Structural Scheme Design Report

for

155, Drummond Street

London, NW1 2FB

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February 2019

Structural Calculations

for

155 Drummond Street
London
NW1 2FB

for

Adrian Cova & Angela Rodriguez-Cova
155 Drummond Street
NW1 2FB

Job No 863

Rev	Date	Notes
0	01.02.19	First Issue

1. CALCULATION PLAN

This report contains the structural engineering calculations for the proposed development at 155 Drummond Street.

The project consists of the entire demolition of three upper floors in an existing four storey building located at No.155 Drummond Street and construction of seven new storey plus a roof terrace. To minimize the weight of the new structures the existing masonry walls will be replaced with a steel frame. The existing RC ground and first floor structure will be maintained, while the mezzanine between ground and first floor will be re-built. It is also proposed to excavate the lower ground floor void to create a new basement to allocate utility spaces and a cycling parking.

1.1 SUMMARY OF THE PROPOSED STRUCTURE

Maximum plan dimensions	10.4m x 9.2m
Storeys	Basement, Ground, Mezzanine, First, Second, Third, Fourth, Fifth, Sixth and Seventh floor and Roof terrace.
Maximum height	26.5m (pavement level to terrace parapet)
Footings	Strip foundations on mass concrete bases and ground bearing slab.
Ground and first floor	Existing RC slabs.
Vertical structure	RC walls at basement level, RC columns and brick walls at ground floor and steel columns above first floor.
New floors and roof structure	Timber joists supported by steel beams
Lateral stability	Masonry wall and RC columns at ground floor and steel bracings above first floor.

1.2 IMPOSED LOADS ON FLOORS

The following imposed loads must be allowed for to BS6399 part 1 categories:

Class A – Domestic and Residential
First to 7th floor (plus partitions allowance) 1.50 kN/m^2 + 1 kN/m^2

Class C3 – Communal areas
First to 5th floor 3.00 kN/m^2

Class F – Parking for cars
Ground floor 2.50 kN/m^2

All internal non-load bearing partitions are metal/timber stud with plasterboard or OSB and an imposed load allowance is shown for these of 1kN/m^2.

1.3 IMPOSED LOADS ON ROOFS

The following imposed loads on roofs must be allowed for to BS6399 part 3.

Roof with access	1.50 kN/m^2

1.4 WIND LOADS

Wind loads are calculated to BS 6399 part 2.

1.5 OVERALL STRUCTURAL STABILITY

From first floor to top level the stability will be guaranteed by steel cross bracings placed mainly around the perimeter, while at ground floor the stability is guaranteed by the existing masonry walls, the existing RC columns and the new RC lift case.

1.6 ROBUSTNESS

The Building Regulations have the following requirement:

Disproportionate Collapse
A3. The building shall be constructed so that in the event of an accident the building will not suffer collapse to an extent disproportionate to the cause.

The Building Regulations Approved Document A – Structure 2010 edition, classifies the building as a Class 2B building and gives the following guidance for compliance with the Building Regulations.

For Class 2B buildings – Provide effective horizontal ties, as described in the Codes and Standards listed under paragraph 5.2 for framed and load-bearing wall construction (the latter being defined in paragraph 5.3 below), together with effective vertical ties, as defined in the Codes and Standards listed under paragraph 5.2, in all supporting columns and walls.

The required vertical and horizontal ties are provided by the steel frame and the steel frame is an inherently robust form of construction.

1.7 FOUNDATIONS

It is proposed to maintain the existing mass concrete strip foundations, enlarging them where necessary. From the original construction drawings and borehole results in the areas nearby the formation level is assumed to be London Clay.

1.8 FLOOR STRUCTURE

Except for the ground and first floor, all the new floors will be of timber joists and steel beams construction. Diaphragm effect will be provided by 18mm plywood boarding screwed and glued on top of joists.

1.9 UNDERGROUND DRAINAGE

All drainage is to be designed in accordance with the Building Regulations requirements.

Storm water and foul water drainage are to be kept separate up to the municipal connection.

Storm water drains will fall at a minimum of 1:100 unless hydraulic calculations justify a lower gradient. Foul water drains will fall at a minimum of 1:40 unless hydraulic calculations justify a lower gradient. Where storm water drains and foul water drains run next to each other, the storm water will be above the foul water.

Drainage system at basement level will be guaranteed with the installation of a new sump pump.

2. RESOURCES

CODES & REFERENCES

BS6399 Pt1 Loadings for buildings. Code of practice for dead and imposed loads.

BS6399 Pt2 Loadings for buildings. Code of practice for wind loads.

BS6399 Pt3 Loadings for buildings. Code of practice for imposed roof loads.

BS5269 Pt2 Structural use of Timber. Code of practice for permissible stress design, materials and workmanship.

BS5628 Pt1 Use of masonry. Structural use of unreinforced masonry.

BS5950 Pt1 Structural use of steelwork in building. Code of practice for design in simple and continuous construction hot rolled sections.

BS8110 Pt1 Structural use of concrete

Manual for the design of plain masonry in building structures – The Institution of Structural Engineers. July 1997.

SOFTWARE

Trimble Tekla suite of design and analysis tools.

Roof

Dead	Finishes and services	0.85 kN/m^2
	Timber joists	0.15 kN/m^2
		1.00 kN/m^2
Imposed		0.75 kN/m^2

Mansard wall (assumed)

Dead	Slates	0.70 kN/m^2
	Battens and felt	0.05 kN/m^2
	12mm plywood sarking	0.08 kN/m^2
	Studs	0.11 kN/m^2
	Insulation	0.01 kN/m^2
	Plasterboard and skim coat	0.18 kN/m^2
		1.13 kN/m^2
Imposed (for angle > 60)		nil

Domestic timber floor

Dead	Finishes and services	1.25 kN/m^2
	Timber joists	0.15 kN/m^2
		1.40 kN/m^2
Imposed		1.50 kN/m^2

Domestic concrete floor

Dead	Finishes and services	2.00 kN/m^2
	200 thk RC slab	4.80 kN/m^2
		6.80 kN/m^2
Imposed		1.50 kN/m^2
	Partition	1.00 kN/m^2
		2.50 kN/m^2

Shops

Dead	Finishes and services	2.00 kN/m^2
	200 thk RC slab	4.80 kN/m^2
		6.80 kN/m^2
Imposed		4.00 kN/m^2
	Partition	1.00 kN/m^2
		5.00 kN/m^2

Balconies

Dead	Finishes and services	3.85 kN/m^2
	200 thk RC slab	4.80 kN/m^2
		8.65 kN/m^2
Imposed		1.50 kN/m^2

External cavity walls

Dead	102.5mm brickwork	2.26 kN/m^2
	Insulation	0.05 kN/m^2
	100mm blockwork	1.80 kN/m^2
	Plasterboard and skim coat	0.25 kN/m^2
		4.36 kN/m^2

Existing ground floor

Dead	50mm asphalt	1.05 kN/m^2
	400 thk RC slab	9.60 kN/m^2
		10.65 kN/m^2
Imposed		2.50 kN/m^2

Proposed Roof Terrace

Dead	Paving	1.20 kN/m^2
	Waterproofing	0.40 kN/m^2
	Boarding	0.13 kN/m^2
	Joists	0.12 kN/m^2
	Insulation	0.05 kN/m^2
	Services	0.05 kN/m^2
	Plasterboard & skim coat	0.25 kN/m^2
		2.20 kN/m^2
Imposed		3.00 kN/m^2

Typical floor

Dead	Finishes	0.15 kN/m^2
	Boarding	0.13 kN/m^2
	Joists	0.12 kN/m^2
	Insulation	0.05 kN/m^2
	Services	0.15 kN/m^2
	Plasterboard & skim coat	0.25 kN/m^2
		0.85 kN/m^2
Imposed		1.50 kN/m^2
	Partitions	1.00 kN/m^2
		2.50 kN/m^2

Proposed first floor

Dead	Finishes	0.15 kN/m^2
	Boarding & battens	0.15 kN/m^2
	Insulation	0.05 kN/m^2
	200mm thk slab	4.80 kN/m^2
	Services	0.15 kN/m^2
	Plasterboard & skim coat	0.25 kN/m^2
		5.55 kN/m^2
Imposed		1.50 kN/m^2
	Partitions	1.00 kN/m^2
		2.50 kN/m^2

Proposed basement floor

Dead	Finishes	0.15 kN/m^2
	Screed	1.00 kN/m^2
	Insulation	0.05 kN/m^2
	Services	0.15 kN/m^2
	400mm thk slab	9.6 kN/m^2
		10.95 kN/m^2
Imposed		1.50 kN/m^2

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Job title: 155 Drummond Street			
Calculations: Proposed Area Loads	Designed: lb	Date: 23/11/2018	Ckd: -

Proposed green roof

Dead		1.93 kN/m2
Imposed (allowing for maintenance of structure above)		1.5 kN/m2

Proposed ground floor (external)

Dead	Paving	1.20 kN/m^2
	Screed	1.80 kN/m^2
	400mm thk slab	9.60 kN/m^2
	Insulation	0.05 kN/m^2
	Services	0.15 kN/m^2
		12.80 kN/m^2
Imposed		2.5 kN/m^2

Proposed ground floor (internal)

Dead	Finishes	0.15 kN/m^2
	Screed	1.80 kN/m^2
	400mm thk slab	9.60 kN/m^2
	Insulation	0.05 kN/m^2
	Services	0.15 kN/m^2
		11.75 kN/m^2
Imposed		2.5 kN/m^2

Internal studwalls

Dead	Plasterboard & skim coat (both sides)	0.50 kN/m^2
	Studs and blocking	0.15 kN/m^2
		0.65 kN/m^2

External walls

Dead	Zinc cladding	0.40 kN/m^2
	Battens and felt	0.13 kN/m^2
	12mm plywood sarking	0.08 kN/m^2
	Studs	0.30 kN/m^2
	Insulation	0.01 kN/m^2
	Plasterboard and skim coat	0.25 kN/m^2
		1.17 kN/m^2

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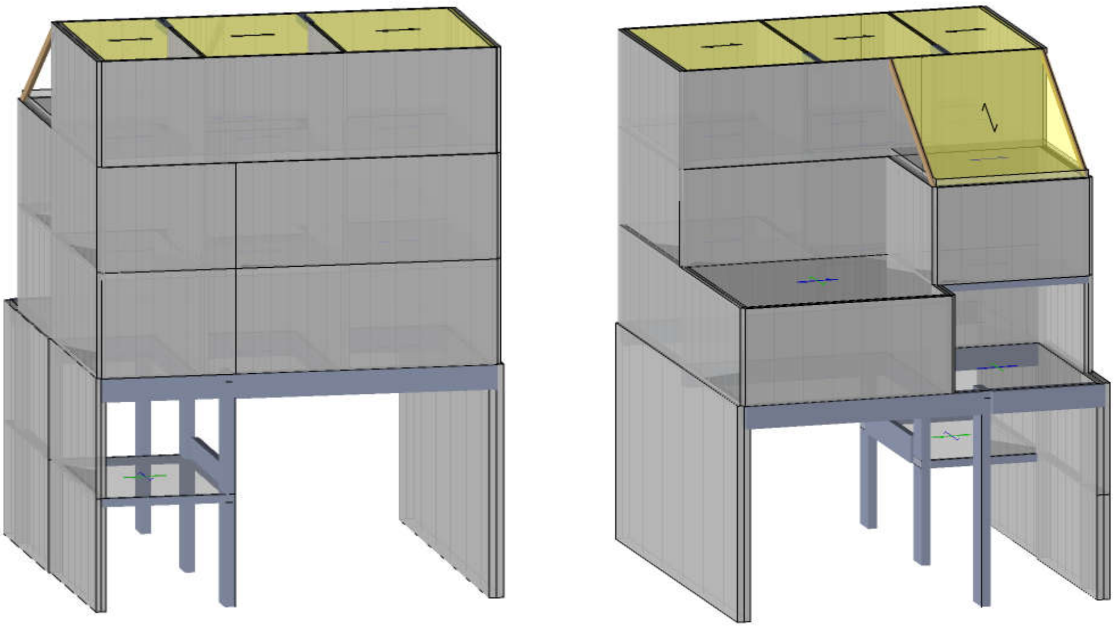
Job title: 155 Drummond Street			
Calculations: Existing loads take down	Designed: ab	Date: 23/11/2018	Ckd: -

The following loads take down is intended to define the loads at the base of the ground floor superstructures.

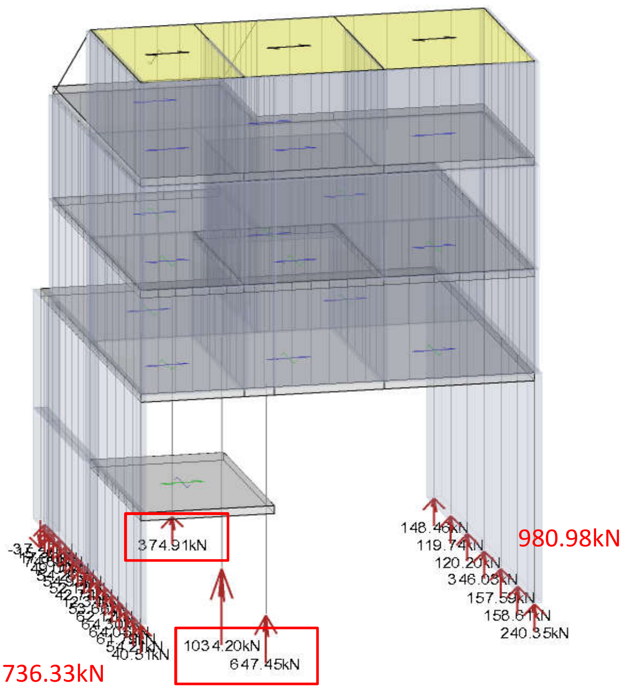
Beam & Load description	Density	Area loads		Height	Length	Area	Loads	
		DL	LL				DL	LL
	kN/mc	kN/m²	kN/m²	m	m	mq	kN	kN
Timber flat roof		1.00	0.75			38.49	38.49	28.87
Timber pitch roof		1.00	0.75			16.15	16.15	12.11
Timber third floor		1.40	1.50			55.54	77.76	83.31
External mandard walls		1.13		2.4	13.49		36.58	
Int 100 blockwalls	18.00	1.8		2.4	10.87		46.96	
Party walls		4.36		2.4	13.02		136.09	
Concrete second floor		6.80	2.50			55.54	377.67	138.85
Balcony		8.65	1.50			28.90	249.99	43.35
Int 100 blockwalls	18.00	1.8		2.4	14.77		63.81	
External cavity walls		4.36		2.4	13.02		136.09	
Concrete first floor		6.80	2.50			95.45	649.06	238.63
400 wide RC down stands	24.00	9.6		0.4	58.26		223.72	
Int 190 blockwalls	18.00	3.42		2.8	35.80		342.82	
External cavity walls		4.36		2.8	10.53		128.34	
Mezzanine floor		6.80	2.50			14.80	100.64	37.00
160 wide RC up stands	24.00	3.84		0.545	4.90		10.25	
330 blockwalls	18.00	5.94		5.6	15.57		517.92	
225 blockwalls	18.00	4.05		5.6	5.32		120.66	
3No. RC columns	24.00			5.6		0.15	20.41	
TOT							3293.4	582.1

The same exercise has been done with a FEM model to compare the overall load at ground floor and check the load under each wall and column at that floor.

3D model view (front and rear elevation)



Vertical reaction at base of ground floor superstructures



Tot reaction from FEM model = 3773.87kN

Tot reaction from sheet 4.1 = 3875.50kN

The comparison shows that the FEM model is enough accurate for further assessments.

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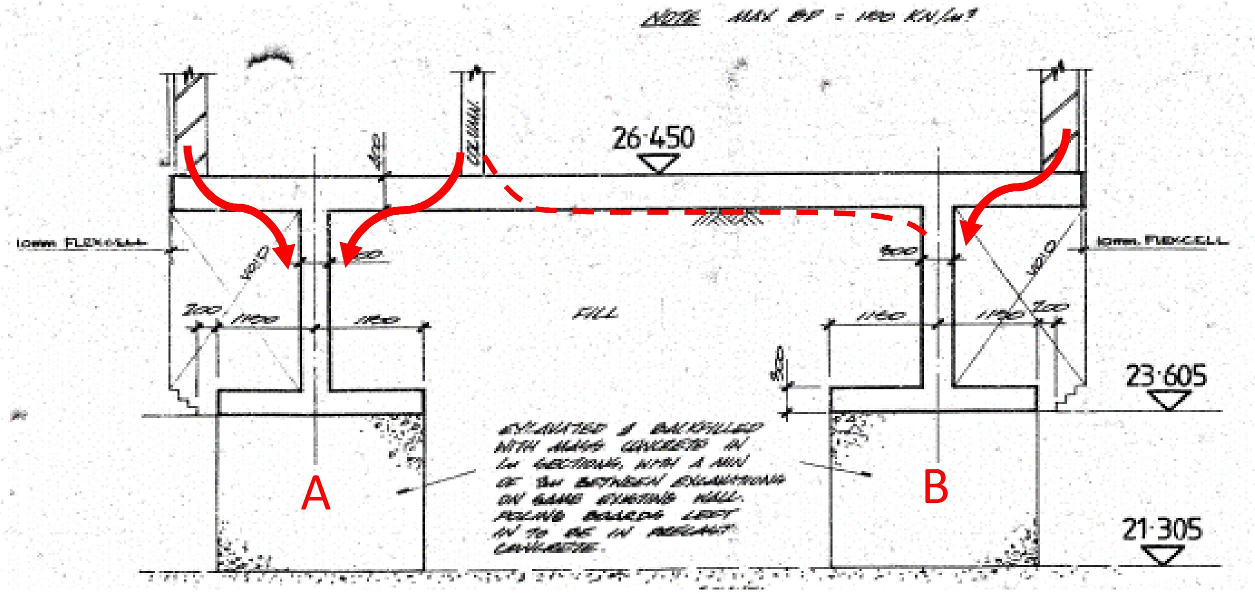
Job title: 155 Drummond Street			
Calculations: Existing loads take down	Designed: ab	Date: 23/11/2018	Ckd: -

The existing ground floor structure is composed by a 400 thk RC slab supporting the sides walls and the three central columns.
The loads from the super-structure and the ground floor slab itself is then tranferred onto two big strip fondations composed by a 300 thick RC wall, a 300 thick 2300 wide RC raft and a 2300x2300 deep mass concrete base. Those two strip foundations extend for the whole depth of the building.
The picture below shows a section from the original contruction drawings and the red arrows show how the loads are traferred to the foundation.



The following assessment is intended to define the bearing pressure under the existing foundations.
The area loads at ground floor are the ones shown in sheet 3.2.
The loads from the super-structure are the ones highlighted in sheet 4.2 as results from the FEM model.

Foundation A

Leght of foundation	10.44	m
Total load from side wall	736.33	kN
Total load from side wall as distributed over foundation leght	70.53	kN/m
Total load from central columns	2056.6	kN
Total load from central columns as distributed over foundation leght	155.6	kN/m
Dead load from ground floor	10.65	kN/m2
Imposed load from ground floor	2.50	kN/m2
Width of ground floor slab supported by foundation A	4.8	m
Total load from ground floor slab	63.12	kN/m
Foundation RC wall self weight assuming 300 thk and 1.9m high	13.68	kN/m
Foundation RC raft self weight assuming 300 thk 2.3m wide	16.56	kN/m
Foundation mass concrete base salf weght assuming 2.3 wide 2.3 deep	126.96	kN/m
Weight of soil excavated in place of foundation raft and wass concrete	107.64	kN/m
Tot bearing pressure - weight of excavated soil	**147.31**	kN/m2
Allowable bearing pressure	**150.00**	kN/m2

Foundation B

Leght of foundation	10.44	m
Total load from side wall	981	kN
Total load from side wall as distributed over foundation leght	93.966	kN/m
Total load from central columns	2056.6	kN
Total load from central columns as distributed over foundation leght	41.384	kN/m
Dead load from ground floor	10.65	kN/m2
Imposed load from ground floor	2.50	kN/m2
Width of ground floor slab supported by foundation A	4.45	m
Total load from ground floor slab	58.518	kN/m
Foundation RC wall self weight assuming 300 thk and 1.9m high	13.68	kN/m
Foundation RC raft self weight assuming 300 thk 2.3m wide	16.56	kN/m
Foundation mass concrete base salf weght assuming 2.3 wide 2.3 deep	126.96	kN/m
Weight of soil excavated in place of foundation raft and wass concrete	107.64	kN/m
Tot bearing pressure - weight of excavated soil	**105.84**	kN/m2
Allowable bearing pressure	**150.00**	kN/m2

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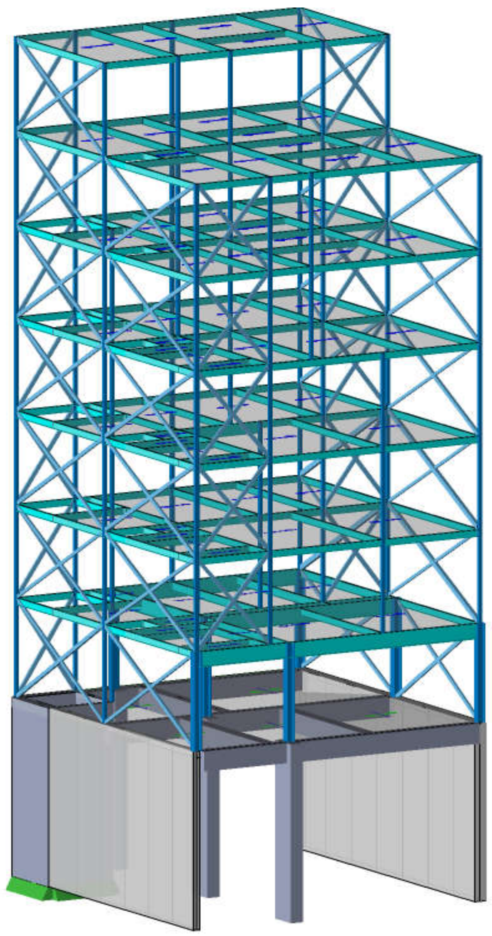
Job title: 155 Drummond Street			
Calculations: Proposed loads take down	Designed: ab	Date: 23/11/2018	Ckd: -

The following pictures are intended to show the proposed loads at the base of the ground floor existing superstructures. This will allow a comparison with the existing condition and it will define the correct structual approach to minimize the impact of the new development on the adjacent properties.

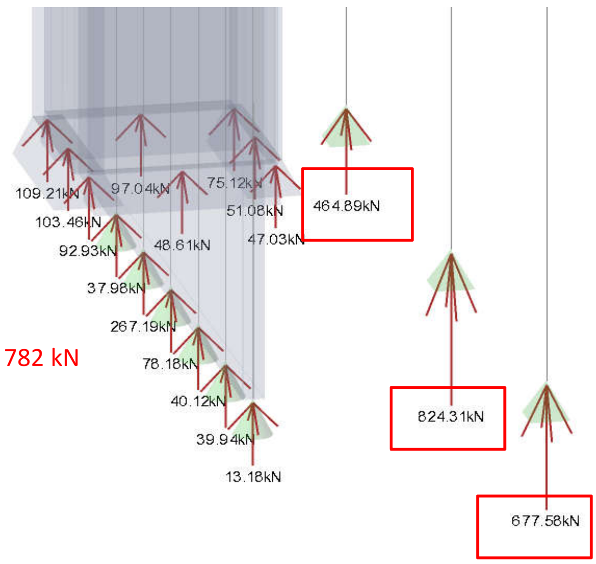
3D model view (front elevation) and vertical reaction at base of ground floor superstructures

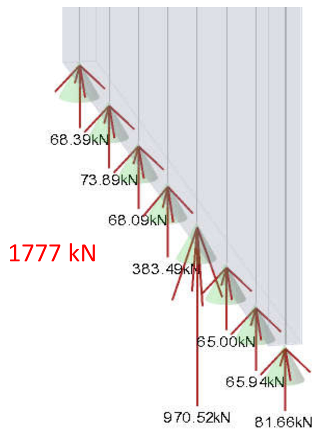


It is proposed to demolish the three existing upper floors, which are of blockwork and timber construction, and replace them with a new lighter structure able to allow four extra floors at the top.

The model shows the existing ground floor RC and masonry structure, with the addition of the RC lift case, and the new upper steel frame with cross bracings.

Referring to the tot reactions highlighted in sheet 4.2 and from the picture below, it is possible to see how the total load in the central existing RC column decreases and how in the other two columns and party wall along staircase it is slightly higher than the existing. But, on the opposite side, it is shown a load increment which needs to be carefully transferred into the nearest existing foundation.





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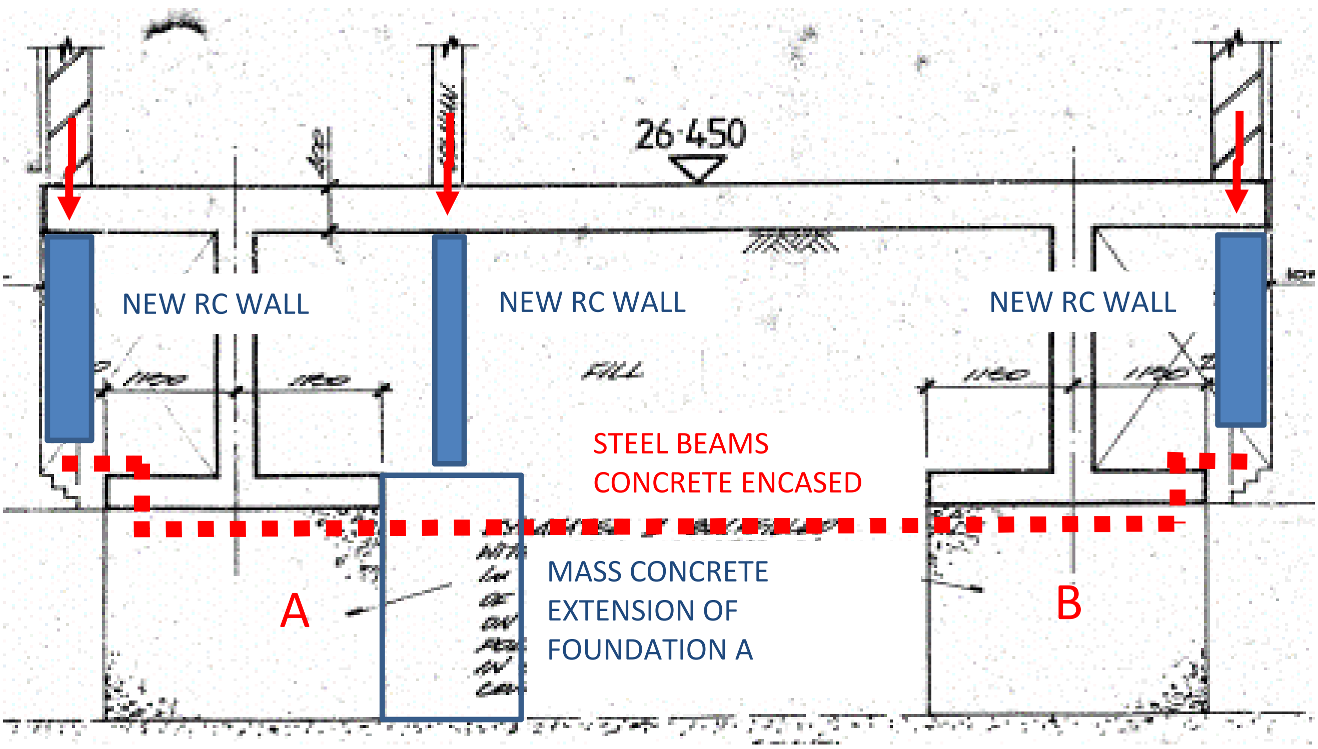
Job title: 155 Drummond Street			
Calculations: Proposed loads take down	Designed: ab	Date: 23/11/2018	Ckd: -

In order to not undermine the neighbours's foundation and to avoid additional loads to be supported by the existing ground floor slab, it is proposed to create a new trasfer structure at the basement level.
The structural approach for this new system is shown in the picture below and it can be compared to the one of sheet 4.3 in which the red arrows show how the loads are proposed to be trasferred down.



New reinforced concrete walls under the main ground floor super structures will trasfer the loads directly to the foundation level, bypassing the ground floor slab which will be affected only by its own area loads. Then cranked steel beams encased in a new basement slab will transfer the loads into the existing strip foundations without touching the neighbours party walls coorbel footings.
To support properly the new RC walls under the existing ground floor columns it is proposed to extend foundation A, while foundation B will not need any additional intervention.

Steel beams concrete encased in the new basement slab are to be considered at 1.5m centre and with the following applied loads.
For beams analysis and design refer to sheet 5.

Beam & Load description	Span	Area loads		Width	Location		UDL		Point loads	
		DL	LL		from	to	DL	LL	DL	LL
	mm	kN/m²	kN/m²	mm	mm	mm	kN/m	kN/m	kN	kN
basement floor	450+2700+5550+150	10.95	1.50	1500			16.43	2.25		
loads from GF wall to stairwell side				1500	0				82.21	30.58
loads from GF wall opposite to stairwell				1500	8850				169.90	86.39
GF	900	11.75	2.50	1500	0				15.86	3.38
	1525	11.75	2.50	1500	1650				26.88	5.72
	3075	11.75	2.50	1500	7550				54.20	11.53
	725	11.75	2.50	1500	8850				12.78	2.72

basem 250 wall	1900	6.00		1500	0				17.10	
basem 250 wall	1900	6.00		1500	8850				17.10	
basem 300 wall	1900	7.20		1500	1650				20.52	
basem 300 wall	1900	7.20		1500	7550				20.52	

Under the new configuration of loads the existing foundations are to be checked again in order to define the new bearing pressure, as it has been done in sheet 4.3.

Foundation A

Leght of foundation	10.44	m
Total load from side wall at GF	782	kN
Total load from side wall as distributed over foundation leght	74.904	kN/m
Total load from central columns	1966.8	kN
Total load from central columns as distributed over foundation leght	188.39	kN/m
Dead load from ground floor	11.75	kN/m2
Imposed load from ground floor	2.50	kN/m2
Width of ground floor slab supported by foundation A	5.4	m
Total load from ground floor slab	76.95	kN/m
Foundation RC wall self weight assuming 300 thk and 1.9m high	13.68	kN/m
Foundation RC raft self weight assuming 300 thk 2.3m wide	16.56	kN/m
New RC side wall self weight assuming 250 thk 1.9 high	11.4	kN/m
New RC wall under columns self weight assuming 300 thk 1.9 high	13.68	kN/m
Foundation mass concrete base self weght assuming 3.1 wide 2.3 deep	171.12	kN/m
Weight of soil excavated in place of foundation raft and wass concrete	145.08	kN/m
Tot bearing pressure - weight of excavated soil	**136**	kN/m2
Allowable bearing pressure	**150.00**	kN/m2

<u>Foundation A requires to be enlarged to maintain the bearing pressure within the limit of 150 kN/m2.</u>

Foundation B

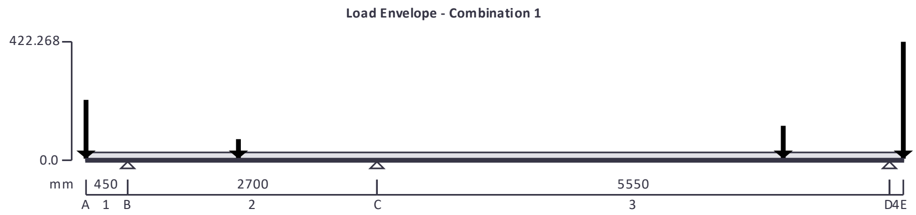
Leght of foundation	10.44	m
Total load from side wall at GF	1777	kN
Total load from side wall as distributed over foundation leght	170.21	kN/m
Dead load from ground floor	11.75	kN/m2
Imposed load from ground floor	2.50	kN/m2
Width of ground floor slab supported by foundation A	3.8	m
Total load from ground floor slab	54.15	kN/m
Foundation RC wall self weight assuming 300 thk and 1.9m high	13.68	kN/m
Foundation RC raft self weight assuming 300 thk 2.3m wide	16.56	kN/m
New RC side wall self weight assuming 250 thk 1.9 high	11.4	kN/m
Foundation mass concrete base self weght assuming 2.3 wide 2.3 deep	126.96	kN/m
Weight of soil excavated in place of foundation raft and wass concrete	107.64	kN/m
Tot bearing pressure - weight of excavated soil	**124.05**	kN/m2
Allowable bearing pressure	**150.00**	kN/m2

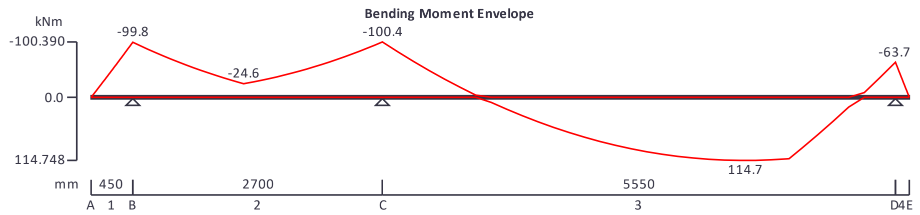
<u>Foundation B is satisfied with the existing width.</u>

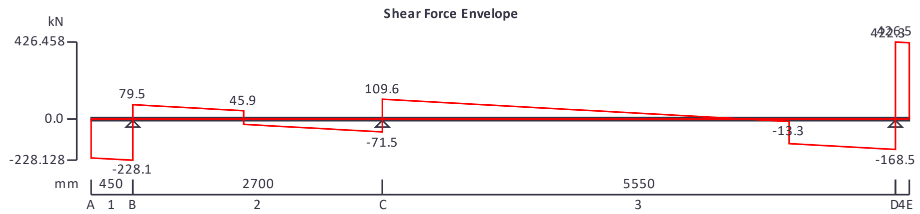
STEEL BEAM ANALYSIS & DESIGN (BS5950)

In accordance with BS5950-1:2000 incorporating Corrigendum No.1

TEDDS calculation version 3.0.07







Support conditions

Support A	Vertically free
	Rotationally free
Support B	Vertically restrained
	Rotationally free
Support C	Vertically restrained
	Rotationally free
Support D	Vertically restrained
	Rotationally free
Support E	Vertically free
	Rotationally free

Applied loading

Beam loads	Dead self weight of beam × 1
	Dead full UDL 16.43 kN/m
	Imposed full UDL 2.25 kN/m
	Dead point load 115.17 kN at 0 mm
	Imposed point load 33.95 kN at 0 mm
	Dead point load 47.4 kN at 1650 mm

Imposed point load 5.72 kN at 1650 mm
Dead point load 74.72 kN at 7550 mm
Imposed point load 11.53 kN at 7550 mm
Dead point load 199.78 kN at 8850 mm
Imposed point load 89.11 kN at 8850 mm

Load combinations

Load combination 1	Support A	Dead × 1.40
		Imposed × 1.60
		Dead × 1.40
		Imposed × 1.60
	Support B	Dead × 1.40
		Imposed × 1.60
		Dead × 1.40
		Imposed × 1.60
	Support C	Dead × 1.40
		Imposed × 1.60
		Dead × 1.40
		Imposed × 1.60
	Support D	Dead × 1.40
		Imposed × 1.60
		Dead × 1.40
		Imposed × 1.60
	Support E	Dead × 1.40
		Imposed × 1.60

Analysis results

Maximum moment	M_{max} = **114.7** kNm	M_{min} = **-100.4** kNm
Maximum moment span 1	M_{s1_max} = **0** kNm	M_{s1_min} = **-99.8** kNm
Maximum moment span 2	M_{s2_max} = **-24.6** kNm	M_{s2_min} = **-100.4** kNm
Maximum moment span 3	M_{s3_max} = **114.7** kNm	M_{s3_min} = **-100.4** kNm
Maximum moment span 4	M_{s4_max} = **0** kNm	M_{s4_min} = **-63.7** kNm
Maximum shear	V_{max} = **426.5** kN	V_{min} = **-228.1** kN
Maximum shear span 1	V_{s1_max} = **-215.6** kN	V_{s1_min} = **-228.1** kN
Maximum shear span 2	V_{s2_max} = **79.5** kN	V_{s2_min} = **-71.5** kN
Maximum shear span 3	V_{s3_max} = **109.6** kN	V_{s3_min} = **-168.5** kN
Maximum shear span 4	V_{s4_max} = **426.5** kN	V_{s4_min} = **422.3** kN
Deflection	δ_{max} = **4.2** mm	δ_{min} = **0.5** mm
Deflection span 1	δ_{s1_max} = **0.5** mm	δ_{s1_min} = **0** mm
Deflection span 2	δ_{s2_max} = **0** mm	δ_{s2_min} = **0.5** mm
Deflection span 3	δ_{s3_max} = **4.2** mm	δ_{s3_min} = **0** mm
Deflection span 4	δ_{s4_max} = **0** mm	δ_{s4_min} = **0.4** mm
Maximum reaction at support A	R_{A_max} = **0** kN	R_{A_min} = **0** kN
Maximum reaction at support B	R_{B_max} = **307.6** kN	R_{B_min} = **307.6** kN
Unfactored dead load reaction at support B	R_{B_Dead} = **168** kN	
Unfactored imposed load reaction at support B	$R_{B\_Imposed}$ = **45.2** kN	
Maximum reaction at support C	R_{C_max} = **181.1** kN	R_{C_min} = **181.1** kN
Unfactored dead load reaction at support C	R_{C_Dead} = **119.5** kN	

Unfactored imposed load reaction at support C	$R_{C\_Imposed}$ = **8.6** kN	
Maximum reaction at support D	R_{D_max} = **594.9** kN	R_{D_min} = **594.9** kN
Unfactored dead load reaction at support D	R_{D_Dead} = **303.3** kN	
Unfactored imposed load reaction at support D	$R_{D\_Imposed}$ = **106.4** kN	
Maximum reaction at support E	R_{E_max} = **0** kN	R_{E_min} = **0** kN

Section details

Section type **UC 305x305x97 (BS4-1)**

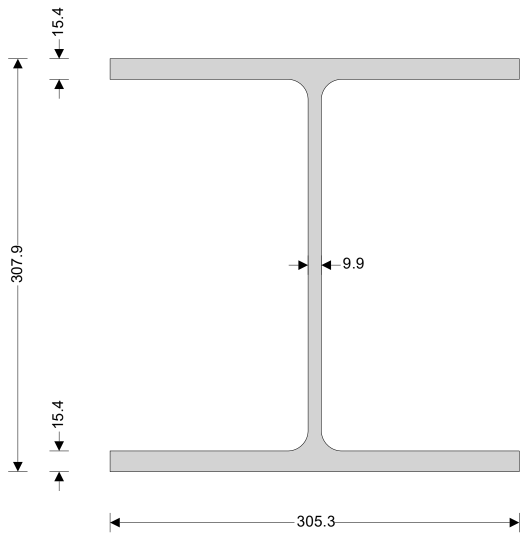
Steel grade **S355**

From table 9: Design strength p_y

Thickness of element max(T, t) = **15.4** mm

Design strength p_y = **355** N/mm^2

Modulus of elasticity E = **205000** N/mm^2



Lateral restraint

Span 1 has full lateral restraint

Span 2 has full lateral restraint

Span 3 has full lateral restraint

Span 4 has full lateral restraint

Cantilever tip is free

Cantilever support is continuous with lateral and torsional restraint

Effective length factors

Effective length factor in major axis K_x = **1.00**

Effective length factor in minor axis K_y = **1.00**

Effective length factor for lateral-torsional buckling $K_{LT.A}$ = **2.00**

$K_{LT.B}$ = **1.00**

$K_{LT.C}$ = **1.00**

$K_{LT.D}$ = **1.00**

$K_{LT.E}$ = **1.00**

Classification of cross sections - Section 3.5

$\varepsilon = \sqrt{[275 \text{ N/mm}^2 / p_y]}$ = **0.88**

Internal compression parts - Table 11

Depth of section d = **246.7** mm

$d / t = 28.3 \times \varepsilon <= 80 \times \varepsilon$ Class 1 plastic

Outstand flanges - Table 11

Width of section $b = B / 2 =$ **152.7** mm

$b / T = 11.3 \times \varepsilon <= 15 \times \varepsilon$ Class 3 semi-compact

Section is class 3 semi-compact

Shear capacity - Section 4.2.3

Design shear force $F_v = \max(\text{abs}(V_{max}), \text{abs}(V_{min})) =$ **426.5** kN

$d / t < 70 \times \varepsilon$

Web does not need to be checked for shear buckling

Shear area $A_v = t \times D =$ **3048** mm^2

Design shear resistance $P_v = 0.6 \times p_y \times A_v =$ **649.3** kN

PASS - Design shear resistance exceeds design shear force

Moment capacity at span 3 - Section 4.2.5

Design bending moment $M = \max(\text{abs}(M_{s3\_max}), \text{abs}(M_{s3\_min})) =$ **114.7** kNm

Effective plastic modulus - Section 3.5.6

Limiting value for class 2 compact flange $\beta_{2f} = 10 \times \varepsilon =$ **8.801**

Limiting value for class 3 semi-compact flange $\beta_{3f} = 15 \times \varepsilon =$ **13.202**

Limiting value for class 2 compact web $\beta_{2w} = 100 \times \varepsilon =$ **88.014**

Limiting value for class 3 semi-compact web $\beta_{3w} = 120 \times \varepsilon =$ **105.617**

Effective plastic modulus - cl.3.5.6.2

$S_{eff} = \min(Z_{xx} + (S_{xx} - Z_{xx}) \times \min([((\beta_{3w} / (d / t))^2 - 1) / ((\beta_{3w} / \beta_{2w})^2 - 1)], [(\beta_{3f} / (b / T) - 1) / (\beta_{3f} / \beta_{2f} - 1)]), S_{xx}) =$ **1542693** mm^3

Reduction factor $\rho_v = [2 \times (F_{v\_s3} / P_v) - 1]^2 =$ **0.231**

Plastic modulus of shear area $S_v = t \times D^2 / 4 =$ **234636** mm^3

Moment capacity high shear - cl.4.2.5.3 $M_c = \min(p_y \times S_{eff}, 1.5 \times p_y \times Z_{xx}) =$ **547.7** kNm

PASS - Moment capacity exceeds design bending moment

Check vertical deflection - Section 2.5.2

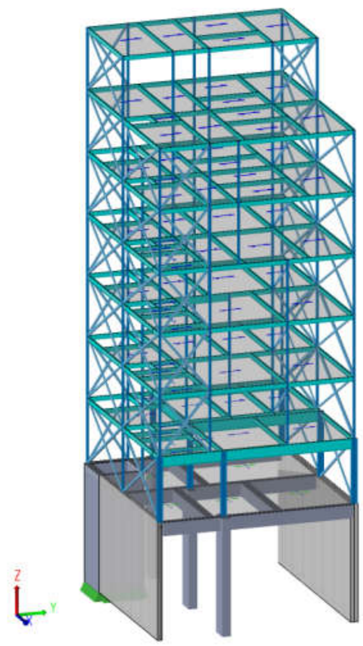
Consider deflection due to dead loads

Limiting deflection $\delta_{lim} = L_{s3} / 500 =$ **11.1** mm

Maximum deflection span 3 $\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) =$ **4.156** mm

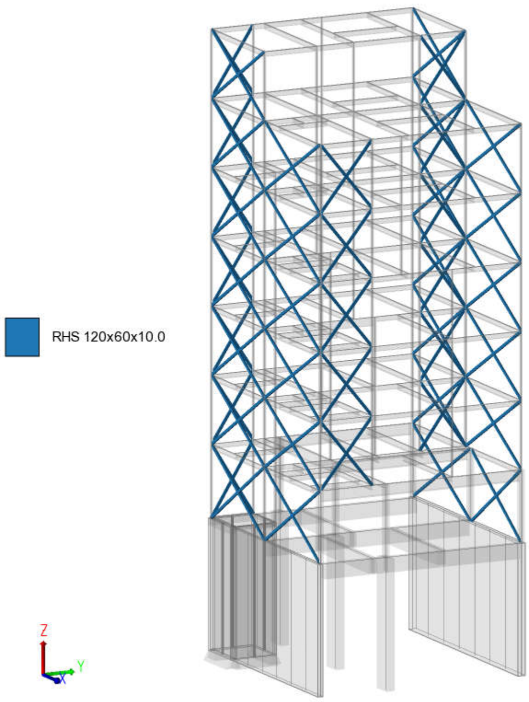
PASS - Maximum deflection does not exceed deflection limit

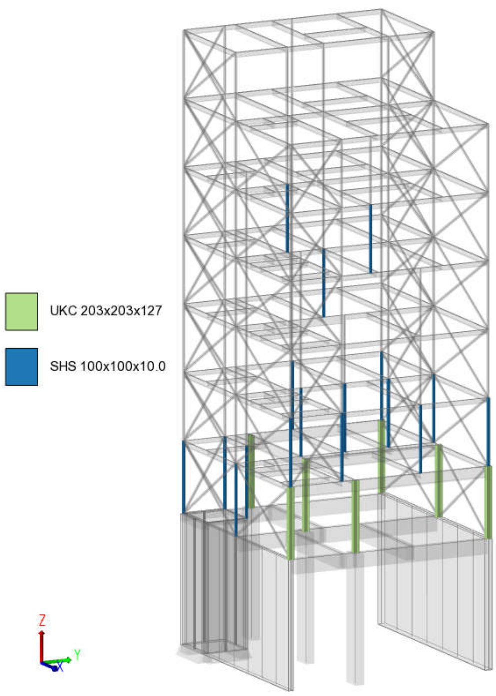
Super-structure 3D model view

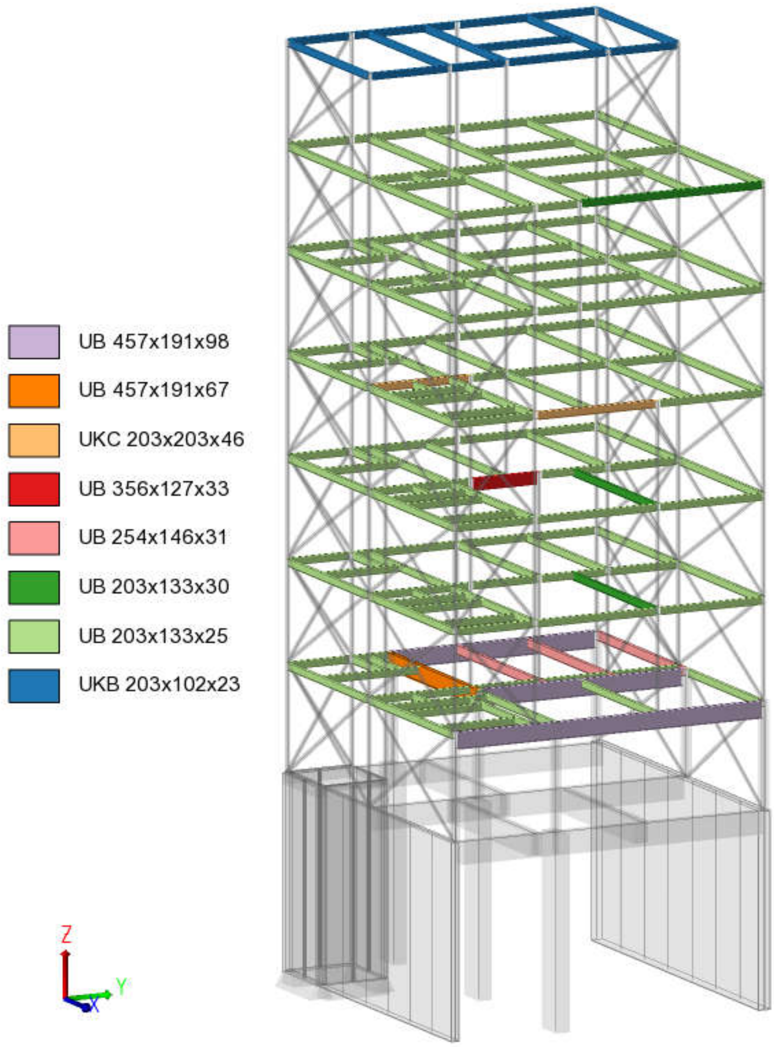


The model is composed by the new braced steel frame from first floor to top, first floor existing reinforced concrete slab 200mm thick supported by existing 400x600 reinforced concrete beams, ground floor existing internal concrete columns which are proposed to be concrete encased, ground floor existing side brick walls (215 and 330mm thk) and new reinforced concrete lift case.

The following pictures show the steel sections used in the model for the bracings, the columns and the beams.







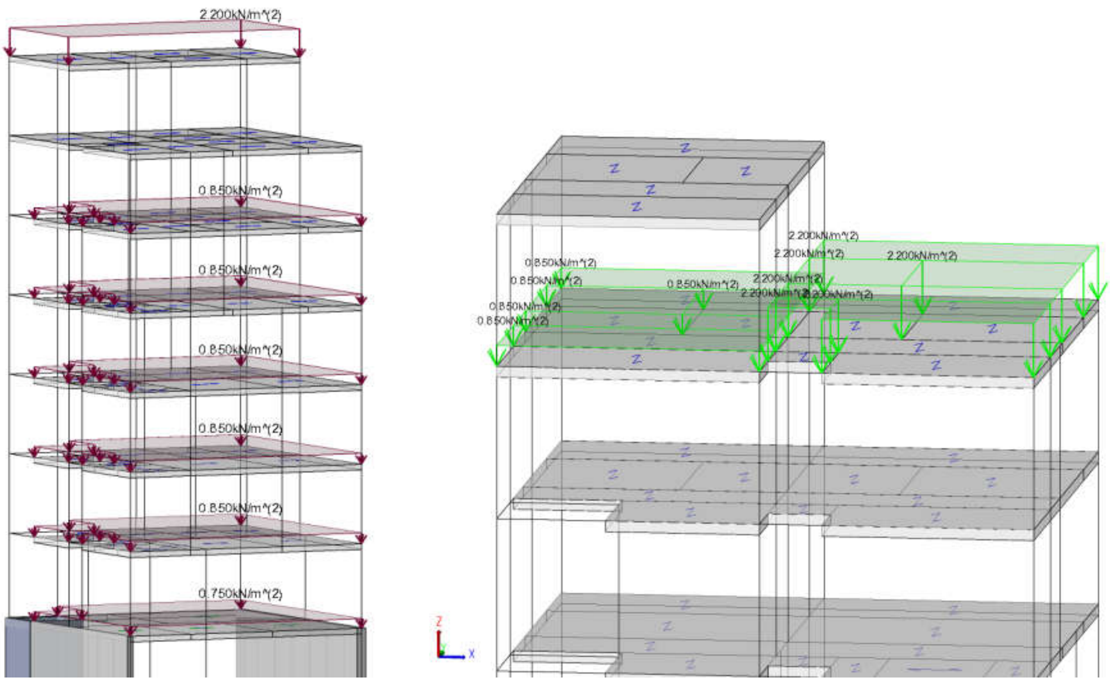
Loadcases

Loadcase Title	Type	Calculate Automatically	Include In Generator	Imposed Load Reductions	Pattern Load
0 Self weight - excluding slabs	SelfWeight	•	•		
1 Slab self weight	Slab Dry	•	•		
2 Dead	Dead		•		
3 Services	Dead		•		
4 Imposed	Imposed		•		
5 Wind 0,Cpi -0.3,-Cpe	Wind	•	•		
6 Wind 90,Cpi -0.3,-Cpe	Wind	•	•		
7 Wind 180,Cpi -0.3,-Cpe	Wind	•	•		
8 Wind 270,Cpi -0.3,-Cpe	Wind	•	•		

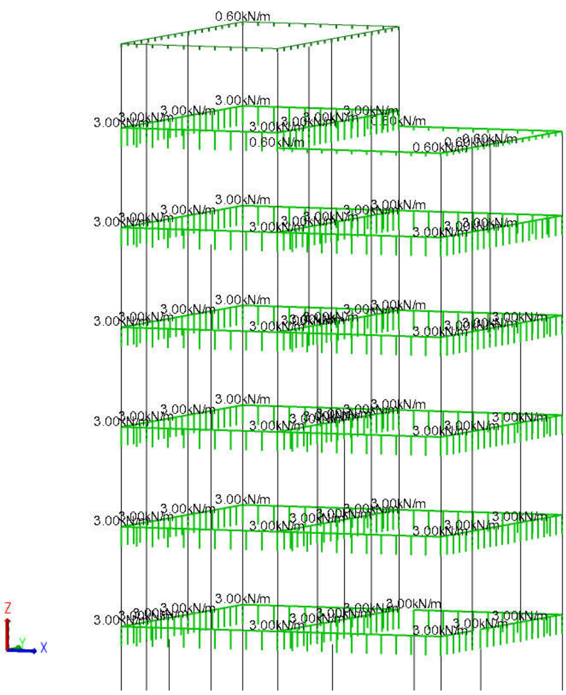
The loads have been applied at each floors as shown in the following pictures.

Tekla Structural Designer	Project 155 Drummond Street			Job Ref. 863	
	Structure steel frame			Sheet no. 6.3	
	Calc. by ab	Date 01/02/2019	Chk'd by	Date 28/01/2019	App'd by	Date 28/01/2019

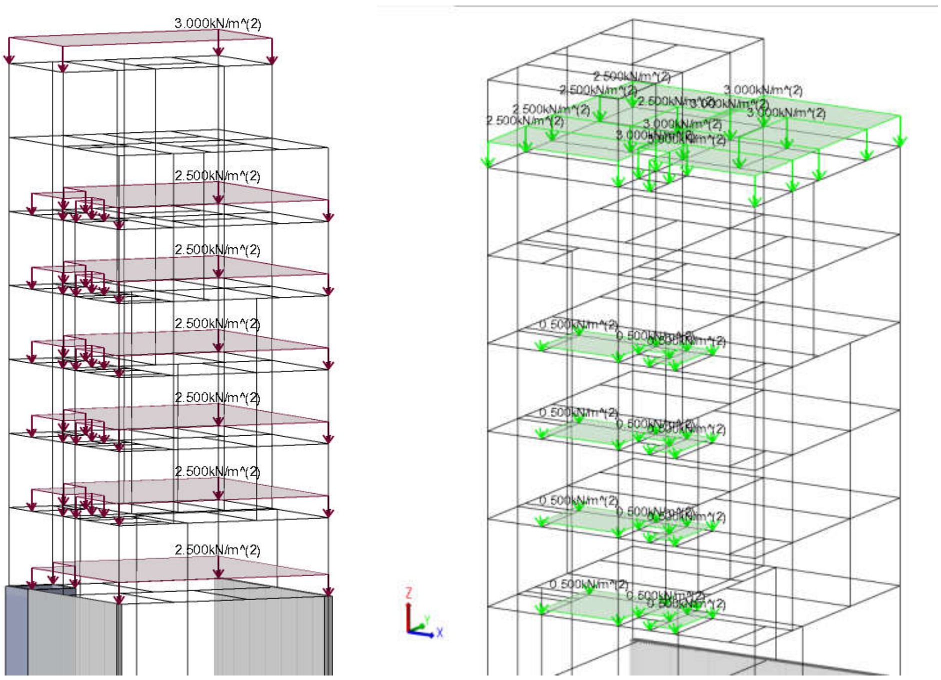
Dead load as level and area load at each floor (kN/m2)



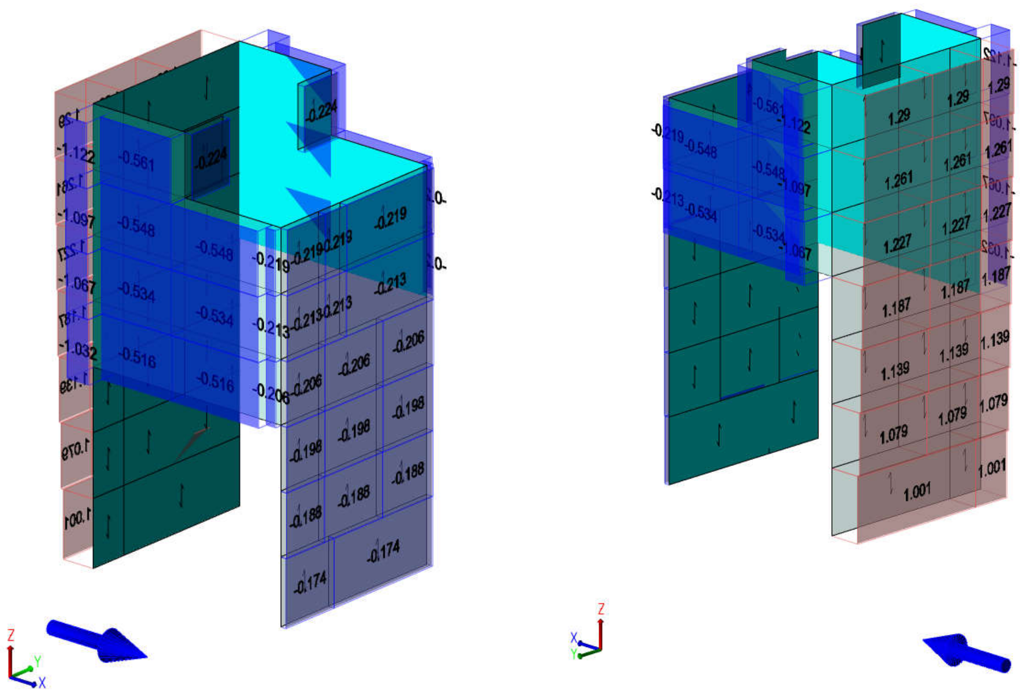
Dead load as perimeter walls and parapet at each floor (kN/m)



Imposed load as level and area load at each floor (kN/m2)

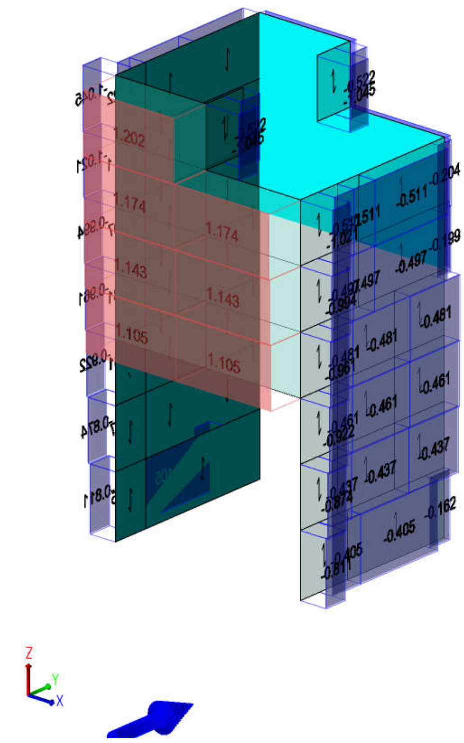


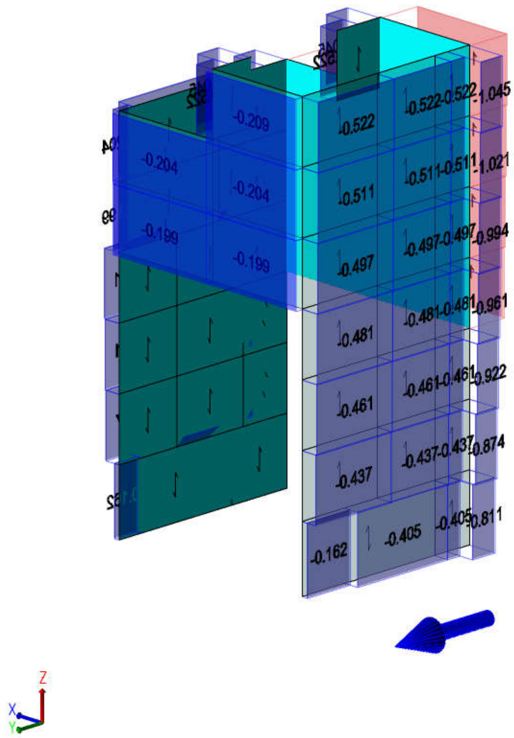
Wind +X net pressure



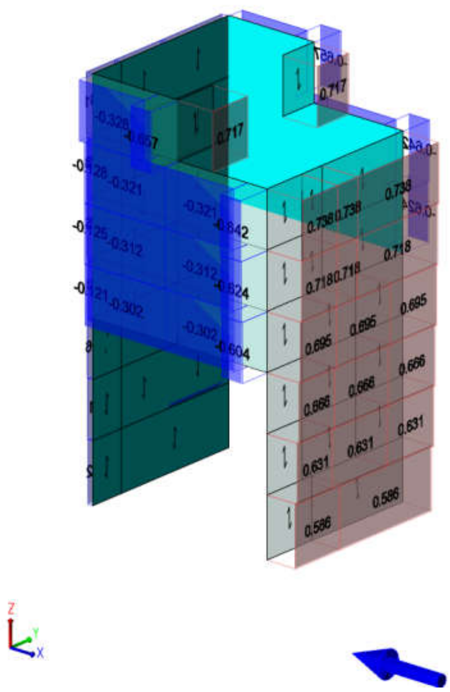
	Project 155 Drummond Street	Job Ref. 863			
	Structure steel frame	Sheet no. 6.5			
Calc. by ab	Date 01/02/2019	Chk'd by	Date 28/01/2019	App'd by	Date 28/01/2019

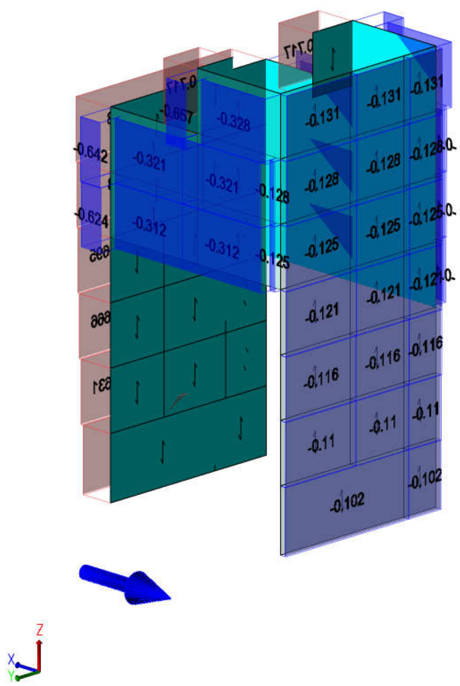
Wind +Y net pressure



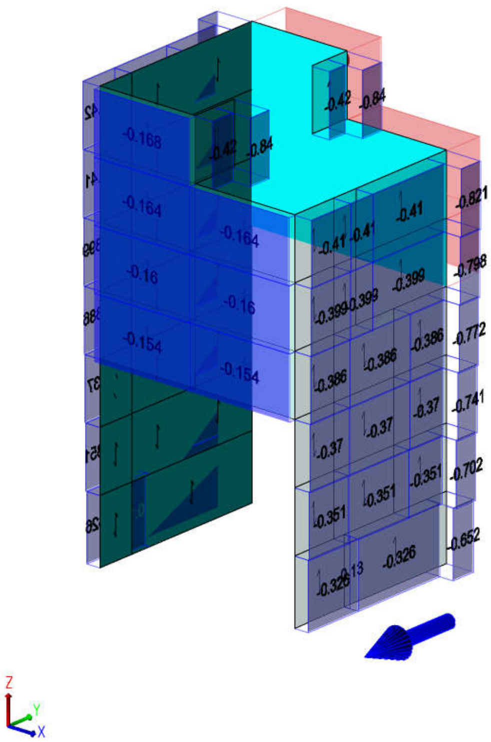


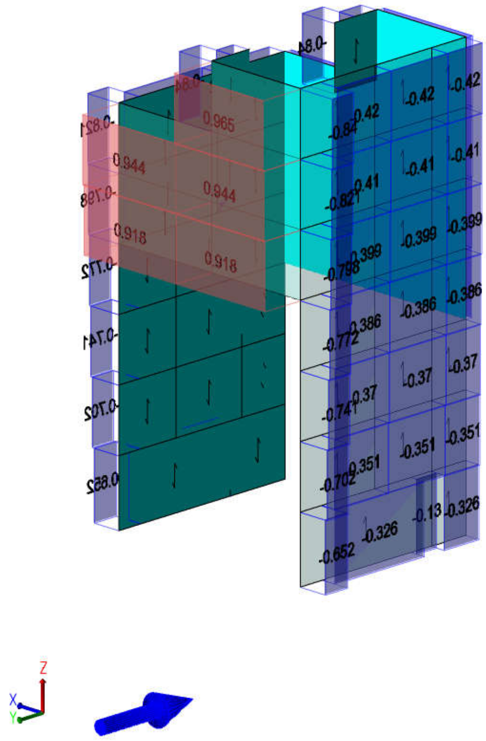
Wind -X net pressure





Wind -Y net pressure





Combinations

Name	Class	Active	Strength	Service
1 Cb_1-1.4D+1.6I+1.6RI	Gravity	●	●	●
2 $Cb_{1.1}$-1.4D+1.6I+1.6RI+NHF_{Dir1+}	Lateral	●	●	●
3 $Cb_{1.2}$-1.4D+1.6I+1.6RI+NHF_{Dir1-}	Lateral	●	●	●
4 $Cb_{1.3}$-1.4D+1.6I+1.6RI+NHF_{Dir2+}	Lateral	●	●	●
5 $Cb_{1.4}$-1.4D+1.6I+1.6RI+NHF_{Dir2-}	Lateral	●	●	●
6 $Cb_{3.1}$-1.2D+1.2I+1.2RI+1.2W	Lateral	●	●	●
7 $Cb_{3.2}$-1.2D+1.2I+1.2RI+1.2W	Lateral	●	●	●
8 $Cb_{3.3}$-1.2D+1.2I+1.2RI+1.2W	Lateral	●	●	●
9 $Cb_{3.4}$-1.2D+1.2I+1.2RI+1.2W	Lateral	●	●	●
10 $Cb_{5.1}$-D+1.4W	Lateral	●	●	●
11 $Cb_{5.2}$-D+1.4W	Lateral	●	●	●
12 $Cb_{5.3}$-D+1.4W	Lateral	●	●	●
13 $Cb_{5.4}$-D+1.4W	Lateral	●	●	●

1 Cb_1-1.4D+1.6I+1.6RI

Loadcase Title	Strength	Service
0 Self weight - excluding slabs	1.400	1.000
1 Slab self weight	1.400	1.000
2 Dead	1.400	1.000
3 Services	1.400	1.000
4 Imposed	1.600	1.000

2 $Cb_{1.1}$-1.4D+1.6I+1.6RI+NHF_{Dir1+}

Loadcase Title	Strength	Service
0 Self weight - excluding slabs	1.400	1.000
1 Slab self weight	1.400	1.000
2 Dead	1.400	1.000
3 Services	1.400	1.000
4 Imposed	1.600	1.000

3 $Cb_{1.2}$-1.4D+1.6I+1.6RI+NHF_{Dir1-}

Loadcase Title	Strength	Service
0 Self weight - excluding slabs	1.400	1.000
1 Slab self weight	1.400	1.000
2 Dead	1.400	1.000
3 Services	1.400	1.000
4 Imposed	1.600	1.000

4 $Cb_{1.3}$-1.4D+1.6I+1.6RI+NHF_{Dir2+}

Loadcase Title	Strength	Service
0 Self weight - excluding slabs	1.400	1.000
1 Slab self weight	1.400	1.000
2 Dead	1.400	1.000
3 Services	1.400	1.000
4 Imposed	1.600	1.000

5 $Cb_{1.4}$-1.4D+1.6I+1.6RI+NHF_{Dir2-}

Loadcase Title	Strength	Service
0 Self weight - excluding slabs	1.400	1.000
1 Slab self weight	1.400	1.000
2 Dead	1.400	1.000
3 Services	1.400	1.000
4 Imposed	1.600	1.000

6 $Cb_{3.1}$-1.2D+1.2I+1.2RI+1.2W

Loadcase Title	Strength	Service
0 Self weight - excluding slabs	1.200	1.000
1 Slab self weight	1.200	1.000
2 Dead	1.200	1.000
3 Services	1.200	1.000
4 Imposed	1.200	0.800
5 Wind 0,Cpi -0.3,-Cpe	1.200	0.800

7 $Cb_{3.2}$-1.2D+1.2I+1.2RI+1.2W

Loadcase Title	Strength	Service
0 Self weight - excluding slabs	1.200	1.000
1 Slab self weight	1.200	1.000
2 Dead	1.200	1.000
3 Services	1.200	1.000
4 Imposed	1.200	0.800
6 Wind 90,Cpi -0.3,-Cpe	1.200	0.800

8 $Cb_{3.3}$-1.2D+1.2I+1.2RI+1.2W

Loadcase Title	Strength	Service
0 Self weight - excluding slabs	1.200	1.000
1 Slab self weight	1.200	1.000
2 Dead	1.200	1.000
3 Services	1.200	1.000

Loadcase Title	Strength	Service
4 Imposed	1.200	0.800
7 Wind 180,Cpi -0.3,-Cpe	1.200	0.800

9 $Cb_{3.4}$-1.2D+1.2I+1.2RI+1.2W

Loadcase Title	Strength	Service
0 Self weight - excluding slabs	1.200	1.000
1 Slab self weight	1.200	1.000
2 Dead	1.200	1.000
3 Services	1.200	1.000
4 Imposed	1.200	0.800
8 Wind 270,Cpi -0.3,-Cpe	1.200	0.800

10 $Cb_{5.1}$-D+1.4W

Loadcase Title	Strength	Service
0 Self weight - excluding slabs	1.000	1.000
1 Slab self weight	1.000	1.000
2 Dead	1.000	1.000
3 Services	1.000	1.000
5 Wind 0,Cpi -0.3,-Cpe	1.400	1.000

11 $Cb_{5.2}$-D+1.4W

Loadcase Title	Strength	Service
0 Self weight - excluding slabs	1.000	1.000
1 Slab self weight	1.000	1.000
2 Dead	1.000	1.000
3 Services	1.000	1.000
6 Wind 90,Cpi -0.3,-Cpe	1.400	1.000

12 $Cb_{5.3}$-D+1.4W

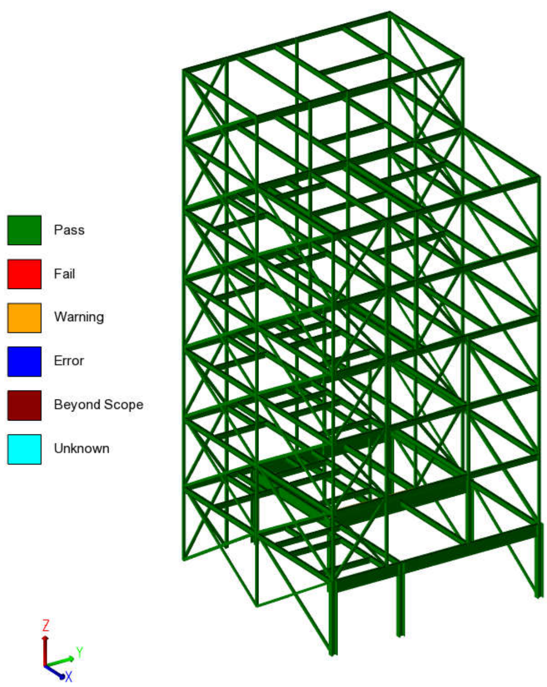
Loadcase Title	Strength	Service
0 Self weight - excluding slabs	1.000	1.000
1 Slab self weight	1.000	1.000
2 Dead	1.000	1.000
3 Services	1.000	1.000
7 Wind 180,Cpi -0.3,-Cpe	1.400	1.000

13 $Cb_{5.4}$-D+1.4W

Loadcase Title	Strength	Service
0 Self weight - excluding slabs	1.000	1.000
1 Slab self weight	1.000	1.000
2 Dead	1.000	1.000
3 Services	1.000	1.000
8 Wind 270,Cpi -0.3,-Cpe	1.400	1.000

Steel frame checks

The picture below shows the design status of the steel elements after the resistance and deflection checks.

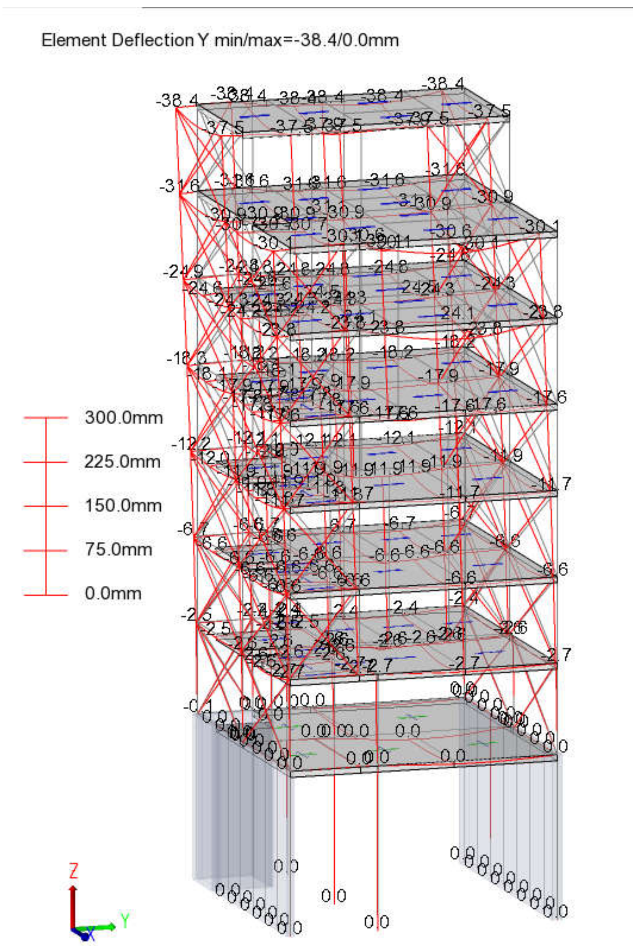


Horizontal deflection

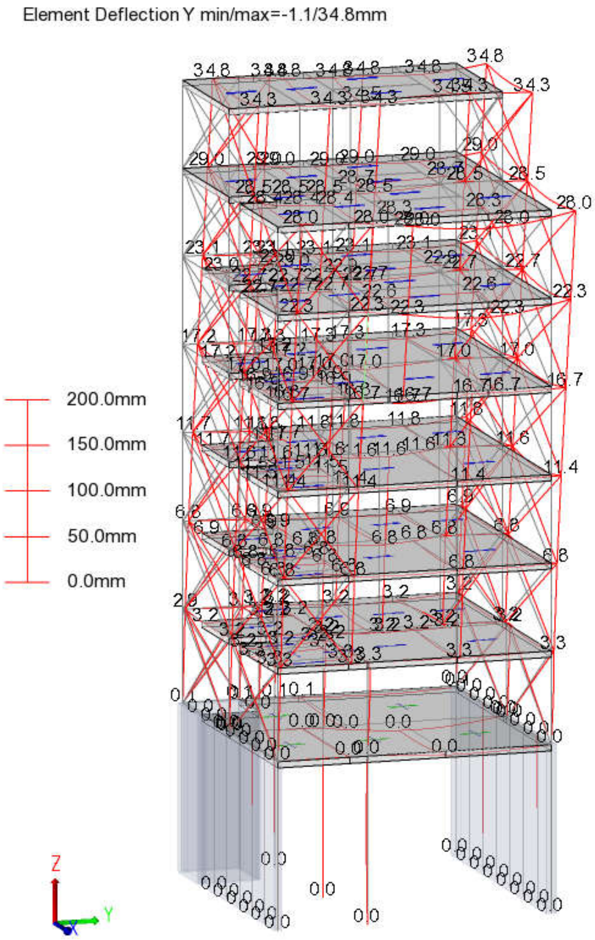
Even if the movements are within the limits it is recommended to provide construction joints between the neighbours' buildings party walls and the new frame/finishes to avoid hammering effects.

In the pictures below are shown the maximum lateral movements at each (from first floor to top terrace).

Combination 9 – deflection -Y (0, 2.5, 6.7, 12.2, 18.3, 24.9, 31.6, 38.4mm)



Combination 11 – deflection +Y (0, 3.3, 6.9, 11.8, 17.3, 23.1, 29, 34.8mm)



7. SUSTAINABILITY STATEMENT

The existing building consists of a three-storey masonry building with the third floor constructed with timber joists and floor boards; the first and second floor constructed in 200mm reinforced concrete slab bearing on a reinforced concrete podium structure providing vehicular and pedestrian access to Tolmer’s Square Estate.

An initial inspection of the substructure revealed an unusual substructure and foundations.

A historic search for documentation revealed that Tolmer’s Square Estate was designed by consulting structural engineers Ove Arup & Partners.

An archive request was made to Ove Arup and drawings from the original development issued between May and November 1980 were made available for viewing and from which copies of relevant drawings were made.

The buildings that constitute 155 Drummond Street replaced terraced houses, with retail at ground floor, and can be seen adjacent to and further along the street.

The terraced houses have basements and historical investigation revealed that the original terraced houses were damaged by bombs dropped during World War II and replaced as part of the Tolmer’s Square Estate development.

The substructure and foundations designed by Ove Arup & Partners were designed to replace the existing terraced house without disturbance of the existing basements each side of 155 Drummond Street.

The reinforced concrete podium structure, which supports the three-storey residential structure is supported by a double-storey brickwork wall along each boundary and three double storey brickwork clad reinforced concrete columns positioned between the vehicular access and the freeholder’s demise.

The brickwork wall and brickwork clad reinforced concrete columns is supported by a 400mm thick reinforced concrete raft slab which is supported by two parallel reinforced concrete spine walls bearing onto reinforced concrete slabs cast onto mass concrete foundations.

The reinforced concrete spine walls supporting the reinforced concrete raft is positioned a couple of metres from the boundary creating a basement between the spine walls and the side walls of the adjacent property’s basements.

The existing substructure, consisting of reinforced concrete raft slab, spine walls and slabs, bearing onto mass concrete foundations were cast in 1980 have been designed to support three storeys of residential accommodation.

Consequently, to support seven storeys of residential accommodation above the podium requires significant strengthening of the existing foundations.

To demolish and replace the existing substructure and foundations would create a significant disturbance to the existing access to the Tolmer’s Square Estate, destroy a structure with significant embodied energy and any new foundations would have a significant carbon footprint.

To mitigate any detrimental effect to the adjacent properties, to significantly reduce the carbon footprint of the proposed development, it was decided to use the existing foundations, and to significantly strengthen the existing foundations.

A detailed assessment was made of the capacity of the existing foundations and strengthening of the existing foundations designed to significantly increase the capacity of the existing foundations. This has been done by propping the edges of the existing raft slab and increasing the existing bearing area of the existing foundations, without undermining the adjacent basement foundations and party walls.

The design principles for the strengthening of the substructure and foundations is the most cost-effective method and energy efficient method of developing the site.

The maintenance of the podium structure allows the vehicular and pedestrian access to be maintained at all times during the demolition and construction period. This facilitates uninterrupted access to the Tolmer’s Square Estate.

It is proposed to demolish the existing building down to podium level, i.e. demolish the existing timber roof and third floor, and reinforced concrete first and second floor, and, all internal and external masonry walls.

The reduction in weight from the removal of the existing building above podium level and strengthening of the existing foundations allows a lightweight steel and timber framed to be constructed.

It would not be possible to retain the existing masonry structure and extend the building by a further four floors. The capacity of the existing masonry cavity wall would not have been enough to support the additional weight of the new structure.

The load from the additional weight of the four floors would have overloaded the existing reinforced concrete podium structure, which would have required substantial strengthening. This would have resulted in temporary closure of the access through the site to the Tolmer’s Square Estate, which is anticipated would not be acceptable to the freeholders of the estate.

The superstructure of the proposed development has been designed with a transfer structure at second floor level, allowing all the load from the six storeys to bear onto the three strengthened reinforced columns and the masonry walls each side of the side, against the party walls. One floor is supported by the existing podium structure at first floor level. The capacity of the podium structure will be adequate as it has been originally designed to support the three-storey masonry structure.

The preservation of the podium structure and strengthening of the existing foundations is the most efficient and lowest embodied energy solution for creating a multi-storey building above the existing podium structure, whilst preserving the vehicular and pedestrian access to Tolmer’s Square Estate.

The efficient use of steelwork for the primary structure and timber for the floors, internal wall and external walls allows a multi-storey building to be constructed with the lowest carbon footprint. The proposed superstructure uses the lightest and most efficient materials to minimize the overall weight of the seven storeys of residential development, and

By insulating the structure using high specification materials the lifespan carbon footprint of the building is also minimized.