Calculation Report: «1750 OX Residences - 1750 N Oxford Ave. - Eau Claire, WI»

XC structural engineering

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1 Introduction and scope

This report describes the calculation procedure and data considered in order to design the structure of a new apartment building in Eau Claire, Wisconsin.

The construction consists in a three-story apartment building with a first-floor footprint of about 19,500 square feet, a below-grade parking garage with a footprint of about 27,200 square feet, perimeter retaining walls, a slab-on-grade, and a conventional foundation system.

The first floor system is precast hollow core concrete plank on precast beams and columns. For the upper floors and roof, the system is wood-framed. Retaining walls and slab on grade are comprised of cast in place concrete, except for three reinforced CMU walls next to the garage aisles, that will be demolished during the second phase of construction.

The foundation uses conventional cast in place concrete footings to transfer axial compression and lateral loads to the ground.

2 Building codes

The following building and material codes were used for the design:

- Building code
  - International Building Code, 2018 Edition (IBC 2018) with reference to Minimum Design Loads for Buildings and Other Structures by the American Society of Civil Engineers, 2016 Edition (ASCE 7).
- Material codes
  - Reinforced Concrete: Building Code Requirements for Structural Concrete and Commentary by the American Concrete Institute, 2019 Edition (ACI 318).
  - Masonry: Building Code Requirements and Specification for Masonry Structures and Companion Commentaries, 2013 Edition (ACI 530/530).

3 Loading criteria

A summary of the project-specific loading criteria follows (see appendix A for a detailed list of load values).

3.1 Gravity loading

The gravity loads listed in Table 1 are in addition to the self weight of the structure. The minimum loading requirements were taken from ASCE 7 as well as the loading criteria supplied by the engineer of record. Loads are given in pounds per square foot (psf).

In addition to these uniform slab loads, a perimeter dead load of 12 psf was applied to the structure to account for the weight of the cladding system.

3.2 Wind design criteria

Wind loading is in accordance with the IBC and ASCE 7 requirements as shown in Table 2.

3.3 Snow loading

Wind loading is in accordance with the ASCE 7 requirements as shown in Table 3.

Table 1: **Gravity Loads**

Use	**Live Loading**	**Superimposed Dead Loading**
Parking Garage	40	3
Storage/HVAC	125	28
Stairways, exits	100	28
Level 1 residential	40	28
Level 1 corridors	100	28
Level 1 office, recreational	100	28
Level 1 courtyard (footprint)	150	150
Elevated levels residential	40	28
Elevated levels corridors	40	28
Cornices	60	-
Balconies	40	28
Roof	20	28

Table 2: **Wind Design Criteria**

Parameter	**Value**
Basic Wind Speed, 3-second gust (ultimate)	115 mph
Basic Wind Speed, 3-second gust (nominal)	90 mph
Exposure	B
Occupancy Category	II
Importance Factor (I_w)	1.0
Topographic Factor (K_{zt})	1.0
Enclosure Classification	Enclosed
Mean Roof Height (h)	33’

Table 3: **Snow Design Criteria**

Parameter	**Value**
Ground snow load p_g	60 psf
Terrain category	B
Exposure factor C_e	1.0
Thermal factor C_t	1.0
Occupancy Category	II
Snow load importance factor I_s	1.0
Snow load flat roof	42 psf

4 Seismic design criteria

Seismic loads are in accordance with the IBC requirements as shown in Table 4.

Table 4: **Seismic Design Criteria**

Parameter	**Value**
Building Latitude/Longitude	44^o49'01.8"N 91^o30'34.8"W
Occupancy Category	II
Importance Factor I_e	1.0
Mapped Spectral Acceleration	$S_s = 0.045$; $S_1 = 0.038$
Site Class	B
Site Class Coefficients	$F_a = 1.0$; $F_v = 1.0$
Spectral Response Coefficients	$S_{DS} = 0.03$; $S_{D1} = 0.025$
Seismic Design Category	A

5 Materials

The material properties used for the design are summarized in Tables 5 and 6.

Table 5: **Concrete properties**

Member	**Nominal f'$_c$**
Footings	3.0 ksi
Basement Walls	4.0 ksi
Foundation frost walls	4.0 ksi
Stair landings and treads	4.0 ksi
Slab on grade	4.0 ksi

Table 6: **Reinforcement properties**

Standard	**Nominal f$_y$**
All ASTM A615 Grade 60	60 ksi

6 Design and analysis software

The computer software employed for the analysis of the structure is the Finite Element Program called **XC** (see program description at `http://xcengineering.xyz/html_files/software.html`).

7 Load combinations

The load combinations shown in tables 7 and 8 follow the strength design load combinations listed in IBC, section 1605.

Table 7: **Combinations Ultimate Limit States**

Identifier	**Load Combination**
ULS01:	1.4*D
ULS02_a:	1.2*D + 1.6*Lru + Lpu + 0.5*S
ULS02_b:	1.2*D + 1.6*Lrs + Lps + 0.5*S
ULS03_a:	1.2*D + 1.6*S + 0.5*Lru + Lpu
ULS03_b:	1.2*D + 1.6*S + 0.5*Lrs + Lps
ULS04_b:	1.2*D + 1.6*S + 0.5*W_NS
ULS04_a:	1.2*D + 1.6*S + 0.5*W_WE
ULS05_a:	1.2*D + W_WE + 0.5*Lru + Lpu
ULS05_b:	1.2*D + W_NS + 0.5*Lru + Lpu
ULS05_c:	1.2*D + W_WE + 0.5*Lrs + Lps
ULS05_d:	1.2*D + W_NS + 0.5*Lrs + Lps
ULS06_a:	1.2*D + 0.5*Lru + Lpu + 0.2*S
ULS06_b:	1.2*D + 0.5*Lrs + Lps + 0.2*S
ULS07_a:	0.9*D + W_WE
ULS07_b:	0.9*D + W_NS

Where:

D = dead load
Lru = live load (uniform on rooms)
Lrs = live load (staggered pattern on rooms)
Lpu = live load (uniform on patios)
Lps = live load (staggered pattern on patios)
S = snow load
W_WE = Wind West-East
W_NS = Wind North-South

Table 8: **Combinations Serviceability Limit States**

Identifier	**Load Combination**
SLS01:	1.0*D
SLS02_a:	1.0*D + 1.0*Lru + Lpu + 0.3*S
SLS02_b:	1.0*D + 1.0*Lrs + Lps + 0.3*S
SLS03_a:	1.0*D + 1.0*S + 0.3*Lru + 0.3*Lpu
SLS03_b:	1.0*D + 1.0*S + 0.3*Lrs + 0.3*Lps
SLS04_a:	1.0*D + W_WE + 1.0*Lru + Lpu
SLS04_b:	1.0*D + W_NS + 1.0*Lru + Lpu
SLS04_c:	1.0*D + W_WE + 1.0*Lrs + Lps
SLS04_d:	1.0*D + W_NS + 1.0*Lrs + Lps
SLS05_a:	1.0*D + W_WE
SLS05_b:	1.0*D + W_NS

Where:

D = dead load
Lru = live load (uniform on rooms)
Lrs = live load (staggered pattern on rooms)
Lpu = live load (uniform on patios)
Lps = live load (staggered pattern on patios)
S = snow load
W_WE = Wind West-East
W_NS = Wind North-South

8 Wood framing

8.1 Gravity

8.2 Trusses

8.2.1 Trusses A and B. Roof

The deflection results for those trusses (see figure 2) are as follows [1]:

Load	truss	deflection		truss	deflection	
EQ1608	roof(A):	-1.94 mm	(L/5782; L= 11.22 m)	roof(B):	-1.12 mm	(L/9586; L= 10.77 m)
EQ1609	roof(A):	-5.63 mm	(L/1994; L= 11.22 m)	roof(B):	-3.82 mm	(L/2819; L= 10.77 m)
EQ1610	roof(A):	-9.66 mm	(L/1161; L= 11.22 m)	roof(B):	-6.76 mm	(L/1591; L= 10.77 m)
EQ1611	roof(A):	-10.49 mm	(L/1069; L= 11.22 m)	roof(B):	-7.38 mm	(L/1459; L= 10.77 m)
EQ1612	roof(A):	0.99 mm	(L/11391; L= 11.22 m)	roof(B):	1.02 mm	(L/10598; L= 10.77 m)
EQ1613	roof(A):	-8.30 mm	(L/1352; L= 11.22 m)	roof(B):	-5.77 mm	(L/1865; L= 10.77 m)
EQ1615	roof(A):	1.76 mm	(L/6370; L= 11.22 m)	roof(B):	1.47 mm	(L/7348; L= 10.77 m)
LIVE	roof(A):	-3.69 mm	(L/3044; L= 11.22 m)	roof(B):	-2.69 mm	(L/3995; L= 10.77 m)

The truss depth is allways greater than 24 inches due to the geometry of the roof. The spacing of the trusses is 24 inches.

$$\Delta_{LL,A} = 3.69\ mm = \frac{L}{3044} < \frac{L}{540} \implies OK \tag{1}$$

$$\Delta_{LL,B} = 2.69\ mm = \frac{L}{3995} < \frac{L}{540} \implies OK \tag{2}$$

$$\Delta_{TL,A} = 10.49\ mm = \frac{L}{1069} < \frac{L}{360} \implies OK \tag{3}$$

$$\Delta_{TL,B} = 7.38\ mm = \frac{L}{1459} < \frac{L}{360} \implies OK \tag{4}$$

8.2.2 Trusses A and B. Third floor

The deflection results for those trusses (see figure 3) are as follows:

Load	truss	deflection		truss	deflection	
EQ1608	A	-9.14 mm	(L/1228; L= 11.22 m)	B	-7.75 mm	(L/1389; L= 10.77 m)
EQ1609	A	-26.45 mm	(L/424; L= 11.22 m)	B	-22.46 mm	(L/479; L= 10.77 m)
EQ1610	A	-9.13 mm	(L/1228; L= 11.22 m)	B	-7.74 mm	(L/1389; L= 10.77 m)
EQ1611	A	-22.12 mm	(L/507; L= 11.22 m)	B	-18.78 mm	(L/573; L= 10.77 m)
EQ1612	A	-9.14 mm	(L/1228; L= 11.22 m)	B	-7.75 mm	(L/1389; L= 10.77 m)
EQ1613	A	-22.12 mm	(L/507; L= 11.22 m)	B	-18.78 mm	(L/573; L= 10.77 m)
EQ1615	A	-5.48 mm	(L/2047; L= 11.22 m)	B	-4.65 mm	(L/2315; L= 10.77 m)
LIVE	A	-17.32 mm	(L/648; L= 11.22 m)	B	-14.71 mm	(L/731; L= 10.77 m)

The truss depth is 24 inches and the spacing of the trusses is 12 inches.

$$\Delta_{LL,A} = 17.32\ mm = \frac{L}{648} < \frac{L}{540} \implies OK \tag{5}$$

$$\Delta_{LL,B} = 14.71\ mm = \frac{L}{731} < \frac{L}{540} \implies OK \tag{6}$$

$$\Delta_{TL,A} = 26.45\ mm = \frac{L}{424} < \frac{L}{360} \implies OK \tag{7}$$

$$\Delta_{TL,B} = 22.46\ mm = \frac{L}{479} < \frac{L}{360} \implies OK \tag{8}$$

[1] The load combinations are listed in 9.3.3.

Figure 1: Trusses key plan.

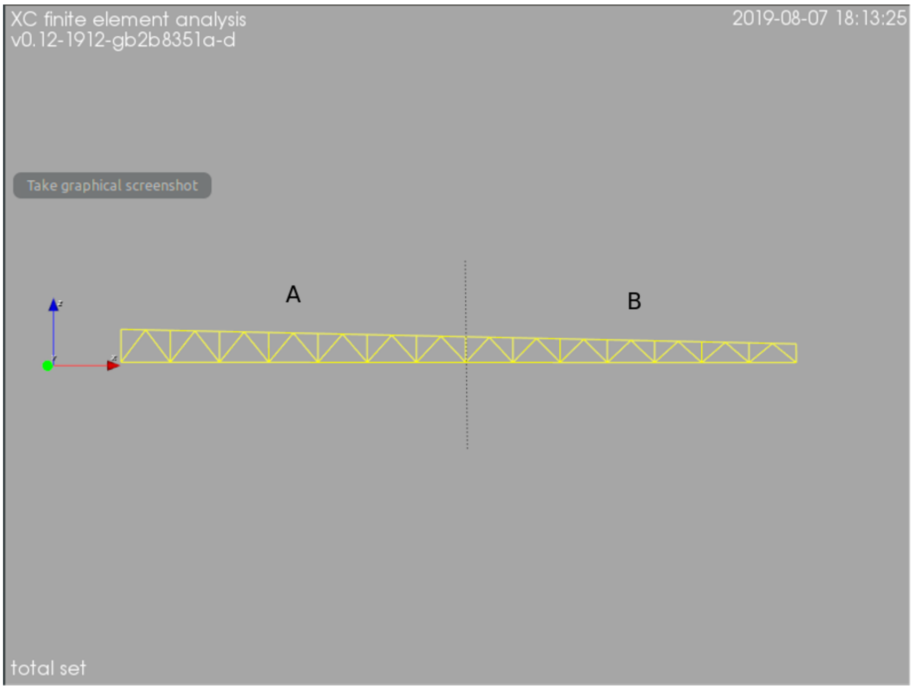


Figure 2: Roof trusses at zones A and B (see key plan in figure 1).

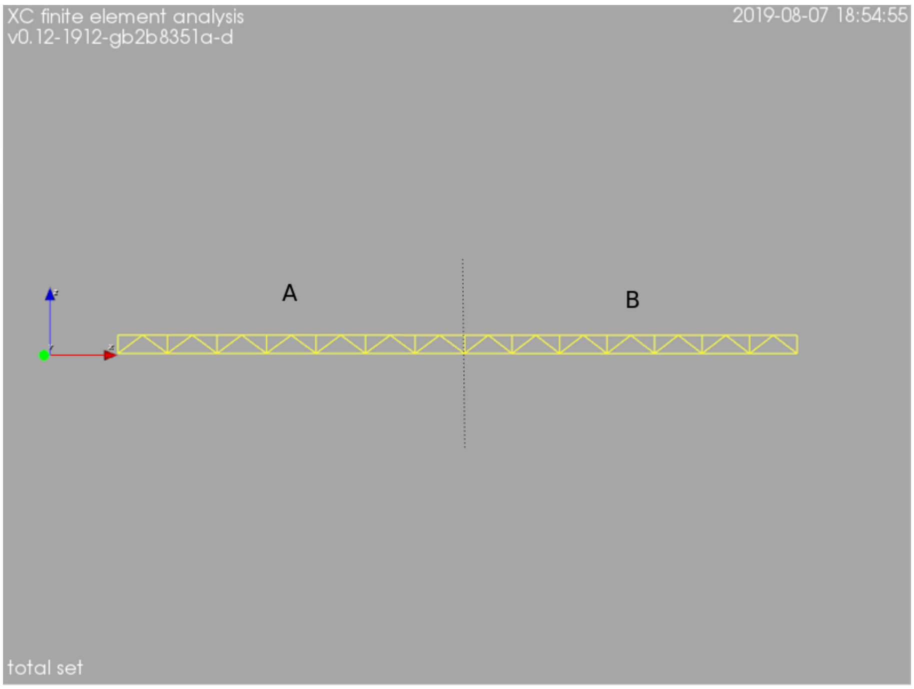


Figure 3: Third floor trusses at zones A and B (see key plan in figure 1).

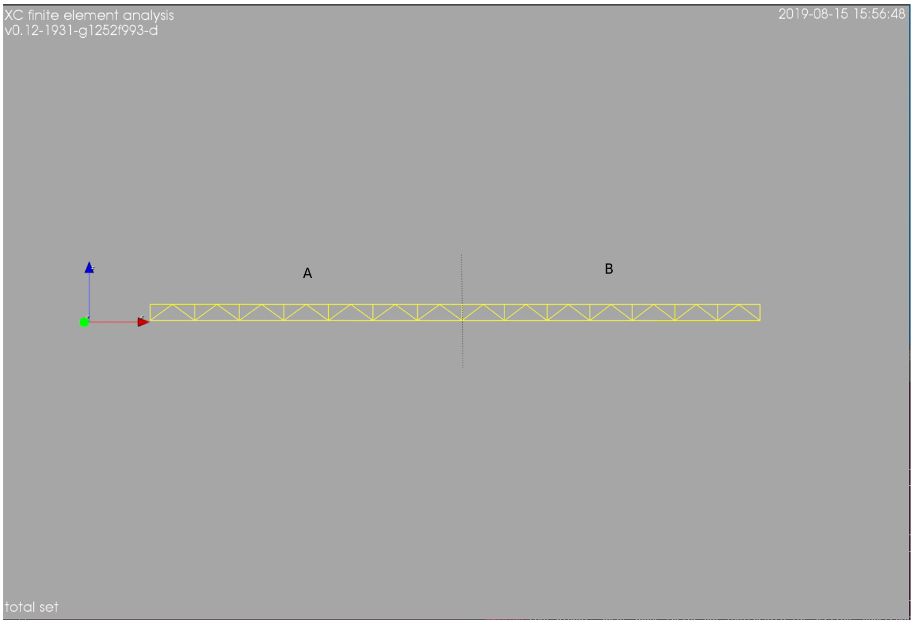


Figure 4: Second floor trusses at zones A and B (see key plan in figure 1).

8.2.3 Trusses A and B. Second floor

The deflection results for those trusses (see figure 4) are as follows:

Load	truss	deflection		truss	deflection	
EQ1608	A	-8.13 mm	(L/1282; L= 10.42 m)	B	-6.81 mm	(L/1464; L= 9.97 m)
EQ1609	A	-23.51 mm	(L/443; L= 10.42 m)	B	-19.71 mm	(L/505; L= 9.97 m)
EQ1610	A	-8.13 mm	(L/1282; L= 10.42 m)	B	-6.81 mm	(L/1464; L= 9.97 m)
EQ1611	A	-19.66 mm	(L/530; L= 10.42 m)	B	-16.48 mm	(L/604; L= 9.97 m)
EQ1612	A	-8.13 mm	(L/1282; L= 10.42 m)	B	-6.81 mm	(L/1464; L= 9.97 m)
EQ1613	A	-19.66 mm	(L/530; L= 10.42 m)	B	-16.48 mm	(L/604; L= 9.97 m)
EQ1615	A	-4.88 mm	(L/2136; L= 10.42 m)	B	-4.08 mm	(L/2440; L= 9.97 m)
LIVE	A	-15.38 mm	(L/677; L= 10.42 m)	B	-12.90 mm	(L/772; L= 9.97 m)

The truss depth is 22 inches and the spacing of the trusses is 12 inches.

$$\Delta_{LL,A} = 15.38\ mm = \frac{L}{677} < \frac{L}{540} \implies OK \tag{9}$$

$$\Delta_{LL,B} = 12.90\ mm = \frac{L}{772} < \frac{L}{540} \implies OK \tag{10}$$

$$\Delta_{TL,A} = 23.51\ mm = \frac{L}{443} < \frac{L}{360} \implies OK \tag{11}$$

$$\Delta_{TL,B} = 19.71\ mm = \frac{L}{505} < \frac{L}{360} \implies OK \tag{12}$$

8.2.4 Trusses C and D. Roof

The deflection results for those trusses (see figure 5) are as follows:

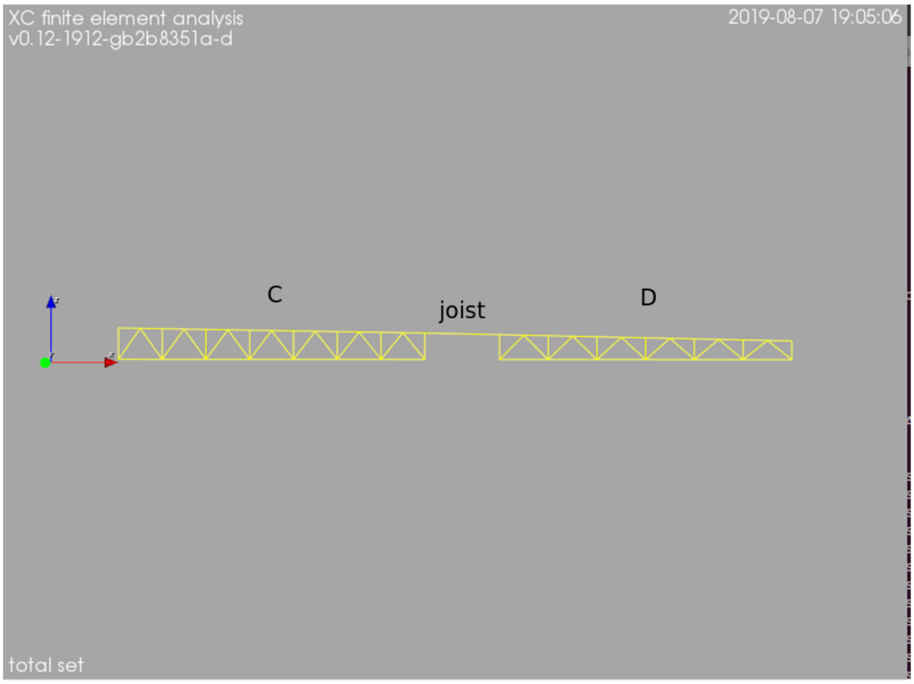


Figure 5: Roof trusses at zones C and D (see key plan in figure 1).

Load	truss	deflection		truss	deflection	
EQ1608	roof(C)	-2.92 mm	(L/3420; L= 10.00 m)	roof(D)	-5.24 mm	(L/1822; L= 9.55 m)
EQ1609	roof(C)	-7.42 mm	(L/1347; L= 10.00 m)	roof(D)	-11.87 mm	(L/803; L= 9.55 m)
EQ1610	roof(C)	-12.34 mm	(L/810; L= 10.00 m)	roof(D)	-19.13 mm	(L/498; L= 9.55 m)
EQ1611	roof(C)	-13.36 mm	(L/748; L= 10.00 m)	roof(D)	-20.64 mm	(L/462; L= 9.55 m)
EQ1612	roof(C)	0.64 mm	(L/15512; L= 10.00 m)	roof(D)	0.03 mm	(L/323264; L= 9.55 m)
EQ1613	roof(C)	-10.68 mm	(L/936; L= 10.00 m)	roof(D)	-16.69 mm	(L/572; L= 9.55 m)
EQ1615	roof(C)	1.81 mm	(L/5512; L= 10.00 m)	roof(D)	2.12 mm	(L/4493; L= 9.55 m)
LIVE	roof(C)	-4.50 mm	(L/2224; L= 10.00 m)	roof(D)	-6.64 mm	(L/1438; L= 9.55 m)

The truss depth is allways greater than 24 inches due to the geometry of the roof. The spacing of the trusses is 24 inches. The spacing of the joists is 32 inches.

$$\Delta_{LL,C} = 4.50\ mm = \frac{L}{2224} < \frac{L}{540} \implies OK \tag{13}$$

$$\Delta_{LL,D} = 6.64\ mm = \frac{L}{772} < \frac{L}{540} \implies OK \tag{14}$$

$$\Delta_{TL,C} = 13.36\ mm = \frac{L}{748} < \frac{L}{360} \implies OK \tag{15}$$

$$\Delta_{TL,D} = 20.64\ mm = \frac{L}{462} < \frac{L}{360} \implies OK \tag{16}$$

8.2.5 Trusses C and D. Third floor

The deflection results for those trusses (see figure 6) are as follows:

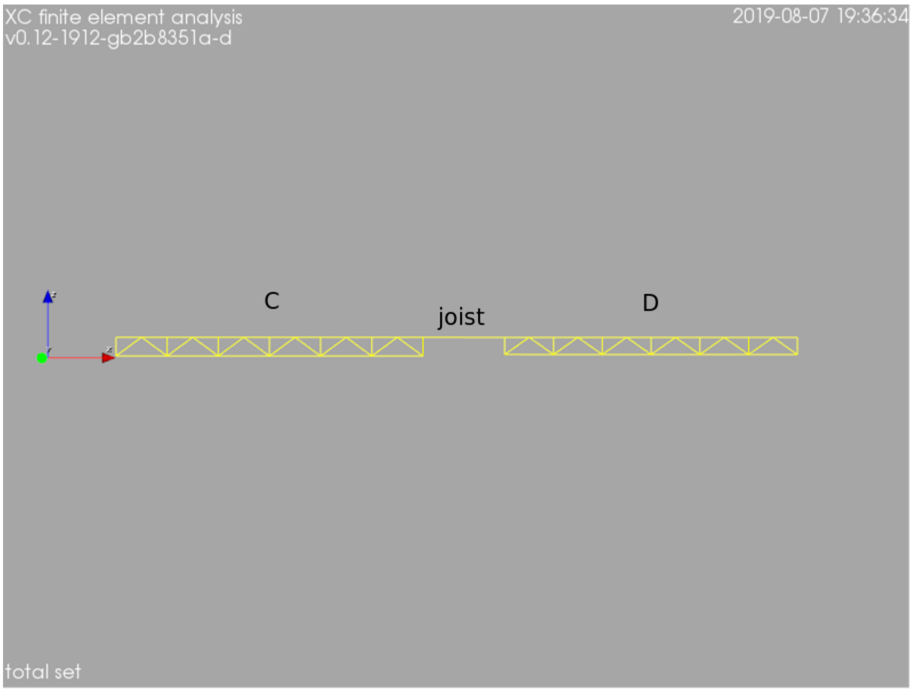


Figure 6: Third floor trusses at zones C and D (see key plan in figure 1).

Load	truss	deflection		truss	deflection	
EQ1608	C	-10.00 mm	(L/982; L= 9.82 m)	D	-8.93 mm	(L/1048; L= 9.37 m)
EQ1609	C	-26.93 mm	(L/364; L= 9.82 m)	D	-24.04 mm	(L/389; L= 9.37 m)
EQ1610	C	-10.00 mm	(L/982; L= 9.82 m)	D	-8.93 mm	(L/1048; L= 9.37 m)
EQ1611	C	-22.70 mm	(L/432; L= 9.82 m)	D	-20.26 mm	(L/462; L= 9.37 m)
EQ1612	C	-10.00 mm	(L/982; L= 9.82 m)	D	-8.93 mm	(L/1048; L= 9.37 m)
EQ1613	C	-22.70 mm	(L/432; L= 9.82 m)	D	-20.26 mm	(L/462; L= 9.37 m)
EQ1615	C	-6.00 mm	(L/1636; L= 9.82 m)	D	-5.36 mm	(L/1747; L= 9.37 m)
LIVE	C	-16.93 mm	(L/580; L= 9.82 m)	D	-15.11 mm	(L/620; L= 9.37 m)

The truss depths are 24 inches for the C truss 22 inches for the D truss. The spacing of the trusses is 24 inches. The spacing of the joists is 32 inches.

$$\Delta_{LL,C} = 16.93\ mm = \frac{L}{580} < \frac{L}{540} \implies OK \tag{17}$$
$$\Delta_{LL,D} = 15.11\ mm = \frac{L}{620} < \frac{L}{540} \implies OK \tag{18}$$
$$\Delta_{TL,C} = 26.93\ mm = \frac{L}{364} < \frac{L}{360} \implies OK \tag{19}$$
$$\Delta_{TL,D} = 24.04\ mm = \frac{L}{389} < \frac{L}{360} \implies OK \tag{20}$$

8.2.6 Trusses C and D. Second floor

The deflection results for those trusses (see figure 7) are as follows:

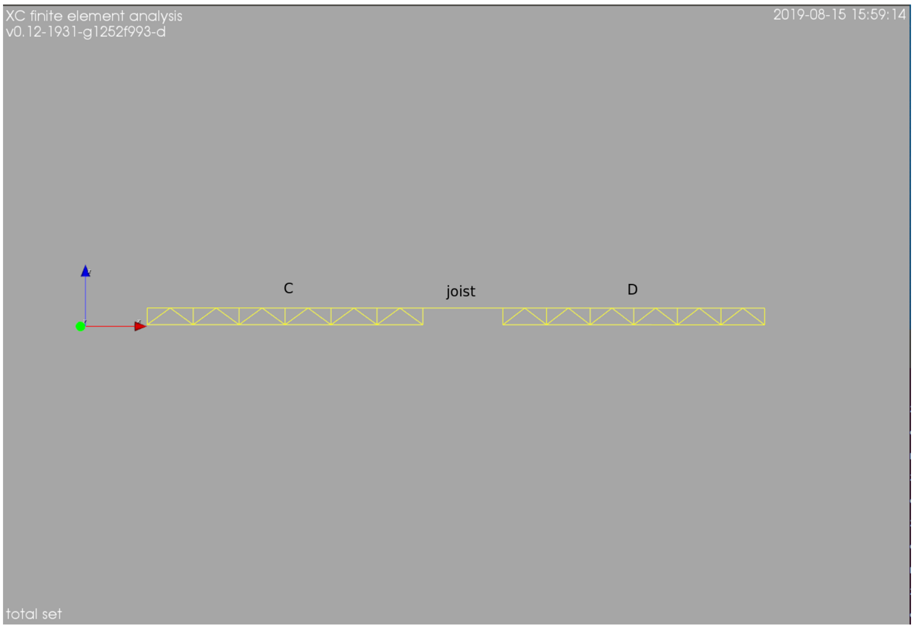


Figure 7: Second floor trusses at zones C and D (see key plan in figure 1).

Load	**truss**	**deflection**		**truss**	**deflection**	
EQ1608	C	-8.43 mm	(L/1070; L= 9.02 m)	D	-9.01 mm	(L/1039; L= 9.37 m)
EQ1609	C	-22.70 mm	(L/397; L= 9.02 m)	D	-24.24 mm	(L/386; L= 9.37 m)
EQ1610	C	-8.43 mm	(L/1070; L= 9.02 m)	D	-9.01 mm	(L/1039; L= 9.37 m)
EQ1611	C	-19.14 mm	(L/471; L= 9.02 m)	D	-20.43 mm	(L/458; L= 9.37 m)
EQ1612	C	-8.43 mm	(L/1070; L= 9.02 m)	D	-9.01 mm	(L/1039; L= 9.37 m)
EQ1613	C	-19.14 mm	(L/471; L= 9.02 m)	D	-20.43 mm	(L/458; L= 9.37 m)
EQ1615	C	-5.06 mm	(L/1783; L= 9.02 m)	D	-5.41 mm	(L/1732; L= 9.37 m)
LIVE	C	-14.27 mm	(L/632; L= 9.02 m)	D	-15.23 mm	(L/614; L= 9.37 m)

The truss depths are 22 inches. The spacing of the trusses is 24 inches. The spacing of the joists is 32 inches.

$$\Delta_{LL,C} = 14.27\ mm = \frac{L}{632} < \frac{L}{540} \implies OK \tag{21}$$

$$\Delta_{LL,D} = 15.23\ mm = \frac{L}{614} < \frac{L}{540} \implies OK \tag{22}$$

$$\Delta_{TL,C} = 22.70\ mm = \frac{L}{397} < \frac{L}{360} \implies OK \tag{23}$$

$$\Delta_{TL,D} = 24.24\ mm = \frac{L}{386} < \frac{L}{360} \implies OK \tag{24}$$

8.2.7 Truss E. Roof

The deflection results for those trusses (see figure 8) are as follows:

Load	**truss**	**deflection**	
EQ1608	3E	-4.67 mm	(L/2025; L= 9.47 m)
EQ1609	3E	-11.56 mm	(L/819; L= 9.47 m)
EQ1610	3E	-19.09 mm	(L/496; L= 9.47 m)
EQ1611	3E	-20.65 mm	(L/458; L= 9.47 m)
EQ1612	3E	0.79 mm	(L/11978; L= 9.47 m)
EQ1613	3E	-16.55 mm	(L/572; L= 9.47 m)
EQ1615	3E	2.66 mm	(L/3559; L= 9.47 m)
LIVE	3E	-6.89 mm	(L/1375; L= 9.47 m)

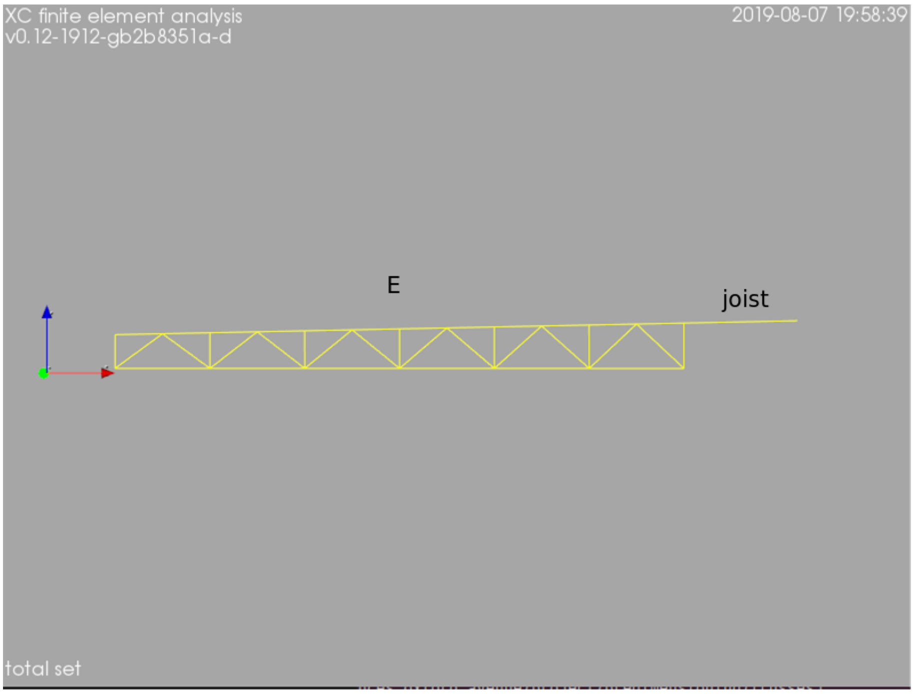


Figure 8: Roof truss at zone E (see key plan in figure 1).

The truss depth is allways greater than 24 inches due to the geometry of the roof. The spacing of the trusses is 24 inches. The spacing of the joists is 32 inches.

$$\Delta_{LL,E} = 6.89\ mm = \frac{L}{1375} < \frac{L}{540} \implies OK \tag{25}$$

$$\Delta_{TL,E} = 20.65\ mm = \frac{L}{458} < \frac{L}{360} \implies OK \tag{26}$$

$$\tag{27}$$

8.2.8 Truss E. Third floor

The deflection results for those trusses (see figure 9) are as follows:

Load	truss	deflection	
EQ1608	2E	-8.65 mm	(L/1095; L= 9.47 m)
EQ1609	2E	-23.30 mm	(L/406; L= 9.47 m)
EQ1610	2E	-8.65 mm	(L/1095; L= 9.47 m)
EQ1611	2E	-19.63 mm	(L/482; L= 9.47 m)
EQ1612	2E	-8.65 mm	(L/1095; L= 9.47 m)
EQ1613	2E	-19.63 mm	(L/482; L= 9.47 m)
EQ1615	2E	-5.19 mm	(L/1825; L= 9.47 m)
LIVE	2E	-14.65 mm	(L/646; L= 9.47 m)

The truss depth 24 inches. The spacing of the trusses is 24 inches and the spacing of the joists is 32 inches.

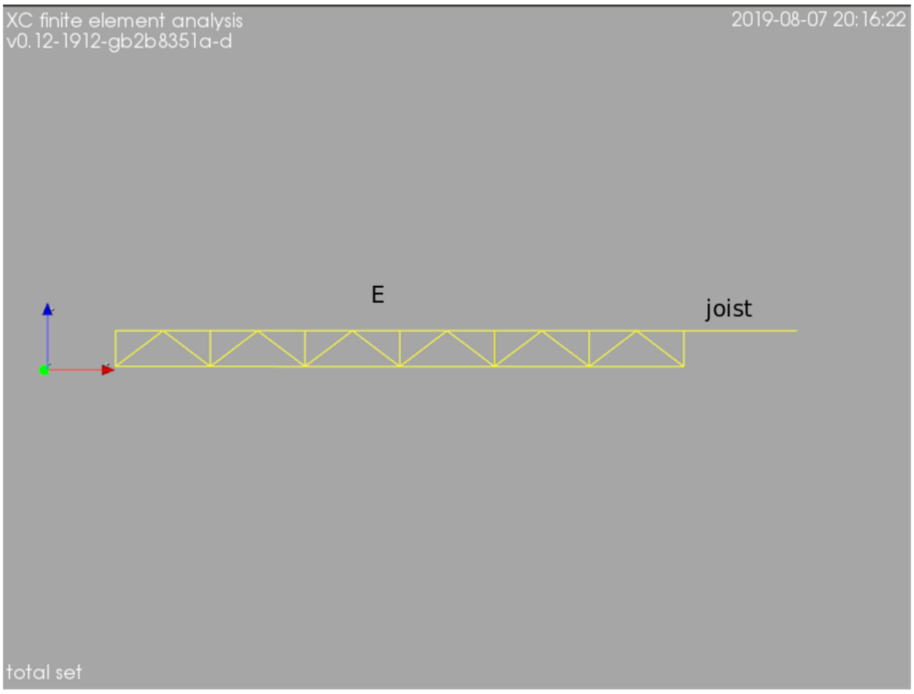


Figure 9: Third floor truss at zone E (see key plan in figure 1).

$$\Delta_{LL,E} = 14.65\ mm = \frac{L}{646} < \frac{L}{540} \implies OK \quad (28)$$
$$\Delta_{TL,E} = 23.30\ mm = \frac{L}{406} < \frac{L}{360} \implies OK \quad (29)$$
$$(30)$$

8.2.9 Truss E. Second floor

The deflection results for those trusses (see figure 10) are as follows:

Load	truss	deflection
EQ1608 2E: -7.18 mm	(L/1208; L= 8.67 m)	
EQ1609 2E: -19.34 mm	(L/448; L= 8.67 m)	
EQ1610 2E: -7.18 mm	(L/1208; L= 8.67 m)	
EQ1611 2E: -16.30 mm	(L/532; L= 8.67 m)	
EQ1612 2E: -7.18 mm	(L/1208; L= 8.67 m)	
EQ1613 2E: -16.30 mm	(L/532; L= 8.67 m)	
EQ1615 2E: -4.31 mm	(L/2013; L= 8.67 m)	
LIVE 2E: -12.16 mm	(L/713; L= 8.67 m)	

The truss depth 22 inches. The spacing of the trusses is 24 inches and the spacing of the joists is 32 inches.

$$\Delta_{LL,E} = 12.16\ mm = \frac{L}{713} < \frac{L}{540} \implies OK \quad (31)$$
$$\Delta_{TL,E} = 19.34\ mm = \frac{L}{448} < \frac{L}{360} \implies OK \quad (32)$$
$$(33)$$

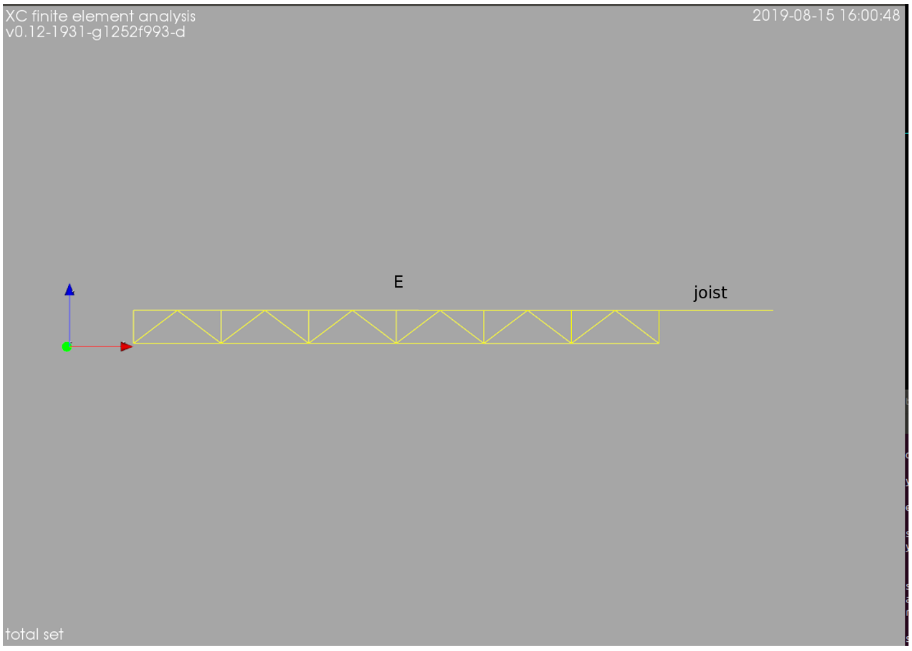


Figure 10: Second floor truss at zone E (see key plan in figure 1).

8.3 Joists

8.3.1 Corridor floor sheathing

Three layers of 4x8 foot, 19/32 plywood panels are installed as corridor floor sheathing over corridors joists (nominal 3.5 inch wide) spaced 32 inches on center. The panels are installed with the long panel direction (strength axis) perpendicular to the corridor joists. The design loads are:

$$q_{live} = 1.92\ kN/m^2 (40\ psf) \tag{34}$$
$$q_{dead} = 0.72\ kN/m^2 (15\ psf) \tag{35}$$

The allowable live load deflection is span/540 and the allowable total load deflection span/360.

Structural design of the panels.

Mechanical properties of the plywood panel. The mechanical properties used to compute the floor deflection are the elastic modulus $E = 4200\ MPa$ and its thickness $t = 15.09\ mm$ (19/32 inch). Each layer works independently, otherwise said, they are connected only over the joists.

Bending stiffness. The deflection obtained under live load is:

$$\Delta_{LL} = 1.34\ mm = \frac{span}{607} < \frac{span}{540} \implies OK \tag{36}$$

and the deflection under total load is:

$$\Delta_{TL} = 1.84\ mm = \frac{span}{441} < \frac{span}{360} \implies OK \tag{37}$$

Bending strength. The allowable bending stress for the 5-ply plywood panel is $F_b = 4.33\ MPa$ (the panel grade and construction factors are already been applied to this capacity). The load duration factor for the live load on the corridor is $C_D = 1.6$. The adjusted allowable bending stress is therefore $F_b' = 6.94\ MPa$.

The maximum bending stress obtained under total load (three-span condition) is:

$$\sigma_{max} = 1.69\ MPa < 6.94\ MPa = F_b' \implies OK \tag{38}$$

Shear strength. The allowable shear stress of the panel is $F_v = 0.2\ MPa$ and the adjusted allowable shear stress is (under the same conditions that we used for the bending stress) $F_v' = 0.33\ MPa$.

The maximum shear stress obtained under total load is:

$$\tau_{max} = 0.04\ MPa < 0.33\ MPa = F_v' \implies OK \tag{39}$$

Fire design of the panels. According the table 9 the time assigned to a 19/32 inch panel is 15 minutes, so after 30 minutes of fire only one of the three panels remains in place. Accordingly, we perform the bending and shear checks to the remaining panel.

Bending strength. The maximum bending stress obtained under total load (three-span condition) is:

$$\sigma_{max} = 4.59\ MPa < 6.94\ MPa = F_b' \implies OK \tag{40}$$

Shear strength. The maximum shear stress obtained under total load is:

$$\tau_{max} = 0.12\ MPa < 0.33\ MPa = F_v' \implies OK \tag{41}$$

8.3.2 Corridor Joists

Simply supported 3.5x6 LVL floor joists span a maximum of $L = 2.49\ m$ (94.25 inches) and are spaced at $s = 0.81\ m$ (32 inches). The design loads are:

$$q_{live} = 1.92\ kN/m^2 (40\ psf) \tag{42}$$

$$q_{dead} = 0.72\ kN/m^2 (15\ psf) \tag{43}$$

Timber decking nailed to the compression edge of the joists provides lateral bracing for at least the same fire resistance time as the joists (i.e. $C_L = 1.0$).

Structural design of the joist.

Loads.

$$w_{load} = s \cdot (q_{dead} + q_{live}) = 1.56\ kN/m \tag{44}$$

722.6.2 Walls, Floors and Roofs

These procedures apply to both load-bearing and nonload-bearing assemblies.

TABLE 722.6.2(1) TIME ASSIGNED TO WALLBOARD MEMBRANES[a, b, c, d]

DESCRIPTION OF FINISH	TIME[e](minutes)
$^3/_8$-inch wood structural panel bonded with exterior glue	5
$^{15}/_{32}$-inch wood structural panel bonded with exterior glue	10
$^{19}/_{32}$-inch wood structural panel bonded with exterior glue	15
$^3/_8$-inch gypsum wallboard	10
$^1/_2$-inch gypsum wallboard	15
$^5/_8$-inch gypsum wallboard	30
$^1/_2$-inch Type X gypsum wallboard	25
$^5/_8$-inch Type X gypsum wallboard	40
Double $^3/_8$-inch gypsum wallboard	25
$^1/_2$-inch + $^3/_8$-inch gypsum wallboard	35
Double $^1/_2$-inch gypsum wallboard	40

For SI: 1 inch = 25.4 mm.

Table 9: Time assigned to wallboard membranes

Internal forces. Maximum induced moment:

$$M_{max} = w_{load}\frac{L^2}{8} = 1.54\ kNm \tag{45}$$

Maximum induced shear:

$$V_{max} = w_{load}\frac{L}{2} = 2.57\ kN \tag{46}$$

Joist mechanical properties. Joist section modulus:

$$S_s = 344.13 \times 10^{-6}\ m^3 \tag{47}$$

Tabulated bending stress:

$$F_b = 21.59\ MPa \tag{48}$$

Adjusted allowable bending stress with $C_r = 1.0$, $C_D = 1.0$,$C_M = 1.0$, $C_t = 1.0$,$C_V = 0.62$:

$$F_b' = 13.59\ MPa \tag{49}$$

Tabulated shear stress:

$$F_v = 1.97\ MPa \tag{50}$$

Adjusted allowable shear stress with $C_D = 1.0$,$C_M = 1.0$, $C_t = 1.0$:

$$F_v' = 1.97\ MPa \tag{51}$$

Structural bending check. Design resisting moment:

$$M_s' = 4.67\ kNm \tag{52}$$

Structural bending check: $M_s' = 4.67 > 1.54 = M_{max} \implies OK$

Structural shear check. Design resisting shear:

$$V_s' = 17.74\ kN \tag{53}$$

Structural shear check: $V_s' = 17.74 > 2.57 = V_{max} \implies OK$

Fire design of the joist. For the fire design of the joist, mass loss due to charring is conservatively neglected, so the loading is unchanged. Therefore, the maximum induced moment and shear are unchanged. The fire resistance must be calculated.

Mechanical properties of the burned section. Effective char depth:

$$a_{eff} = 0.7 \times 10^{-3} \times 30 + 7 \times 10^{-3} = 28\ mm \tag{54}$$

section modulus for a joist exposed on three sides:

$$S_s = 84.86 \times 10^{-6}\ m^3 \tag{55}$$

shear area for a beam exposed on three sides:

$$A_f = 40.93\ cm^2 \tag{56}$$

Adjusted allowable bending stress with $C_{fire} = 2.85$, $C_r = 1.0$, $C_D = 1.0$,$C_M = 1.0$, $C_t = 1.0$,$C_V = 0.62$:

$$F_{b,f}' = 38.74\ MPa \tag{57}$$

Adjusted allowable shear stress with $C_{fire} = 2.85$, $C_D = 1.0$,$C_M = 1.0$, $C_t = 1.0$:

$$F_{v,f}' = 5.40\ MPa \tag{58}$$

Structural bending check. Design resisting moment:

$$M'_f = 3.28\ kNm \tag{59}$$

Structural bending check: $M'_s = 3.28 > 1.54 = M_{max} \implies OK$

Structural shear check. Design resisting shear:

$$V'_f = 14.74\ kN \tag{60}$$

Structural shear check: $V'_s = 14.74 > 2.57 = V_{max} \implies OK$

8.3.3 Joists under storage/HVAC floor

Simply supported 3.5x9.25 LVL floor joists span a maximum of $L = 2.9\ m$ and are spaced at $s = 0.81\ m$ (32 inches). The design loads are:

$$q_{live} = 5.99\ kN/m^2 (125\ psf) \tag{61}$$

$$q_{dead} = 0.72\ kN/m^2 (15\ psf) \tag{62}$$

Timber decking nailed to the compression edge of the joists provides lateral bracing (i.e. $C_L = 1.0$).

Structural design of the joist.

Loads.

$$w_{load} = s \cdot (q_{dead} + q_{live}) = 5.45\ kN/m \tag{63}$$

Internal forces. Maximum induced moment:

$$M_{max} = w_{load}\frac{L^2}{8} = 5.79\ kNm \tag{64}$$

Maximum induced shear:

$$V_{max} = w_{load}\frac{L}{2} = 7.94\ kN \tag{65}$$

Joist mechanical properties. Joist section modulus:

$$S_s = 817.90 \times 10^{-6}\ m^3 \tag{66}$$

Tabulated bending stress:

$$F_b = 20.58\ MPa \tag{67}$$

Adjusted allowable bending stress with $C_r = 1.0$, $C_D = 1.0$,$C_M = 1.0$, $C_t = 1.0$

$$F'_b = 20.58\ MPa \tag{68}$$

Tabulated shear stress:

$$F_v = 1.97\ MPa \tag{69}$$

Adjusted allowable shear stress with $C_D = 1.0$,$C_M = 1.0$, $C_t = 1.0$:

$$F'_v = 1.97\ MPa \tag{70}$$

Structural bending check. Design resisting moment:

$$M'_s = 16.83\ kNm \tag{71}$$

Structural bending check: $M'_s = 16.83 > 5.79 = M_{max} \implies OK$

Structural shear check. Design resisting shear:

$$V'_s = 27.36\ kN \tag{72}$$

Structural shear check: $V'_s = 27.36 > 7.94 = V_{max} \implies OK$

Bending stiffness. The deflection obtained under live load is:

$$\Delta_{LL} = 3.45\ mm = \frac{span}{845} < \frac{span}{540} \implies OK \tag{73}$$

and the deflection under total load is:

$$\Delta_{TL} = 3.86\ mm = \frac{span}{754} < \frac{span}{360} \implies OK \tag{74}$$

8.4 Headers

8.4.1 Third floor enclosed balconies headers (H3.1 to H3.3)

Simply supported 1.75x14 LSL 1.55E headers.

Structural design of the header.

Design load.

$$w_{load} = 1.75\ kN/m \tag{75}$$

Internal forces. Maximum induced moment:

$$M_{max} = w_{load}\frac{L^2}{8} = 1.65\ kNm \tag{76}$$

Maximum induced shear:

$$V_{max} = w_{load}\frac{L}{2} = 2.40\ kN \tag{77}$$

Structural bending check. Design resisting moment:

$$M'_s = 14.96\ kNm \tag{78}$$

Structural bending check: $M'_s = 14.96 > 1.65 = M_{max} \implies OK$

Structural shear check. Design resisting shear:

$$V'_s = 29.79\ kN \tag{79}$$

Structural shear check: $V'_s = 29.79 > 2.40 = V_{max} \implies OK$

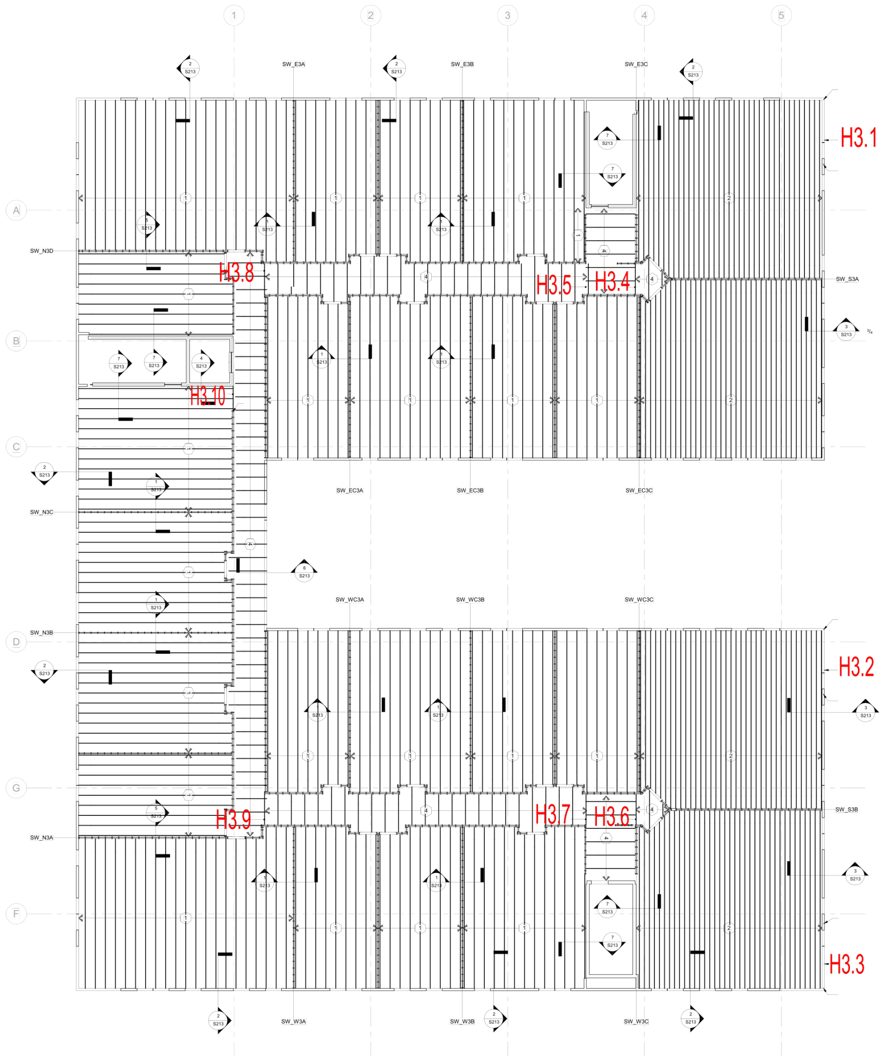


Figure 11: Headers key plan. Roof

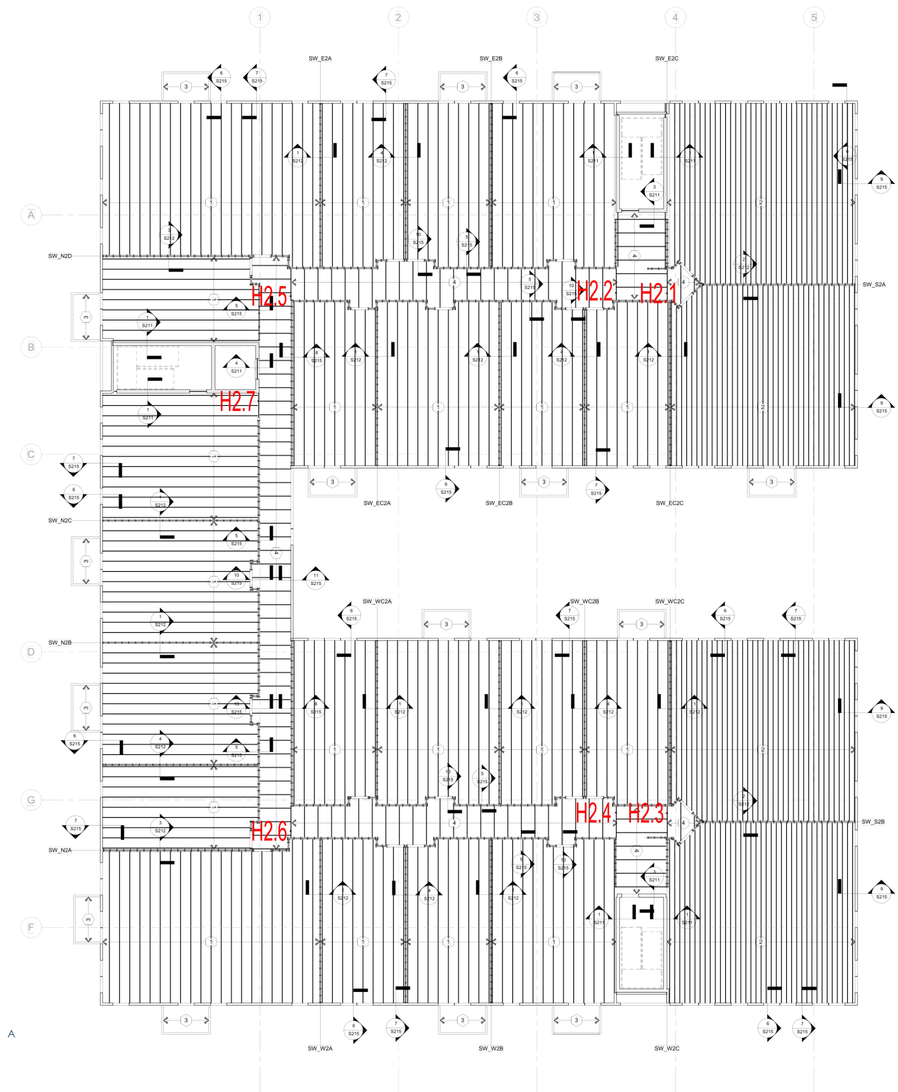


Figure 12: Headers key plan. Third floor

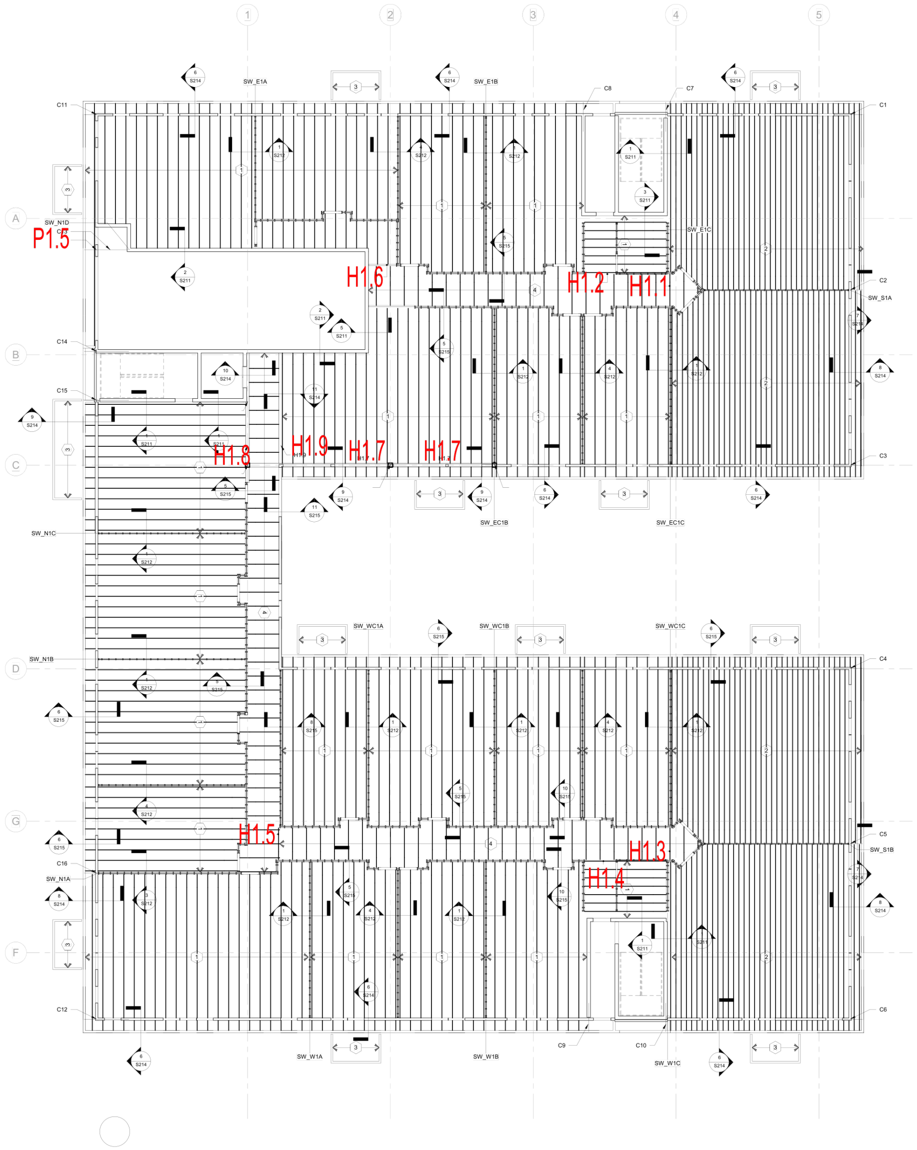


Figure 13: Headers key plan. Second floor

Bending stiffness. The deflection obtained is:

$$\Delta_{TL} = 0.72\ mm = \frac{L}{3781} < \frac{L}{360} \implies OK \tag{80}$$

8.4.2 Corridor headers (H3.4 to H3.9, H2.1 to H2.6 and H1.1 to H1.6)

Simply supported 3.5x7-1/4” LVL headers.

Structural design of the header.

Design load.

$$w_{load} = 5.26\ kN/m \tag{81}$$

Internal forces. Maximum induced moment:

$$M_{max} = w_{load}\frac{L^2}{8} = 2.29\ kNm \tag{82}$$

Maximum induced shear:

$$V_{max} = w_{load}\frac{L}{2} = 5.21\ kN \tag{83}$$

Structural bending check. Design resisting moment:

$$M_s' = 10.63\ kNm \tag{84}$$

Structural bending check: $M_s' = 10.63 > 2.29 = M_{max} \implies OK$

Structural shear check. Design resisting shear:

$$V_s' = 21.44\ kN \tag{85}$$

Structural shear check: $V_s' = 21.44 > 5.21 = V_{max} \implies OK$

Bending stiffness. The deflection obtained is:

$$\Delta_{TL} = 0.07\ mm = \frac{L}{27939} < \frac{L}{360} \implies OK \tag{86}$$

Fire design of the header. For the fire design of the header, mass loss due to charring is conservatively neglected, so the loading is unchanged. Therefore, the maximum induced moment and shear are unchanged. The fire resistance must be calculated.

Mechanical properties of the burned section. Effective char depth:

$$a_{eff} = 0.7 \times 10^{-3} \times 30 + 7 \times 10^{-3} = 28\ mm \tag{87}$$

section modulus for a joist exposed on three sides:

$$S_s = 133.70 \times 10^{-6}\ m^3 \tag{88}$$

shear area for a beam exposed on three sides:

$$A_f = 51.37\ cm^2 \tag{89}$$

Adjusted allowable bending stress with $C_{fire} = 2.85$, $C_r = 1.0$, $C_D = 1.0$,$C_M = 1.0$, $C_t = 1.0$

$$F'_{b,f} = 60.26\ MPa \tag{90}$$

Adjusted allowable shear stress with $C_{fire} = 2.85$, $C_D = 1.0$,$C_M = 1.0$, $C_t = 1.0$:

$$F'_{v,f} = 5.40\ MPa \tag{91}$$

Structural bending check. Design resisting moment:

$$M'_f = 8.06\ kNm \tag{92}$$

Structural bending check: $M'_s = 8.06 > 2.29 = M_{max} \implies OK$

Structural shear check. Design resisting shear:

$$V'_f = 18.51\ kN \tag{93}$$

Structural shear check: $V'_s = 18.51 > 5.21 = V_{max} \implies OK$

8.4.3 Headers H3.10 and H2.7

Simply supported 3.5x14" LVL header.

Structural design of the header.

Design load.

$$w_{load} = 18.11\ kN/m \tag{94}$$

Internal forces. Maximum induced moment:

$$M_{max} = w_{load}\frac{L^2}{8} = 10.94\ kNm \tag{95}$$

Maximum induced shear:

$$V_{max} = w_{load}\frac{L}{2} = 27.61\ kN \tag{96}$$

Structural bending check. Design resisting moment:

$$M'_s = 36.65\ kNm \tag{97}$$

Structural bending check: $M'_s = 36.65 > 10.94 = M_{max} \implies OK$

Structural shear check. Design resisting shear:

$$V'_s = 41.41\ kN \tag{98}$$

Structural shear check: $V'_s = 41.41 > 27.61 = V_{max} \implies OK$

Bending stiffness. The deflection obtained is:

$$\Delta_{TL} = 0.33\ mm = \frac{L}{6027} < \frac{L}{360} \implies OK \tag{99}$$

Fire design of the header. For the fire design of the header, mass loss due to charring is conservatively neglected, so the loading is unchanged. Therefore, the maximum induced moment and shear are unchanged. The fire resistance must be calculated.

Mechanical properties of the burned section. Effective char depth:

$$a_{eff} = 0.7 \times 10^{-3} \times 30 + 7 \times 10^{-3} = 28\ mm \tag{100}$$

section modulus for a joist exposed on three sides:

$$S_s = 588.48 \times 10^{-6}\ m^3 \tag{101}$$

shear area for a beam exposed on three sides:

$$A_f = 107.78\ cm^2 \tag{102}$$

Adjusted allowable bending stress with $C_{fire} = 2.85$, $C_r = 1.0$, $C_D = 1.0$,$C_M = 1.0$, $C_t = 1.0$

$$F'_{b,f} = 55.74\ MPa \tag{103}$$

Adjusted allowable shear stress with $C_{fire} = 2.85$, $C_D = 1.0$,$C_M = 1.0$, $C_t = 1.0$:

$$F'_{v,f} = 5.40\ MPa \tag{104}$$

Structural bending check. Design resisting moment:

$$M'_f = 32.8\ kNm \tag{105}$$

Structural bending check: $M'_s = 32.80 > 10.93 = M_{max} \implies OK$

Structural shear check. Design resisting shear:

$$V'_f = 38.82\ kN \tag{106}$$

Structural shear check: $V'_s = 38.82 > 27.605 = V_{max} \implies OK$

8.4.4 Header H1.9

Simply supported 5.25x18” LVL 1.55E beam.

Structural design of the header.

Design load.

$$w_{load} = 4.77\ kN/m \tag{107}$$

Internal forces. Maximum induced moment:

$$M_{max} = w_{load}\frac{L^2}{8} = 25.63\ kNm \tag{108}$$

Maximum induced shear:

$$V_{max} = w_{load}\frac{L}{2} = 15.73\ kN \tag{109}$$

Structural bending check. Design resisting moment:

$$M_s' = 87.66\ kNm \tag{110}$$

Structural bending check: $M_s' = 87.66 > 25.63 = M_{max} \implies OK$

Structural shear check. Design resisting shear:

$$V_s' = 79.87\ kN \tag{111}$$

Structural shear check: $V_s' = 79.87 > 15.73 = V_{max} \implies OK$

Bending stiffness. The deflection obtained is:

$$\Delta_{TL} = 8.97\ mm = \frac{L}{735} < \frac{L}{600} \implies OK \tag{112}$$

8.4.5 Facade headers

Simply supported 3.5x11 7/8 LSL 1.55E header.

Structural design of the header.

Design load.

$$w_{load} = 75.91\ kN/m \tag{113}$$

Internal forces. Maximum induced moment:

$$M_{max} = w_{load}\frac{L^2}{8} = 8.26\ kNm \tag{114}$$

Maximum induced shear:

$$V_{max} = w_{load}\frac{L}{2} = 43.38\ kN \tag{115}$$

Structural bending check. Design resisting moment:

$$M_s' = 21.96\ kNm \tag{116}$$

Structural bending check: $M_s' = 21.96 > 8.26 = M_{max} \implies OK$

Structural shear check. Design resisting shear:

$$V_s' = 50.53\ kN \tag{117}$$

Structural shear check: $V_s' = 50.53 > 43.38 = V_{max} \implies OK$

Bending stiffness. The deflection obtained is:

$$\Delta_{TL} = 0.41\ mm = \frac{L}{2758} < \frac{L}{600} \implies OK \tag{118}$$

8.4.6 Corridor headers

Simply supported 3.5x16 LSL 1.55E header.

Structural design of the header.

Design load.

$$w_{load} = 118.11\ kN/m \tag{119}$$

Internal forces. Maximum induced moment:

$$M_{max} = w_{load}\frac{L^2}{8} = 14.63\ kNm \tag{120}$$

Maximum induced shear:

$$V_{max} = w_{load}\frac{L}{2} = 72.0\ kN \tag{121}$$

Structural bending check. Design resisting moment:

$$M'_s = 48.00\ kNm \tag{122}$$

Structural bending check: $M'_s = 48.00 > 14.63 = M_{max} \implies OK$

Structural shear check. Design resisting shear:

$$V'_s = 76.60\ kN \tag{123}$$

Structural shear check: $V'_s = 76.60 > 72.0 = V_{max} \implies OK$

Bending stiffness. The deflection obtained is:

$$\Delta_{TL} = 0.26\ mm = \frac{L}{4581} < \frac{L}{600} \implies OK \tag{124}$$

8.5 Steel beams

8.5.1 Steel beam at courtyard facade

Continuous beam supporting the second floor trusses between the axes 1, 2 and 3 (see figure 14). The beam has two equal spans; L= 8.31 m ($27' - 3''$).

Design loads. The design loads are show in table 10. The beam then carries a load of 19.785 kN each $0.6m(24'')$.

Structural design of the beam.

Loads.

$$w_{load} = 19.785\ kN/m \tag{125}$$

Internal forces on each structural channel. Maximum induced moment:

$$M_{max} = 170.61\ kNm \tag{126}$$

Maximum induced shear:

$$V_{max} = 102.71\ kN \tag{127}$$

CD_reactions

comb	floor	truss name	Rz (kN)	Rz (kN)	truss name	Rz (kN)	Rz (kN)
EQ1608	roof	3C	2.25	3.08	3D	2.91	2.11
EQ1609	roof	3C	5.18	6.84	3D	6.42	4.78
EQ1610	roof	3C	8.38	10.96	3D	10.26	7.70
EQ1611	roof	3C	9.04	11.81	3D	11.05	8.30
EQ1612	roof	3C	-0.07	0.09	3D	0.13	-0.02
EQ1613	roof	3C	7.30	9.57	3D	8.96	6.71
EQ1615	roof	3C	-0.97	-1.14	3D	-1.04	-0.86
			9.04	**11.81**		**11.05**	**8.30**
		kN/m	**14.84**	**19.37**		**18.13**	**13.62**
EQ1608	3rd floor	2C	3.78	4.86	2D	4.69	3.62
EQ1609	3rd floor	2C	9.64	12.14	2D	11.71	9.20
EQ1610	3rd floor	2C	3.78	4.86	2D	4.69	3.62
EQ1611	3rd floor	2C	8.17	10.32	2D	9.95	7.81
EQ1612	3rd floor	2C	3.78	4.86	2D	4.69	3.62
EQ1613	3rd floor	2C	8.17	10.32	2D	9.95	7.81
EQ1615	3rd floor	2C	2.27	2.92	2D	2.82	2.17
			9.64	**12.14**		**11.71**	**9.20**
		kN/m	**15.81**	**19.92**		**19.20**	**15.10**
EQ1608		3C+2C	6.03	7.94	3D+2D	7.61	5.72
EQ1609		3C+2C	14.82	18.98	3D+2D	18.13	13.98
EQ1610		3C+2C	12.16	15.82	3D+2D	14.95	11.31
EQ1611		3C+2C	17.22	22.13	3D+2D	21.00	16.11
EQ1612		3C+2C	3.71	4.95	3D+2D	4.82	3.60
EQ1613		3C+2C	15.47	19.89	3D+2D	18.92	14.52
EQ1615		3C+2C	1.30	1.78	3D+2D	1.78	1.31
			17.22	**22.13**		**21.00**	**16.11**
		kN/m	**28.24**	**36.30**		**34.46**	**26.43**
EQ1608	2nd floor	1C	3.04	5.17	1D	5.29	3.16
EQ1609	2nd floor	1C	8.19	13.24	1D	13.56	8.52
EQ1610	2nd floor	1C	3.04	5.17	1D	5.29	3.16
EQ1611	2nd floor	1C	6.90	11.22	1D	11.49	7.18
EQ1612	2nd floor	1C	3.04	5.17	1D	5.29	3.16
EQ1613	2nd floor	1C	6.90	11.22	1D	11.49	7.18
EQ1615	2nd floor	1C	1.82	3.10	1D	3.17	1.90
LIVE	2nd floor	1C	5.15	8.08	1D	8.27	5.36
			8.19	**13.24**		**13.56**	**8.52**
		kN/m	**13.44**	**21.72**		**22.24**	**13.98**
EQ1608		3C+2C+1C	9.07	13.10	3D+2D+1D	12.89	8.88
EQ1609		3C+2C+1C	23.01	32.22	3D+2D+1D	31.68	22.50
EQ1610		3C+2C+1C	15.20	20.98	3D+2D+1D	20.23	14.48
EQ1611		3C+2C+1C	24.12	33.35	3D+2D+1D	32.49	23.29
EQ1612		3C+2C+1C	6.75	10.12	3D+2D+1D	10.11	6.76
EQ1613		3C+2C+1C	22.38	31.11	3D+2D+1D	30.41	21.70
EQ1615		3C+2C+1C	3.12	4.87	3D+2D+1D	4.95	3.21
			24.12	**33.35**		**32.49**	**23.29**
		kN/m	**39.57**	**54.71**		**53.30**	**38.21**

Table 10: Steel beam at courtyard facade.Trusses reactions.

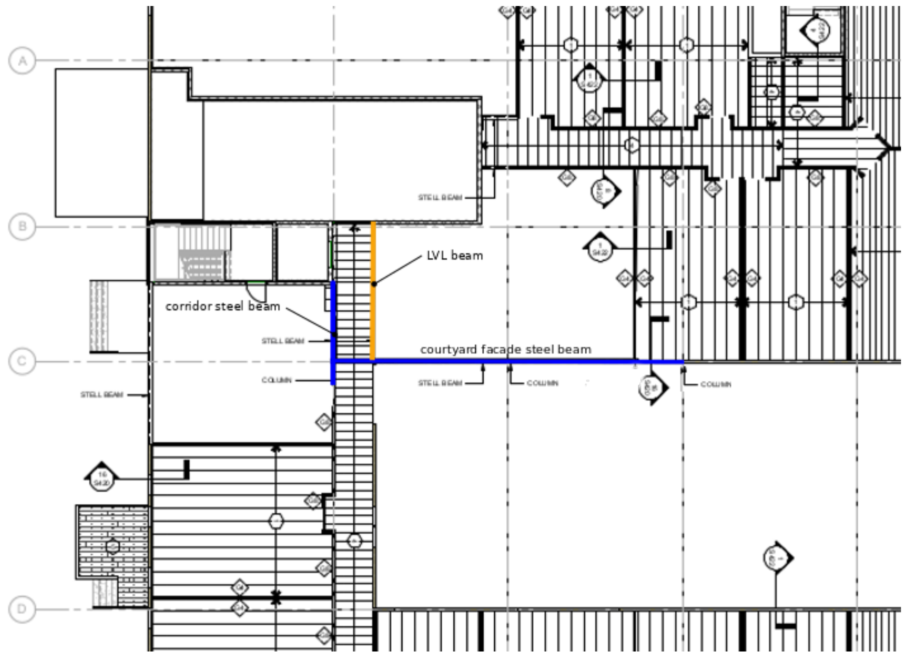


Figure 14: Second floor beams key plan.

Structural channel (C380X50.4) mechanical properties. Steel: ASTM A-572
Shear strength:

$$V_u = 463.52\ kN \tag{128}$$

Structural shear check: $V_u = 463.52 > 102.71 = V_{max} \implies OK$
Resisting moment:

$$M_u = 171.88\ kN \cdot m \tag{129}$$

Structural bending check: $M_U = 171.88 > 170.61 = M_{max} \implies OK$

Bending stiffness. The deflection obtained is:

$$\Delta_{TL} = 18.72\ mm = \frac{span}{444} < \frac{L}{360} \implies OK \tag{130}$$

8.5.2 Steel beam at corridor

This beam supports the second floor trusses near the elevator well (see figure 14) . It has a main span of 3.97 m long ($13' - 5/16''$) and a cantilever that spans 1.04 m ($3' - 4''$).

Design loads. The design loads are show in table 11. The beam then carries a load of 31.66 kN each $0.6m(24'')$.

Structural design of the beam.

Loads.

$$w_{load} = 51.94\ kN/m \tag{131}$$

					roof_truss_E_reactions			
comb	floor	truss name	Rx (kN)	Ry (kN)	Rz (kN)	Rx (kN)	Ry (kN)	Rz (kN)
EQ1608	roof	3E	0.00	0.00	2.19	0.00	0.00	2.90
EQ1609	roof	3E	0.00	0.00	4.96	0.00	0.00	6.41
EQ1610	roof	3E	0.00	0.00	7.99	0.00	0.00	10.25
EQ1611	roof	3E	0.00	0.00	8.62	0.00	0.00	11.04
EQ1612	roof	3E	0.00	0.00	-0.01	0.00	0.00	0.12
EQ1613	roof	3E	0.00	0.00	6.97	0.00	0.00	8.95
EQ1615	roof	3E	0.00	0.00	-0.89	0.00	0.00	-1.04
					8.62			**11.04**
				kN/m	**14.14**			**18.11**
EQ1608	3rd floor	2E	0.00	0.00	3.59	0.00	0.00	4.68
EQ1609	3rd floor	2E	0.00	0.00	9.13	0.00	0.00	11.69
EQ1610	3rd floor	2E	0.00	0.00	3.59	0.00	0.00	4.68
EQ1611	3rd floor	2E	0.00	0.00	7.74	0.00	0.00	9.94
EQ1612	3rd floor	2E	0.00	0.00	3.59	0.00	0.00	4.68
EQ1613	3rd floor	2E	0.00	0.00	7.74	0.00	0.00	9.94
EQ1615	3rd floor	2E	0.00	0.00	2.15	0.00	0.00	2.81
					9.13			**11.69**
				kN/m	**14.97**			**19.18**
EQ1608		3E+2E	0.00	0.00	5.78	0.00	0.00	7.58
EQ1609		3E+2E	0.00	0.00	14.09	0.00	0.00	18.10
EQ1610		3E+2E	0.00	0.00	11.58	0.00	0.00	14.92
EQ1611		3E+2E	0.00	0.00	16.36	0.00	0.00	20.98
EQ1612		3E+2E	0.00	0.00	3.58	0.00	0.00	4.79
EQ1613		3E+2E	0.00	0.00	14.71	0.00	0.00	18.89
EQ1615		3E+2E	0.00	0.00	1.27	0.00	0.00	1.76
					16.36			**20.98**
				kN/m	**26.84**			**34.41**
EQ1608	2nd floor	1E	0.00	0.00	2.91	0.00	0.00	4.88
EQ1609	2nd floor	1E	0.00	0.00	7.85	0.00	0.00	12.61
EQ1610	2nd floor	1E	0.00	0.00	2.91	0.00	0.00	4.88
EQ1611	2nd floor	1E	0.00	0.00	6.62	0.00	0.00	10.68
EQ1612	2nd floor	1E	0.00	0.00	2.91	0.00	0.00	4.88
EQ1613	2nd floor	1E	0.00	0.00	6.62	0.00	0.00	10.68
EQ1615	2nd floor	1E	0.00	0.00	1.75	0.00	0.00	2.93
LIVE	2nd floor	1E	0.00	0.00	4.94	0.00	0.00	7.73
					7.85			**12.61**
				kN/m	**12.88**			**20.69**
EQ1608		3E+2E+1E	0.00	0.00	8.69	0.00	0.00	12.46
EQ1609		3E+2E+1E	0.00	0.00	21.94	0.00	0.00	30.71
EQ1610		3E+2E+1E	0.00	0.00	14.49	0.00	0.00	19.80
EQ1611		3E+2E+1E	0.00	0.00	22.98	0.00	0.00	31.66
EQ1612		3E+2E+1E	0.00	0.00	6.49	0.00	0.00	9.67
EQ1613		3E+2E+1E	0.00	0.00	21.33	0.00	0.00	29.57
EQ1615		3E+2E+1E	0.00	0.00	3.01	0.00	0.00	4.69
					22.98			**31.66**
				kN/m	**37.70**			**51.93**

Table 11: Steel beam at corridor. Trusses reactions.

Internal forces on each structural channel. Maximum induced moment:

$$M_{max} = 147.94\ kNm \tag{132}$$

Maximum induced shear:

$$V_{max} = 140.13\ kN \tag{133}$$

Structural shape (W14X30) mechanical properties. Steel: ASTM A-572
Shear strength:

$$V_u = 254.63\ kN \tag{134}$$

Structural shear check: $V_u = 254.63 > 140.13 = V_{max} \implies OK$
Resisting moment:

$$M_u = 160.10\ kN \cdot m \tag{135}$$

Structural bending check: $M_U = 160.10 > 147.94 = M_{max} \implies OK$

Bending stiffness. The deflection obtained is:

$$\Delta_{TL} = 9.04\ mm = \frac{L}{439} < \frac{span}{360} \implies OK \tag{136}$$

8.6 Cantilevers

8.6.1 Cantilevers C1, C3, C4, C6, C7 and C8

Loads.

- Load from trusses (three trusses at 12 inches): 13.21 kN/truss
- Facade weight: 25.76 kN
- Total load: 65.41 kN

Internal forces. Maximum induced moment:

$$M_{max} = 37.98\ kNm \tag{137}$$

Maximum induced shear:

$$V_{max} = 65.41\ kN \tag{138}$$

Structural bending check. LSL 1.55E 5.25x14” design resisting moment:

$$M'_s = 44.89\ kNm \tag{139}$$

Structural bending check: $M'_s = 44.89 > 37.98 = M_{max} \implies OK$

Structural shear check. LSL 1.55E 5.25x14” design resisting shear:

$$V'_s = 89.36\ kN \tag{140}$$

Structural shear check: $V'_s = 89.36 > 65.41 = V_{max} \implies OK$

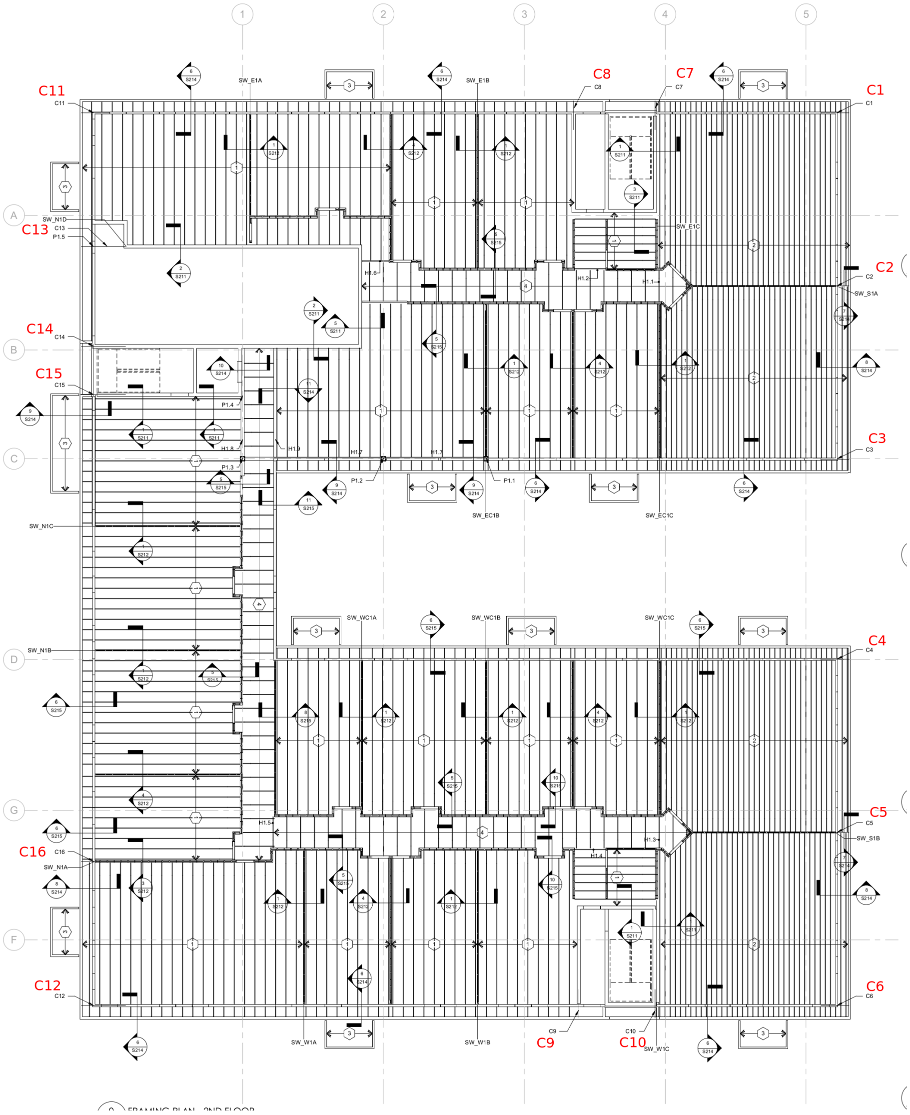


Figure 15: Second floor cantilevers key plan.

Bending stiffness. The deflection obtained under total load is:

$$\Delta_{TL} = 1.88\ mm = \frac{L}{506} < \frac{L}{360} \implies OK \tag{141}$$

8.6.2 Cantilevers C2 and C5

Loads.

- Load from trusses (2x3 trusses at 12 inches): 13 kN/truss
- Facade weight: 51.55 kN
- Total load: 129.25 kN

Internal forces. Maximum induced moment:

$$M_{max} = 63.79\ kNm \tag{142}$$

Maximum induced shear:

$$V_{max} = 129.25\ kN \tag{143}$$

Structural bending check. (2) LSL 1.55E 5.25x14" design resisting moment:

$$M'_s = 65.88\ kNm \tag{144}$$

Structural bending check: $M'_s = 65.88 > 63.79 = M_{max} \implies OK$

Structural shear check. (2) LSL 1.55E 5.25x14" design resisting shear:

$$V'_s = 151.60\ kN \tag{145}$$

Structural shear check: $V'_s = 151.60 > 129.25 = V_{max} \implies OK$

Bending stiffness. The deflection obtained under total load is:

$$\Delta_{TL} = 1.95\ mm = \frac{L}{442} < \frac{L}{360} \implies OK \tag{146}$$

8.6.3 Cantilever C9

Loads.

- Floor load: 16.25 kN
- Facade weight: 36.37 kN
- Total load: 52.62 kN

Internal forces. Maximum induced moment:

$$M_{max} = 49.99\ kNm \tag{147}$$

Maximum induced shear:

$$V_{max} = 52.62\ kN \tag{148}$$

Structural bending check. LSL 1.55E 5.25x18” design resisting moment:

$$M'_s = 72.00\ kNm \tag{149}$$

Structural bending check: $M'_s = 72.00 > 49.99 = M_{max} \implies OK$

Structural shear check. LSL 1.55E 5.25x18” design resisting shear:

$$V'_s = 114.90\ kN \tag{150}$$

Structural shear check: $V'_s = 114.90 > 52.62 = V_{max} \implies OK$

Bending stiffness. The deflection obtained under total load is:

$$\Delta_{TL} = 1.5\ mm = \frac{L}{717} < \frac{L}{360} \implies OK \tag{151}$$

8.6.4 Cantilever C10

Loads.

- Floor load: 10.84 kN
- Facade weight: 24.25 kN
- Total load: 35.08 kN

Internal forces. Maximum induced moment:

$$M_{max} = 33.33\ kNm \tag{152}$$

Maximum induced shear:

$$V_{max} = 35.08\ kN \tag{153}$$

Structural bending check. LSL 1.55E 3.5x18” design resisting moment:

$$M'_s = 48.00\ kNm \tag{154}$$

Structural bending check: $M'_s = 48.00 > 33.33 = M_{max} \implies OK$

Structural shear check. LSL 1.55E 3.5x18” design resisting shear:

$$V'_s = 76.60\ kN \tag{155}$$

Structural shear check: $V'_s = 76.60 > 35.08 = V_{max} \implies OK$

Bending stiffness. The deflection obtained under total load is:

$$\Delta_{TL} = 1.33\ mm = \frac{L}{717} < \frac{L}{360} \implies OK \tag{156}$$

Cantilevers					
C2,C5	LVL	2x5.25	14	SW_S1A and SW_S1B shear walls	
C1, C3, C4, C6	LVL	5.25	14	facade bearing walls	
C7, C8, C9, C10, C15	LVL	3.5	14	CMU wall	Bolted to masonry
C11, C12	LVL	5.25	14	facade bearing walls	
C13	LVL	5.25	18	SW_N2D shear wall	At shear wall bottom
C14	LVL	3.5	18	CMU wall	Bolted to masonry
C16	LVL	5.25	14	SW_N1A shear wall	

Table 12: Cantilever schedule.

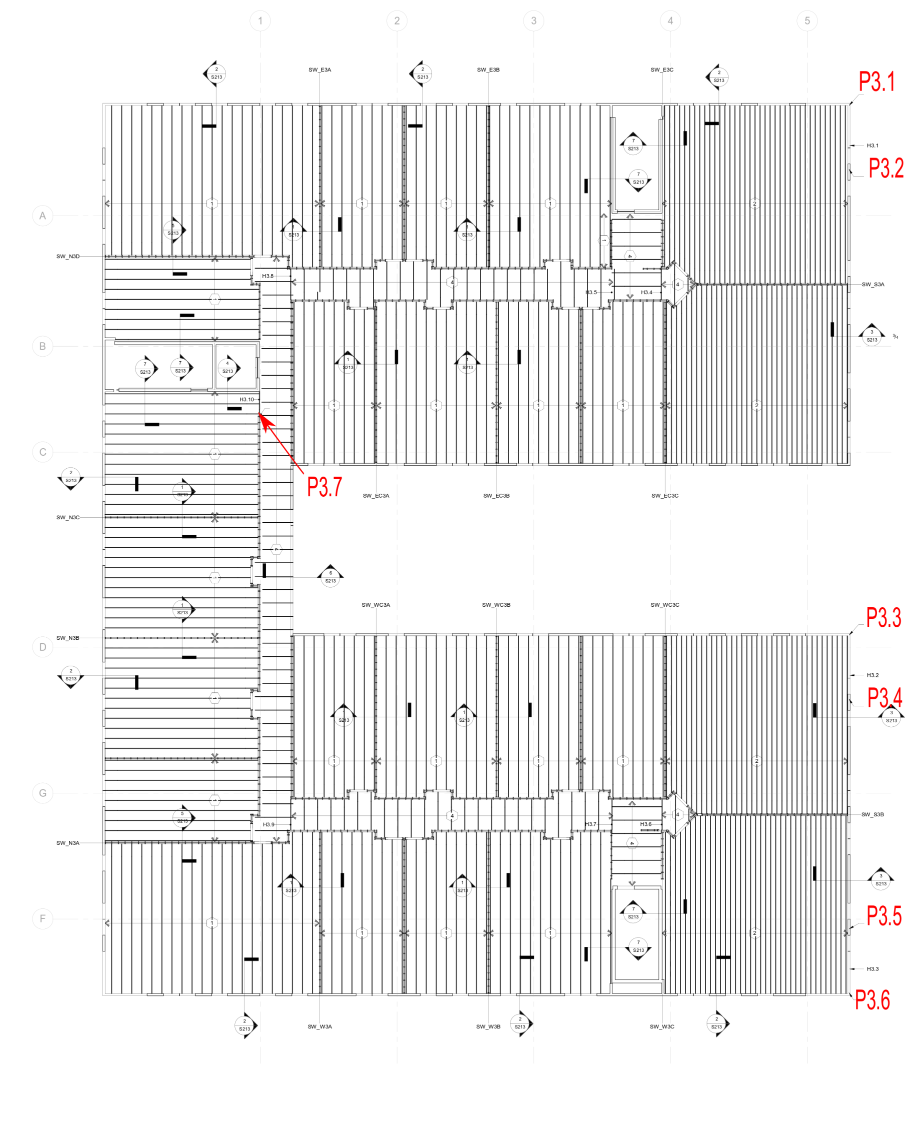


Figure 16: Columns key plan. Roof

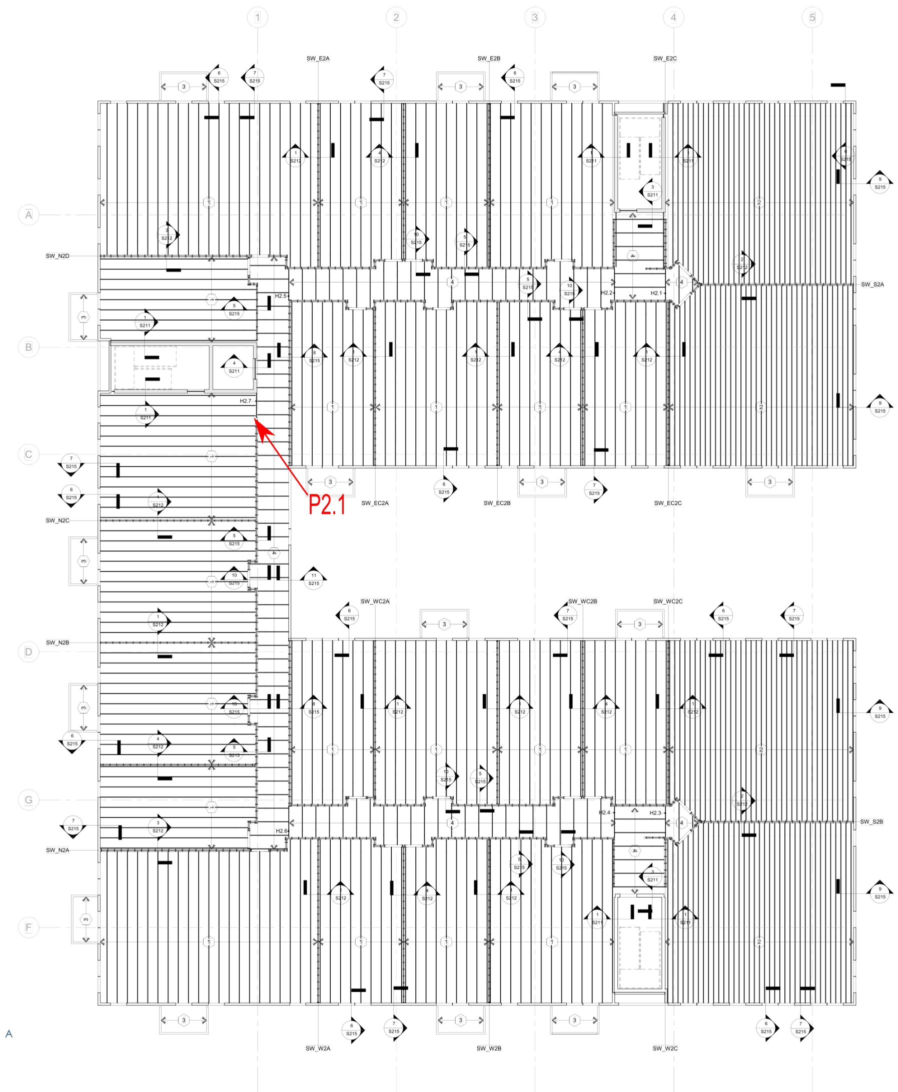


Figure 17: Columns key plan. Third floor

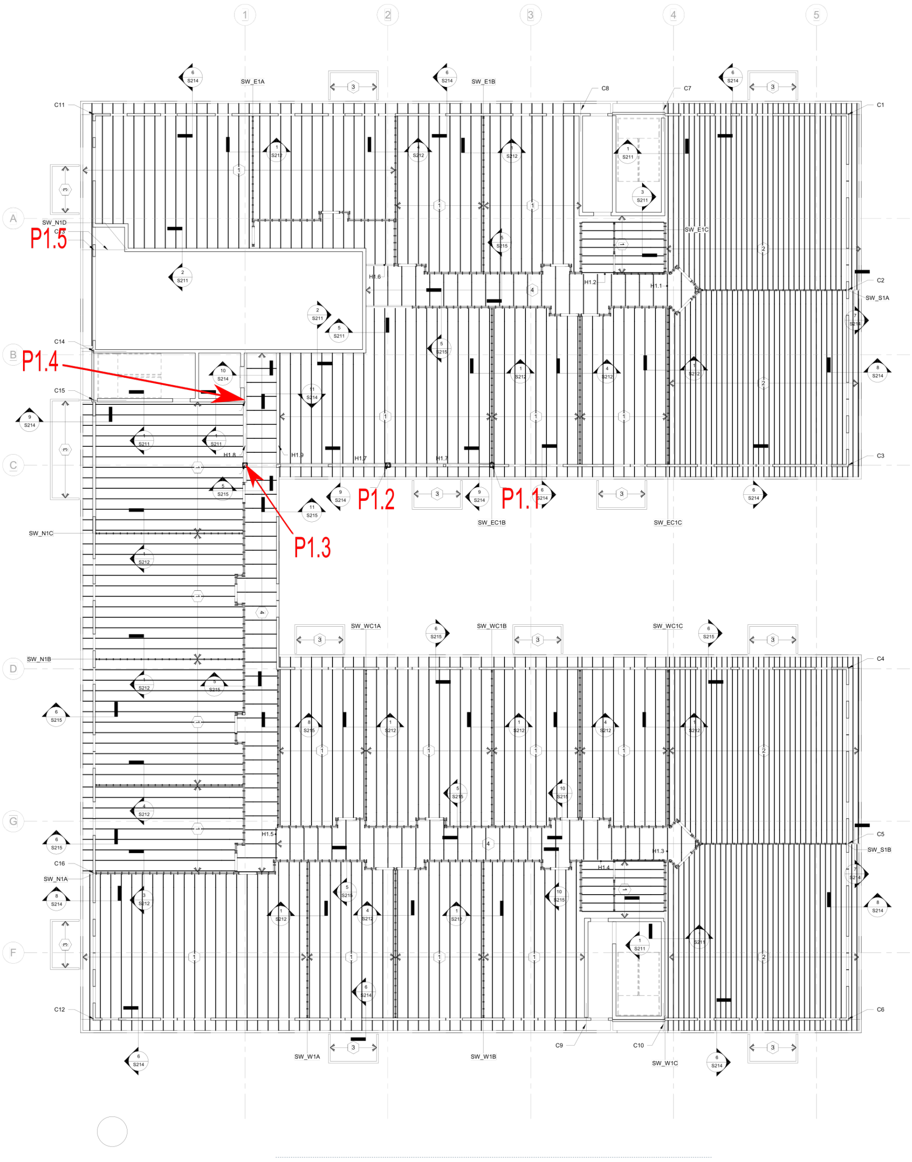


Figure 18: Columns key plan. Second floor

8.7 Columns

8.7.1 Columns P3.1 to P3.6

Loads.

- Load from header: 4.8 kN

Internal forces. Maximum induced axial force:

$$N_{max} = 4.8\ kN \tag{157}$$

Compression check.

Mechanical properties

- Species: Spruce-pine-fir No.2
- $E_{min} = 2.55\ GPa$
- $F_c = 2.93\ MPa.$
- Sections dimensions: (6x6”), effective (5.5x5.5”)= 139.7 x 139.7 mm.
- Unbraced lenght x axis: 3.45 m
- Unbraced lenght y axis: 2.59 m
- Column stability factor: $C_P = 0.74$

$$N'_s = 42.46\ kN \tag{158}$$

Structural compression check: $N'_s = 42.46 > 4.8 = N_{max} \implies OK$

8.7.2 Column P3.7

Loads.

- Load from header 3.10: 27.61 kN

Internal forces. Maximum induced axial force:

$$N_{max} = 27.61\ kN \tag{159}$$

Compression check.

Mechanical properties

- Species: Douglas fir-Larch dense structural
- $E_{min} = 4.27\ GPa$
- $F_c = 9.31\ MPa.$
- Sections dimensions: (4x6”), effective (3.5x5.5”)= 88.9 x 139.7 mm.
- Unbraced lenght x axis: 2.902 m

- Unbraced lenght y axis: 0.5 m
- Column stability factor: $C_P = 0.64$

$$N'_s = 74.34\ kN \tag{160}$$

Structural compression check: $N'_s = 74.34 > 27.61 = N_{max} \implies OK$

8.7.3 Column P2.1

Loads.

- Load from header 2.07 and post P3.10: 55.22 kN

Internal forces. Maximum induced axial force:

$$N_{max} = 55.22\ kN \tag{161}$$

Compression check.

Mechanical properties

- Species: Douglas fir-Larch dense structural
- $E_{min} = 4.27\ GPa$
- $F_c = 9.31\ MPa$.
- Sections dimensions: (4x6”), effective (3.5x5.5”)= 88.9 x 139.7 mm.
- Unbraced lenght x axis: 2.902 m
- Unbraced lenght y axis: 0.5 m
- Column stability factor: $C_P = 0.64$

$$N'_s = 74.34\ kN \tag{162}$$

Structural compression check: $N'_s = 74.34 > 55.22 = N_{max} \implies OK$

8.7.4 2x6 wood stud capacity

Compression capacity.

Mechanical properties.

- Species: Hem-fir stud select structural.
- $E_{min} = 3.24\ GPa$
- $F_c = 8.27\ MPa$.
- Sections dimensions: (2x6”), effective (1.5x5.5”)= 38.1 x 139.7 mm.
- Unbraced lenght x axis: 0.3 m
- Unbraced lenght y axis: 3.45 m
- Column stability factor: $C_P = 0.45$

$$N'_s = 19.90\ kN \tag{163}$$

8.7.5 Columns P1.1, P1.2 and P1.3

Loads.

- Column P1.3: load from corridor beam and courtyard beam: $192.97\ kN + 65.99\ kN = 258.96\ kN$
- Column P1.2: load from courtyard beam: $219.96\ kN$
- Column P1.1: load from courtyard beam: $65.99\ kN$

Internal forces. Maximum induced axial force:

$$N_{max} = 258.96\ kN M_{y,max} = 5.18\ kNm M_{z,max} = 12.95\ kNm \tag{164}$$

Mechanical properties.

- Structural shape: HSS8x8x3/16
- Height: 3.51 m
- Steel: ASTM A500 Grade B ($F_y = 315.0\ MPa$)
- Effective length buckling coefficient: $K = 1$
- Compression
  - Limiting width-to-thickness ratio $\lambda_r = 35.28$
  - Classification of walls: slender

Compression check.

- Nominal compressive strength: $P_n = \frac{844.37}{1.67} = 505.61\ kN$
- Nominal flexural strength: $M_n = \frac{58.55}{1.67} = 35.06\ kNm$

Capacity factor according to equation H1-1 of AISC 360-16:

$$\frac{P_d}{P_n} + \frac{8}{9}(\frac{M_{yd}}{M_{yn}} + \frac{M_{zd}}{M_{zn}}) = 0.97 < 1.0 \implies OK \tag{165}$$

8.7.6 Column P1.4

Loads.

- Column P1.4: load from corridor: $125.49\ kN$

Internal forces. Maximum induced axial force:

$$N_{max} = 125.49\ kN M_{y,max} = 7.33\ kNm M_{z,max} = 6.27\ kNm \tag{166}$$

Mechanical properties.

- Structural shape: HSS7X7X3/16
- Height: 3.51 m
- Steel: ASTM A500 Grade B ($F_y = 315.0\ MPa$)
- Effective length buckling coefficient: $K = 1$
- Compression
  - Limiting width-to-thickness ratio $\lambda_r = 35.28$
  - Classification of walls: slender

Compression check.

- Nominal compressive strength: $P_n = \frac{774.19}{1.67} = 463.59\ kN$
- Nominal flexural strength: $M_n = \frac{51.02}{1.67} = 30.55\ kNm$

Capacity factor according to equation H1-1 of AISC 360-16:

$$\frac{P_d}{P_n} + \frac{8}{9}(\frac{M_{yd}}{M_{yn}} + \frac{M_{zd}}{M_{zn}}) = 0.67 < 1.0 \implies OK \tag{167}$$

8.8 Bearing walls

8.8.1 Facade bearing walls at first floor

Loads.

- Vertical load: 43.34 kN/m
- Vertical load on each stud: 21.14 kN
- Wind load on stud height: 0.42 kN/m

Internal forces. Internal forces:

$$N_{max} = 21.14\ kN \tag{168}$$
$$M_{max} = 0.36\ kN \cdot m \tag{169}$$

Bending and axial compression check.

Mechanical properties

- Species: Hem-fir stud
- Spacing: 0.49 m
- Stud height: 2.64 m
- Repetitive member factor: $C_r = 1.15$
- Size factor: $C_F = 1.3$

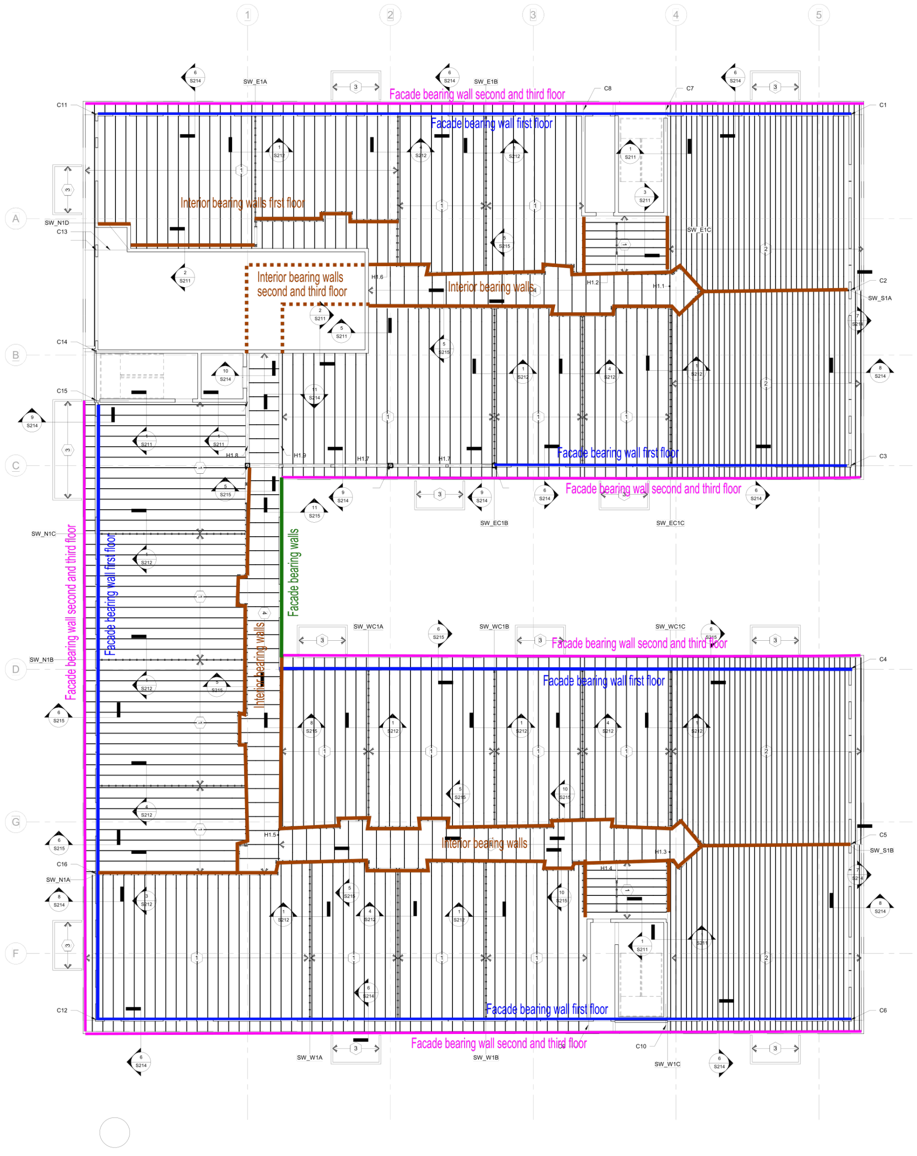


Figure 19: Bearing walls key plan. Roof

- $E_{min} = 3.03\ GPa$
- $F'_c = 6.09\ MPa$.
- $F'_b = 6.97\ MPa$.
- Sections dimensions: (2x8"), effective (1.5x7.5")= $38.1 \times 190.5\ mm$.
- Unbraced length x axis: 2.64 m
- Unbraced length y axis: 0.3 m
- Stud stability factor: $C_P = 0.85$

$$N'_s = 44.21\ kN \tag{170}$$

Capacity factor according to section 3.9.2 of NDS-2018 (equations 3.9.3 and 3.9.4):

$$CF = 0.52 < 1 \implies OK \tag{171}$$

8.8.2 Facade bearing walls at second floor

Loads.

- Vertical load: 30.52 kN/m
- Vertical load on each stud: 14.88 kN
- Wind load on stud height: 0.42 kN/m

Internal forces. Internal forces:

$$N_{max} = 14.88\ kN \tag{172}$$
$$M_{max} = 0.36\ kN \cdot m \tag{173}$$

Bending and axial compression check.

Mechanical properties

- Species: Hem-fir stud
- Spacing: 0.49 m
- Stud height: 2.64 m
- Repetitive member factor: $C_r = 1.15$
- Size factor: $C_F = 1.3$
- $E_{min} = 3.03\ GPa$
- $F'_c = 6.09\ MPa$.
- $F'_b = 6.95\ MPa$.
- Sections dimensions: (2x8"), effective (1.5x7.5")= $38.1 \times 190.5\ mm$.
- Unbraced length x axis: 2.64 m

- Unbraced length y axis: 0.3 m
- Stud stability factor: $C_P = 0.85$

$$N'_s = 44.21\ kN \tag{174}$$

Capacity factor according to section 3.9.2 of NDS-2018 (equations 3.9.3 and 3.9.4):

$$CF = 0.38 < 1 \implies OK \tag{175}$$

8.8.3 Facade bearing walls at third floor

Loads.

- Vertical load: 16.01 kN/m
- Vertical load on each stud: 7.81 kN
- Wind load on stud height: 0.42 kN/m

Internal forces. Internal forces:

$$N_{max} = 7.81\ kN \tag{176}$$
$$M_{max} = 0.36\ kN \cdot m \tag{177}$$

Bending and axial compression check.

Mechanical properties

- Species: Hem-fir stud
- Spacing: 0.49 m
- Stud height: 2.64 m
- Repetitive member factor: $C_r = 1.15$
- Size factor: $C_F = 1.3$
- $E_{min} = 3.03\ GPa$
- $F'_c = 6.09\ MPa.$
- $F'_b = 6.96\ MPa.$
- Sections dimensions: (2x8”), effective (1.5x7.5”)= $38.1 \times 190.5\ mm.$
- Unbraced length x axis: 2.64 m
- Unbraced length y axis: 0.3 m
- Stud stability factor: $C_P = 0.85$

$$N'_s = 44.21\ kN \tag{178}$$

Capacity factor according to section 3.9.2 of NDS-2018 (equations 3.9.3 and 3.9.4):

$$CF = 0.28 < 1 \implies OK \tag{179}$$

8.8.4 Top plates

Loads

- Load from trusses: 9.64 kN/truss.
- Truss spacing: 0.61 m
- Stud spacing: 0.49 m

Bending strength checking: Maximum induced moment:

$$M_{max} = 0.42kN \cdot m \quad (180)$$
$$\sigma_{max} = 9.21\ MPa \quad (181)$$

Bending strength:

$$F_b' = 10.05\ MPa \quad (182)$$

Structural bending check:

$$F_b' = 10.05 > 9.21 = \sigma_{max} \implies OK \quad (183)$$

Perpendicular to grain strength checking: Maximum induced reaction:

$$R_{max} = 9.38\ kN \quad (184)$$
$$\sigma_{max,perp} = 1.29\ MPa \quad (185)$$

$$F'_{c,perp} = 2.79 > 1.29 = \sigma_{max,perp} \implies OK \quad (186)$$

8.8.5 Interior bearing walls at first floor

Loads.

- Vertical load: 84.99 kN/m
- Vertical load on each stud: 25.9 kN
- Wind load on stud height: 0.0 kN/m

Internal forces. Internal forces:

$$N_{max} = 25.9\ kN \quad (187)$$
$$M_{max} = 0.0\ kN \cdot m \quad (188)$$

Bending and axial compression check.

Mechanical properties

- Species: Hem-fir stud
- Spacing: 0.30 m
- Stud height: 2.64 m
- Repetitive member factor: $C_r = 1.15$
- Size factor: $C_F = 1.3$
- $E_{min} = 3.03\ GPa$
- $F'_c = 4.89\ MPa$.
- $F'_b = 6.96\ MPa$.
- Sections dimensions: (2x6"), effective (1.5x5.5")= 38.1 x 139.7 mm.
- Unbraced length x axis: 2.64 m
- Unbraced length y axis: 0.3 m
- Stud stability factor: $C_P = 0.68$

$$N'_s = 26.00\ kN \tag{189}$$

Capacity factor according to section 3.9.2 of NDS-2018 (equations 3.9.3 and 3.9.4):

$$CF = 0.99 < 1 \implies OK \tag{190}$$

8.8.6 Interior bearing walls at second floor

Loads.

- Vertical load: 59.93 kN/m
- Vertical load on each stud: 18.26 kN
- Wind load on stud height: 0.0 kN/m

Internal forces. Internal forces:

$$N_{max} = 18.26\ kN \tag{191}$$
$$M_{max} = 0.0\ kN \cdot m \tag{192}$$

Bending and axial compression check.

Mechanical properties

- Species: Hem-fir stud
- Spacing: 0.30 m
- Stud height: 2.64 m
- Repetitive member factor: $C_r = 1.15$
- Size factor: $C_F = 1.3$
- $E_{min} = 3.03\ GPa$
- $F_c' = 4.89\ MPa.$
- $F_b' = 6.96\ MPa.$
- Sections dimensions: (2x6”), effective (1.5x5.5”)= 38.1 x 139.7 mm.
- Unbraced length x axis: 2.64 m
- Unbraced length y axis: 0.3 m
- Stud stability factor: $C_P = 0.68$

$$N_s' = 26.00\ kN \tag{193}$$

Capacity factor according to section 3.9.2 of NDS-2018 (equations 3.9.3 and 3.9.4):

$$CF = 0.49 < 1 \implies OK \tag{194}$$

8.8.7 Interior bearing walls at third floor

Loads.

- Vertical load: 33.7 kN/m
- Vertical load on each stud: 20.54 kN
- Wind load on stud height: 0.0 kN/m

Internal forces. Internal forces:

$$N_{max} = 20.54\ kN \tag{195}$$
$$M_{max} = 0.00\ kN \cdot m \tag{196}$$

Bending and axial compression check.

Mechanical properties

- Species: Hem-fir stud
- Spacing: 0.61 m
- Stud height: 2.64 m
- Repetitive member factor: $C_r = 1.15$
- Size factor: $C_F = 1.3$
- $E_{min} = 3.03\ GPa$
- $F_c' = 4.89\ MPa$.
- $F_b' = 6.96\ MPa$.
- Sections dimensions: (2x6"), effective (1.5x5.5")= 38.1 x 139.7 mm.
- Unbraced length x axis: 2.64 m
- Unbraced length y axis: 0.3 m
- Stud stability factor: $C_P = 0.68$

$$N_s' = 26.00\ kN \tag{197}$$

Capacity factor according to section 3.9.2 of NDS-2018 (equations 3.9.3 and 3.9.4):

$$CF = 0.62 < 1 \implies OK \tag{198}$$

8.8.8 Top plates

Loads

- Load from trusses: 11.81 kN/truss.
- Truss spacing: 0.61 m
- Stud spacing: 0.49 m

Bending strength checking: Maximum induced moment:

$$M_{max} = 0.24 kN \cdot m \tag{199}$$
$$\sigma_{max} = 7.15\ MPa \tag{200}$$

Bending strength:

$$F_b' = 10.05\ MPa \tag{201}$$

Structural bending check:

$$F_b' = 10.05 > 7.15 = \sigma_{max} \implies OK \tag{202}$$

Perpendicular to grain strength checking: Maximum induced reaction:

$$R_{max} = 8.56\ kN \tag{203}$$
$$\sigma_{max,perp} = 1.61\ MPa \tag{204}$$

$$F'_{c,perp} = 2.79 > 1.61 = \sigma_{max,perp} \implies OK \tag{205}$$

8.9 Lateral. Diaphragms/Shear walls

8.9.1 East and West facades shear walls

The shear walls of the East facade are those denoted by the letters "E" and "W" in figures 20 to 22. The wind load on each floor per unit length is as follows:

floor	**wind force** (kN/m)
roof	2.34
third	1.67
second	1.71

The shear values obtained for each wall are as follows:

floor	**shear force** (kN)		
	EA/WA	EB/WB	EC/WC
roof	68.76	-21.54	59.39
third	48.93	-15.32	42.26
second	118.86	44.95	31.49

And the cumulated values are:

floor	**shear force** (kN)		
	EA/WA	EB/WB	EC/WC
roof	68.76	-21.54	59.39
third	117.70	-36.86	101.65
second	118.86	8.09	133.14

leading to the following dimensions:

											Bottom plate attachment (foundation)		Bottom plate attachment (floor to floor)
Shear wall	Sheathing material	Panel thickness	Blocking	Minimum fastener penetration	Fastener type and size	Panel edge fastener spacing	Nominal unit shear capacity v_w	Hold-down anchor capacity	Hold-down studs	Hold-down anchor type	Number of bolts	Bolt spacing	
ID		(in)		(in)		(in)	(plf)	(kip)		HD		(in)	
E3A	WSP – sheathing	19/32	Y	1-1/2	10d	4	1430	3	(1)	U4-SDS2.5	-	-	wood screws 20 (d= 0.32 in) at 16 in. o/c; 46 fasteners in 2 rows.

E3B	WSP – sheathing	3/8	N	1-3/8	8d	6	560	-	-		-	-	16d (d= 0.268 in) nails at 12 in. o/c; 30 fasteners in 1 row.
E3C	WSP – sheathing	19/32	Y	1-1/2	10d	4	1430	6	(2)	U11-SDS2.5	-	-	SDWS log screw (d= 0.197 in) at 15 in. o/c; 32 fasteners in 2 rows.
E2A	WSP – sheathing	19/32	Y	1-1/2	10d	3	1860	7	(3)	U11-SDS2.5	-	-	SDWS log screw (d= 0.197 in) at 11 in. o/c; 64 fasteners in 2 rows.
E2B	WSP – sheathing	3/8	N	1-3/8	8d	6	560	1	(1)	U4-SDS2.5	-	-	16d (d= 0.268 in) nails at 14 in. o/c; 51 fasteners in 2 rows.
E2C	WSP – sheathing	19/32	Y	1-1/2	10d	2	2435	11	(4)	19	-	-	SDWS log screw (d= 0.197 in) at 9 in. o/c; 54 fasteners in 2 rows.
E1A	WSP – sheathing	19/32	Y	1-1/2	10d	2	2435	13	(4)	19	7	36	SDWS log screw (d= 0.197 in) at 7 in. o/c; 64 fasteners in 2 rows.
E1B	WSP – sheathing	3/8	N	1-3/8	8d	6	560	-	-		11	36	16d (d= 0.268 in) nails at 32 in. o/c; 12 fasteners in 1 row.
E1C	WSP – sheathing	19/32	Y	1-1/2	10d	2	2435	9	(3)	19	11	36	SDWS log screw (d= 0.197 in) at 10 in. o/c; 72 fasteners in 2 rows.
W3A	WSP – sheathing	19/32	Y	1-1/2	10d	4	1430	3	(1)	U4-SDS2.5	-	-	wood screws 20 (d= 0.32 in) at 16 in. o/c; 46 fasteners in 2 rows.
W3B	WSP – sheathing	3/8	N	1-3/8	8d	6	560	-	-		-	-	16d (d= 0.268 in) nails at 12 in. o/c; 30 fasteners in 1 row.
W3C	WSP – sheathing	19/32	Y	1-1/2	10d	4	1430	6	(2)	U11-SDS2.5	-	-	SDWS log screw (d= 0.197 in) at 15 in. o/c; 32 fasteners in 2 rows.
W2A	WSP – sheathing	19/32	Y	1-1/2	10d	3	1860	7	(3)	U11-SDS2.5	-	-	SDWS log screw (d= 0.197 in) at 11 in. o/c; 64 fasteners in 2 rows.
W2B	WSP – sheathing	3/8	N	1-3/8	8d	6	560	1	(1)	U4-SDS2.5	-	-	16d (d= 0.268 in) nails at 14 in. o/c; 51 fasteners in 2 rows.
W2C	WSP – sheathing	19/32	Y	1-1/2	10d	2	2435	11	(4)	19	-	-	SDWS log screw (d= 0.197 in) at 9 in. o/c; 54 fasteners in 2 rows.
W1A	WSP – sheathing	19/32	Y	1-1/2	10d	2	2435	13	(4)	19	9	30	SDWS log screw (d= 0.197 in) at 7 in. o/c; 64 fasteners in 2 rows.
W1B	WSP – sheathing	3/8	N	1-3/8	8d	6	560	-	-		11	36	16d (d= 0.268 in) nails at 32 in. o/c; 12 fasteners in 1 row.
W1C	WSP – sheathing	19/32	Y	1-1/2	10d	2	2435	9	(3)	19	11	36	SDWS log screw (d= 0.197 in) at 10 in. o/c; 72 fasteners in 2 rows.

8.9.2 Courtyard facades shear walls

The shear walls of the courtyard East and West facades are those denoted by the letters "EC" or "WC" in figures 20 to 22. The wind load on each floor per unit length is as follows:

floor	**wind force** (kN/m)
roof	2.50
third	1.98
second	2.03

The shear values obtained for each wall are as follows:

floor	**shear force** (kN)		
	ECA/WCA	ECB/WCB	ECC/WCC
roof	30.35	-4.77	59.26
third	24.06	-3.78	46.97
second	24.61	-3.87	48.04

And the cumulated values are:

floor	**shear force** (kN)		
	ECA/WCA	ECB/WCB	ECC/WCC
roof	30.35	-4.77	59.26
third	54.41	-8.56	106.22
second	79.02	-12.43	154.27

leading to the following dimensions:

											Bottom plate attachment (foundation)		Bottom plate attachment (floor to floor)
Shear wall	Sheathing material	Panel thickness	Blocking	Minimum fastener penetration	Fastener type and size	Panel edge fastener spacing	Nominal unit shear capacity v_w	Hold-down anchor capacity	Hold-down studs	Hold-down anchor type	Number of bolts	Bolt spacing	
ID		(in)		(in)		(in)	(plf)	(kip)		HD		(in)	
EC3A	WSP – sheathing	19/32	Y	1-1/2	10d	6	950	0	-		-	-	16d (d= 0.268 in) nails at 18 in. o/c; 42 fasteners in 2 rows.
EC3B	WSP – sheathing	3/8	N	1-3/8	8d	6	560	-	-		-	-	16d (d= 0.268 in) nails at 60 in. o/c; 7 fasteners in 1 row.
EC3C	WSP – sheathing	19/32	Y	1-1/2	10d	6	950	3	(1)	U4-SDS2.5	-	-	wood screws 20 (d= 0.32 in) at 19 in. o/c; 40 fasteners in 2 rows.
EC2A	WSP – sheathing	19/32	Y	1-1/2	10d	3	1860	2	(1)	U4-SDS2.5	-	-	wood screws 20 (d= 0.32 in) at 21 in. o/c; 36 fasteners in 2 rows.
EC2B	WSP – sheathing	3/8	N	1-3/8	8d	6	560	-	-		-	-	16d (d= 0.268 in) nails at 32 in. o/c; 12 fasteners in 1 row.
EC2C	WSP – sheathing	19/32	Y	1-1/2	10d	3	1860	6	(2)	U11-SDS2.5	-	-	SDWS log screw (d= 0.197 in) at 12 in. o/c; 58 fasteners in 2 rows.
EC1A	WSP – sheathing	19/32	Y	1-1/2	10d	2	2435	11	(4)	19	6	36	SDWS log screw (d= 0.197 in) at 9 in. o/c; 42 fasteners in 2 rows.
EC1B	WSP – sheathing	3/8	N	1-3/8	8d	6	560	-	-		11	36	16d (d= 0.268 in) nails at 22 in. o/c; 17 fasteners in 1 row.
EC1C	WSP – sheathing	19/32	Y	1-1/2	10d	2	2435	11	(4)	19	11	36	SDWS log screw (d= 0.197 in) at 9 in. o/c; 82 fasteners in 2 rows.
WC3A	WSP – sheathing	19/32	Y	1-1/2	10d	6	950	0	-		-	-	16d (d= 0.268 in) nails at 18 in. o/c; 42 fasteners in 2 rows.
WC3B	WSP – sheathing	3/8	N	1-3/8	8d	0	560	-	-		-	-	16d (d= 0.268 in) nails at 60 in. o/c; 7 fasteners in 1 row.
WC3C	WSP – sheathing	19/32	Y	1-1/2	10d	6	950	3	(1)	U4-SDS2.5	-	-	wood screws 20 (d= 0.32 in) at 19 in. o/c; 40 fasteners in 2 rows.

WC2A	WSP – sheathing	19/32	Y	1-1/2	10d	3	1860	2	(1)	U4-SDS2.5	-	-	wood screws 20 (d= 0.32 in) at 21 in. o/c; 36 fasteners in 2 rows.
WC2B	WSP – sheathing	3/8	N	1-3/8	8d	6	560	-	-		-	-	16d (d= 0.268 in) nails at 32 in. o/c; 12 fasteners in 1 row.
WC2C	WSP – sheathing	19/32	Y	1-1/2	10d	3	1860	6	(2)	U11-SDS2.5	-	-	SDWS log screw (d= 0.197 in) at 12 in. o/c; 58 fasteners in 2 rows.
WC1A	WSP – sheathing	19/32	Y	1-1/2	10d	2	2435	11	(4)	19	6	36	SDWS log screw (d= 0.197 in) at 9 in. o/c; 42 fasteners in 2 rows.
WC1B	WSP – sheathing	3/8	N	1-3/8	8d	6	560	-	-		11	36	16d (d= 0.268 in) nails at 22 in. o/c; 17 fasteners in 1 row.
WC1C	WSP – sheathing	19/32	Y	1-1/2	10d	2	2435	11	(4)	19	11	36	SDWS log screw (d= 0.197 in) at 9 in. o/c; 82 fasteners in 2 rows.

8.9.3 South facades shear walls

The shear walls of the South facade are those denoted by the letter “S” in figures 20 to 22. The wind load on each floor per unit length is as follows:

floor	**wind force** (kN/m)
roof	2.50
third	1.98
second	2.03

The shear values obtained for each wall are as follows:

floor	**shear force** (kN) SA/SB
roof	54.95
third	43.56
second	44.55

And the cumulated values are:

floor	**shear force** (kN) SA/SB
roof	54.95
third	98.51
second	143.06

leading to the following dimensions:

											Bottom plate attachment (foundation)		Bottom plate attachment (floor to floor)
Shear wall	Sheathing material	Panel thickness	Blocking	Minimum fastener penetration	Fastener type and size	Panel edge fastener spacing	Nominal unit shear capacity v_w	Hold-down anchor capacity	Hold-down studs	Hold-down anchor type	Number of bolts	Bolt spacing	

ID		(in)		(in)		(in)	(plf)	(kip)		HD		(in)	
S3A	WSP – sheathing	19/32	Y	1-1/2	10d	6	950	2	(1)	U4-SDS2.5	-	-	wood screws 20 (d= 0.32 in) at 21 in. o/c; 36 fasteners in 2 rows.
S3B	WSP – sheathing	19/32	Y	1-1/2	10d	6	950	2	(1)	U4-SDS2.5	-	-	wood screws 20 (d= 0.32 in) at 21 in. o/c; 36 fasteners in 2 rows.
S2A	WSP – sheathing	19/32	Y	1-1/2	10d	3	1860	6	(2)	U11-SDS2.5	-	-	SDWS log screw (d= 0.197 in) at 13 in. o/c; 54 fasteners in 2 rows.
S2B	WSP – sheathing	19/32	Y	1-1/2	10d	3	1860	6	(2)	U11-SDS2.5	-	-	SDWS log screw (d= 0.197 in) at 13 in. o/c; 54 fasteners in 2 rows.
S1A	WSP – sheathing	19/32	Y	1-1/2	10d	2	2435	11	(4)	19	10	36	SDWS log screw (d= 0.197 in) at 8 in. o/c; 76 fasteners in 2 rows.
S1B	WSP – sheathing	19/32	Y	1-1/2	10d	2	2435	11	(4)	19	10	36	SDWS log screw (d= 0.197 in) at 8 in. o/c; 76 fasteners in 2 rows.

8.9.4 North facade shear walls

The shear walls of the North facade are those denoted by the letter “N” in figures 20 to 22. The wind load on each floor per unit length is as follows:

floor	**wind force** (kN/m)
roof	2.34
third	1.67
second	1.71

The shear values obtained for each wall are as follows:

floor	**shear force** (kN)			
	NA	NB	NC	ND
roof	44.84	11.72	25.01	45.63
third	31.91	8.34	17.79	32.47
second	32.64	8.53	18.20	33.22

And the cumulated values are:

floor	**shear force** (kN)			
	NA	NB	NC	ND
roof	44.84	11.72	25.01	45.63
third	76.75	20.06	42.80	78.11
second	109.39	28.59	61.01	111.33

leading to the following dimensions:

	Bottom plate attachment (foundation)	Bottom plate attachment (floor to floor)

Shear wall	Sheathing material	Panel thickness	Blocking	Minimum fastener penetration	Fastener type and size	Panel edge fastener spacing	Nominal unit shear capacity v_w	Hold-down anchor capacity	Hold-down studs	Hold-down anchor type	Number of bolts	Bolt spacing	
ID		(in)		(in)		(in)	(plf)	(kip)		HD		(in)	
N3A	WSP – sheathing	3/8	Y	1-3/8	8d	4	840	2	(1)	U4-SDS2.5	-	-	wood screws 20 (d= 0.32 in) at 25 in. o/c; 30 fasteners in 2 rows.
N3B	WSP – sheathing	3/8	N	1-3/8	8d	6	560	-	-		-	-	16d (d= 0.268 in) nails at 24 in. o/c; 16 fasteners in 1 row.
N3C	WSP – sheathing	3/8	N	1-3/8	8d	6	560	-	-		-	-	16d (d= 0.268 in) nails at 21 in. o/c; 35 fasteners in 2 rows.
N3D	WSP – sheathing	3/8	Y	1-3/8	8d	4	840	2	(1)	U4-SDS2.5	-	-	wood screws 20 (d= 0.32 in) at 25 in. o/c; 30 fasteners in 2 rows.
N2A	WSP – sheathing	19/32	Y	1-1/2	10d	4	1430	4	(2)	U4-SDS2.5	-	-	wood screws 20 (d= 0.32 in) at 14 in. o/c; 52 fasteners in 2 rows.
N2B	WSP – sheathing	19/32	Y	1-1/2	10d	6	950	-	-		-	-	16d (d= 0.268 in) nails at 13 in. o/c; 28 fasteners in 1 row.
N2C	WSP – sheathing	19/32	Y	1-1/2	10d	6	950	1	(1)	U4-SDS2.5	-	-	16d (d= 0.268 in) nails at 12 in. o/c; 59 fasteners in 2 rows.
N2D	WSP – sheathing	19/32	Y	1-1/2	10d	4	1430	4	(2)	U4-SDS2.5	-	-	wood screws 20 (d= 0.32 in) at 14 in. o/c; 52 fasteners in 2 rows.
N1A	WSP – sheathing	19/32	Y	1-1/2	10d	3	1860	7	(3)	U11-SDS2.5	10	36	SDWS log screw (d= 0.197 in) at 12 in. o/c; 58 fasteners in 2 rows.
N1B	WSP – sheathing	19/32	Y	1-1/2	10d	6	950	-	-		11	36	16d (d= 0.268 in) nails at 19 in. o/c; 39 fasteners in 2 rows.
N1C	WSP – sheathing	19/32	Y	1-1/2	10d	6	950	3	(1)	U4-SDS2.5	11	36	wood screws 20 (d= 0.32 in) at 19 in. o/c; 40 fasteners in 2 rows.
N1D	WSP – sheathing	19/32	Y	1-1/2	10d	3	1860	7	(3)	U11-SDS2.5	10	36	SDWS log screw (d= 0.197 in) at 12 in. o/c; 60 fasteners in 2 rows.

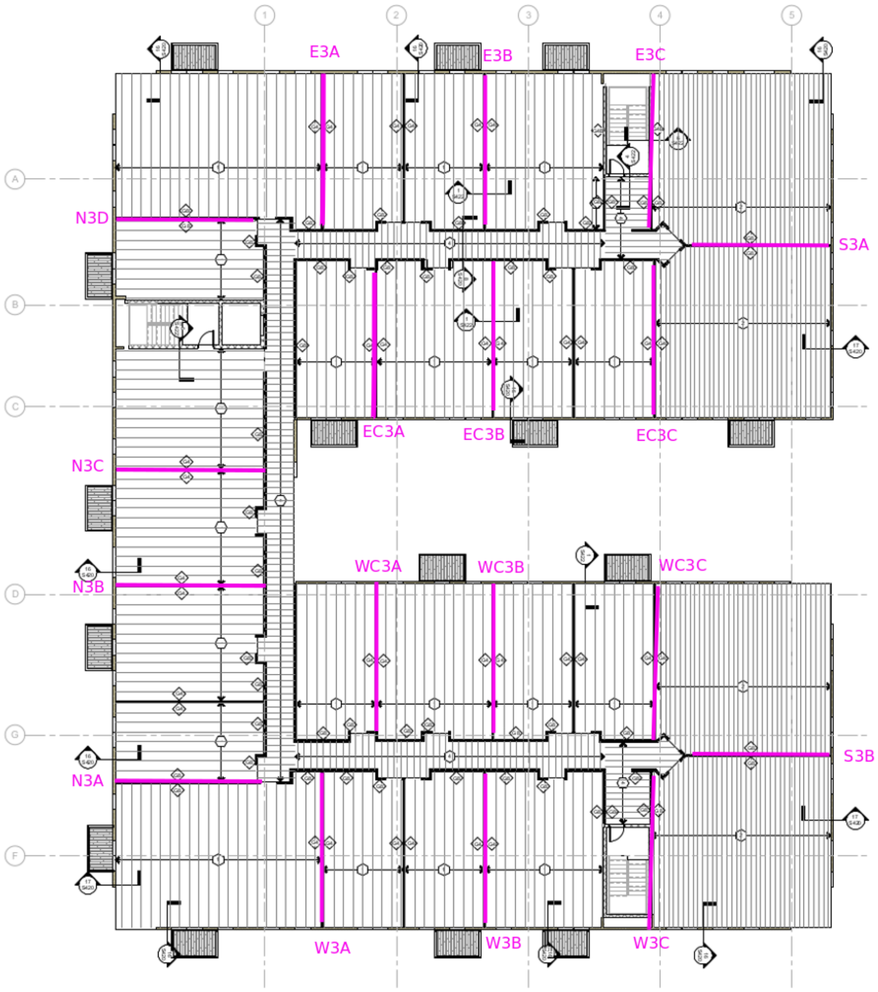


Figure 20: Shear walls on the third floor.

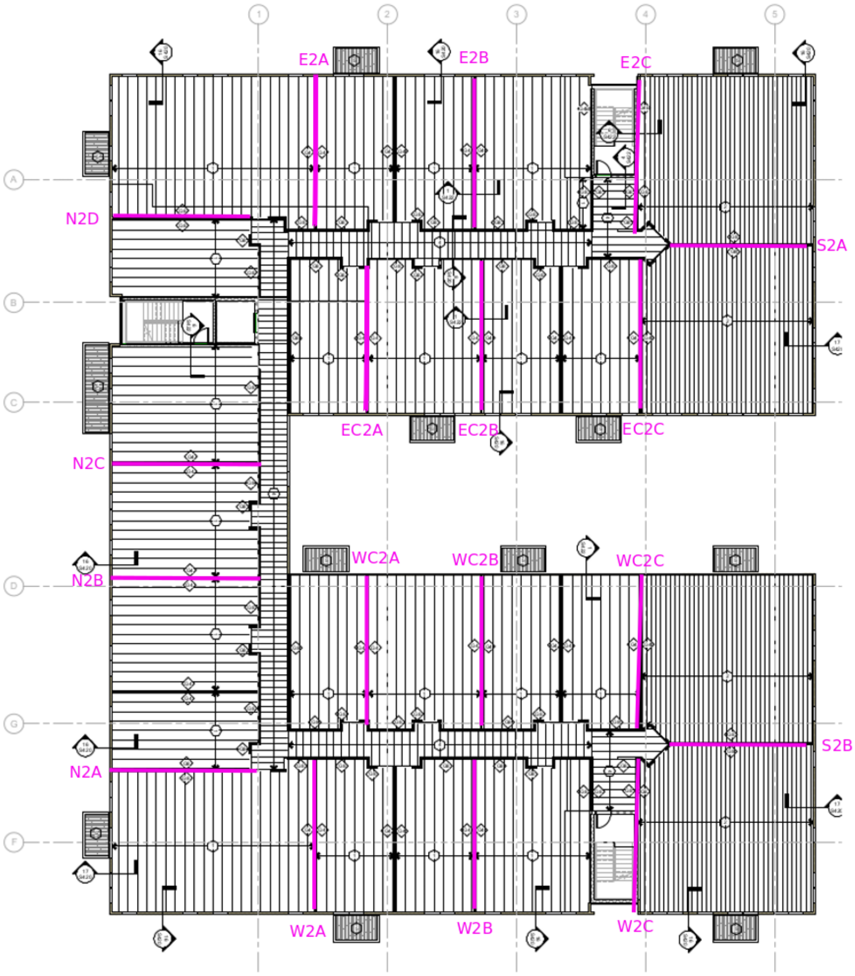


Figure 21: Shear walls on the second floor.

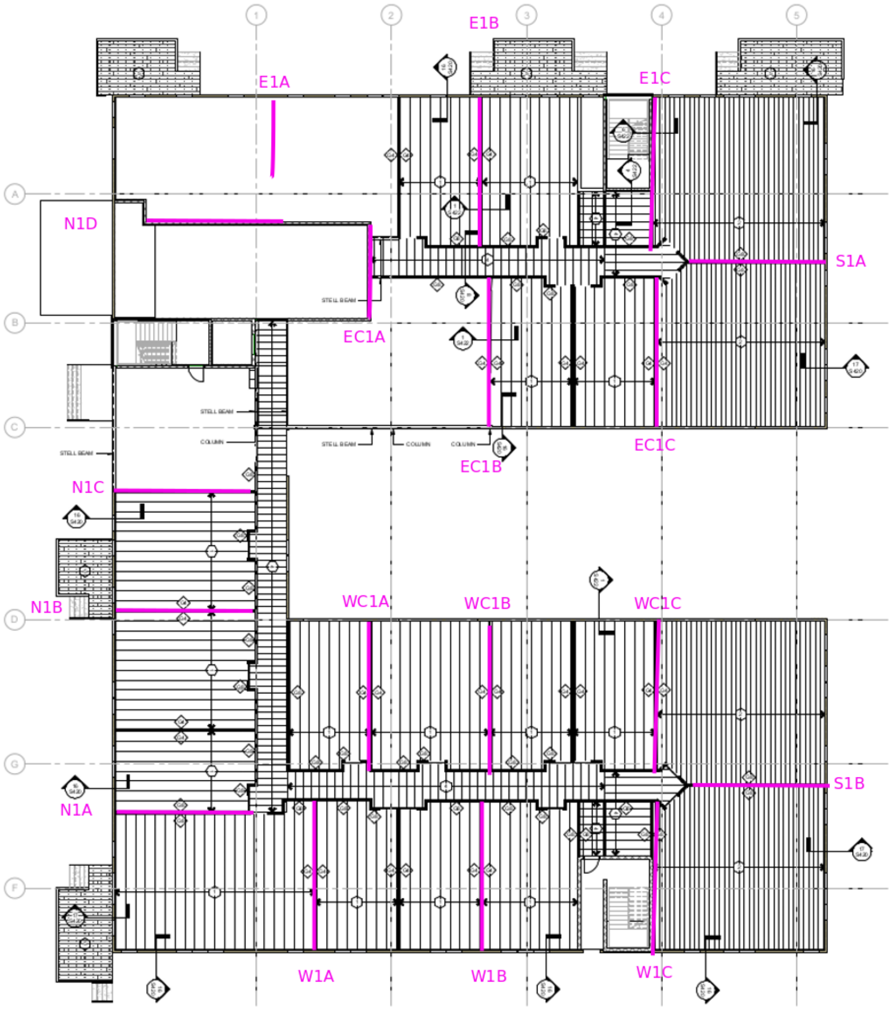


Figure 22: Shear walls on the first floor.

9 Basement

9.1 Structural model

A three-dimensional elastic computer model of the substructure is analyzed using XC. The model includes first floor frame and columns (see figure 23). The hollow core planks ar modelled using shell elements, while beams and columns are modelled using frame elements. Loads transmited by 2^{nd}, 3^{rd} floors and roof are applied to the 1^{st}. Load layout is shown in figure 32. See in figures 24 to 31 load distribution for each load case.

Linear loads are expressed in kN/m and surface loads in kN/m^2, where:

1 kN/m =	68.52178 lb/ft
1 kN/m^2 =	20.885434 psf

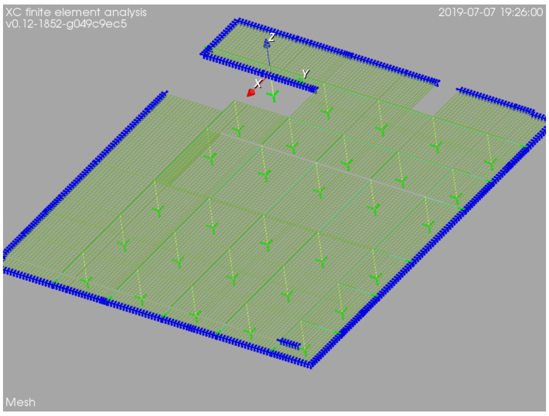


Figure 23: Elastic model, mesh.

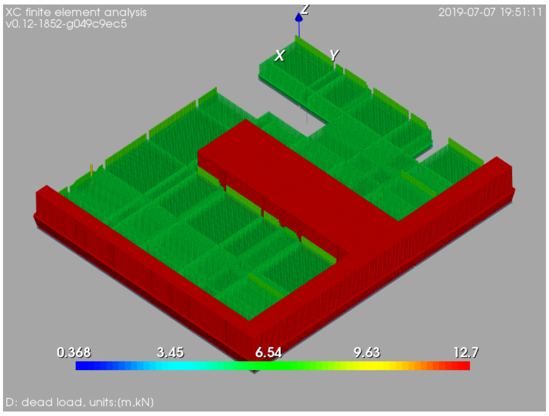


Figure 24: Load case D: dead load (include slab self-weight) [units: kN,m].

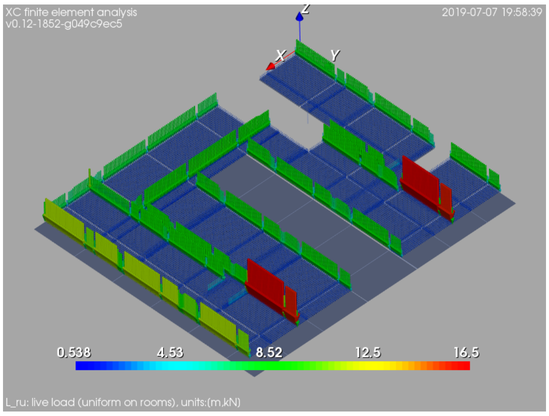


Figure 25: Load case Lru: live load (uniform on rooms) [units: kN,m].

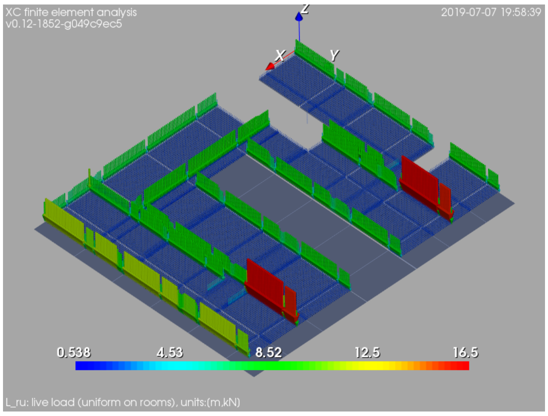


Figure 26: Load case Lrs: live load (staggered pattern on rooms) [units: kN,m].

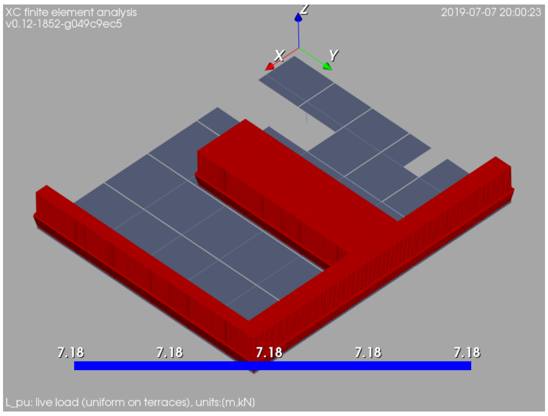


Figure 27: Load case Lpu: live load (uniform on patios) [units: kN,m].

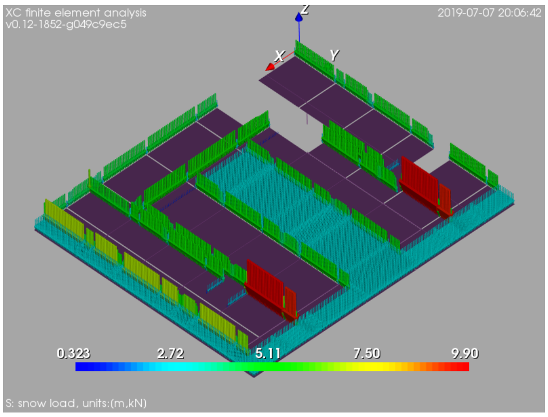


Figure 28: Load case S: snow [units: kN,m].

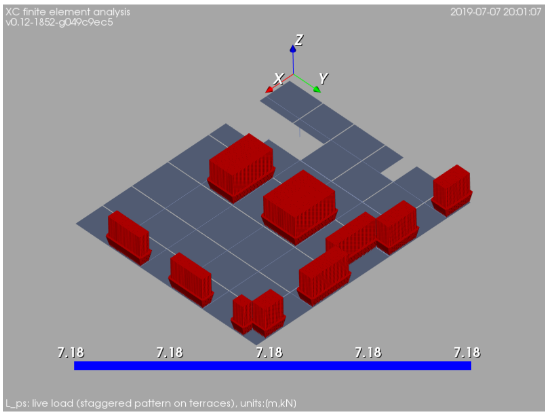


Figure 29: Load case Lps: live load (staggered pattern on patios) [units: kN,m].

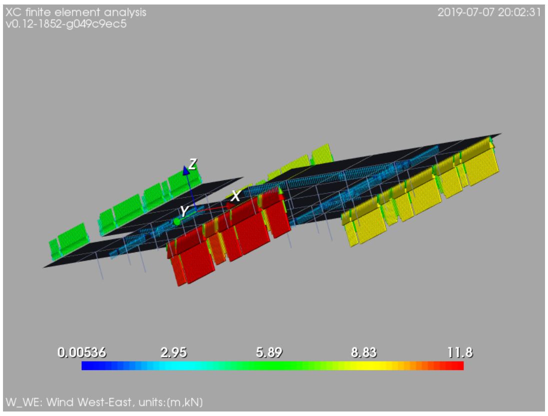


Figure 30: Load case W_WE: wind West-East [units: kN,m].

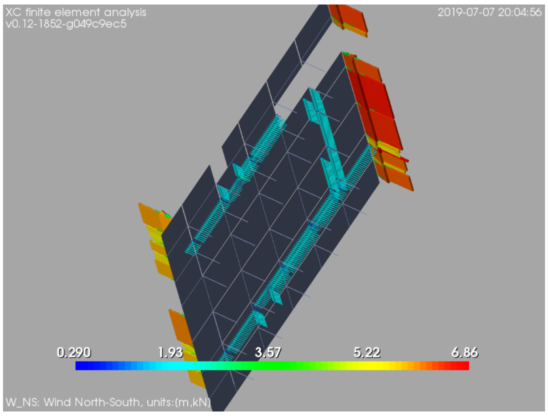
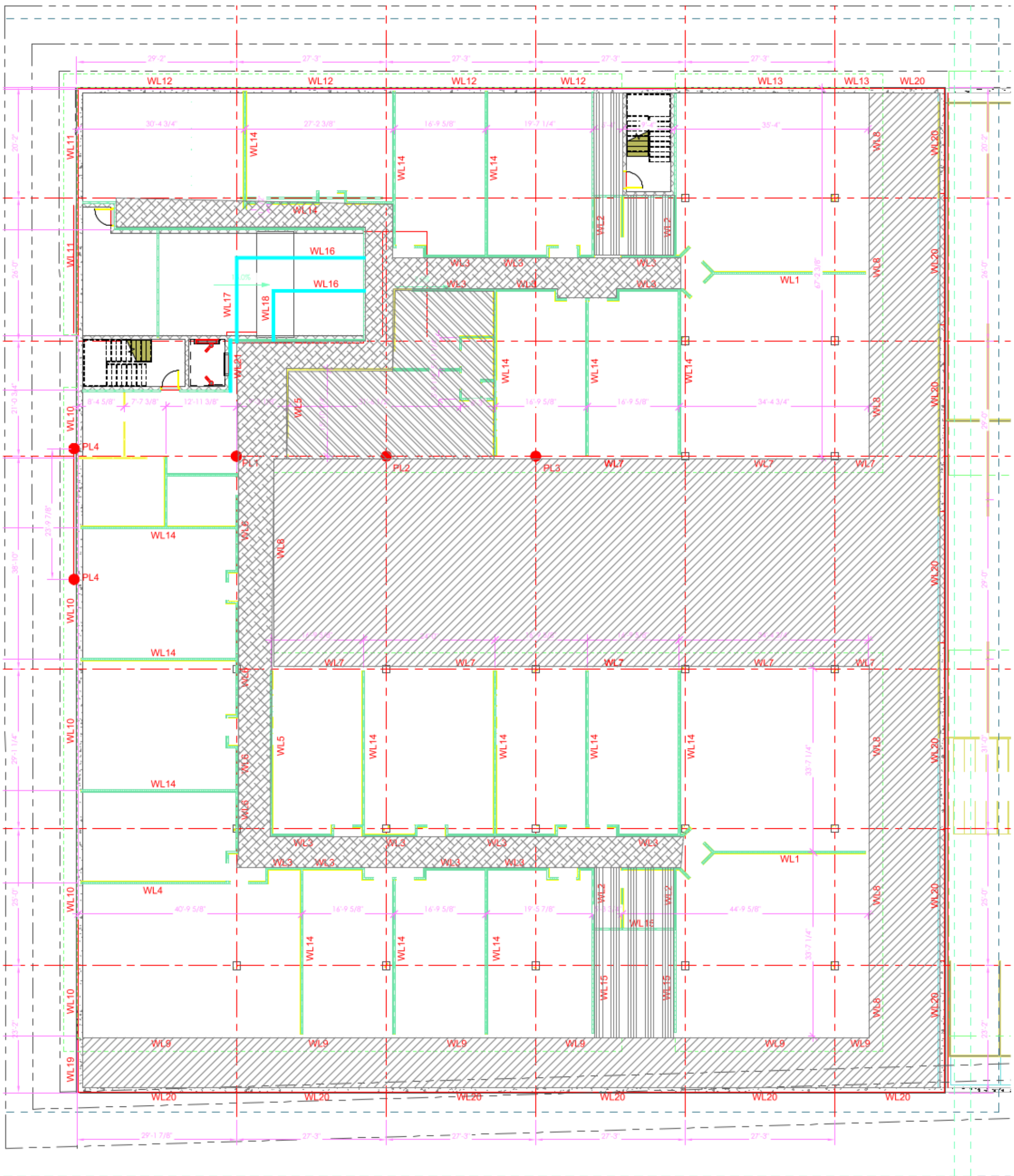


Figure 31: Load case W_NS: wind North-South [units: kN,m].



WALL LOAD SCHEDULE													
MARK	ELEV.	DEAD LOAD Klb/ft	LIVE LOAD Klb/ft	SNOW LOAD Klb/ft	W-E WIND LOAD			N-S WIND LOAD			EARTH PRES.	CHORD LOAD	
					UPLIFT Klb/ft	DOWN LIFT Klb/ft	IN PLANE Klb/ft	UPLIFT Klb/ft	DOWN LIFT Klb/ft	IN PLANE Klb/ft	IN PLANE Klb/ft	DEAD IN PLANE Klb/ft	LIVE IN PLANE Klb/ft
WL1	-0'-4"	1.51	3.35	2.01	0.40	N/A	N/A	0.40	N/A	0.15	N/A	N/A	N/A
WL2	-0'-4"	0.83	0.74	0.44	0.40	N/A	N/A	0.40	N/A	N/A	N/A	N/A	N/A
WL3	-0'-4"	1.12	1.84	1.10	0.40	N/A	N/A	0.40	N/A	0.15	N/A	N/A	N/A
WL4	-0'-4"	1.05	1.57	0.94	0.40	N/A	N/A	0.40	N/A	0.15	N/A	N/A	N/A
WL5	-0'-4"	0.72	0.33	0.20	0.40	N/A	0.39	0.40	N/A	N/A	N/A	N/A	N/A
WL6	-0'-4"	1.11	1.79	1.07	0.40	N/A	0.39	0.40	N/A	N/A	N/A	N/A	N/A
WL7	-0'-4"	1.58	1.67	1.00	1.61	2.39	N/A	N/A	N/A	0.15	N/A	N/A	N/A
WL8	-0'-4"	0.50	N/A	N/A	N/A	N/A	0.39	N/A	1.20	N/A	N/A	N/A	N/A
WL9	-0'-4"	1.73	2.26	1.36	1.86	N/A	N/A	N/A	N/A	0.15	N/A	N/A	N/A
WL10	-0'-4"	1.52	1.46	0.88	N/A	N/A	0.39	1.28	N/A	N/A	2.30	N/A	N/A
WL11	-0'-4"	0.50	N/A	N/A	N/A	N/A	0.39	1.28	N/A	N/A	2.30	N/A	N/A
WL12	-0'-4"	1.55	1.57	0.94	N/A	1.08	N/A	N/A	N/A	0.15	2.30	N/A	N/A
WL13	-0'-4"	1.58	1.67	1.00	N/A	1.08	N/A	N/A	N/A	0.15	2.30	N/A	N/A
WL14	-0'-4"	0.27	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
WL15	-0'-4"	1.92	1.33	0.44	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
WL16	11'-2"	0.95	1.10	1.10	0.40	N/A	N/A	0.40	N/A	0.16	N/A	N/A	N/A
WL17	11'-2"	0.94	1.07	1.07	0.40	N/A	0.39	0.40	N/A	N/A	N/A	N/A	N/A
WL18	11'-2"	0.69	0.20	0.20	0.40	N/A	0.39	0.40	N/A	N/A	N/A	N/A	N/A
WL19	-1'-8"	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	2.30	N/A	N/A
WL20	-1'-8"	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	2.30	N/A	N/A
WL21	-0'-4"	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	1.94	1.76

POINT LOAD SCHEDULE			
MARK	DEAD LOAD Klb	LIVE LOAD Klb	SNOW LOAD Klb
PL1	15.24	25.01	15.01
PL2	30.47	50.02	30.02
PL3	15.24	25.01	15.01
PL4	17.84	17.40	10.50

SUPERIMPOSED UNIFORM LOAD SCHEDULE				
MARK	DEAD LOAD PSF	LIVE LOAD PSF	SNOW LOAD PSF	COMMENTS
	150.00	150.00	42.00	YARDS AND TERRACES PEDESTRIAN
	20.00	100.00	N/A	CORRIDORS FIRST FLOOR
	20.00	100.00	N/A	STAIRS AND EXITS
	20.00	100.00	N/A	STORE FIRST FLOOR
	20.00	40.00	N/A	PRIVATE ROOMS MULTIFAMILY DWELLING

Figure 32: Load layout on first floor.

COLUMN FOOTING SCHEDULE

MARK	DIMENSIONS			BOTTOM REINFORCING		COLUMNS
	W	L	D	LONG	SHORT	
FT90a	9'-0"	9'-0"	1'-8"	(10)-#7	(10)-#7	A1 A2
FT90b	9'-0"	9'-0"	1'-8"	(10)-#8	(10)-#8	A3 A4 A5
FT96a	9'-6"	9'-6"	1'-8"	(10)-#7	(10)-#7	B2
FT96b	9'-6"	9'-6"	1'-8"	(10)-#8	(10)-#8	B3 B4 B5
FT100	10'-0"	10'-0"	2'-1"	(11)-#8	(11)-#8	F1 F2 F3 F4 F5
FT106	10'-6"	10'-6"	2'-3"	(11)-#8	(11)-#8	C1
FT110a	11'-0"	11'-0"	2'-1"	(12)-#8	(12)-#8	G2 G3 G4 G5
FT110b	11'-0"	11'-0"	2'-3"	(12)-#8	(12)-#8	D1 G1
FT116	11'-6"	11'-6"	2'-1"	(12)-#8	(12)-#8	C2 C3 C4 C5
FT120	12'-0"	12'-0"	2'-3"	(13)-#8	(13)-#8	D2 D3 D4 D5

COLUMN FOOTING SCHEDULE:
1. REFER TO STRUCTURAL NOTES SHEET FOR LAPS IN STEEL REINFORCEMENT.
2. REFER TO FUNDATION PLAN FOR TOP OF FOOTING ELEVATIONS.
3. ALL FOOTINF EXCAVATIONS SHALL BE INSPECTED AND APPROVED BY THE GEOTECHNICAL ENGINEER PRIOR TO PLACING CONCRETE

Table 17: Column footing schedule.

9.2 Footings

9.2.1 Loads

The loads acting on the footings are shown in §B.

9.2.2 Load combinations

The load combinations are shown in tables 7 and 8.

9.2.3 Footing dimensions and reinforcement

The dimensions and the reinforcement of the footings are indicated in the table 17. The position of the footing in the building grid is indicated at the last column.

9.2.4 Limit state checking

Allowable soil-bearing pressures. The results obtained for the verification of the soil-bearing capacity are shown in the table 18.

Flexure design. The capacity factor for the bending in the longitudinal and transverse directions are shown in figures 33 and 34.

Shear design. The results of the shear strength verification are shown in the table 19. The results of the punching shear strength verification are shown in table 20.

9.3 Basement walls

9.3.1 Introduction

The design is based in the following assumptions:

- Design wall with pinned base and pinned top.
- Neglect corner regions (wall spans one-way only).
- Top slab is in place and has achieved full strength prior to back-filling.
- Vehicular traffic around the building is represented by a uniform load of 250 psf (11.97 kN/m^2).

Foundation	Worst combination	Vertical load (kN)	Capacity factor
A1	SLS04_a	-356.20	0.33
A2	SLS02_a	-644.78	0.60
A3	SLS02_a	-950.92	0.88
A4	SLS02_a	-881.82	0.82
A5	SLS04_b	-933.03	0.86
B2	SLS02_a	-670.79	0.56
B3	SLS02_a	-1,030.65	0.86
B4	SLS02_a	-968.24	0.80
B5	SLS04_b	-972.32	0.81
C1	SLS02_a	-1,460.67	0.99
C2	SLS02_a	-1,742.45	0.99
C3	SLS02_a	-1,750.31	0.99
C4	SLS02_a	-1,751.62	0.99
C5	SLS02_a	-1,660.02	0.94
D1	SLS02_a	-1,648.12	1.02
D2	SLS04_a	-1,950.01	1.01
D3	SLS04_a	-1,959.12	1.02
D4	SLS04_a	-1,960.14	1.02
D5	SLS04_a	-1,838.69	0.96
F1	SLS02_a	-1,090.88	0.82
F2	SLS02_a	-997.32	0.75
F3	SLS02_a	-1,011.87	0.76
F4	SLS02_a	-1,005.30	0.75
F5	SLS04_b	-741.90	0.56
G1	SLS02_a	-1,496.86	0.93
G2	SLS02_a	-1,256.06	0.78
G3	SLS02_a	-1,227.94	0.76
G4	SLS02_a	-1,137.75	0.70
G5	SLS04_b	-1,167.26	0.72

Table 18: Soil bearing pressures. Capacity factors

Footing	Worst combination	Vertical load (kN)	thickness (m)	l (m)	d (m)	c (m)	Vd/l kN/m	Vu kN/m	CF
A1	SLS04_a	-356.20	0.51	2.74	0.46	0.41	33.66	280.00	0.12
A2	SLS02_a	-644.78	0.51	2.74	0.46	0.41	60.94	280.00	0.22
A3	SLS02_a	-950.92	0.51	2.74	0.46	0.41	89.87	280.00	0.32
A4	SLS02_a	-881.82	0.51	2.74	0.46	0.41	83.34	280.00	0.30
A5	SLS04_b	-933.03	0.51	2.74	0.46	0.41	88.18	280.00	0.31
B2	SLS02_a	-670.79	0.51	2.90	0.46	0.41	63.00	280.00	0.22
B3	SLS02_a	-1,030.65	0.51	2.90	0.46	0.41	96.79	280.00	0.35
B4	SLS02_a	-968.24	0.51	2.90	0.46	0.41	90.93	280.00	0.32
B5	SLS04_b	-972.32	0.51	2.90	0.46	0.41	91.31	280.00	0.33
C1	SLS02_a	-1,460.67	0.69	3.20	0.62	0.41	111.20	378.00	0.29
C2	SLS02_a	-1,742.45	0.64	3.51	0.57	0.41	138.68	350.00	0.40
C3	SLS02_a	-1,750.31	0.64	3.51	0.57	0.41	139.31	350.00	0.40
C4	SLS02_a	-1,751.62	0.64	3.51	0.57	0.41	139.42	350.00	0.40
C5	SLS02_a	-1,660.02	0.64	3.51	0.57	0.41	132.12	350.00	0.38
D1	SLS02_a	-1,648.12	0.69	3.35	0.62	0.41	125.50	378.00	0.33
D2	SLS04_a	-1,950.01	0.64	3.66	0.57	0.41	153.65	350.00	0.44
D3	SLS04_a	-1,959.12	0.64	3.66	0.57	0.41	154.37	350.00	0.44
D4	SLS04_a	-1,960.14	0.64	3.66	0.57	0.41	154.45	350.00	0.44
D5	SLS04_a	-1,838.69	0.64	3.66	0.57	0.41	144.88	350.00	0.41
F1	SLS02_a	-1,090.88	0.64	3.05	0.57	0.41	87.98	350.00	0.25
F2	SLS02_a	-997.32	0.64	3.05	0.57	0.41	80.44	350.00	0.23
F3	SLS02_a	-1,011.87	0.64	3.05	0.57	0.41	81.61	350.00	0.23
F4	SLS02_a	-1,005.30	0.64	3.05	0.57	0.41	81.08	350.00	0.23
F5	SLS04_b	-741.90	0.64	3.05	0.57	0.41	59.84	350.00	0.17
G1	SLS02_a	-1,496.86	0.69	3.35	0.62	0.41	113.98	378.00	0.30
G2	SLS02_a	-1,256.06	0.64	3.35	0.57	0.41	100.75	350.00	0.29
G3	SLS02_a	-1,227.94	0.64	3.35	0.57	0.41	98.50	350.00	0.28
G4	SLS02_a	-1,137.75	0.64	3.35	0.57	0.41	91.26	350.00	0.26
G5	SLS04_b	-1,167.26	0.64	3.35	0.57	0.41	93.63	350.00	0.27

Table 19: Shear design. Capacity factors

Footing	Worst combination	Vertical load (kN)	thickness (m)	L (m)	d (m)	c (m)	Vd/l kN/m	Vu kN/m	CF
A1	SLS04_a	-356.20	0.51	2.74	0.46	0.41	92.90	517.97	0.18
A2	SLS02_a	-644.78	0.51	2.74	0.46	0.41	168.16	517.97	0.32
A3	SLS02_a	-950.92	0.51	2.74	0.46	0.41	248.00	517.97	0.48
A4	SLS02_a	-881.82	0.51	2.74	0.46	0.41	229.98	517.97	0.44
A5	SLS04_b	-933.03	0.51	2.74	0.46	0.41	243.33	517.97	0.47
B1	SLS02_a	-429.53	0.51	2.90	0.46	0.41	113.28	517.97	0.22
B2	SLS02_a	-670.79	0.51	2.90	0.46	0.41	176.91	517.97	0.34
B3	SLS02_a	-1,030.65	0.51	2.90	0.46	0.41	271.82	517.97	0.52
B4	SLS02_a	-968.24	0.51	2.90	0.46	0.41	255.36	517.97	0.49
B5	SLS04_b	-972.32	0.51	2.90	0.46	0.41	256.43	517.97	0.50
C1	SLS02_a	-1,460.67	0.69	3.20	0.62	0.41	320.25	699.26	0.46
C2	SLS02_a	-1,742.45	0.64	3.51	0.57	0.41	410.79	647.47	0.63
C3	SLS02_a	-1,750.31	0.64	3.51	0.57	0.41	412.64	647.47	0.64
C4	SLS02_a	-1,751.62	0.64	3.51	0.57	0.41	412.95	647.47	0.64
C5	SLS02_a	-1,660.02	0.64	3.51	0.57	0.41	391.35	647.47	0.60
D1	SLS02_a	-1,648.12	0.69	3.35	0.62	0.41	365.00	699.26	0.52
D2	SLS04_a	-1,950.01	0.64	3.66	0.57	0.41	462.88	647.47	0.71
D3	SLS04_a	-1,959.12	0.64	3.66	0.57	0.41	465.05	647.47	0.72
D4	SLS04_a	-1,960.14	0.64	3.66	0.57	0.41	465.29	647.47	0.72
D5	SLS04_a	-1,838.69	0.64	3.66	0.57	0.41	436.46	647.47	0.67
F1	SLS02_a	-1,090.88	0.64	3.05	0.57	0.41	250.18	647.47	0.39
F2	SLS02_a	-997.32	0.64	3.05	0.57	0.41	228.72	647.47	0.35
F3	SLS02_a	-1,011.87	0.64	3.05	0.57	0.41	232.06	647.47	0.36
F4	SLS02_a	-1,005.30	0.64	3.05	0.57	0.41	230.55	647.47	0.36
F5	SLS04_b	-741.90	0.64	3.05	0.57	0.41	170.14	647.47	0.26
G1	SLS02_a	-1,496.86	0.69	3.35	0.62	0.41	331.50	699.26	0.47
G2	SLS02_a	-1,256.06	0.64	3.35	0.57	0.41	293.79	647.47	0.45
G3	SLS02_a	-1,227.94	0.64	3.35	0.57	0.41	287.22	647.47	0.44
G4	SLS02_a	-1,137.75	0.64	3.35	0.57	0.41	266.12	647.47	0.41
G5	SLS04_b	-1,167.26	0.64	3.35	0.57	0.41	273.02	647.47	0.42

Table 20: Two-way shear design. Capacity factors

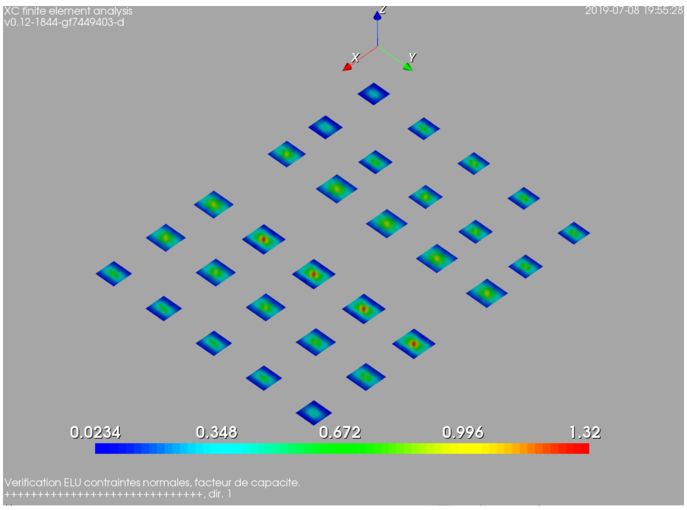


Figure 33: Flexure in the longitudinal direction. Capacity factor.

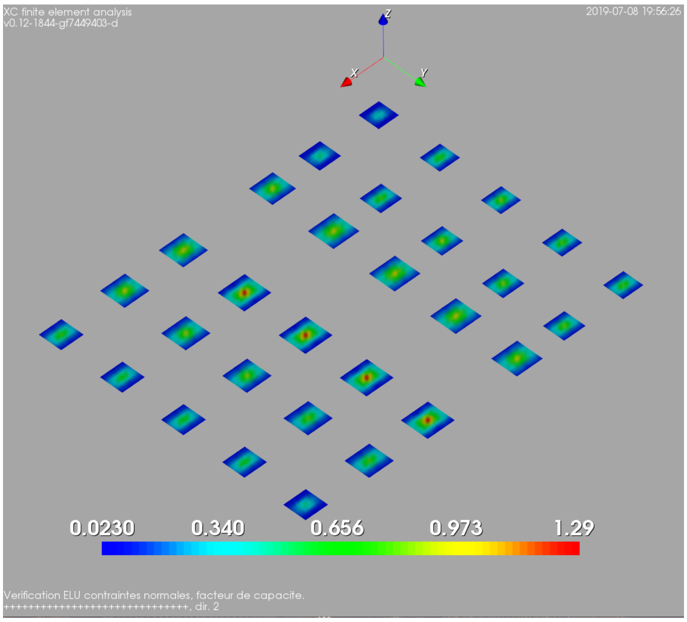


Figure 34: Flexure in the transverse direction. Capacity factor.

- The vertical response of the soil calculated using a Winkler model with a sub-grade reaction module of set of 200 pounds per cubic inch ($54.29 \times 10^6\ N/m^3$).

- Water table deep below structure.

9.3.2 Load determination

Self weight. The self weight of the reinforced concrete is calculated from its density: $2500\ kg/m^3$.

Axial loads from building. The loads transferred by the top slab to the wall are as follows:

Building side	Load	Phase 1 (kN/m)	Phase 2 (kN/m)
North	SnowL	10.06	10.06
North	LiveL	21.67	21.67
North	Wind_NS	-15.12	-15.12
North	Wind_WE	-1.33	-1.33
North	DeadL	31.54	31.54
South	SnowL	8.04	16.08
South	LiveL	14.22	28.44
South	Wind_NS	4.97	9.95
South	Wind_WE	-0.23	-0.46
South	DeadL	20.58	41.15
East	SnowL	11.96	11.96
East	LiveL	23.75	23.75
East	Wind_NS	-0.07	-0.07
East	Wind_WE	12.97	12.97
East	DeadL	30.87	30.87
West	SnowL	15.02	15.02
West	LiveL	27.15	27.15
West	Wind_NS	-0.20	-0.20
West	Wind_WE	-13.20	-13.20
West	DeadL	29.81	29.81

9.3.3 Load combinations

Serviceability limit states		
Equation 16-8	EQ1608	1.0*selfWeight+1.0*deadLoad
Equation 16-9	EQ1609A	1.0*selfWeight+1.0*deadLoad+1.0*trafficLoad
Equation 16-9	EQ1609B	1.0*selfWeight+1.0*deadLoad+1.0*liveLoad
Equation 16-10	EQ1610	1.0*selfWeight+1.0*deadLoad+1.0*snowLoad
Equation 16-11	EQ1611A	1.0*selfWeight+1.0*deadLoad+0.75*trafficLoad+0.75*snowLoad
Equation 16-11	EQ1611B	1.0*selfWeight+1.0*deadLoad+0.75*liveLoad+0.75*snowLoad
Equation 16-12	EQ1612	1.0*selfWeight+1.0*deadLoad+0.6*windLoad
Equation 16-13	EQ1613A	1.0*selfWeight+1.0*deadLoad+0.45*windLoad+0.75*trafficLoad+0.75*snowLoad
Equation 16-13	EQ1613B	1.0*selfWeight+1.0*deadLoad+0.45*windLoad+0.75*liveLoad+0.75*snowLoad
Equation 16-14	doesn't apply	
Equation 16-15	EQ1615	0.6*selfWeight+0.6*deadLoad+0.6*windLoad
Equation 16-16	doesn't apply	

Ultimate limit states.		
Equation 16-1	EQ1601	1.4*selfWeight+1.4*deadLoad
Equation 16-2	EQ1602A	1.2*selfWeight+1.2*deadLoad+1.6*trafficLoad+0.5*snowLoad
Equation 16-2	EQ1602B	1.2*selfWeight+1.2*deadLoad+1.6*liveLoad+0.5*snowLoad
Equation 16-3	EQ1603A	1.2*selfWeight+1.2*deadLoad+1.6*snowLoad+0.5*trafficLoad
Equation 16-3	EQ1603B	1.2*selfWeight+1.2*deadLoad+1.6*snowLoad+0.5*liveLoad
Equation 16-3	EQ1603C	1.2*selfWeight+1.2*deadLoad+1.6*snowLoad+0.5*windLoad
Equation 16-4	EQ1604A	1.2*selfWeight+1.2*deadLoad+1.0*windLoad+0.5*trafficLoad+0.5*snowLoad
Equation 16-4	EQ1604B	1.2*selfWeight+1.2*deadLoad+1.0*windLoad+0.5*liveLoad+0.5*snowLoad
Equation 16-5	EQ1605A	1.2*selfWeight+1.2*deadLoad+0.5*trafficLoad+0.7*snowLoad
Equation 16-5	EQ1605B	1.2*selfWeight+1.2*deadLoad+0.5*liveLoad+0.7*snowLoad
Equation 16-6	doesn't apply	
Equation 16-7	doesn't apply	

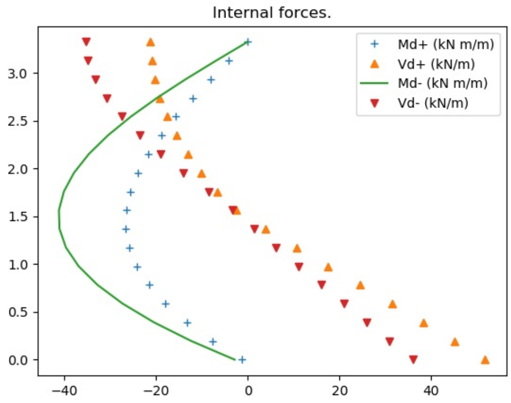
Earth pressure. The soil pressure over the wall has been calculated using the lateral pressure at rest with a coefficient $K_0 = 0.5$.

CONCRETE WALL REINFORCING SCHEDULE					
MARK	TYPE	THICKNESS	REINFORCEMENT VERTICAL	REINFORCEMENT HORIZONTAL	REMARKS
W1	CONCRETE	10"	5#'s AT 18"o.c.	5#'s AT 12"o.c.	inside face
W2	CONCRETE	10"	5#'s AT 12"o.c.	5#'s AT 12"o.c.	inside face
W3	CONCRETE	10"	6#'s AT 12"o.c.	5#'s AT 12"o.c.	inside face
W4	CONCRETE	8"	4#'s AT 12"o.c.	3#'s AT 12"o.c.	centered in wall thickness

CONCRETE WALL REINFORCING SCHEDULE NOTES:
1. REFER TO STRUCTURAL NOTES SHEET FOR LAPS IN STEEL REINFORCEMENT.
2. COORDINATE AND VERIFY ALL DIMENSIONS WITH ARCHITECTURAL DRAWINGS AND EXIST. CONDITIONS

Table 21: Concrete walls reinforcing schedule

T1



Wall geometry
Stem top thickness:
$b_{top} = 0.25\ m$
Stem height:
$h_{stem} = 3.15\ m$
Stem bottom thickness:
$b_{bottom} = 0.25\ m$
Footing thickness:
$b_{footing} = 0.36\ m$

Materials
Concrete: C4000 Steel: A615G60 Concrete cover: 55 mm

Table 22: Wall materials and dimensions T1

9.3.4 Stem dimensions and reinforcement

The thickness and the reinforcement for the walls are indicated in the table 21.

Wall types. For analysis purposes we have considered the following wall types:

Wall	Stem height (m)
T1	3.15
T2	2.74
T3	3.53
T4	3.12
T5	2.51
T6	3.43

Internal forces. The envelope of internal forces envelope for each of the walls are given in tables 22 to 27.

<table>
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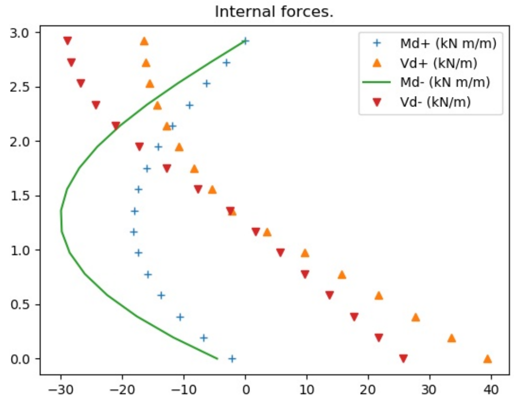
<tr><td>
</td><td>WALL GEOMETRY
Stem top thickness:
$b_{top} = 0.25\ m$
Stem height:
$h_{stem} = 2.74\ m$
Stem bottom thickness:
$b_{bottom} = 0.25\ m$
Footing thickness:
$b_{footing} = 0.36\ m$</td></tr>
<tr><td colspan="2">MATERIALS
Concrete: C4000 Steel: A615G60 Concrete cover: 55 mm</td></tr>
</table>

Table 23: Wall materials and dimensions T2

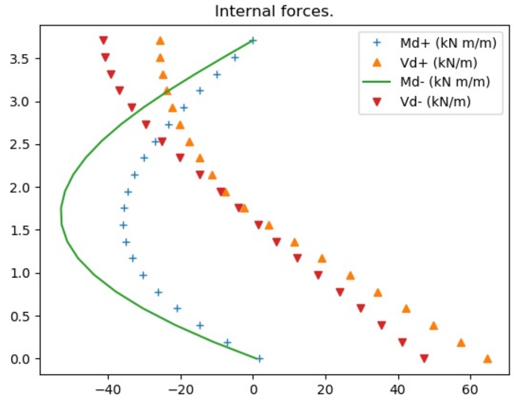
<table>
<tr><th colspan="2">T3</th></tr>
<tr><td>
</td><td>WALL GEOMETRY
Stem top thickness:
$b_{top} = 0.25\ m$
Stem height:
$h_{stem} = 3.53\ m$
Stem bottom thickness:
$b_{bottom} = 0.25\ m$
Footing thickness:
$b_{footing} = 0.36\ m$</td></tr>
<tr><td colspan="2">MATERIALS
Concrete: C4000 Steel: A615G60 Concrete cover: 55 mm</td></tr>
</table>

Table 24: Wall materials and dimensions T3

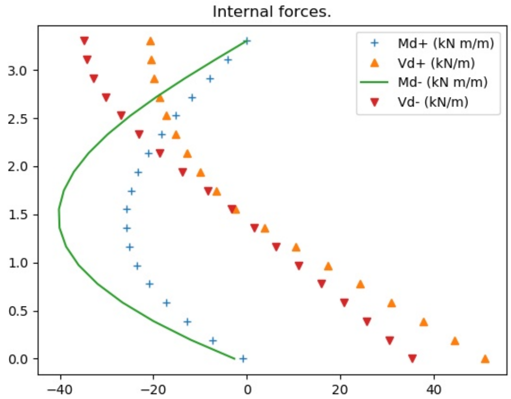
<table>
<tr><th colspan="2">T4</th></tr>
<tr><td>
</td><td>WALL GEOMETRY
Stem top thickness:
$b_{top} = 0.25\ m$
Stem height:
$h_{stem} = 3.12\ m$
Stem bottom thickness:
$b_{bottom} = 0.25\ m$
Footing thickness:
$b_{footing} = 0.36\ m$</td></tr>
<tr><td colspan="2">MATERIALS
Concrete: C4000 Steel: A615G60 Concrete cover: 55 mm</td></tr>
</table>

Table 25: Wall materials and dimensions T4

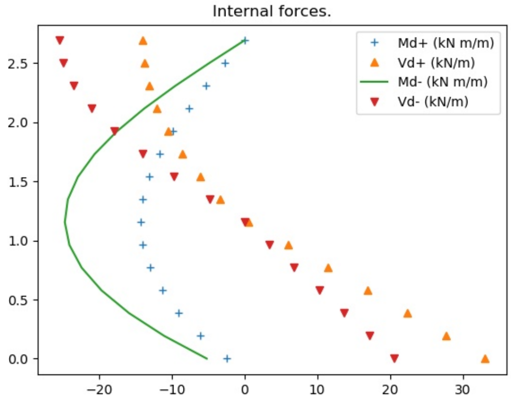
<table>
<tr><th colspan="2">T5</th></tr>
<tr><td>
</td><td>WALL GEOMETRY
Stem top thickness:
$b_{top} = 0.25\ m$
Stem height:
$h_{stem} = 2.51\ m$
Stem bottom thickness:
$b_{bottom} = 0.25\ m$
Footing thickness:
$b_{footing} = 0.36\ m$</td></tr>
<tr><td colspan="2">MATERIALS
Concrete: C4000 Steel: A615G60 Concrete cover: 55 mm</td></tr>
</table>

Table 26: Wall materials and dimensions T5

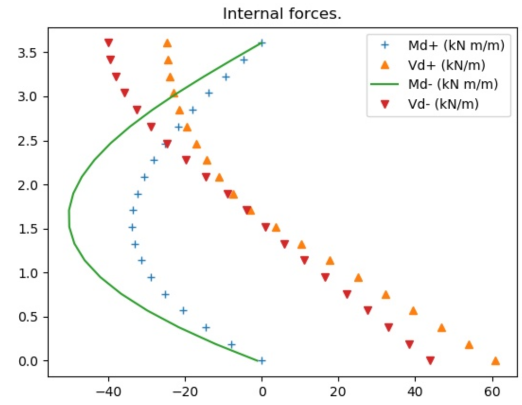
<table>
<tr><th colspan="2">T6</th></tr>
<tr><td>
</td><td>WALL GEOMETRY
Stem top thickness:
$b_{top} = 0.25\ m$
Stem height:
$h_{stem} = 3.43\ m$
Stem bottom thickness:
$b_{bottom} = 0.25\ m$
Footing thickness:
$b_{footing} = 0.36\ m$</td></tr>
<tr><td colspan="2">MATERIALS
Concrete: C4000 Steel: A615G60 Concrete cover: 55 mm</td></tr>
</table>

Table 27: Wall materials and dimensions T6

Reinforcement checks.

WALL VERTICAL REINFORCEMENTS
T1 wall. Inside stem reinforcement: RC section dimensions; b= 1.00 m, h= 0.25 m diam: 16 mm, spacing: 300 mm reinf. development L=0.34 m (22 diameters). area: As= 6.67 cm2/m areaMin: 4.56 cm2/m F(As)= 1.46 OK! Bending check: Md= 40.09 kN m, MR= 41.36kN m F(M)= 1.03 OK! Shear check: Vd= 7.61 kN, VR= 199.37 kN F(V)= 26.21 OK!
T2 wall. Inside stem reinforcement: RC section dimensions; b= 1.00 m, h= 0.25 m diam: 16 mm, spacing: 400 mm reinf. development L=0.34 m (22 diameters). area: As= 5.00 cm2/m areaMin: 4.56 cm2/m F(As)= 1.10 OK! Bending check: Md= 29.02 kN m, MR= 31.02kN m F(M)= 1.07 OK! Shear check: Vd= 6.84 kN, VR= 199.37 kN F(V)= 29.15 OK!
T3 wall. Inside stem reinforcement: RC section dimensions; b= 1.00 m, h= 0.25 m diam: 19 mm, spacing: 300 mm reinf. development L=0.61 m (32 diameters). area: As= 9.47 cm2/m areaMin: 4.56 cm2/m F(As)= 2.08 OK! Bending check: Md= 51.88 kN m, MR= 58.26kN m F(M)= 1.12 OK! Shear check: Vd= 7.98 kN, VR= 199.37 kN F(V)= 24.98 OK!
T4 wall. Inside stem reinforcement: RC section dimensions; b= 1.00 m, h= 0.25 m diam: 16 mm, spacing: 300 mm reinf. development L=0.34 m (22 diameters). area: As= 6.67 cm2/m areaMin: 4.56 cm2/m F(As)= 1.46 OK! Bending check: Md= 39.19 kN m, MR= 41.36kN m F(M)= 1.06 OK!
../..

wall vertical reinforcements (cont.)
Shear check: Vd= 7.46 kN, VR= 199.37 kN F(V)= 26.73 OK!
T5 wall. Inside stem reinforcement: RC section dimensions; b= 1.00 m, h= 0.25 m diam: 16 mm, spacing: 400 mm reinf. development L=0.34 m (22 diameters). area: As= 5.00 cm2/m areaMin: 4.56 cm2/m F(As)= 1.10 OK! Bending check: Md= 23.62 kN m, MR= 31.02kN m F(M)= 1.31 OK! Shear check: Vd= 6.42 kN, VR= 199.37 kN F(V)= 31.05 OK!
T6 wall. Inside stem reinforcement: RC section dimensions; b= 1.00 m, h= 0.25 m diam: 19 mm, spacing: 300 mm reinf. development L=0.61 m (32 diameters). area: As= 9.47 cm2/m areaMin: 4.56 cm2/m F(As)= 2.08 OK! Bending check: Md= 49.02 kN m, MR= 58.26kN m F(M)= 1.19 OK! Shear check: Vd= 8.13 kN, VR= 199.37 kN F(V)= 24.54 OK!

Shear check
T1 wall. Shear check: Shear check: Vd= 42.99 kN, VR= 199.37 kN F(V)= 4.64 OK!
T2 wall. Shear check: Shear check: Vd= 31.78 kN, VR= 199.37 kN F(V)= 6.27 OK!
T3 wall. Shear check: Shear check: Vd= 55.00 kN, VR= 199.37 kN F(V)= 3.63 OK!
T4 wall. Shear check: Shear check: Vd= 42.35 kN, VR= 199.37 kN F(V)= 4.71 OK!
T5 wall. Shear check: Shear check: Vd= 26.03 kN, VR= 199.37 kN F(V)= 7.66 OK!
T6 wall. Shear check: Shear check: Vd= 51.43 kN, VR= 199.37 kN F(V)= 3.88 OK!

Wall foundations The results obtained for the verifications of the footing stability and the soil-bearing capacity. According to the geotechnical report the allowable soil bearing pressure is 3000 psf (143.64 kN/m^2).

WALL FOUNDATION: T1			
Verification:	F_{disp}	F_{req}	Combination
Overturning:	$\gg 1$	1.00	EQ1609A
Sliding:	1.23	1.00	EQ1609A
Adm. pressure:	1.09	1.00	EQ1613B
WALL FOUNDATION: T2			
Verification:	F_{disp}	F_{req}	Combination
Overturning:	$\gg 1$	1.00	EQ1613B
Sliding:	1.46	1.00	EQ1609A
Adm. pressure:	1.13	1.00	EQ1613B
WALL FOUNDATTION: T3			
Verification:	F_{disp}	F_{req}	Combination
Overturning:	$\gg 1$	1.00	EQ1609A
Sliding:	1.13	1.00	EQ1609A
Adm. pressure:	1.12	1.00	EQ1613B
WALL FOUNDATION: T4			
Verification:	F_{disp}	F_{req}	Combination
Overturning:	$\gg 1$	1.00	EQ1613B
Sliding:	1.45	1.00	EQ1609A
Adm. pressure:	1.08	1.00	EQ1613B
WALL FOUNDATION: T5			
Verification:	F_{disp}	F_{req}	Combination
Overturning:	$\gg 1$	1.00	EQ1613B
Sliding:	1.69	1.00	EQ1609A
Adm. pressure:	1.22	1.00	EQ1613B
WALL FOUNDATION: T6			
Verification:	F_{disp}	F_{req}	Combination
Overturning:	$\gg 1$	1.00	EQ1609A
Sliding:	1.10	1.00	EQ1609A
Adm. pressure:	1.03	1.00	EQ1613B
$F_{avail.}$: available security. F_{req}: required security.			

9.4 Ramp

9.4.1 Design criteria

Materials	Concrete: $f'_c = 4.0$ ksi
	Reinforcing steel: $f_y = 60$ ksi
Structural loads	Self weight reinforced concrete: 2500 kg/m^3
	Live load garages (passenger vehicles): 40 psf
	Concentrated load vehicle : 3000 pound
Load cases	D: dead load (see fig. 37)
	Lunif: uniform live load (see fig. 38)
	LconcSpan1: concentrated live load on mid-span 1 (see fig. 39)
	LconcSpan2: concentrated live load on mid-span 2 (see fig. 40)
	LconcSpan3: concentrated live load on mid-span 3 (see fig. 41)
Ultim. Limit States	ULS01: 1.4*D
	ULS02: 1.2*D + 1.6*Lunif
	ULS03: 1.2*D + 1.6*LconcSpan1
	ULS04: 1.2*D + 1.6*LconcSpan2
	ULS05: 1.2*D + 1.6*LconcSpan3
Structural model	3D elastic computer model (see fig. 36) analyzed using XC

9.4.2 Acceptance criteria

Figure 35 shows the design thickness of the ramp slab and the reinforcing layout.

The slab is checked for the load combinations summarized in section 9.4.1. The limit state checking was performed in general compliance with ACI 318, using the program XC. The representative plots for the results obtained are shown in figs. 42 to 44 for the normal stresses check, and figs. 45 to 48 for the shear check. In every case, all the elements have a demand to capacity ratio of 1.0 or less.

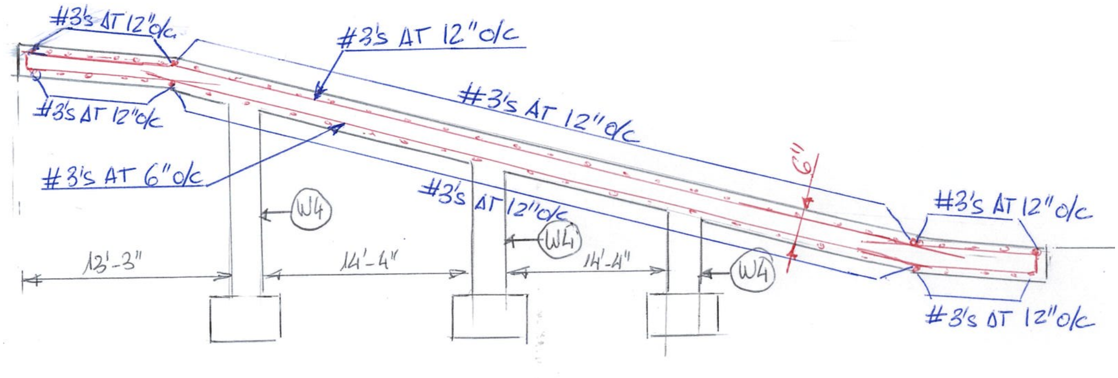


Figure 35: Ramp. Reinforcing layout.

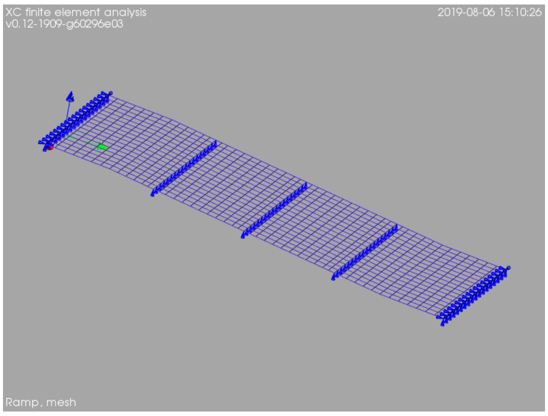


Figure 36: Ramp elastic model, mesh.

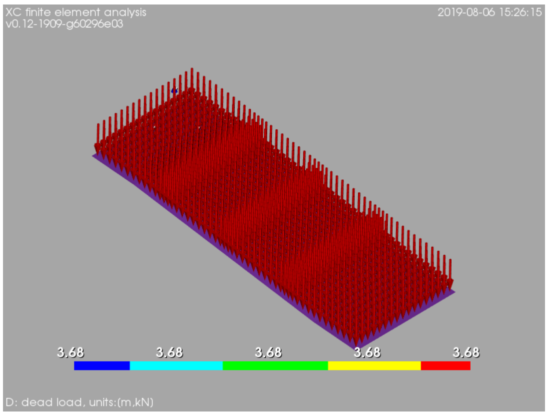


Figure 37: Load case D: dead load (include slab self-weight) [units: kN/m^2].

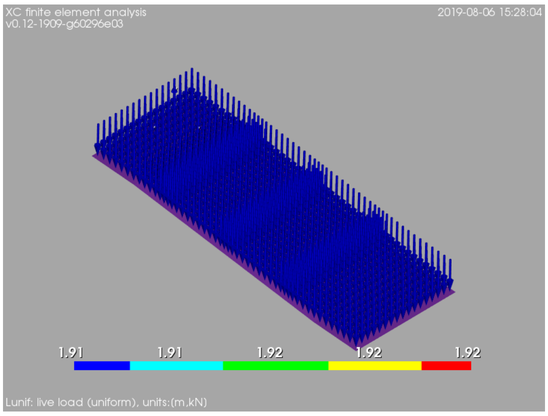


Figure 38: Load case Lv_{unif}: uniform live load (vehicles) [units: kN/m^2].

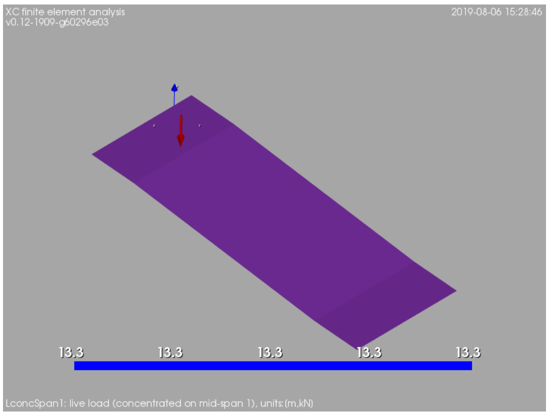


Figure 39: Load case $\mathrm{Lv}_{conc,s1}$: concentrated live load (vehicles) on mid-span 1 [units: kN].

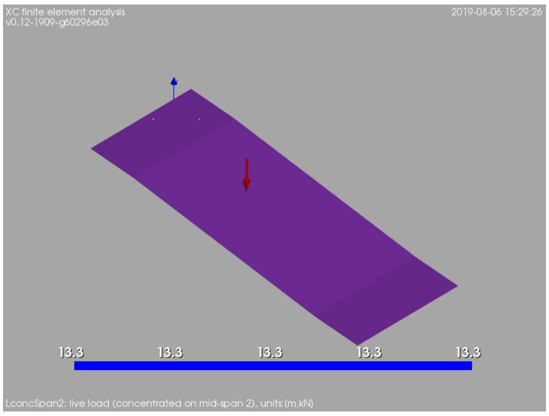


Figure 40: Load case $\mathrm{Lv}_{conc,s2}$: concentrated live load (vehicles) on mid-span 2 [units: kN].

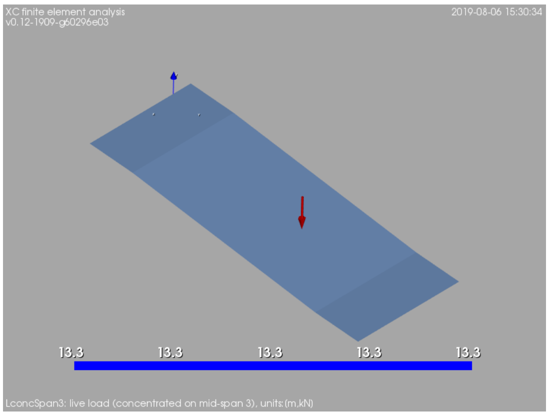


Figure 41: Load case $\mathrm{Lv}_{conc,s3}$: concentrated live load (vehicles) on mid-span 3 [units: kN].

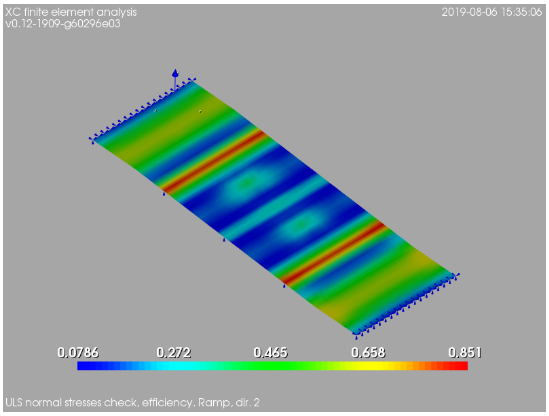


Figure 42: ULS normal stresses check. Efficiency in longitudinal direction

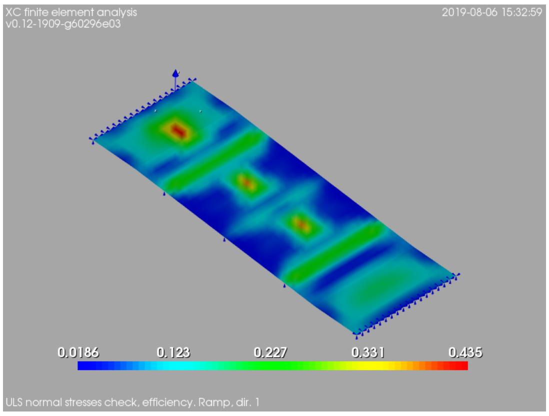


Figure 43: ULS normal stresses check. Efficiency in transversal direction

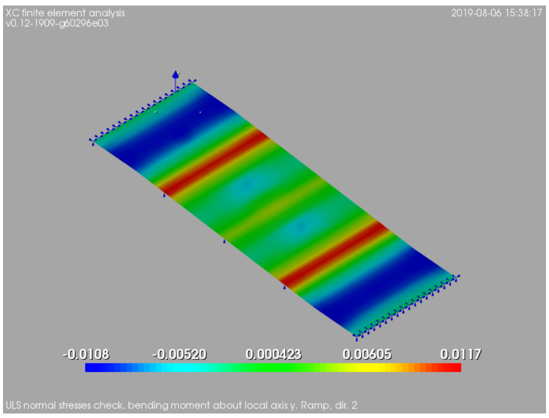


Figure 44: ULS normal stresses check. Bending moment in longitudinal direction [units: kN.m].

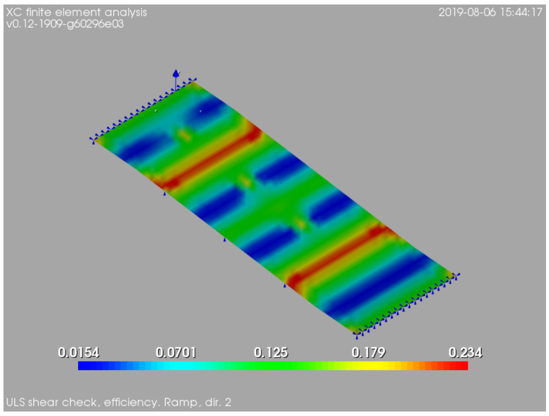


Figure 45: ULS shear check. Efficiency in longitudinal direction

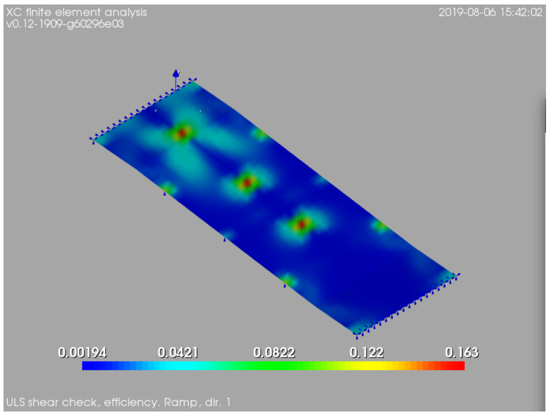


Figure 46: ULS shear check. Efficiency in transversal direction

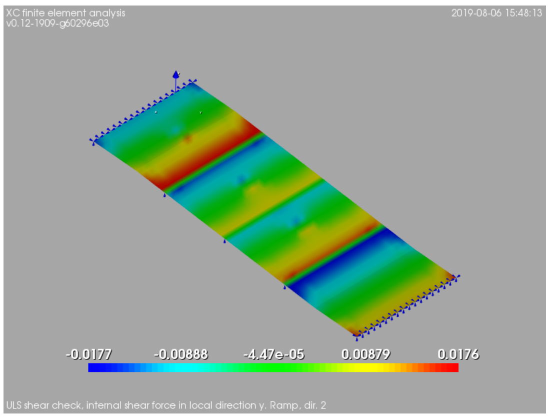


Figure 47: ULS shear check. Internal shear force in longitudinal direction [units: kN].

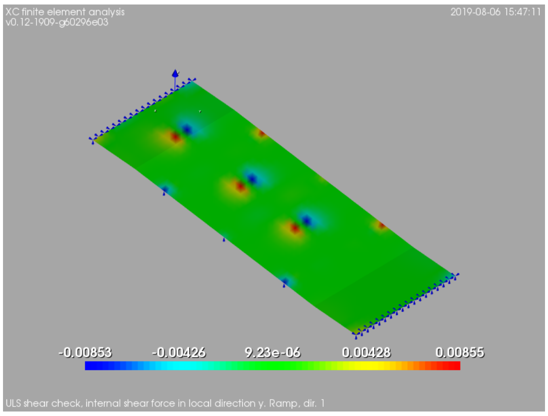


Figure 48: ULS shear check. Internal shear force in transversal direction [units: kN].

Appendices

A Loading criteria

A.1 Dead loads

Materials	
Wood structural panel	36.0 pcf = 5655 $\frac{\text{newton}}{\text{meter}^3}$
Concrete reinforced stone (including gravel)	150.0 pcf = 23563 $\frac{\text{newton}}{\text{meter}^3}$
Steel	489.0 pcf = 76816 $\frac{\text{newton}}{\text{meter}^3}$
Gypsum crete	115.0 pcf = 18065 $\frac{\text{newton}}{\text{meter}^3}$
Gypsum,loose	70.0 pcf = 10996 $\frac{\text{newton}}{\text{meter}^3}$
Earth (not submerged) sand and gravel (wet)	120.0 pcf = 18850 $\frac{\text{newton}}{\text{meter}^3}$
Water	62.4 pcf = 9802 $\frac{\text{newton}}{\text{meter}^3}$
Frame partitions	
Wood or steel studs, $\frac{1}{2}$in gypsum board inside	8 psf = 383 pascal
Wood studs, 2x4 unplastered	4 psf = 192 pascal
Wood studs, 2x4 plastered one side	12 psf = 575 pascal
Wood studs, 2x4 plastered two sides	20 psf = 958 pascal
Movable steel partitions	4 psf = 192 pascal
Frame walls	
Exterior stud wall 2x4 @ 16in, $\frac{5}{8}$ gypsum insulated, $\frac{3}{8}$ in siding	11 psf = 526 pascal
Exterior stud wall 2x6 @ 16in, $\frac{5}{8}$ gypsum insulated, $\frac{3}{8}$ in siding	12 psf = 575 pascal
Exterior stud wall with brick veneer	48 psf = 2298 pascal
CMU wall 8in	60 psf = 9425 pascal
Window, glass, frame and sash	8 psf = 383 pascal
Cladding	
Fiber cement panels, large format 38.4in × 102in	3.2 psf = 153 pascal
Fiber cement panels, small scale 9.6in × 102in	3.2 psf = 153 pascal
Perforated metal panel at exterior HVAC location	
Floor truss	
Single chord @ 24in o.c. spacing	3.2 psf = 153 pascal
Double chord @ 24in o.c. spacing	4.25 psf = 203 pascal
Sheating	
Roof sheating	3.5 psf = 167 pascal
Floor sheating	2.5 psf = 120 pascal
Ceilings	2.5 psf = 120 pascal
Deck composite sleeperes (3in)	9.00 psf = 431 pascal

A.2 Live loads

Occupancy or use	**Uniform**	**Concentrated**	**Notes**
Private rooms and corridors serving them in multifamily dwelling	40.0 psf = 1915 pascal	-	*IBC-2018 Table 1607.1*
Stairs and exits	100.0 psf = 4788 pascal	300 pound = 1334 newton	*IBC-2018 Table 1607.1. Concentrated load on stair treads applied on an area of 2 inches by 2 inches*
Balconies and decks	same as occupancy served	-	*IBC-2018 Table 1607.1*
Garages (passenger vehicles only)	40.0 psf = 1915 pascal	-	*IBC-2018 Table 1607.1*
Cornices	60.0 psf = 2873 pascal	-	*IBC-2018 Table 1607.1*
Elevator machine room and control room grating	-	300 pound = 1334 newton	*IBC-2018 Table 1607.1. Concentrated load applied on an area of 2 inches by 2 inches*
Flat roof (not occupiable) + maintenace	20.0 psf = 958 pascal	300 pound = 1334 newton	*IBC-2018 Table 1607.1*
Yards and terraces, pedestrians	100.0 psf = 4788 pascal	-	*IBC-2018 Table 1607.1*
Sidewalks, vehicular driveways and yards, subject to trucking	250.0 psf = 11970 pascal	8000 pound = 35586 newton	*IBC-2018 Table 1607.1*
Corridors first floor	100.0 psf = 4788 pascal	-	*IBC-2018 Table 1607.1*
Store first floor	100.0 psf = 4788 pascal	-	*IBC-2018 Table 1607.1*

A.3 Snow loads

Ground snow load	$p_g = 60.0$ psf = 2873 pascal	*ASCE 7. Figure 7.1*
Exposure factor	$C_e = 1.0$	*ASCE 7. Table 7-2. Terrain category B, roof partially exposed*
Thermal factor	$C_t = 1.0$	*ASCE 7. Table 7-3.*
Snow load importance factor	$I_s = 1.0$	*ASCE 7. Table 7-4. Structure risk category II*
Snow load flat roof	$p_f = 0.7 \times C_e \times C_t \times I_s \times p_g = 0.7 \times 1.0 \times 1.0 \times 1.0 \times 60.0 = 42.0$ psf = 2873 pascal	*ASCE 7. Sect. 7.3*

A.4 Wind loads

Alternate all-heights method.		*IBC-2018, sect. 1609.6. Regularly shaped building, less than 75 feet in height, not sensitive to dynamic effects, not channeling effects or buffeting, simple diaphragm building*
Ultimate design wind speed	$V_{ult} = 115 \frac{\text{miles}}{\text{hour}} = 51 \frac{\text{meters}}{\text{second}}$	*IBC-2018, figure 1609.3(1). Risk category II building*
Velocity pressure exposure coefficient	$K_z = 0.72$	*ASCE 7, table 27.3.1. Exposure B, height above ground level $z \approx$ 33 feet*
Topographic factor	$K_{zt} = 1.0$	*ASCE 7, sect. 26.8*

Net pressure coefficients C_{net}. Main windforce-resisting frames and systems

IBC-2018, Table 1609.6.2, enclosed

Description	C_{net} + Internal pressure	C_{net} - Internal presure
Windward wall	0.43	0.73
Leeward wall	-0.51	-0.21
Sidewall	-0.66	-0.35
Parapet windward wall	1.28	
Parapet leeward wall	-0.85	
Flat roof	-1.09	-0.79

Design wind pressures P_{net}. Main windforce-resisting frames and systems

IBC-2018, sect. 1609.6.3

$P_{net} = 0.00256 \times V^2 \times K_z \times C_{net} \times K_{zt}$

Description	P_{net} + Internal pressure	P_{net} - Internal presure
Windward wall	10.5 psf = 501 pascal	17.8 psf = 852 pascal
Leeward wall	-12.4 psf = -595 pascal	-5.1 psf = -245 pascal
Sidewall	-16.1 psf = -770 pascal	-8.5 psf = -409 pascal
Parapet windward wall	31.2 psf = 1494 pascal	
Parapet leeward wall	-20.7 psf = -992 pascal	
Flat roof	-26.6 psf = -1272 pascal	-19.3 psf = -992 pascal

A.5 Earthquake loads

Parameter 0.2-second spectral response acceleration	$S_s = 0.045$	*IBC-2018, figure 1613.3.1(1). Site class B*
Parameter 1-second spectral response acceleration	$S_1 = 0.038$	*IBC-2018, figure 1613.3.1(2). Site class B*
Seismic design category	$S_1 \leq 0.04$ *and* $S_s \leq 0.15 \rightarrow$ SDS A	*IBC-2018, sect. 1613.3.1*
Site coefficients	$F_a = 1.0$, $F_v = 1.0$	*IBC-2018, tables 1613.3.3(1) and 1613.3.3(2). Site class B*
Maximum considered earthquake spectral response acceleration for short periods	$S_{MS} = F_a \cdot S_s = 0.045$	*IBC-2018, sect. 163.3.3*
	$S_{M1} = F_a \cdot S_1 = 0.038$	*IBC-2018, sect. 163.3.3*
Design spectral response acceleration parameters	$S_{DS} = \frac{2}{3} S_{MS} = 0.03$	*IBC-2018, sect. 163.3.4*
	$S_{D1} = \frac{2}{3} S_{M1} = 0.025$	*IBC-2018, sect. 163.3.4*

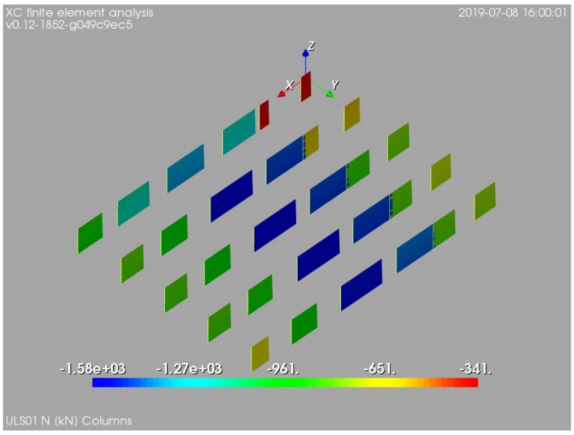


Figure 49: ULS01: 1.4*D. Columns, internal axial force [kN]

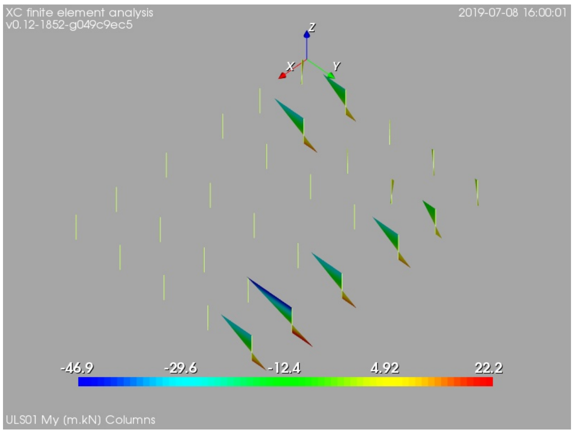


Figure 50: ULS01: 1.4*D. Columns, bending moment about local axis y [m.kN]

B Calculation results. Internal forces on columns

B.1 Ultimate limit states

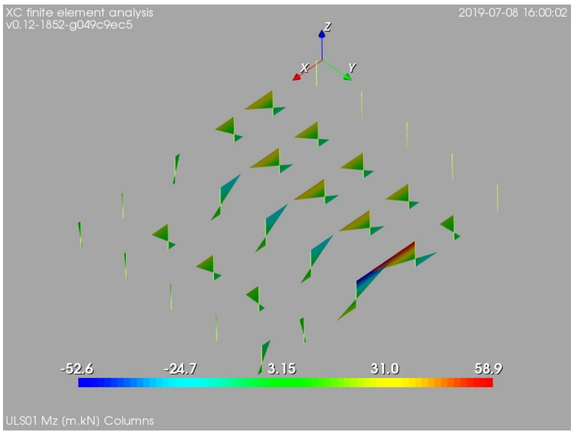


Figure 51: ULS01: 1.4*D. Columns, bending moment about local axis z [m.kN]

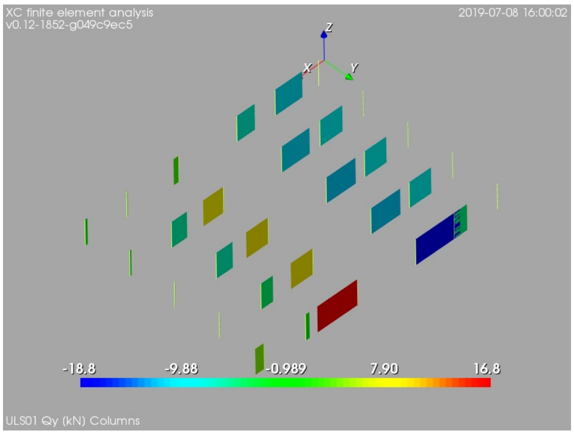


Figure 52: ULS01: 1.4*D. Columns, internal shear force in local direction y [kN]

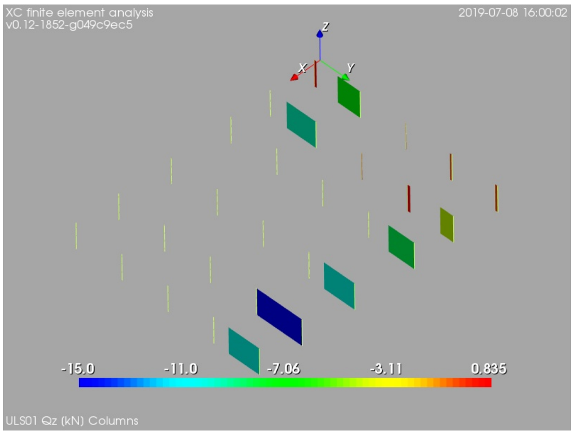


Figure 53: ULS01: 1.4*D. Columns, internal shear force in local direction z [kN]

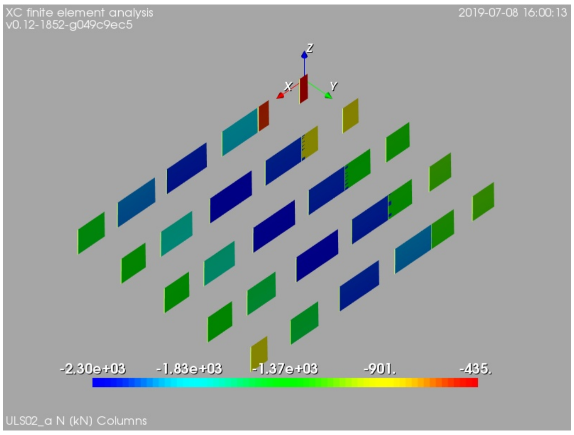


Figure 54: ULS02_a: 1.2*D + 1.6*Lru + Lpu + 0.5*S. Columns, internal axial force [kN]

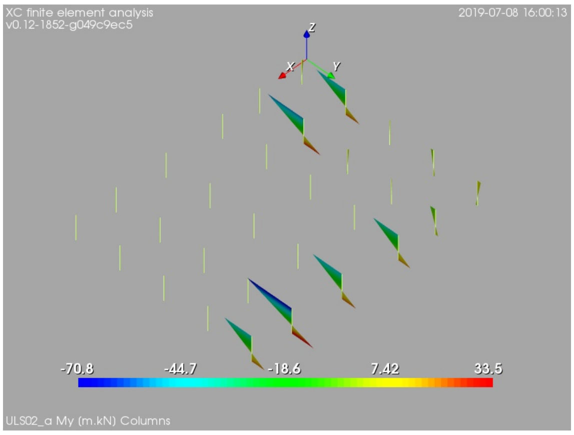


Figure 55: ULS02_a: 1.2*D + 1.6*Lru + Lpu + 0.5*S. Columns, bending moment about local axis y [m.kN]

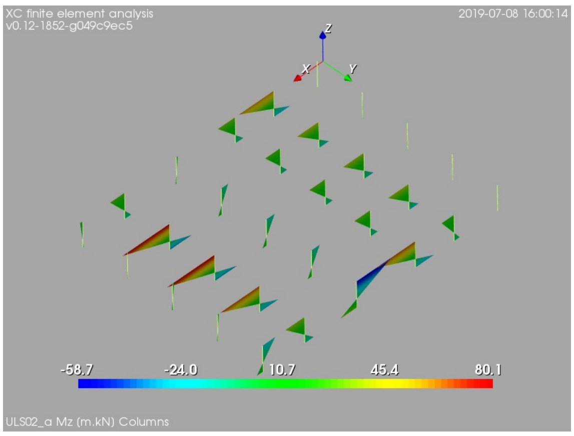


Figure 56: ULS02_a: 1.2*D + 1.6*Lru + Lpu + 0.5*S. Columns, bending moment about local axis z [m.kN]

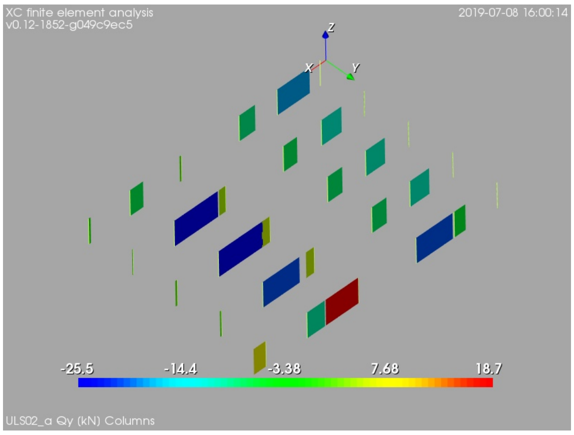


Figure 57: ULS02_a: 1.2*D + 1.6*Lru + Lpu + 0.5*S. Columns, internal shear force in local direction y [kN]

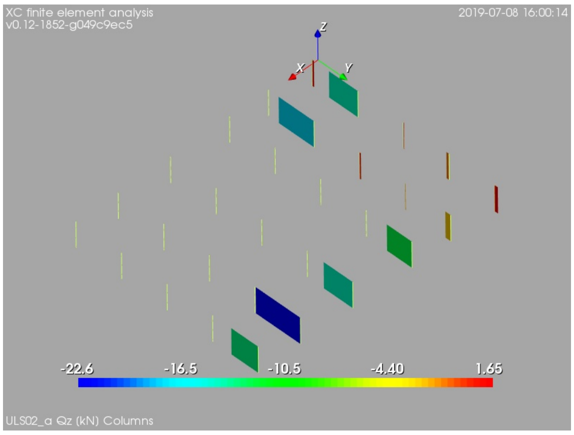


Figure 58: ULS02_a: 1.2*D + 1.6*Lru + Lpu + 0.5*S. Columns, internal shear force in local direction z [kN]

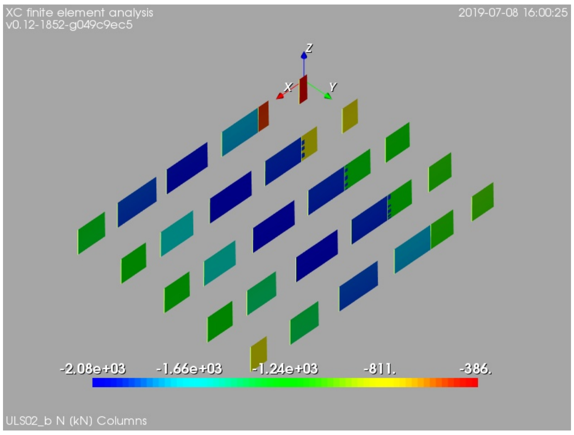


Figure 59: ULS02_b: 1.2*D + 1.6*Lrs + Lps + 0.5*S. Columns, internal axial force [kN]

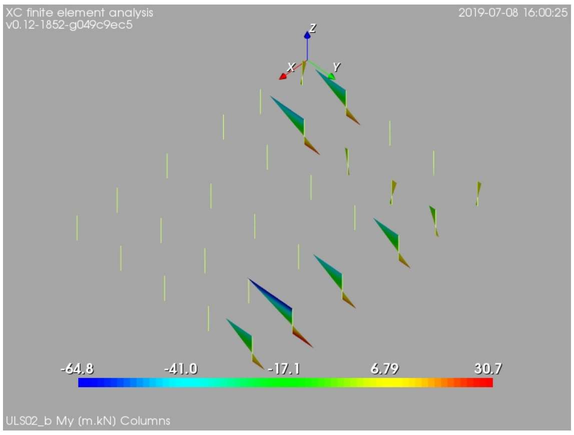


Figure 60: ULS02_b: 1.2*D + 1.6*Lrs + Lps + 0.5*S. Columns, bending moment about local axis y [m.kN]

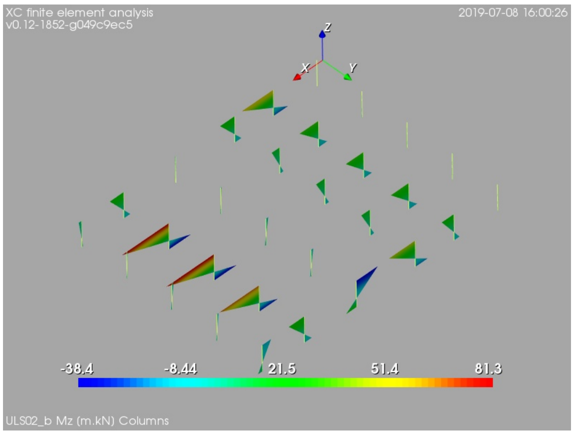


Figure 61: ULS02_b: 1.2*D + 1.6*Lrs + Lps + 0.5*S. Columns, bending moment about local axis z [m.kN]

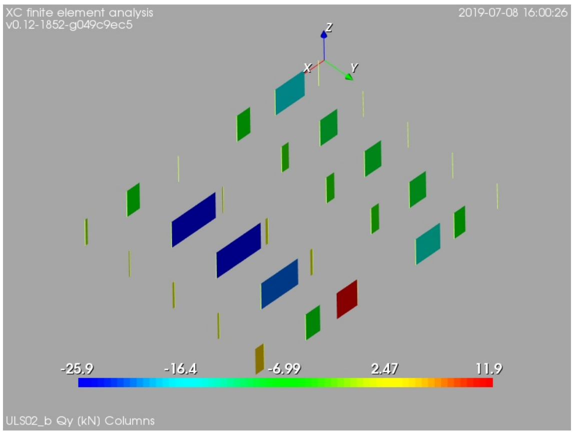


Figure 62: ULS02_b: 1.2*D + 1.6*Lrs + Lps + 0.5*S. Columns, internal shear force in local direction y [kN]

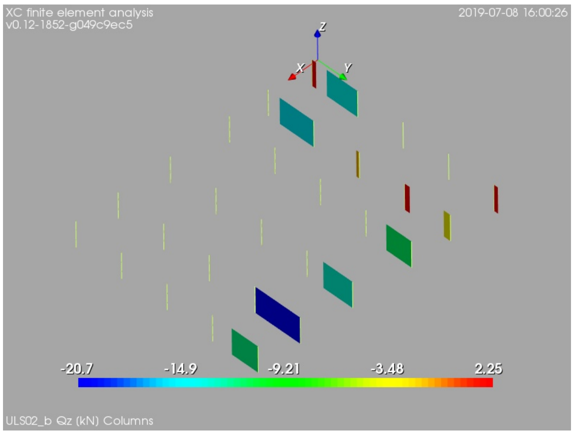


Figure 63: ULS02_b: 1.2*D + 1.6*Lrs + Lps + 0.5*S. Columns, internal shear force in local direction z [kN]

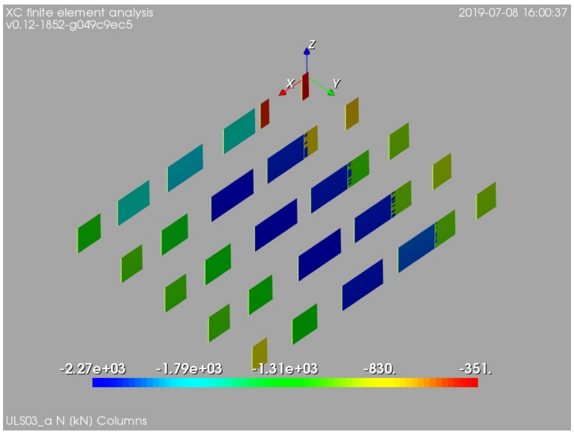


Figure 64: ULS03_a: 1.2*D + 1.6*S + 0.5*Lru + Lpu. Columns, internal axial force [kN]

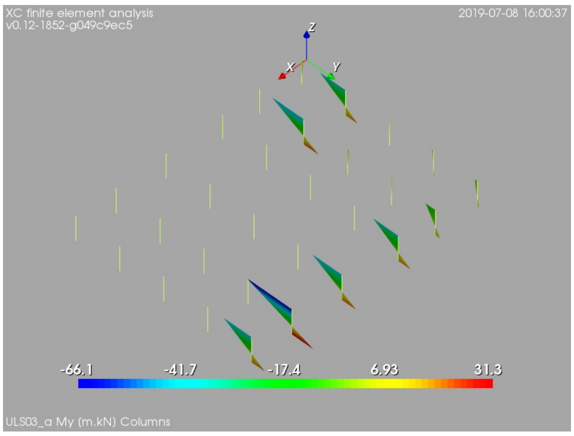


Figure 65: ULS03_a: 1.2*D + 1.6*S + 0.5*Lru + Lpu. Columns, bending moment about local axis y [m.kN]

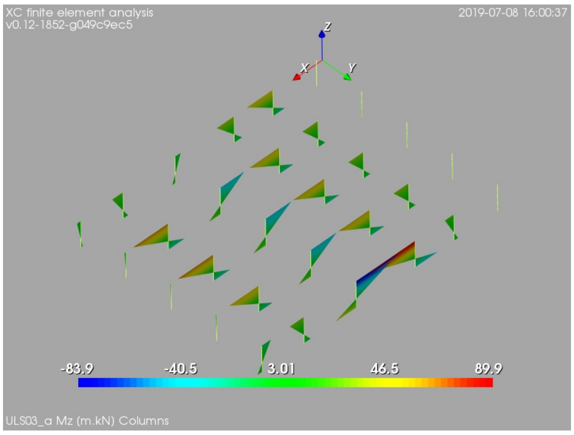


Figure 66: ULS03_a: 1.2*D + 1.6*S + 0.5*Lru + Lpu. Columns, bending moment about local axis z [m.kN]

Figure 67: ULS03_a: 1.2*D + 1.6*S + 0.5*Lru + Lpu. Columns, internal shear force in local direction y [kN]

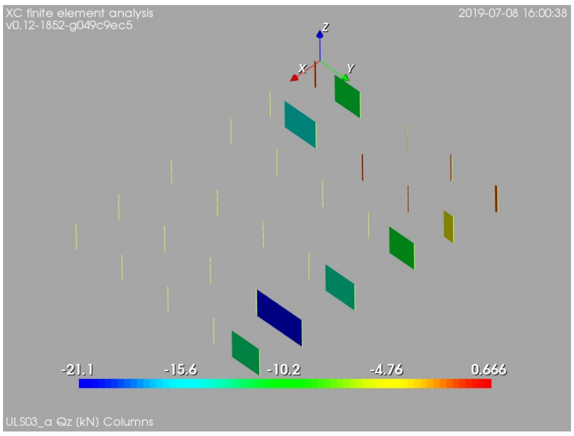


Figure 68: ULS03_a: 1.2*D + 1.6*S + 0.5*Lru + Lpu. Columns, internal shear force in local direction z [kN]

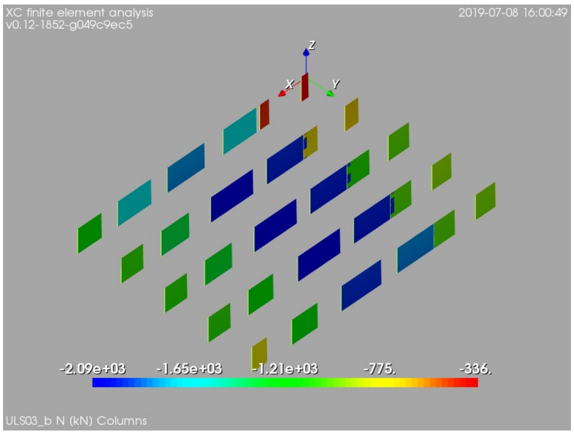


Figure 69: ULS03_b: 1.2*D + 1.6*S + 0.5*Lrs + Lps. Columns, internal axial force [kN]

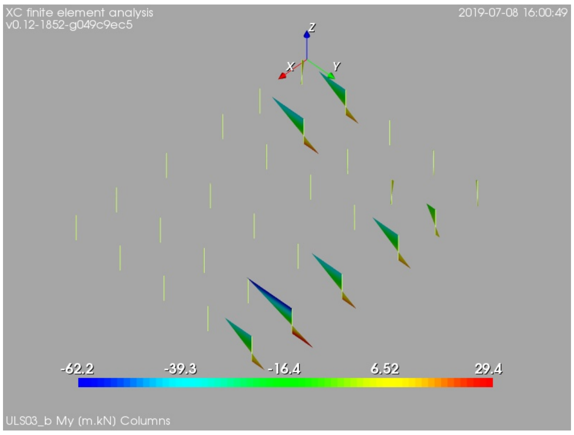


Figure 70: ULS03_b: 1.2*D + 1.6*S + 0.5*Lrs + Lps. Columns, bending moment about local axis y [m.kN]

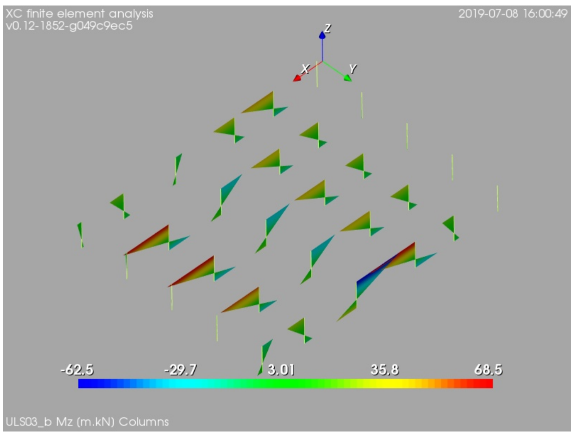


Figure 71: ULS03_b: 1.2*D + 1.6*S + 0.5*Lrs + Lps. Columns, bending moment about local axis z [m.kN]

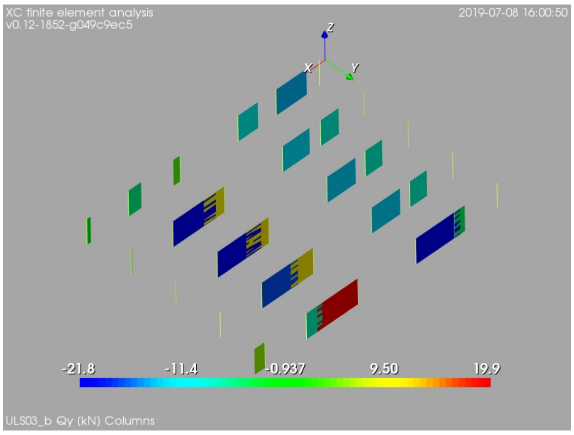


Figure 72: ULS03_b: 1.2*D + 1.6*S + 0.5*Lrs + Lps. Columns, internal shear force in local direction y [kN]

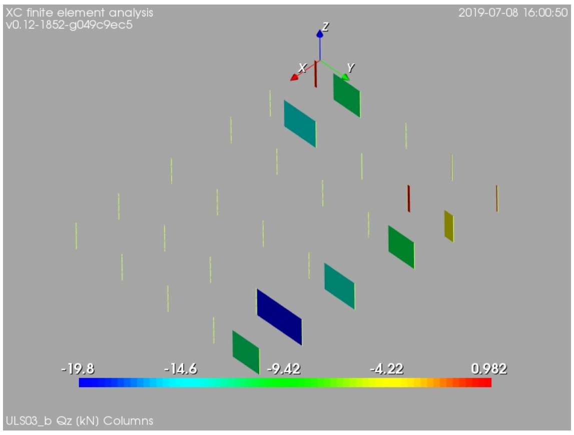


Figure 73: ULS03_b: 1.2*D + 1.6*S + 0.5*Lrs + Lps. Columns, internal shear force in local direction z [kN]

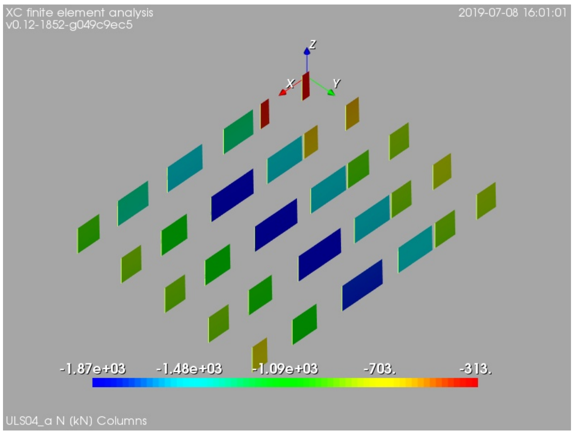


Figure 74: ULS04_a: 1.2*D + 1.6*S + 0.5*W_WE. Columns, internal axial force [kN]

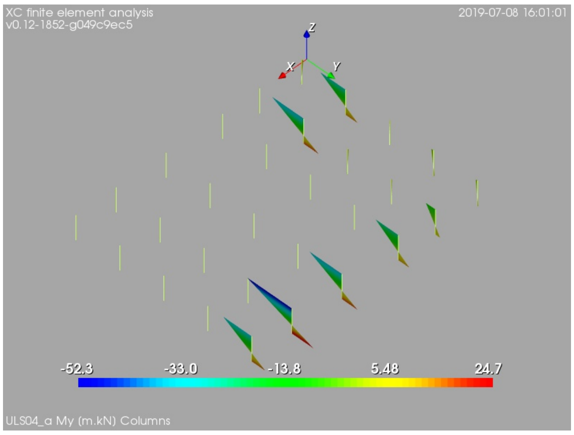


Figure 75: ULS04_a: 1.2*D + 1.6*S + 0.5*W_WE. Columns, bending moment about local axis y [m.kN]

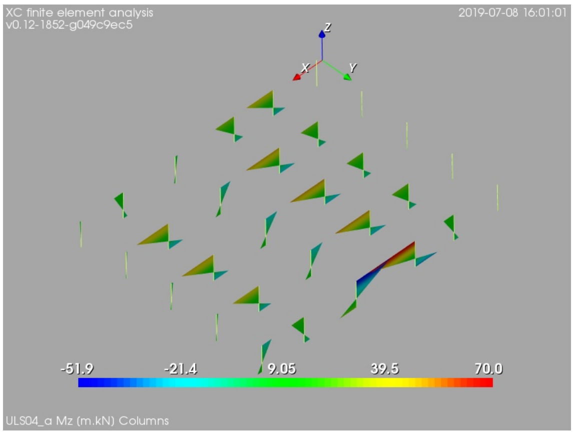


Figure 76: ULS04_a: 1.2*D + 1.6*S + 0.5*W_WE. Columns, bending moment about local axis z [m.kN]

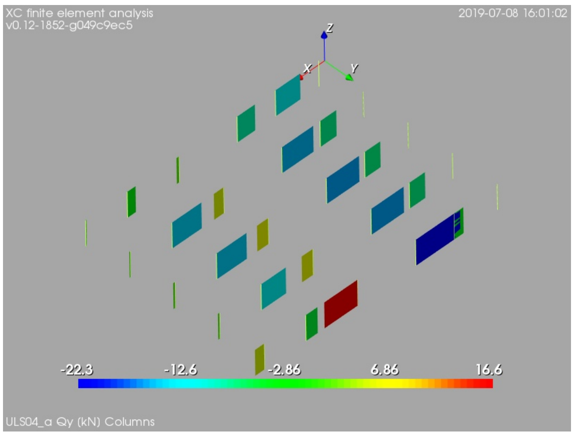


Figure 77: ULS04_a: 1.2*D + 1.6*S + 0.5*W_WE. Columns, internal shear force in local direction y [kN]

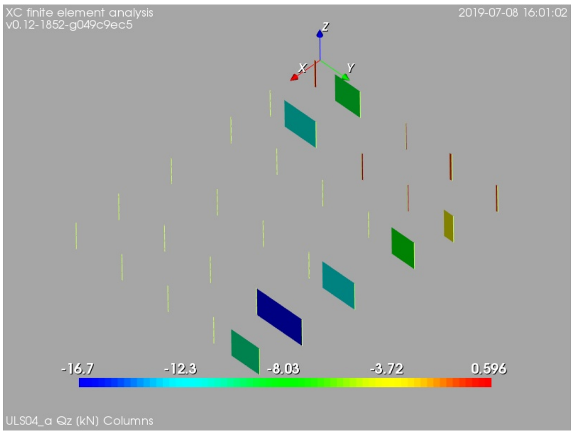


Figure 78: ULS04_a: 1.2*D + 1.6*S + 0.5*W_WE. Columns, internal shear force in local direction z [kN]

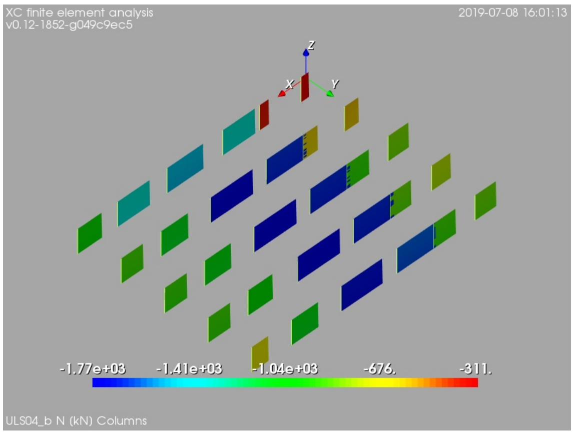


Figure 79: ULS04_b: 1.2*D + 1.6*S + 0.5*W_NS. Columns, internal axial force [kN]

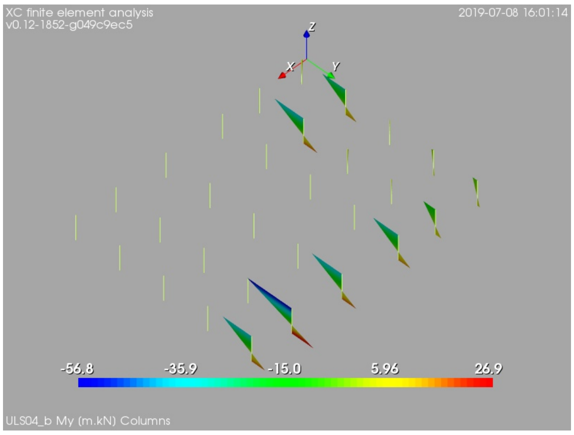


Figure 80: ULS04_b: 1.2*D + 1.6*S + 0.5*W_NS. Columns, bending moment about local axis y [m.kN]

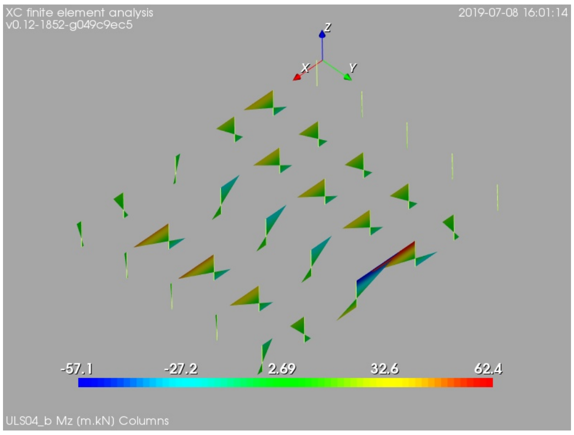


Figure 81: ULS04_b: 1.2*D + 1.6*S + 0.5*W_NS. Columns, bending moment about local axis z [m.kN]

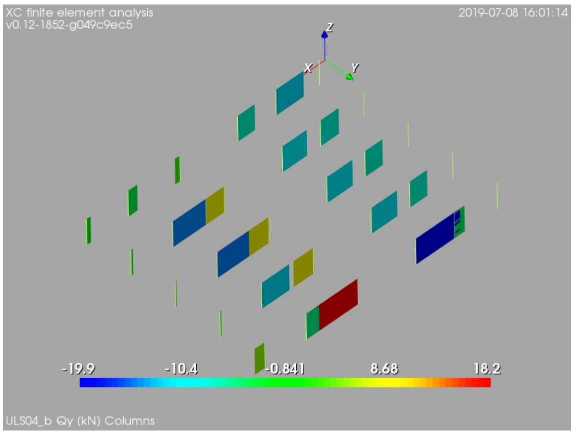


Figure 82: ULS04_b: 1.2*D + 1.6*S + 0.5*W_NS. Columns, internal shear force in local direction y [kN]

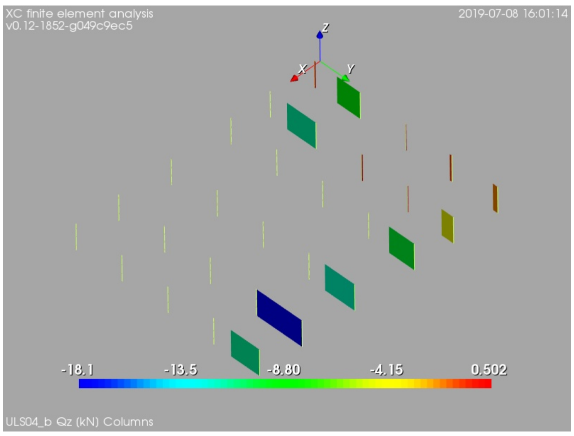


Figure 83: ULS04_b: 1.2*D + 1.6*S + 0.5*W_NS. Columns, internal shear force in local direction z [kN]

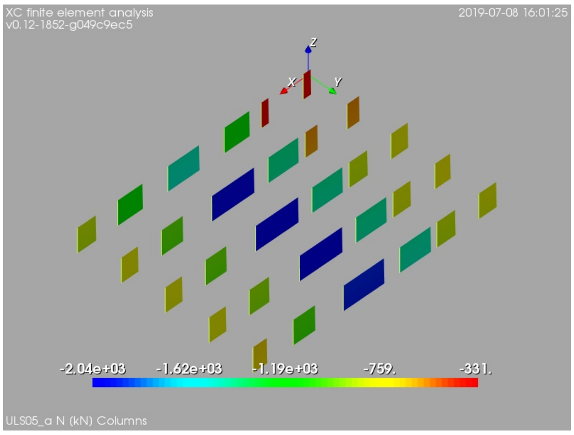


Figure 84: ULS05_a: 1.2*D + W_WE + 0.5*Lru + Lpu. Columns, internal axial force [kN]

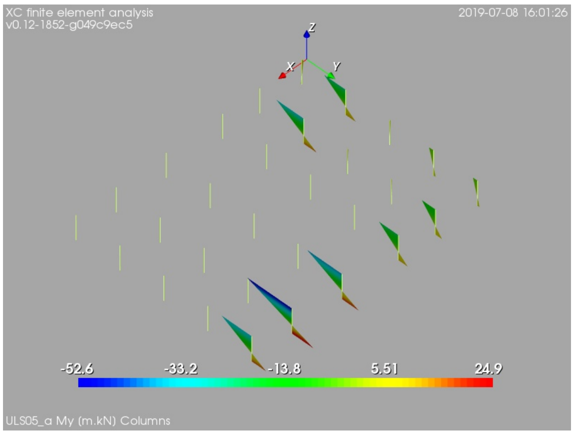


Figure 85: ULS05_a: 1.2*D + W_WE + 0.5*Lru + Lpu. Columns, bending moment about local axis y [m.kN]

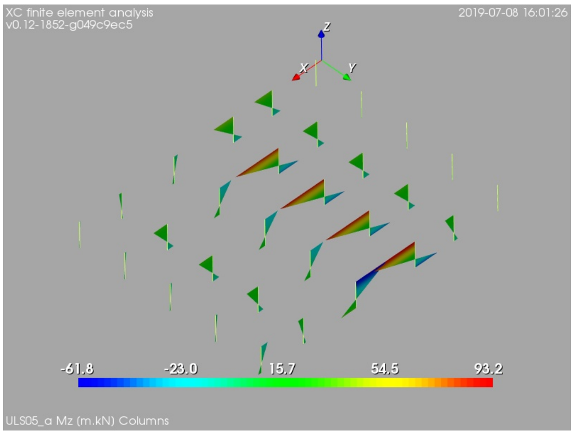


Figure 86: ULS05_a: 1.2*D + W_WE + 0.5*Lru + Lpu. Columns, bending moment about local axis z [m.kN]

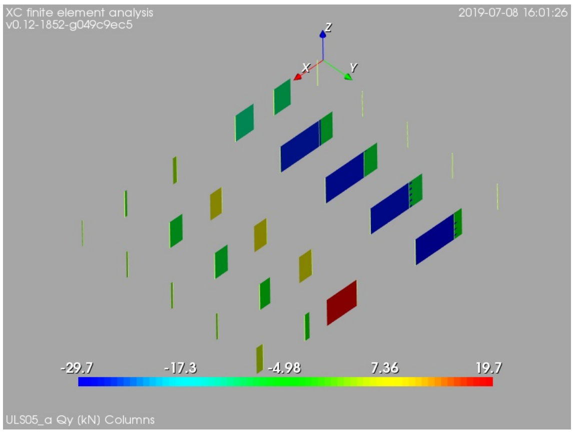


Figure 87: ULS05_a: 1.2*D + W_WE + 0.5*Lru + Lpu. Columns, internal shear force in local direction y [kN]

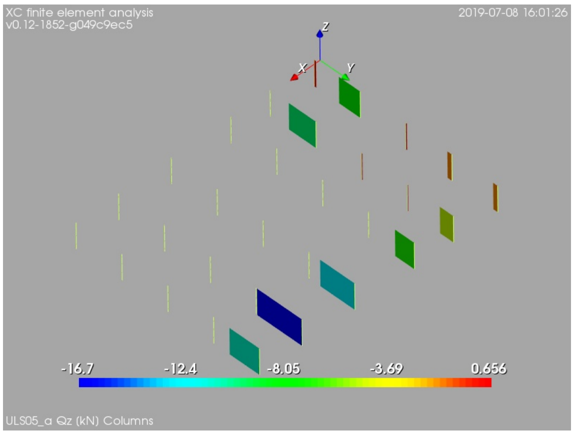


Figure 88: ULS05_a: 1.2*D + W_WE + 0.5*Lru + Lpu. Columns, internal shear force in local direction z [kN]

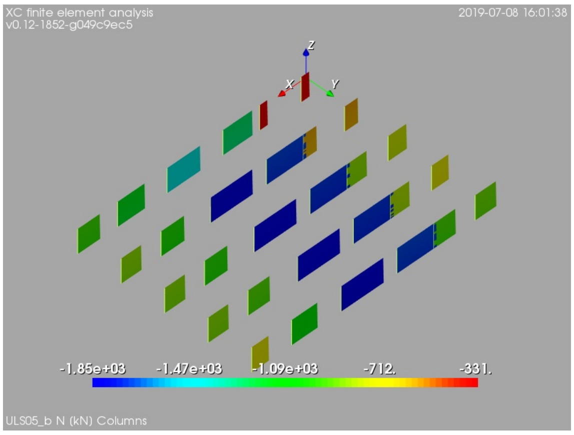


Figure 89: ULS05_b: 1.2*D + W_NS + 0.5*Lru + Lpu. Columns, internal axial force [kN]

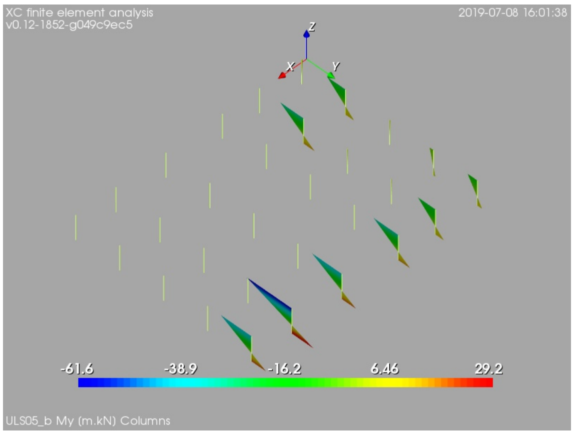


Figure 90: ULS05_b: 1.2*D + W_NS + 0.5*Lru + Lpu. Columns, bending moment about local axis y [m.kN]

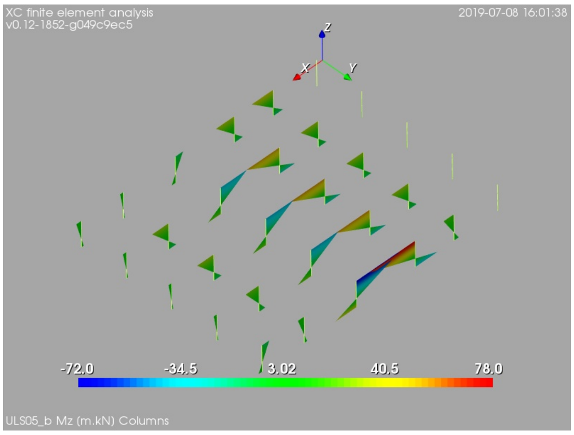


Figure 91: ULS05_b: 1.2*D + W_NS + 0.5*Lru + Lpu. Columns, bending moment about local axis z [m.kN]

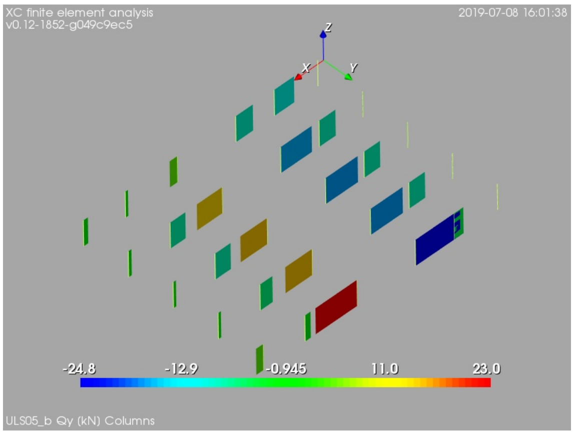


Figure 92: ULS05_b: 1.2*D + W_NS + 0.5*Lru + Lpu. Columns, internal shear force in local direction y [kN]

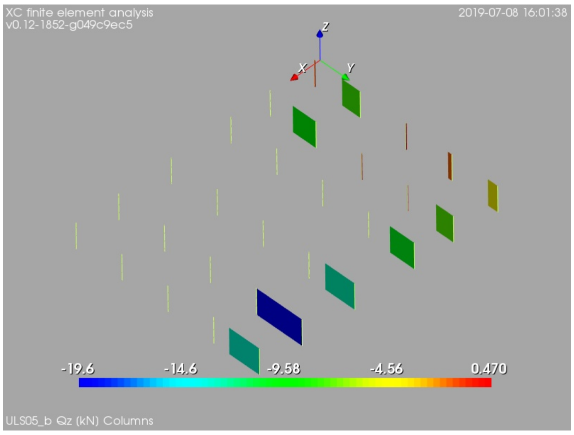


Figure 93: ULS05_b: 1.2*D + W_NS + 0.5*Lru + Lpu. Columns, internal shear force in local direction z [kN]

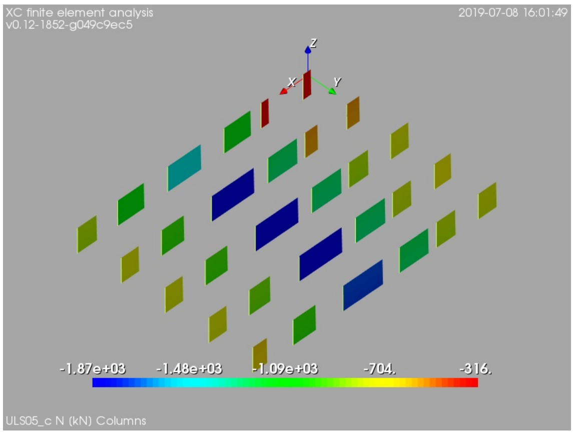


Figure 94: ULS05_c: 1.2*D + W_WE + 0.5*Lrs + Lps. Columns, internal axial force [kN]

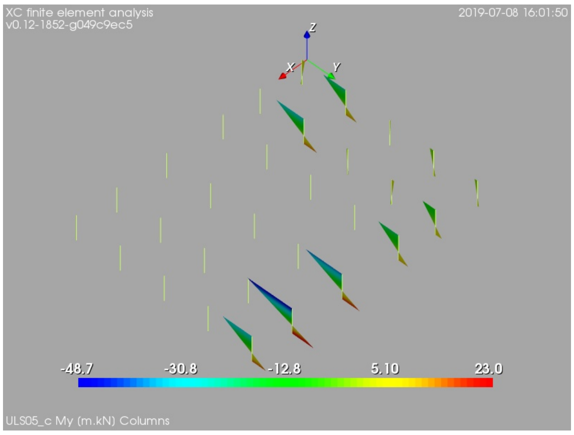


Figure 95: ULS05_c: 1.2*D + W_WE + 0.5*Lrs + Lps. Columns, bending moment about local axis y [m.kN]

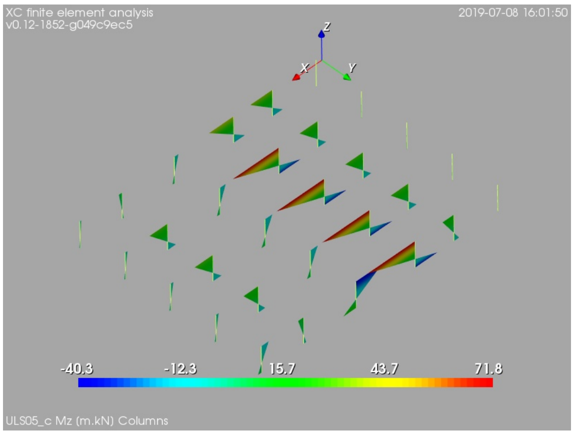


Figure 96: ULS05_c: 1.2*D + W_WE + 0.5*Lrs + Lps. Columns, bending moment about local axis z [m.kN]

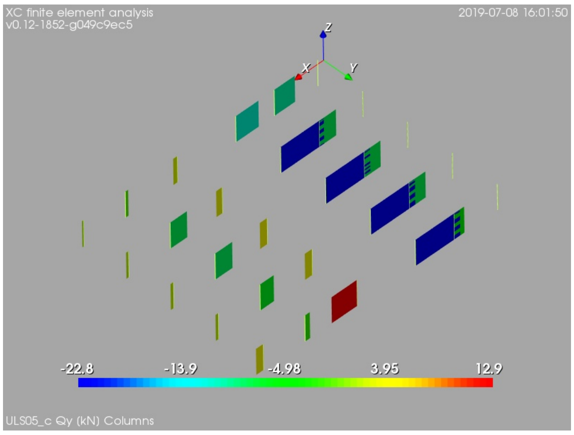


Figure 97: ULS05_c: 1.2*D + W_WE + 0.5*Lrs + Lps. Columns, internal shear force in local direction y [kN]

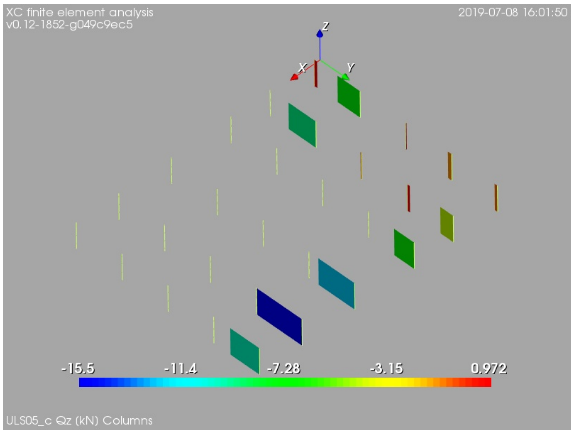


Figure 98: ULS05_c: 1.2*D + W_WE + 0.5*Lrs + Lps. Columns, internal shear force in local direction z [kN]

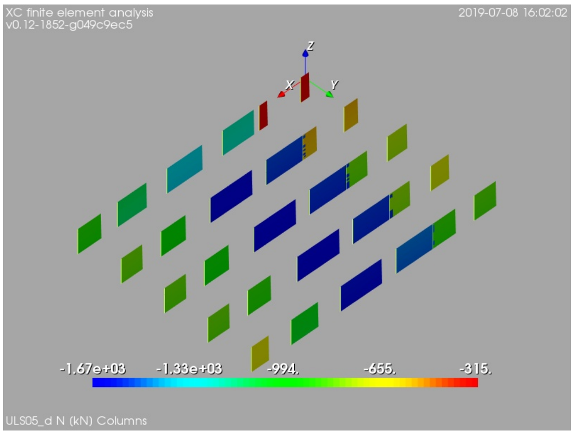


Figure 99: ULS05_d: 1.2*D + W_NS + 0.5*Lrs + Lps. Columns, internal axial force [kN]

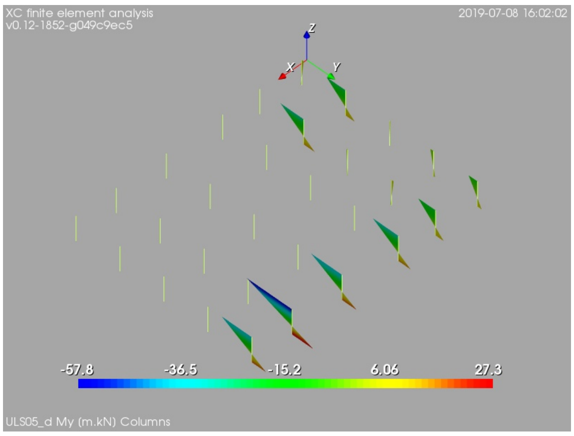


Figure 100: ULS05_d: 1.2*D + W_NS + 0.5*Lrs + Lps. Columns, bending moment about local axis y [m.kN]

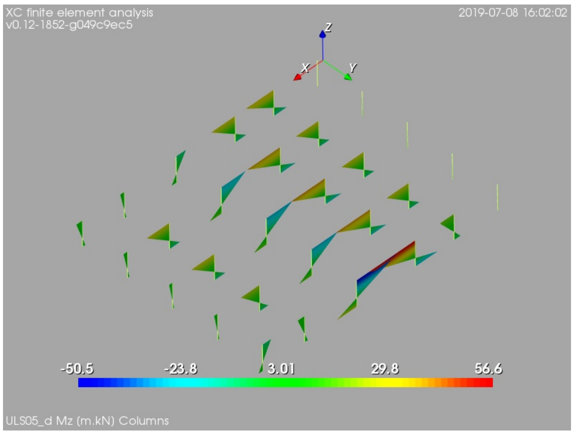


Figure 101: ULS05_d: 1.2*D + W_NS + 0.5*Lrs + Lps. Columns, bending moment about local axis z [m.kN]

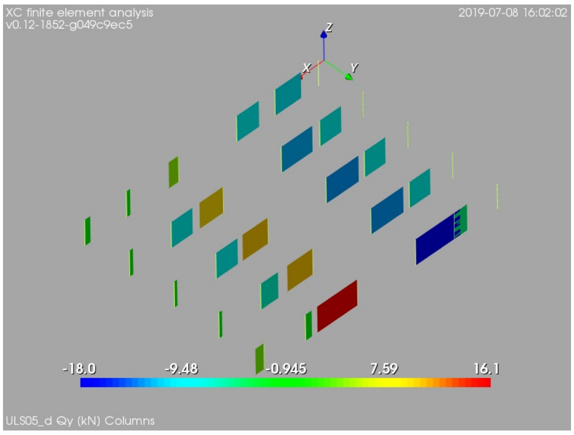


Figure 102: ULS05_d: 1.2*D + W_NS + 0.5*Lrs + Lps. Columns, internal shear force in local direction y [kN]

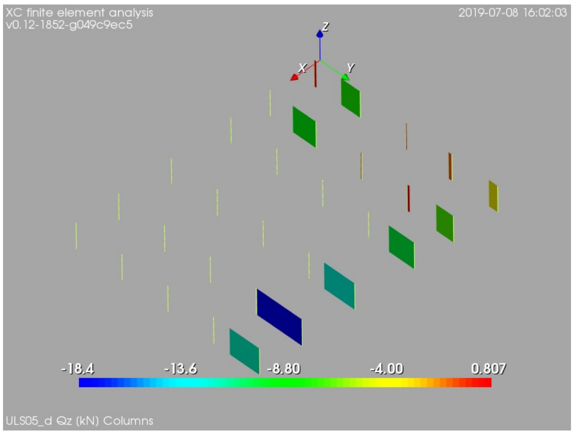


Figure 103: ULS05_d: 1.2*D + W_NS + 0.5*Lrs + Lps. Columns, internal shear force in local direction z [kN]

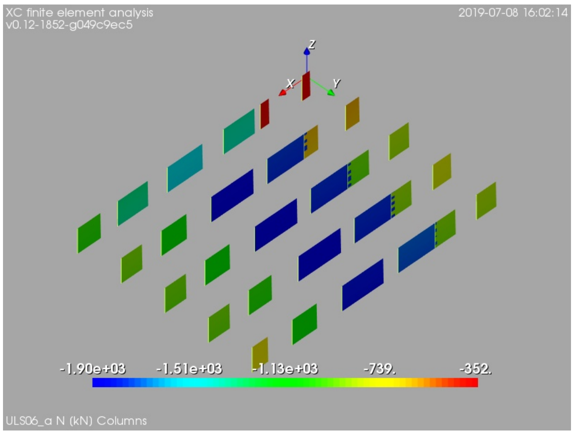


Figure 104: ULS06_a: 1.2*D + 0.5*Lru + Lpu + 0.2*S. Columns, internal axial force [kN]

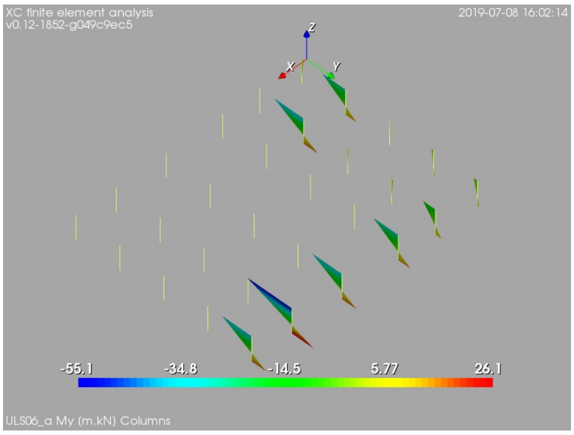


Figure 105: ULS06_a: 1.2*D + 0.5*Lru + Lpu + 0.2*S. Columns, bending moment about local axis y [m.kN]

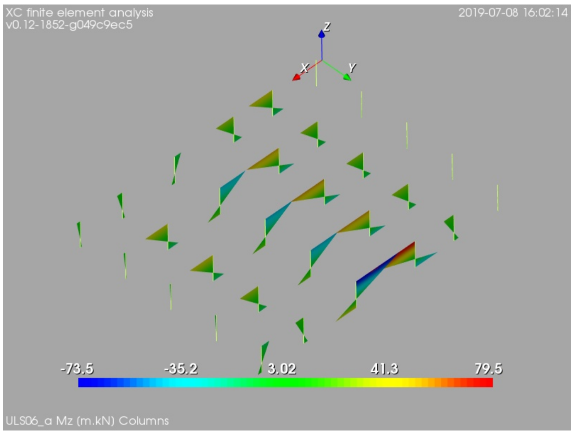


Figure 106: ULS06_a: 1.2*D + 0.5*Lru + Lpu + 0.2*S. Columns, bending moment about local axis z [m.kN]

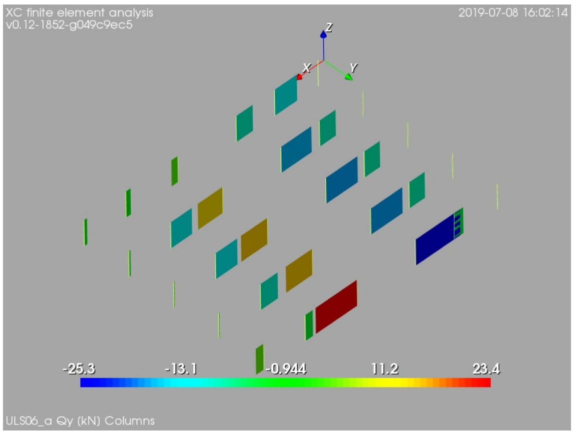


Figure 107: ULS06_a: 1.2*D + 0.5*Lru + Lpu + 0.2*S. Columns, internal shear force in local direction y [kN]

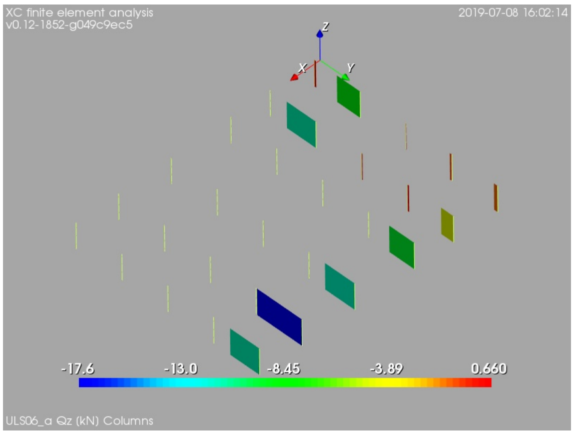


Figure 108: ULS06_a: 1.2*D + 0.5*Lru + Lpu + 0.2*S. Columns, internal shear force in local direction z [kN]

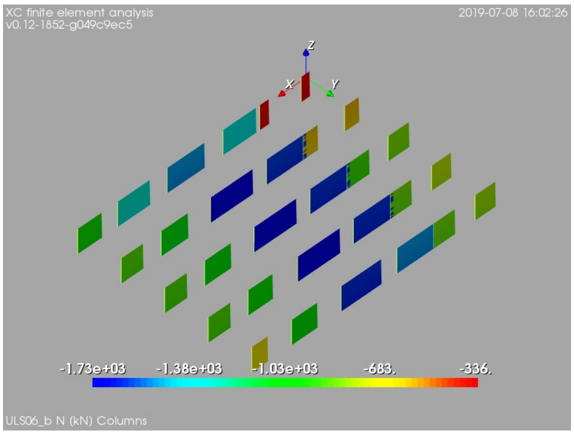


Figure 109: ULS06_b: 1.2*D + 0.5*Lrs + Lps + 0.2*S. Columns, internal axial force [kN]

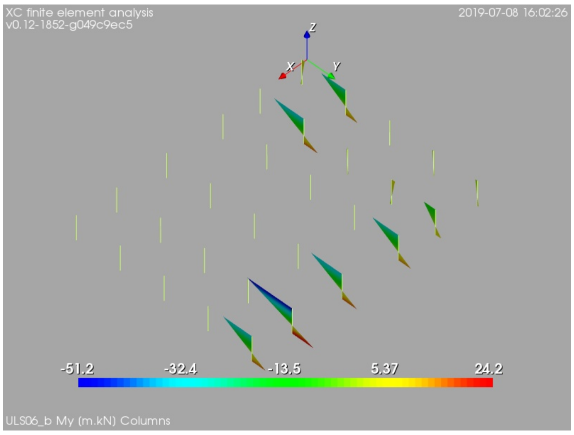


Figure 110: ULS06_b: 1.2*D + 0.5*Lrs + Lps + 0.2*S. Columns, bending moment about local axis y [m.kN]

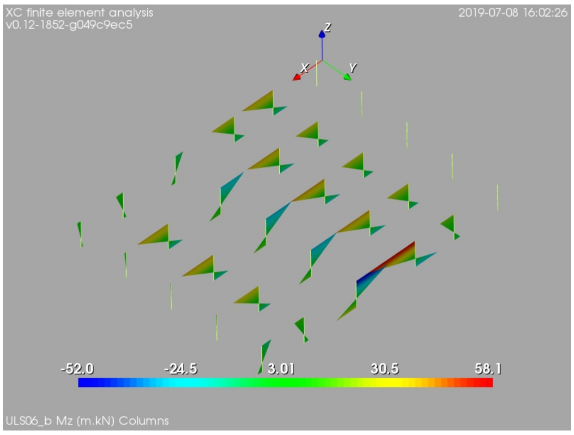


Figure 111: ULS06_b: 1.2*D + 0.5*Lrs + Lps + 0.2*S. Columns, bending moment about local axis z [m.kN]

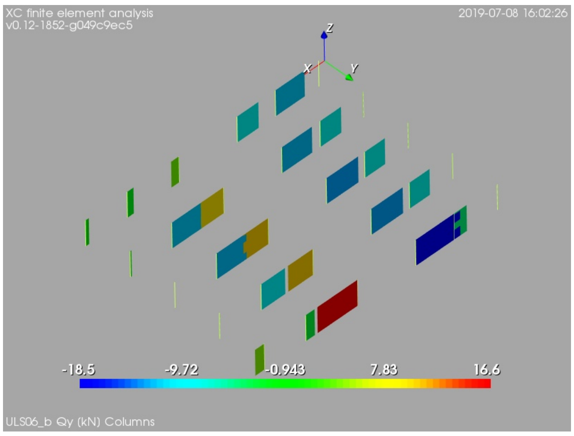


Figure 112: ULS06_b: 1.2*D + 0.5*Lrs + Lps + 0.2*S. Columns, internal shear force in local direction y [kN]

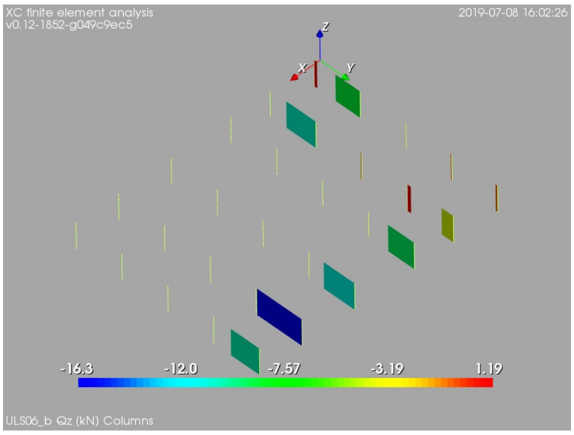


Figure 113: ULS06_b: 1.2*D + 0.5*Lrs + Lps + 0.2*S. Columns, internal shear force in local direction z [kN]

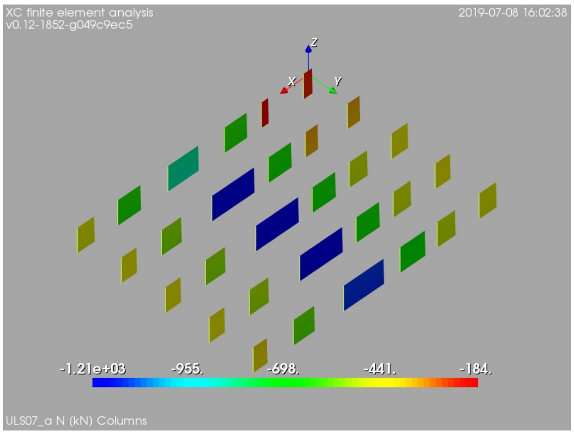


Figure 114: ULS07_a: 0.9*D + W_WE. Columns, internal axial force [kN]

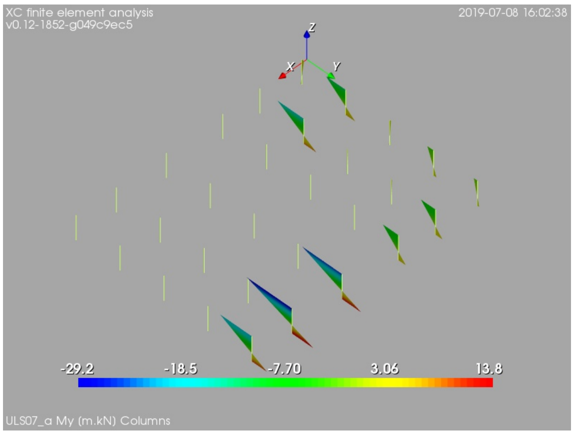


Figure 115: ULS07_a: 0.9*D + W_WE. Columns, bending moment about local axis y [m.kN]

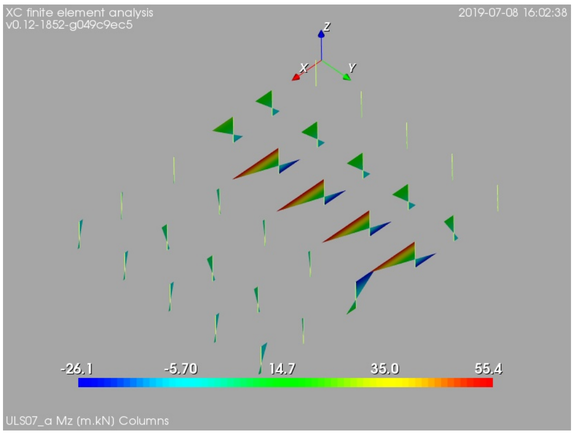


Figure 116: ULS07_a: 0.9*D + W_WE. Columns, bending moment about local axis z [m.kN]

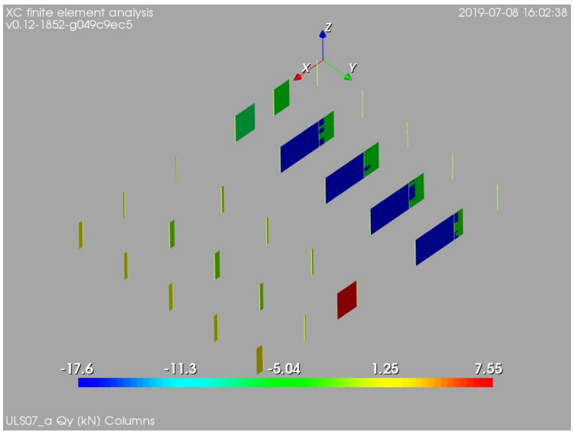


Figure 117: ULS07_a: 0.9*D + W_WE. Columns, internal shear force in local direction y [kN]

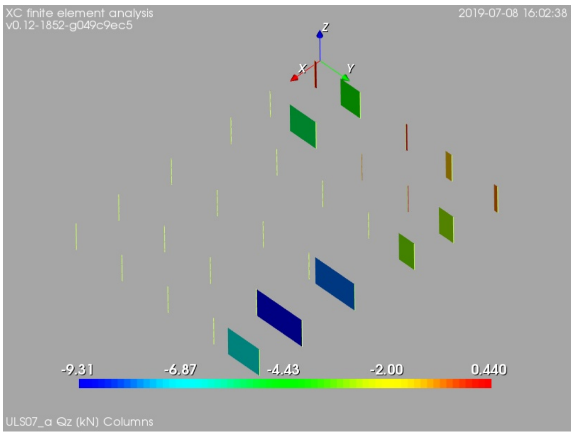


Figure 118: ULS07_a: 0.9*D + W_WE. Columns, internal shear force in local direction z [kN]

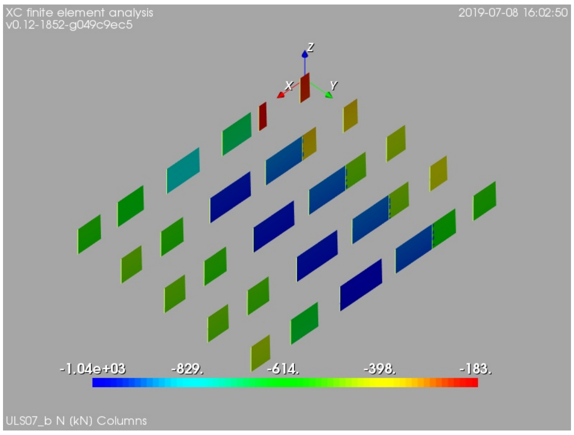


Figure 119: ULS07_b: 0.9*D + W_NS. Columns, internal axial force [kN]

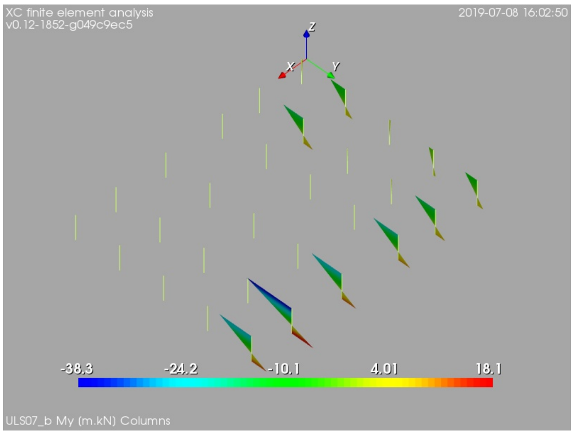


Figure 120: ULS07_b: 0.9*D + W_NS. Columns, bending moment about local axis y [m.kN]

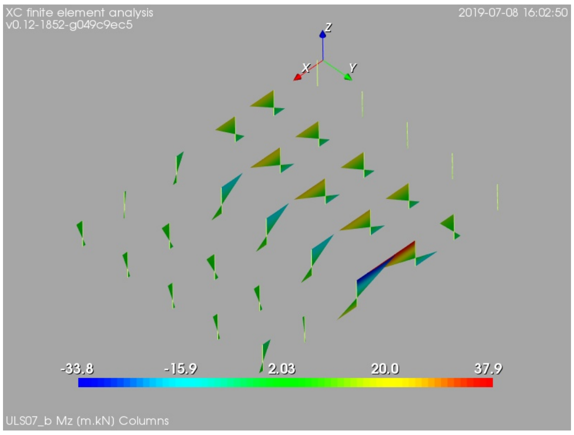


Figure 121: ULS07_b: 0.9*D + W_NS. Columns, bending moment about local axis z [m.kN]

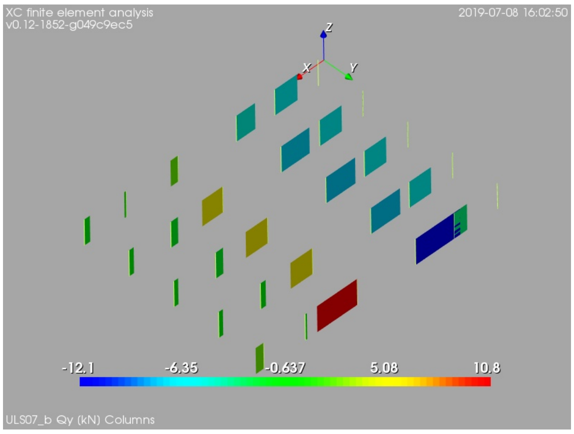


Figure 122: ULS07_b: 0.9*D + W_NS. Columns, internal shear force in local direction y [kN]

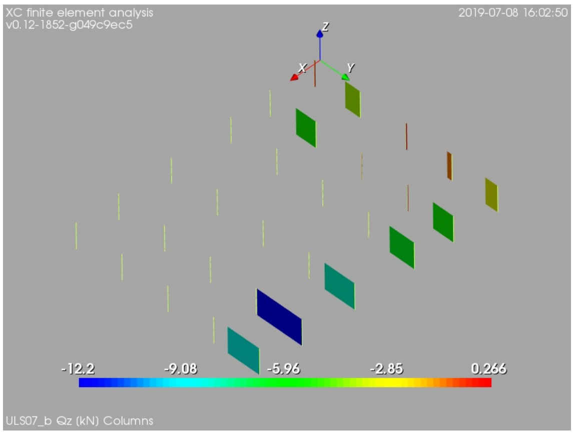


Figure 123: ULS07_b: 0.9*D + W_NS. Columns, internal shear force in local direction z [kN]

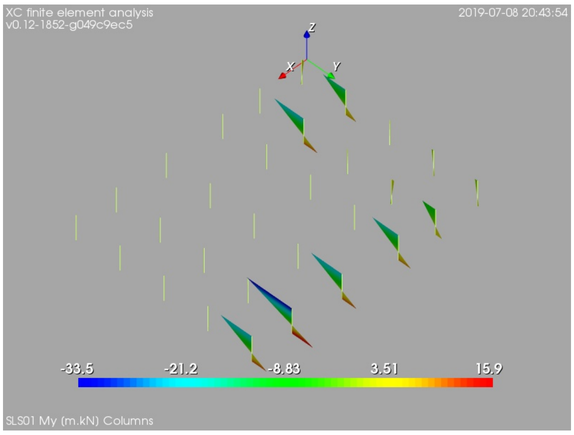


Figure 124: SLS01: 1.0*D. Columns, bending moment about local axis y [m.kN]

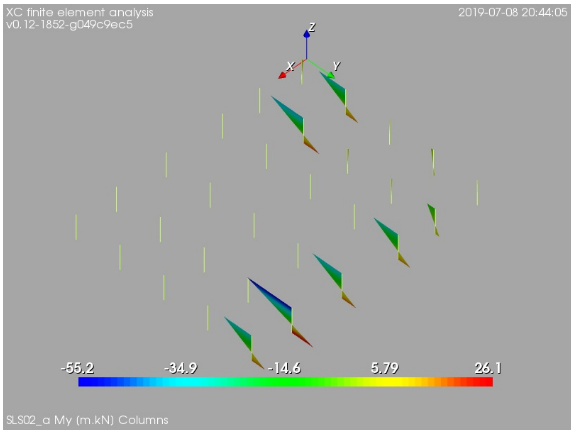


Figure 125: SLS02_a: 1.0*D + 1.0*Lru + Lpu + 0.3*S. Columns, bending moment about local axis y [m.kN]

B.2 Serviceability limit states

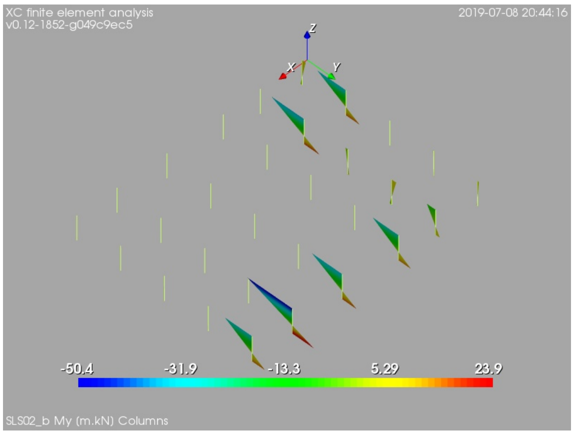


Figure 126: SLS02_b: 1.0*D + 1.0*Lrs + Lps + 0.3*S. Columns, bending moment about local axis y [m.kN]

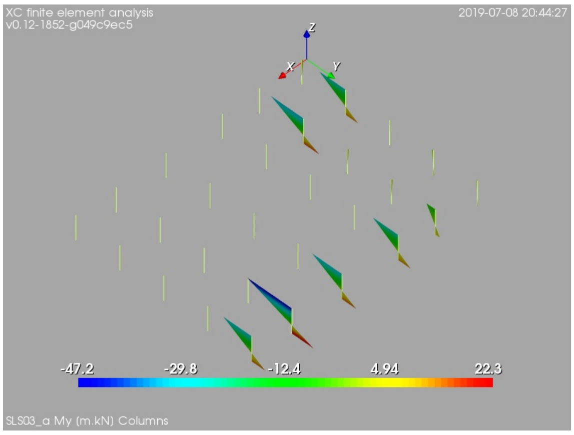


Figure 127: SLS03_a: 1.0*D + 1.0*S + 0.3*Lru + 0.3*Lpu. Columns, bending moment about local axis y [m.kN]

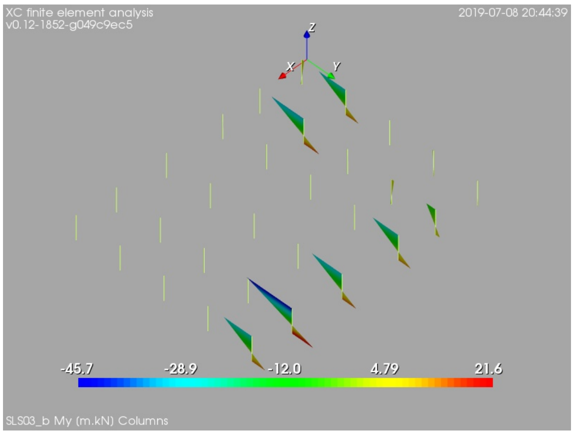


Figure 128: SLS03_b: 1.0*D + 1.0*S + 0.3*Lrs + 0.3*Lps. Columns, bending moment about local axis y [m.kN]

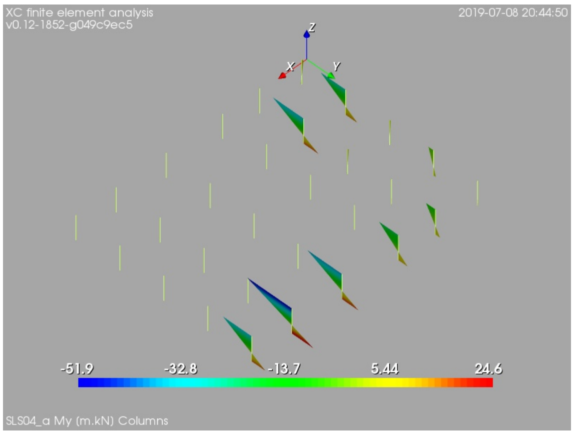


Figure 129: SLS04_a: 1.0*D + W_WE + 1.0*Lru + Lpu. Columns, bending moment about local axis y [m.kN]

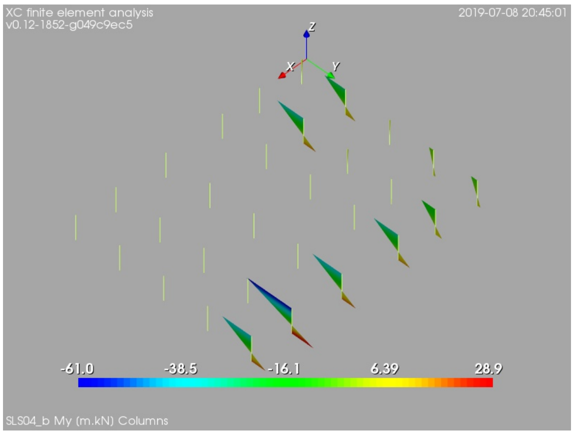


Figure 130: SLS04_b: 1.0*D + W_NS + 1.0*Lru + Lpu. Columns, bending moment about local axis y [m.kN]

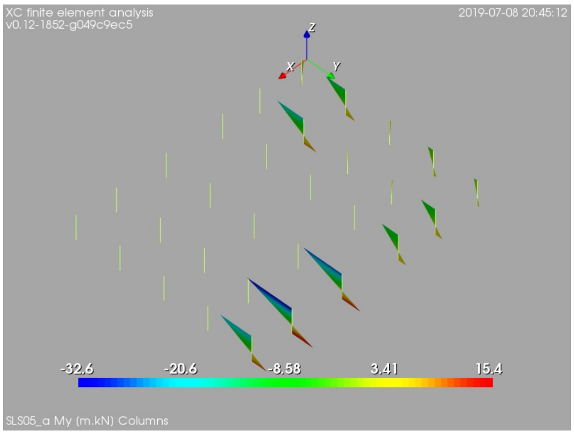


Figure 131: SLS05_a: 1.0*D + W_WE. Columns, bending moment about local axis y [m.kN]

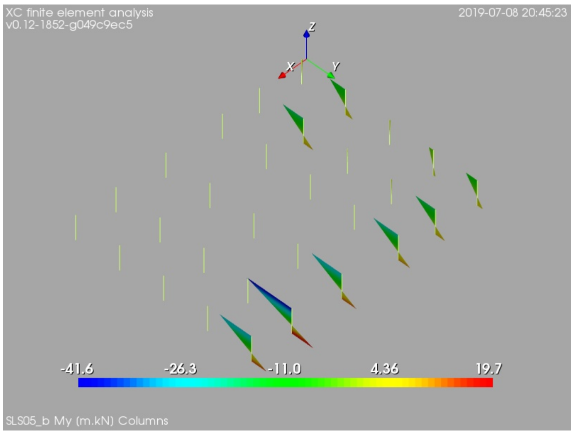


Figure 132: SLS05_b: 1.0*D + W_NS. Columns, bending moment about local axis y [m.kN]