	$M_{y,Ed1}$ = **0.0** kNm
Moment about y axis at end 2	$M_{y,Ed2}$ = **0.0** kNm
Moment about z axis at end 1	$M_{z,Ed1}$ = **0.0** kNm
Moment about z axis at end 2	$M_{z,Ed2}$ = **0.0** kNm
Shear force parallel to z axis	$V_{z,Ed}$ = **0** kN
Shear force parallel to y axis	$V_{y,Ed}$ = **0** kN

Material details

Steel grade	**S355**
Yield strength	f_y = **355** N/mm^2
Ultimate strength	f_u = **470** N/mm^2
Modulus of elasticity	E = **210** kN/mm^2
Poisson's ratio	ν = **0.3**
Shear modulus	$G = E / [2 \times (1 + \nu)]$ = **80.8** kN/mm^2

Buckling length for flexural buckling about y axis

End restraint factor	K_y = **1.000**
Buckling length	$L_{cr\_y} = L_y \times K_y$ = **3000** mm

Buckling length for flexural buckling about z axis

End restraint factor	K_z = **1.000**
Buckling length	$L_{cr\_z} = L_z \times K_z$ = **3000** mm

Section classification

Web section classification (Table 5.2)

Coefficient depending on f_y	$\varepsilon = \sqrt{(235 \text{ N/mm}^2 / f_y)}$ = **0.814**
Depth between fillets	$c_w = h - 2 \times (t_f + r)$ = **160.8** mm
Ratio of c/t	$ratio_w = c_w / t_w$ = **17.11**
Length of web taken by axial load	$l_w = \min(N_{Ed} / (f_y \times t_w), c_w)$ = **112.4** mm
For class 1 & 2 proportion in compression	$\alpha = (c_w/2 + l_w/2) / c_w$ = **0.849**
Limit for class 1 web	$Limit_{1w} = (396 \times \varepsilon) / (13 \times \alpha - 1)$ = **32.08**

The web is class 1

Flange section classification (Table 5.2)

Outstand length	$c_f = (b - t_w) / 2 - r$ = **88.0** mm
Ratio of c/t	$ratio_f = c_f / t_f$ = **6.20**
Limit for class 1 flange	$Limit_{1f} = 9 \times \varepsilon$ = **7.32**
Limit for class 2 flange	$Limit_{2f} = 10 \times \varepsilon$ = **8.14**
Limit for class 3 flange	$Limit_{3f} = 14 \times \varepsilon$ = **11.39**

The flange is class 1

Overall section classification

The section is class 1

Resistance of cross section (cl. 6.2)

Compression (cl. 6.2.4)

Design force	N_{Ed} = **375** kN
Design resistance	$N_{c,Rd} = N_{pl,Rd} = A \times f_y / \gamma_{M0}$ = **2711** kN

PASS - The compression design resistance exceeds the design force

Buckling resistance (cl. 6.3)

Yield strength for buckling resistance	f_y = **355** N/mm^2

Flexural buckling about y axis

Elastic critical buckling force	$N_{cr,y} = \pi^2 \times E \times I_y / L_{cr\_y}^2$ = **14104** kN
Non-dimensional slenderness	$\bar{\lambda}_y = \sqrt{(A \times f_y / N_{cr,y})}$ = **0.438**
Buckling curve (Table 6.2)	**b**
Imperfection factor (Table 6.1)	α_y = **0.34**
Parameter Φ	$\Phi_y = 0.5 \times [1 + \alpha_y \times (\bar{\lambda}_y - 0.2) + \bar{\lambda}_y^2]$ = **0.637**
Reduction factor	$\chi_y = \min(1.0, 1 / [\Phi_y + \sqrt{(\Phi_y^2 - \bar{\lambda}_y^2)}])$ = **0.911**
Design buckling resistance	$N_{b,y,Rd} = \chi_y \times A \times f_y / \gamma_{M1}$ = **2468.7** kN

PASS - The flexural buckling resistance about the y axis exceeds the design axial load

Flexural buckling about z axis

Elastic critical buckling force	$N_{cr,z} = \pi^2 \times E \times I_z / L_{cr\_z}^2$ = **4755** kN
Non-dimensional slenderness	$\bar{\lambda}_z = \sqrt{(A \times f_y / N_{cr,z})}$ = **0.755**
Buckling curve (Table 6.2)	**c**
Imperfection factor (Table 6.1)	α_z = **0.49**
Parameter Φ	$\Phi_z = 0.5 \times [1 + \alpha_z \times (\bar{\lambda}_z - 0.2) + \bar{\lambda}_z^2]$ = **0.921**
Reduction factor	$\chi_z = \min(1.0, 1 / [\Phi_z + \sqrt{(\Phi_z^2 - \bar{\lambda}_z^2)}])$ = **0.690**
Design buckling resistance	$N_{b,z,Rd} = \chi_z \times A \times f_y / \gamma_{M1}$ = **1871.6** kN

PASS - The flexural buckling resistance about the z axis exceeds the design axial load

Torsional and torsional-flexural buckling (cl. 6.3.1.4)

Torsional buckling length factor: K_T = **1.00**

Effective buckling length: $L_{cr\_T} = K_T \times \max(L_y, L_z)$ = **3000** mm

Distance from shear ctr to centroid along y axis: y_0 = **0.0** mm

Distance from shear ctr to centroid along z axis: z_0 = **0.0** mm

$i_0 = \sqrt{(i_y^2 + i_z^2 + y_0^2 + z_0^2)}$ = **103.5** mm

$\beta_T = 1 - (y_0 / i_0)^2$ = **1.000**

Elastic critical torsional buckling force: $N_{cr,T} = 1 / i_0^2 \times (G \times I_t + \pi^2 \times E \times I_w / L_{cr\_T}^2)$ = **7790** kN

Elastic critical torsional-flexural buckling force: $N_{cr,TF} = N_{cr,y}/(2\times\beta_T)\times[1+N_{cr,T}/N_{cr,y}-\sqrt{[(1-N_{cr,T}/N_{cr,y})^2 + 4\times(y_0/i_0)^2\times N_{cr,T}/N_{cr,y}]}]$

$N_{cr,TF}$ = **7790** kN

Non-dimensional slenderness: $\bar{\lambda}_T = \sqrt{(A \times f_y / \min(N_{cr,T}, N_{cr,TF}))}$ = **0.590**

Buckling curve (Table 6.2): **c**

Imperfection factor (Table 6.1): α_T = **0.49**

Parameter Φ: $\Phi_T = 0.5 \times [1 + \alpha_T \times (\bar{\lambda}_T - 0.2) + \bar{\lambda}_T^2]$ = **0.770**

Reduction factor: $\chi_T = \min(1.0, 1 / [\Phi_T + \sqrt{(\Phi_T^2 - \bar{\lambda}_T^2)}])$ = **0.791**

Design buckling resistance: $N_{b,T,Rd} = \chi_T \times A \times f_y / \gamma_{M1}$ = **2145.5** kN

PASS - The torsional/torsional-flexural buckling resistance exceeds the design axial load

Minimum buckling resistance

Minimum buckling resistance: $N_{b,Rd} = \min(N_{b,y,Rd}, N_{b,z,Rd}, N_{b,T,Rd})$ = **1871.6** kN

PASS - The axial load buckling resistance exceeds the design axial load

It is considered that the ground floor slab is suspended. Therefore, to transfer the large vertical loads from the delivery opening columns to the lower ground level, additional 450x300 RC columns will be provided with reinforcement details equivalent to that of the RC stub columns detailed in section 2.2. For additional information see UCL-WSP-00-GF-DR-S-200102.