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 12 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.

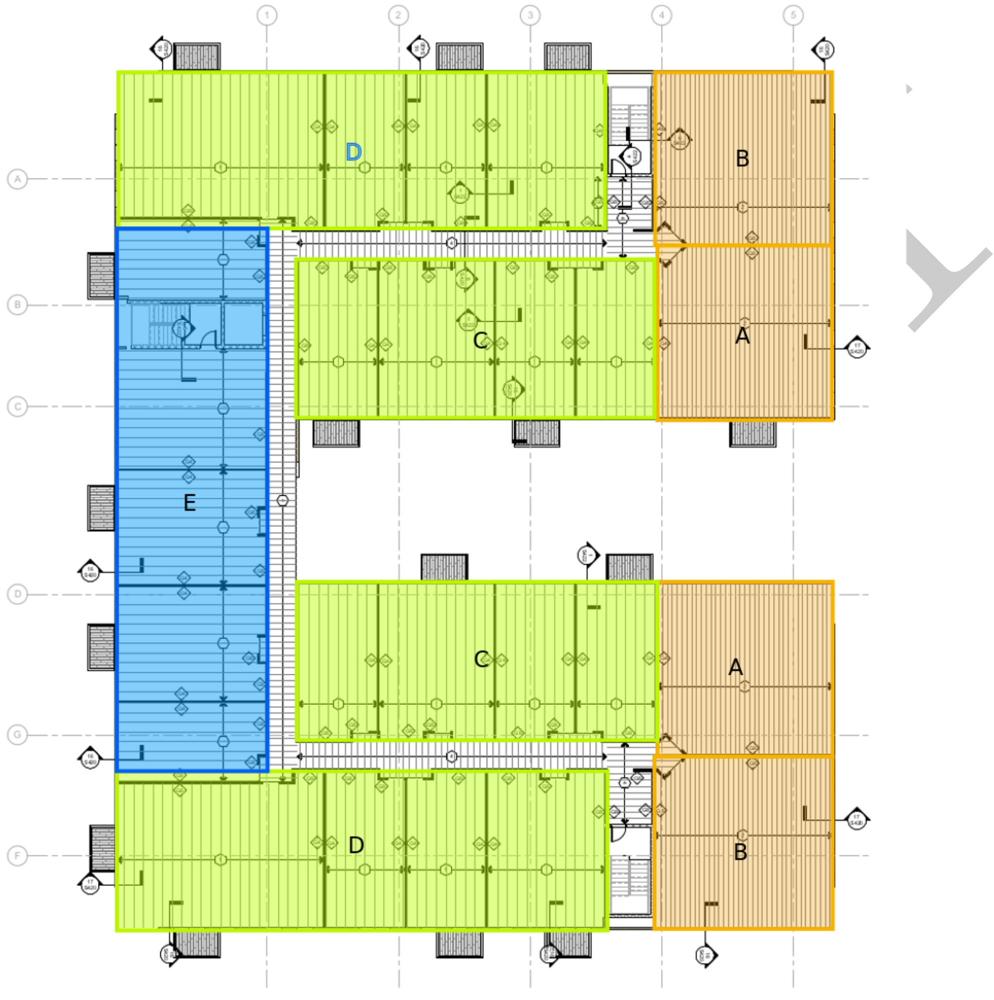


Figure 1: Trusses key plan.

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

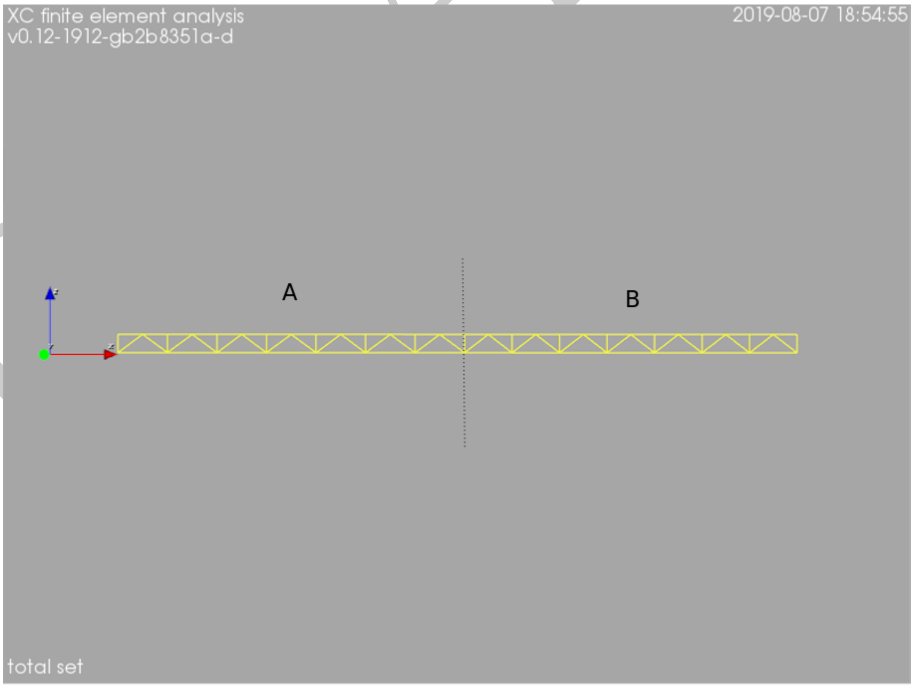


Figure 3: Third floor trusses at zones A and B (see key plan in figure 1).

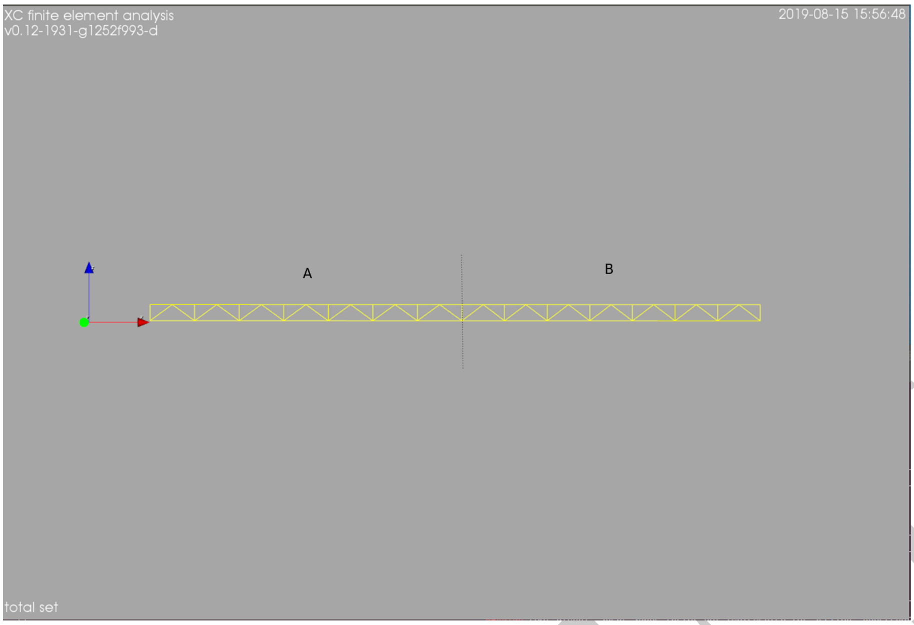


Figure 4: Second floor trusses at zones A and B (see key plan in figure 1).

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:

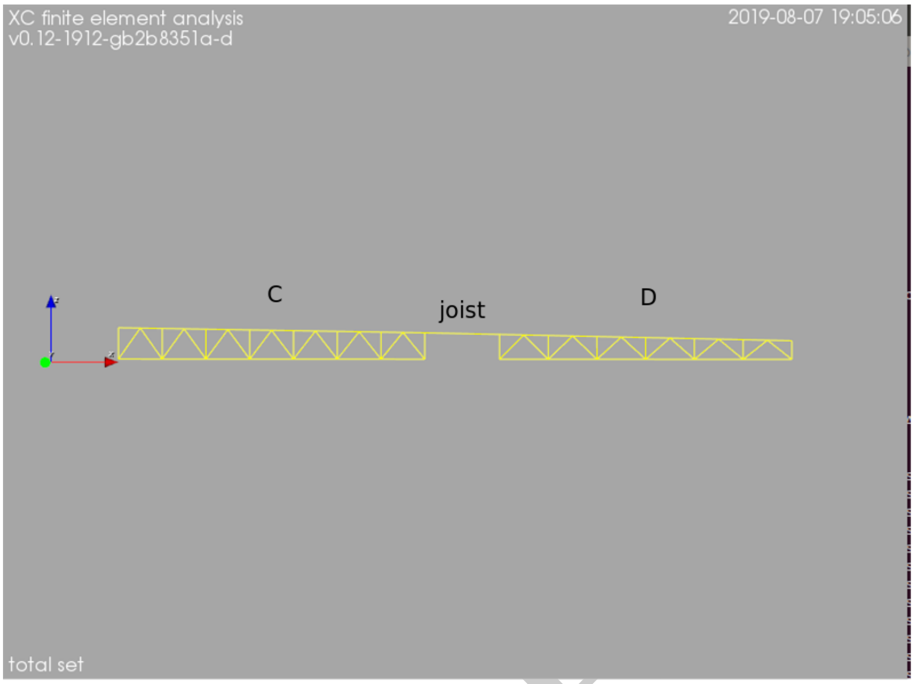


Figure 5: Roof trusses at zones C and D (see key plan in figure 1).

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:

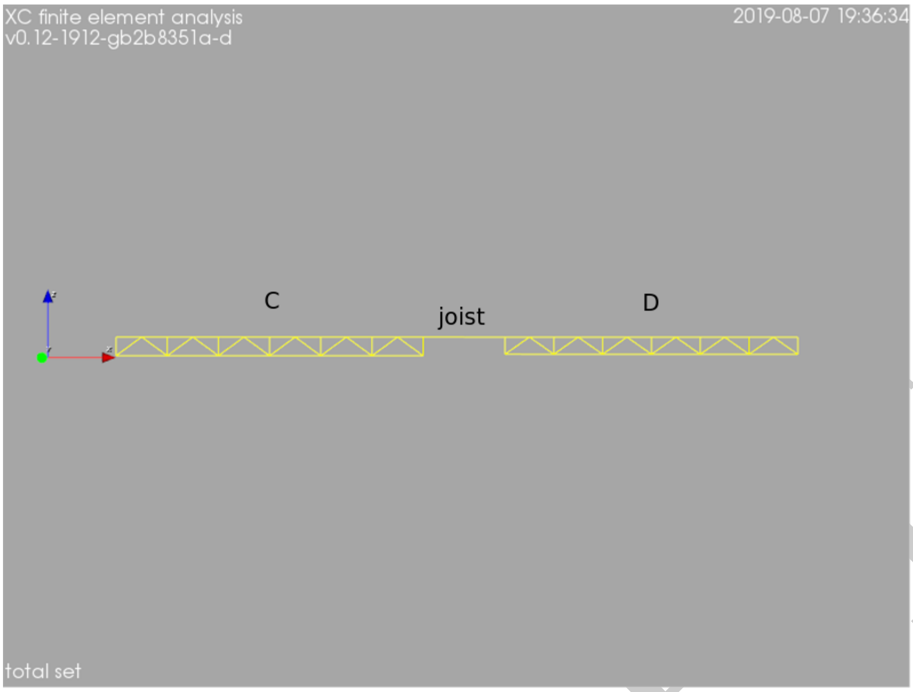


Figure 6: Third floor trusses at zones C and D (see key plan in figure 1).

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.

$$\Delta_{LL,C} = 16.93\ mm = \frac{L}{580} < \frac{L}{540} \implies OK \quad (17)$$

$$\Delta_{LL,D} = 15.11\ mm = \frac{L}{620} < \frac{L}{540} \implies OK \quad (18)$$

$$\Delta_{TL,C} = 26.93\ mm = \frac{L}{364} < \frac{L}{360} \implies OK \quad (19)$$

$$\Delta_{TL,D} = 24.04\ mm = \frac{L}{389} < \frac{L}{360} \implies OK \quad (20)$$

8.2.6 Trusses C and D. Second floor

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

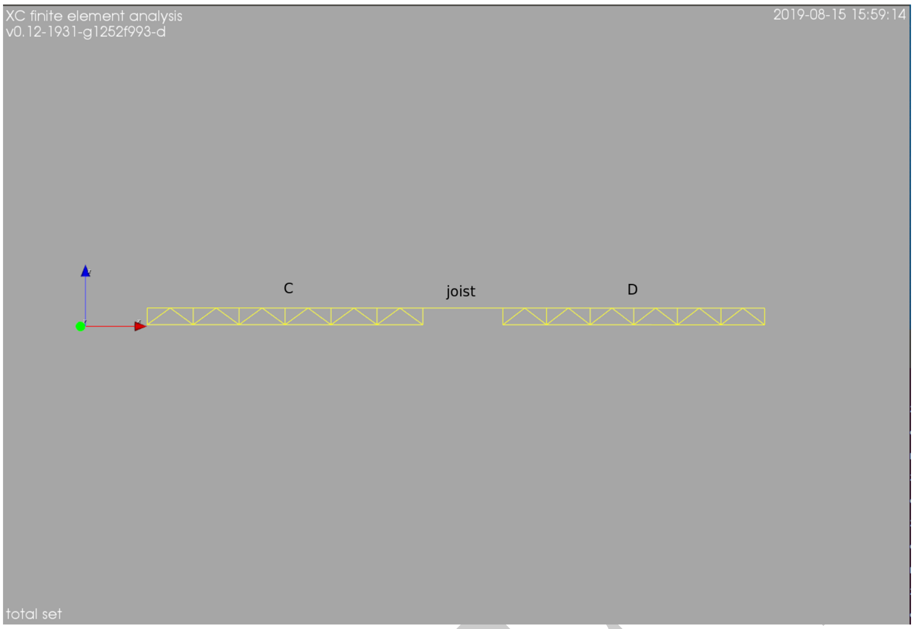


Figure 7: Second floor trusses at zones C and D (see key plan in figure 1).

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.

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

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

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

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

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)

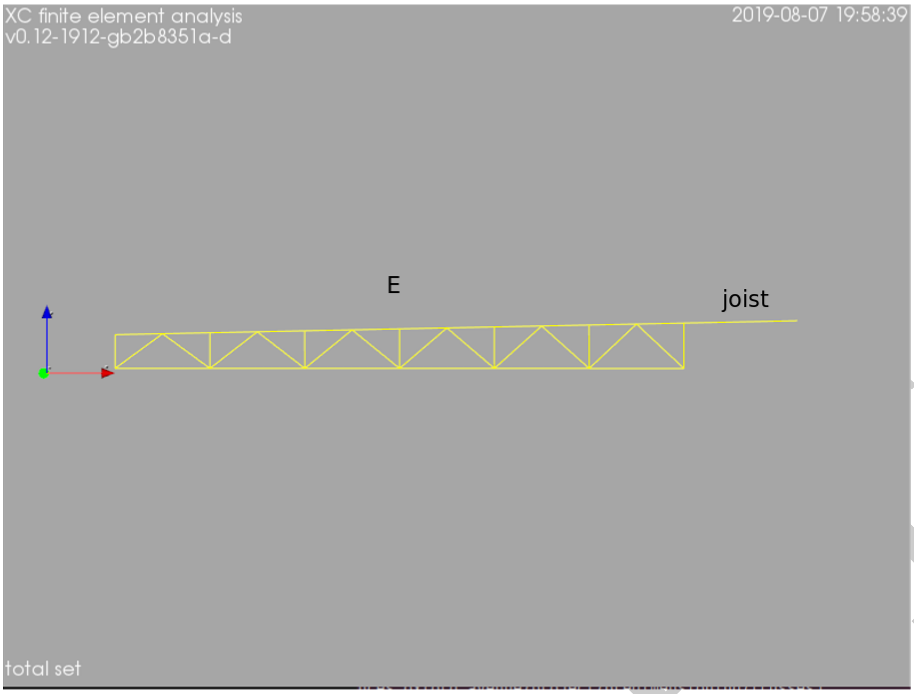


Figure 8: Roof truss at zone E (see key plan in figure 1).

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.

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

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

$$(27)$$

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.

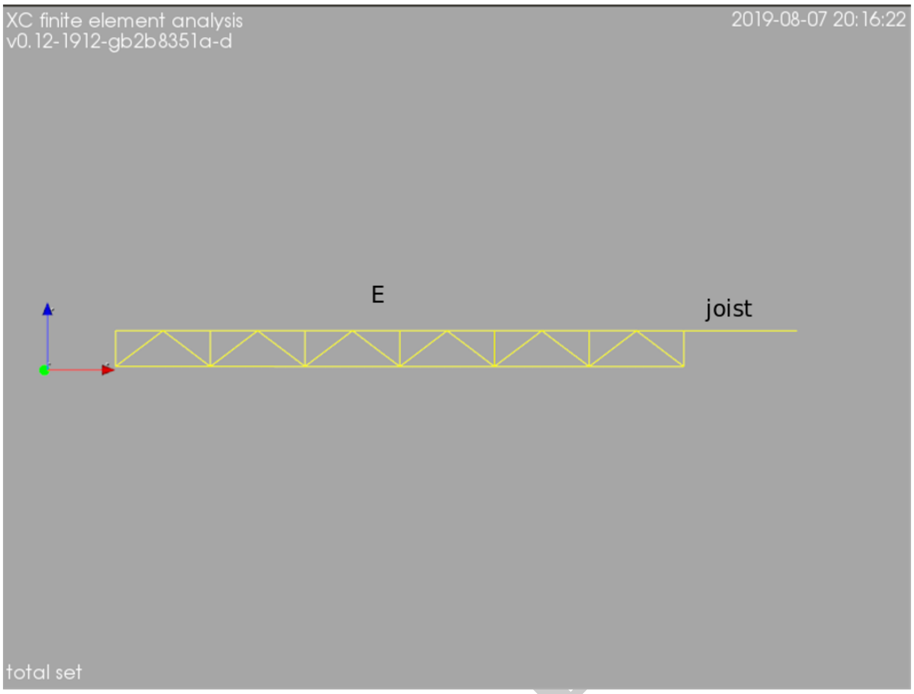


Figure 9: Third floor truss at zone E (see key plan in figure 1).

$$\Delta_{LL,E} = 14.65\ mm = \frac{L}{646} < \frac{L}{540} \implies OK \tag{28}$$

$$\Delta_{TL,E} = 23.30\ mm = \frac{L}{406} < \frac{L}{360} \implies OK \tag{29}$$

(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.

$$\Delta_{LL,E} = 12.16\ mm = \frac{L}{713} < \frac{L}{540} \implies OK \tag{31}$$

$$\Delta_{TL,E} = 19.34\ mm = \frac{L}{448} < \frac{L}{360} \implies OK \tag{32}$$

(33)

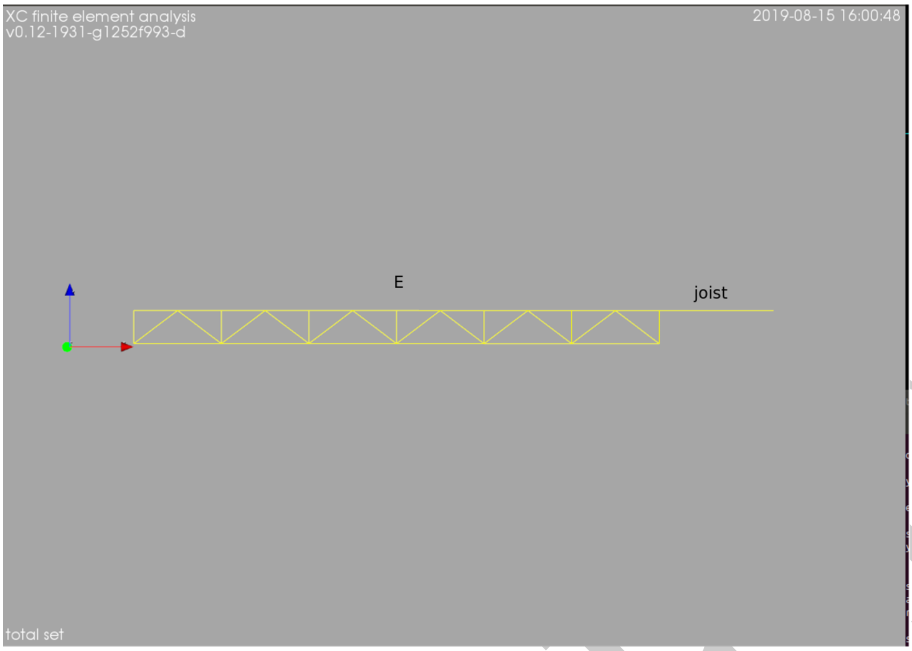


Figure 10: Second floor truss at zone E (see key plan in figure 1).

8.3 Joists and headers

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 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:

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

8.3.4 Facade headers

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

Structural design of the header.

Design load.

$$w_{load} = 75.91\ kN/m \tag{75}$$

Internal forces. Maximum induced moment:

$$M_{max} = w_{load}\frac{L^2}{8} = 8.26\ kNm \tag{76}$$

Maximum induced shear:

$$V_{max} = w_{load}\frac{L}{2} = 43.38\ kN \tag{77}$$

Structural bending check. Design resisting moment:

$$M'_s = 21.96\ kNm \tag{78}$$

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

Structural shear check. Design resisting shear:

$$V'_s = 50.53\ kN \tag{79}$$

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

Bending stiffness. The deflection obtained is:

$$\Delta_{TL} = 0.41\ mm = \frac{span}{2758} < \frac{span}{600} \implies OK \tag{80}$$

8.3.5 Corridor headers

Simply supported 3.5x16 LSL 1.55E header.

Structural design of the header.

Design load.

$$w_{load} = 118.11\ kN/m \tag{81}$$

Internal forces. Maximum induced moment:

$$M_{max} = w_{load}\frac{L^2}{8} = 14.63\ kNm \tag{82}$$

Maximum induced shear:

$$V_{max} = w_{load}\frac{L}{2} = 72.0\ kN \tag{83}$$

Structural bending check. Design resisting moment:

$$M'_s = 48.00\ kNm \tag{84}$$

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

Structural shear check. Design resisting shear:

$$V'_s = 76.60\ kN \tag{85}$$

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

Bending stiffness. The deflection obtained is:

$$\Delta_{TL} = 0.26\ mm = \frac{span}{4581} < \frac{span}{600} \implies OK \tag{86}$$

8.4 Steel beams

8.4.1 Steel beam at courtyard facade

Continuous beam supporting the second floor trusses between the axes 1, 2 and 3 (see figure 11). 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 17.22 kN each $0.6m(24'')$.

Structural design of the beam.

Loads.

$$w_{load} = 28.24\ kN/m \tag{87}$$

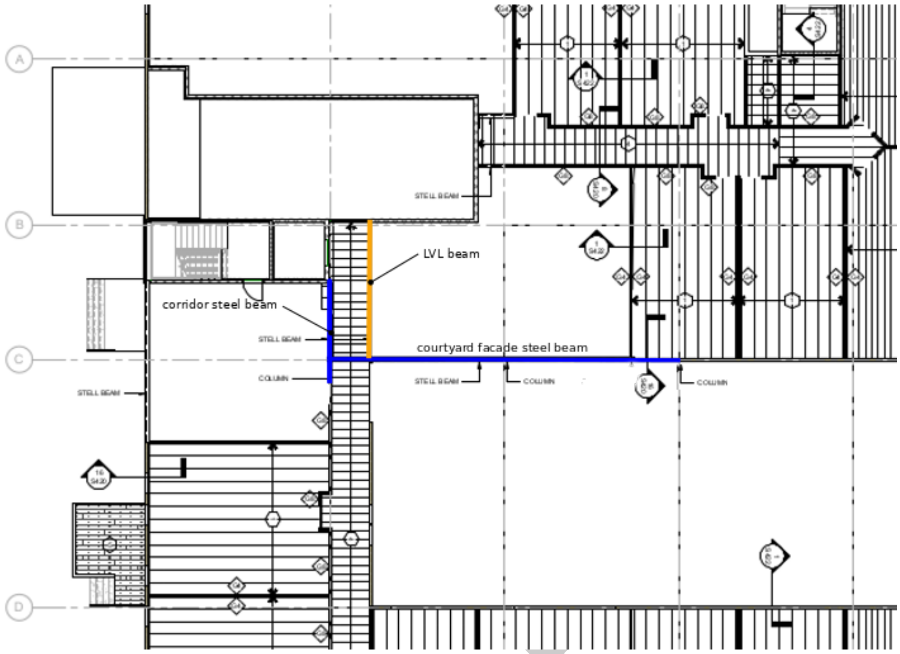


Figure 11: Second floor beams key plan.

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**
EQ1608	2nd floor	2C	3.78	4.86	2D	4.69	3.62
EQ1609	2nd floor	2C	9.64	12.14	2D	11.71	9.20
EQ1610	2nd floor	2C	3.78	4.86	2D	4.69	3.62
EQ1611	2nd floor	2C	8.17	10.32	2D	9.95	7.81
EQ1612	2nd floor	2C	3.78	4.86	2D	4.69	3.62
EQ1613	2nd floor	2C	8.17	10.32	2D	9.95	7.81
EQ1615	2nd floor	2C	2.27	2.92	2D	2.82	2.17
			9.64	**12.14**		**11.71**	**9.20**
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**

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

Internal forces on each structural channel. Maximum induced moment:

$$M_{max} = 60.90\ kNm \tag{88}$$

Maximum induced shear:

$$V_{max} = 146.64\ kN \tag{89}$$

Structural channel (C380X50.4) mechanical properties. Shear strength:

$$V_u = 560.92\ kN \tag{90}$$

Structural shear check: $V_u = 560.92 > 146.64 = V_{max} \implies OK$
Resisting moment:

$$M_u = 208.0\ kN \cdot m \tag{91}$$

Structural bending check: $M_U = 208.0 > 60.90 = M_{max} \implies OK$

Bending stiffness. The deflection obtained is:

$$\Delta_{TL} = 13.36\ mm = \frac{span}{622} < \frac{span}{600} \implies OK \tag{92}$$

8.4.2 Steel beam at corridor

This beam supports supporting the second floor trusses near the elevator well (see figure 11) . 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 20.98 kN each $0.6m(24'')$.

Structural design of the beam.

Loads.

$$w_{load} = 34.41\ kN/m \tag{93}$$

Internal forces on each structural channel. Maximum induced moment:

$$M_{max} = 58.56\ kNm \tag{94}$$

Maximum induced shear:

$$V_{max} = 108.65\ kN \tag{95}$$

Structural channel (C380X50.4) mechanical properties. Shear strength:

$$V_u = 213.77\ kN \tag{96}$$

Structural shear check: $V_u = 213.77 > 108.64 = V_{max} \implies OK$
Resisting moment:

$$M_u = 128.25\ kN \cdot m \tag{97}$$

Structural bending check: $M_U = 128.25 > 58.56 = M_{max} \implies OK$

comb	floor	truss name	Rz (kN)	Rz (kN)
EQ1608	roof	3E	2.19	2.90
EQ1609	roof	3E	4.96	6.41
EQ1610	roof	3E	7.99	10.25
EQ1611	roof	3E	8.62	11.04
EQ1612	roof	3E	-0.01	0.12
EQ1613	roof	3E	6.97	8.95
EQ1615	roof	3E	-0.89	-1.04
			8.619	**11.0419**
EQ1608	2nd floor	2E	3.59	4.68
EQ1609	2nd floor	2E	9.13	11.69
EQ1610	2nd floor	2E	3.59	4.68
EQ1611	2nd floor	2E	7.74	9.94
EQ1612	2nd floor	2E	3.59	4.68
EQ1613	2nd floor	2E	7.74	9.94
EQ1615	2nd floor	2E	2.15	2.81
			9.129	**11.6896**
EQ1608		3E+2E	5.78	7.58
EQ1609		3E+2E	14.09	18.10
EQ1610		3E+2E	11.58	14.92
EQ1611		3E+2E	16.36	20.98
EQ1612		3E+2E	3.58	4.79
EQ1613		3E+2E	14.71	18.89
EQ1615		3E+2E	1.27	1.76
			16.36	**20.978**

Table 11: Steel beam at corridor. Trusses reactions.

Bending stiffness. The deflection obtained is:

$$\Delta_{TL} = 7.77\ mm = \frac{span}{510} < \frac{span}{480} \implies OK \tag{98}$$

8.5 Cantilevers

8.5.1 Cantilevers C1, C3, C4, C6, C11 and C12

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:

$$M_{max} = 37.98\ kNm \tag{99}$$

Maximum induced shear:

$$V_{max} = 65.41\ kN \tag{100}$$

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

$$M'_s = 44.89\ kNm \tag{101}$$

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

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

$$V'_s = 89.36\ kN \tag{102}$$

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

Bending stiffness. The deflection obtained under total load is:

$$\Delta_{TL} = 1.88\ mm = \frac{L}{506} < \frac{L}{360} \implies OK \tag{103}$$

8.5.2 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:

$$M_{max} = 63.79\ kNm \tag{104}$$

Maximum induced shear:

$$V_{max} = 129.25\ kN \tag{105}$$

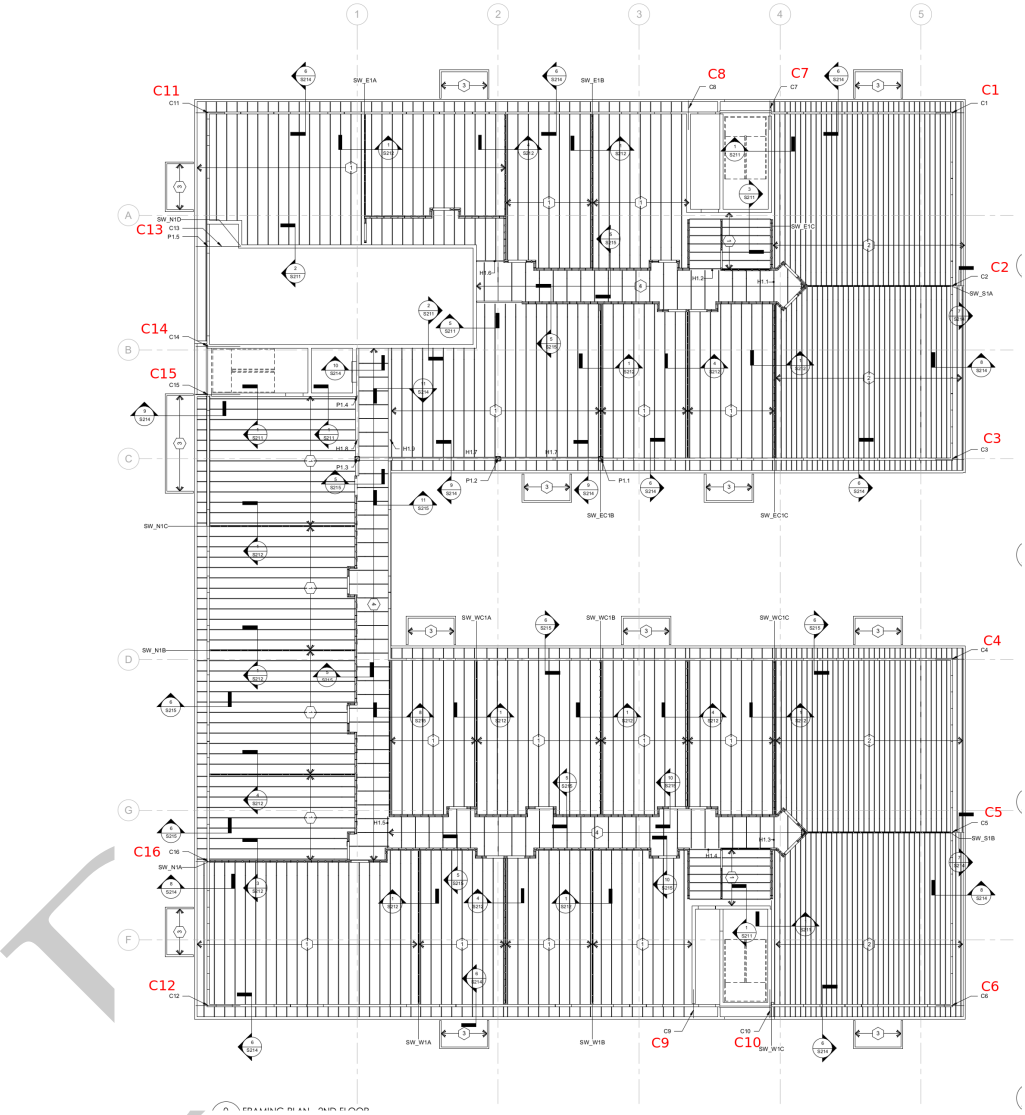


Figure 12: Second floor cantilevers key plan.

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

$$M_s' = 65.88 \ kNm \tag{106}$$

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:

$$V_s' = 151.60 \ kN \tag{107}$$

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

Bending stiffness. The deflection obtained under total load is:

$$\Delta_{TL} = 1.95 \ mm = \frac{L}{442} < \frac{L}{360} \implies OK \tag{108}$$

8.5.3 Cantilevers C7, C8, C9 and C10

Loads.

- Floor load: 5.78 kN
- Facade weight: 12.93 kN
- Total load: 18.71 kN

Internal forces. Maximum induced moment:

$$M_{max} = 17.77 \ kNm \tag{109}$$

Maximum induced shear:

$$V_{max} = 18.71 \ kN \tag{110}$$

Structural bending check. LSL 1.55E 3.5x14” design resisting moment:

$$M_s' = 29.93 \ kNm \tag{111}$$

Structural bending check: $M_s' = 29.93 > 17.77 = M_{max} \implies OK$

Structural shear check. LSL 1.55E 3.5x14” design resisting shear:

$$V_s' = 59.58 \ kN \tag{112}$$

Structural shear check: $V_s' = 59.57 > 18.71 = V_{max} \implies OK$

Bending stiffness. The deflection obtained under total load is:

$$\Delta_{TL} = 1.5 \ mm = \frac{L}{632} < \frac{L}{360} \implies OK \tag{113}$$

8.5.4 Cantilever C13

Loads.

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

Internal forces. Maximum induced moment:

$$M_{max} = 49.99\ kNm \tag{114}$$

Maximum induced shear:

$$V_{max} = 52.62\ kN \tag{115}$$

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

$$M'_s = 72.00\ kNm \tag{116}$$

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

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

$$V'_s = 114.90\ kN \tag{117}$$

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

Bending stiffness. The deflection obtained under total load is:

$$\Delta_{TL} = 1.5\ mm = \frac{L}{717} < \frac{L}{360} \implies OK \tag{118}$$

8.5.5 Cantilever C14

Loads.

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

Internal forces. Maximum induced moment:

$$M_{max} = 33.33\ kNm \tag{119}$$

Maximum induced shear:

$$V_{max} = 35.08\ kN \tag{120}$$

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

$$M'_s = 48.00\ kNm \tag{121}$$

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

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.

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

$$V'_s = 76.60\ kN \tag{122}$$

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

Bending stiffness. The deflection obtained under total load is:

$$\Delta_{TL} = 1.33\ mm = \frac{L}{717} < \frac{L}{360} \implies OK \tag{123}$$

8.6 Lateral. Diaphragms/Shear walls

8.6.1 East and West facades shear walls

The shear walls of the East facade are those denoted by the letters "E" and "W" in figures 13 to 15. 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	Number of bolts	Bolt spacing	
ID		(in)		(in)		(in)	(plf)	(kip)		(in)	
SW_E3A	Wood structural panels – sheathing	19/32	YES	1-1/2	10d	4	1430	3	-	-	wood screws 20 (d= 0.32 in) at 16 in. o/c; 46 fasteners in 2 rows.
SW_E3B	Wood structural panels – sheathing	3/8	NO	1-3/8	8d	6	560	-	-	-	16d (d= 0.268 in) nails at 12 in. o/c; 30 fasteners in 1 row.
SW_E3C	Wood structural panels – sheathing	19/32	YES	1-1/2	10d	4	1430	6	-	-	SDWS log screw (d= 0.197 in) at 15 in. o/c; 32 fasteners in 2 rows.
SW_E2A	Wood structural panels – sheathing	19/32	YES	1-1/2	10d	3	1860	7	-	-	SDWS log screw (d= 0.197 in) at 11 in. o/c; 64 fasteners in 2 rows.
SW_E2B	Wood structural panels – sheathing	3/8	NO	1-3/8	8d	6	560	1	-	-	16d (d= 0.268 in) nails at 14 in. o/c; 51 fasteners in 2 rows.
SW_E2C	Wood structural panels – sheathing	19/32	YES	1-1/2	10d	2	2435	11	-	-	SDWS log screw (d= 0.197 in) at 9 in. o/c; 54 fasteners in 2 rows.
SW_E1A	Wood structural panels – sheathing	19/32	YES	1-1/2	10d	2	2435	13	7	36	SDWS log screw (d= 0.197 in) at 7 in. o/c; 64 fasteners in 2 rows.
SW_E1B	Wood structural panels – sheathing	3/8	NO	1-3/8	8d	6	560	-	11	36	16d (d= 0.268 in) nails at 32 in. o/c; 12 fasteners in 1 row.
SW_E1C	Wood structural panels – sheathing	19/32	YES	1-1/2	10d	2	2435	9	11	36	SDWS log screw (d= 0.197 in) at 10 in. o/c; 72 fasteners in 2 rows.
SW_W3A	Wood structural panels – sheathing	19/32	YES	1-1/2	10d	4	1430	3	-	-	wood screws 20 (d= 0.32 in) at 16 in. o/c; 46 fasteners in 2 rows.
SW_W3B	Wood structural panels – sheathing	3/8	NO	1-3/8	8d	6	560	-	-	-	16d (d= 0.268 in) nails at 12 in. o/c; 30 fasteners in 1 row.
SW_W3C	Wood structural panels – sheathing	19/32	YES	1-1/2	10d	4	1430	6	-	-	SDWS log screw (d= 0.197 in) at 15 in. o/c; 32 fasteners in 2 rows.
SW_W2A	Wood structural panels – sheathing	19/32	YES	1-1/2	10d	3	1860	7	-	-	SDWS log screw (d= 0.197 in) at 11 in. o/c; 64 fasteners in 2 rows.
SW_W2B	Wood structural panels – sheathing	3/8	NO	1-3/8	8d	6	560	1	-	-	16d (d= 0.268 in) nails at 14 in. o/c; 51 fasteners in 2 rows.
SW_W2C	Wood structural panels – sheathing	19/32	YES	1-1/2	10d	2	2435	11	-	-	SDWS log screw (d= 0.197 in) at 9 in. o/c; 54 fasteners in 2 rows.
SW_W1A	Wood structural panels – sheathing	19/32	YES	1-1/2	10d	2	2435	13	9	30	SDWS log screw (d= 0.197 in) at 7 in. o/c; 64 fasteners in 2 rows.
SW_W1B	Wood structural panels – sheathing	3/8	NO	1-3/8	8d	6	560	-	11	36	16d (d= 0.268 in) nails at 32 in. o/c; 12 fasteners in 1 row.
SW_W1C	Wood structural panels – sheathing	19/32	YES	1-1/2	10d	2	2435	9	11	36	SDWS log screw (d= 0.197 in) at 10 in. o/c; 72 fasteners in 2 rows.

8.6.2 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 13 to 15. 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	Number of bolts	Bolt spacing	
ID		(in)		(in)		(in)	(plf)	(kip)		(in)	
SW_EC3A	Wood structural panels – sheathing	19/32	YES	1-1/2	10d	6	950	0	-	-	16d (d= 0.268 in) nails at 18 in. o/c; 42 fasteners in 2 rows.
SW_EC3B	Wood structural panels – sheathing	3/8	NO	1-3/8	8d	6	560	-	-	-	16d (d= 0.268 in) nails at 60 in. o/c; 7 fasteners in 1 row.
SW_EC3C	Wood structural panels – sheathing	19/32	YES	1-1/2	10d	6	950	3	-	-	wood screws 20 (d= 0.32 in) at 19 in. o/c; 40 fasteners in 2 rows.
SW_EC2A	Wood structural panels – sheathing	19/32	YES	1-1/2	10d	3	1860	2	-	-	wood screws 20 (d= 0.32 in) at 21 in. o/c; 36 fasteners in 2 rows.
SW_EC2B	Wood structural panels – sheathing	3/8	NO	1-3/8	8d	6	560	-	-	-	16d (d= 0.268 in) nails at 32 in. o/c; 12 fasteners in 1 row.
SW_EC2C	Wood structural panels – sheathing	19/32	YES	1-1/2	10d	3	1860	6	-	-	SDWS log screw (d= 0.197 in) at 12 in. o/c; 58 fasteners in 2 rows.
SW_EC1A	Wood structural panels – sheathing	19/32	YES	1-1/2	10d	2	2435	11	6	36	SDWS log screw (d= 0.197 in) at 9 in. o/c; 42 fasteners in 2 rows.
SW_EC1B	Wood structural panels – sheathing	3/8	NO	1-3/8	8d	6	560	-	11	36	16d (d= 0.268 in) nails at 22 in. o/c; 17 fasteners in 1 row.
SW_EC1C	Wood structural panels – sheathing	19/32	YES	1-1/2	10d	2	2435	11	11	36	SDWS log screw (d= 0.197 in) at 9 in. o/c; 82 fasteners in 2 rows.
SW_WC3A	Wood structural panels – sheathing	19/32	YES	1-1/2	10d	6	950	0	-	-	16d (d= 0.268 in) nails at 18 in. o/c; 42 fasteners in 2 rows.
SW_WC3B	Wood structural panels – sheathing	3/8	NO	1-3/8	8d	0	560	-	-	-	16d (d= 0.268 in) nails at 60 in. o/c; 7 fasteners in 1 row.
SW_WC3C	Wood structural panels – sheathing	19/32	YES	1-1/2	10d	6	950	3	-	-	wood screws 20 (d= 0.32 in) at 19 in. o/c; 40 fasteners in 2 rows.
SW_WC2A	Wood structural panels – sheathing	19/32	YES	1-1/2	10d	3	1860	2	-	-	wood screws 20 (d= 0.32 in) at 21 in. o/c; 36 fasteners in 2 rows.
SW_WC2B	Wood structural panels – sheathing	3/8	NO	1-3/8	8d	6	560	-	-	-	16d (d= 0.268 in) nails at 32 in. o/c; 12 fasteners in 1 row.
SW_WC2C	Wood structural panels – sheathing	19/32	YES	1-1/2	10d	3	1860	6	-	-	SDWS log screw (d= 0.197 in) at 12 in. o/c; 58 fasteners in 2 rows.
SW_WC1A	Wood structural panels – sheathing	19/32	YES	1-1/2	10d	2	2435	11	6	36	SDWS log screw (d= 0.197 in) at 9 in. o/c; 42 fasteners in 2 rows.

SW_WC1B	Wood structural panels – sheathing	3/8	NO	1-3/8	8d	6	560	-	11	36	16d (d= 0.268 in) nails at 22 in. o/c; 17 fasteners in 1 row.
SW_WC1C	Wood structural panels – sheathing	19/32	YES	1-1/2	10d	2	2435	11	11	36	SDWS log screw (d= 0.197 in) at 9 in. o/c; 82 fasteners in 2 rows.

8.6.3 South facades shear walls

The shear walls of the South facade are those denoted by the letter “S” in figures 13 to 15. 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	Number of bolts	Bolt spacing	
ID		(in)		(in)		(in)	(plf)	(kip)		(in)	
SW_S3A	Wood structural panels – sheathing	19/32	YES	1-1/2	10d	6	950	2	-	-	wood screws 20 (d= 0.32 in) at 21 in. o/c; 36 fasteners in 2 rows.
SW_S3B	Wood structural panels – sheathing	19/32	YES	1-1/2	10d	6	950	2	-	-	wood screws 20 (d= 0.32 in) at 21 in. o/c; 36 fasteners in 2 rows.
SW_S2A	Wood structural panels – sheathing	19/32	YES	1-1/2	10d	3	1860	6	-	-	SDWS log screw (d= 0.197 in) at 13 in. o/c; 54 fasteners in 2 rows.
SW_S2B	Wood structural panels – sheathing	19/32	YES	1-1/2	10d	3	1860	6	-	-	SDWS log screw (d= 0.197 in) at 13 in. o/c; 54 fasteners in 2 rows.
SW_S1A	Wood structural panels – sheathing	19/32	YES	1-1/2	10d	2	2435	11	10	36	SDWS log screw (d= 0.197 in) at 8 in. o/c; 76 fasteners in 2 rows.
SW_S1B	Wood structural panels – sheathing	19/32	YES	1-1/2	10d	2	2435	11	10	36	SDWS log screw (d= 0.197 in) at 8 in. o/c; 76 fasteners in 2 rows.

8.6.4 North facade shear walls

The shear walls of the North facade are those denoted by the letter "N" in figures 13 to 15. 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	Number of bolts	Bolt spacing	
ID		(in)		(in)		(in)	(plf)	(kip)		(in)	
SW_N3A	Wood structural panels – sheathing	3/8	YES	1-3/8	8d	4	840	2	-	-	wood screws 20 (d= 0.32 in) at 25 in. o/c; 30 fasteners in 2 rows.
SW_N3B	Wood structural panels – sheathing	3/8	NO	1-3/8	8d	6	560	-	-	-	16d (d= 0.268 in) nails at 24 in. o/c; 16 fasteners in 1 row.
SW_N3C	Wood structural panels – sheathing	3/8	NO	1-3/8	8d	6	560	-	-	-	16d (d= 0.268 in) nails at 21 in. o/c; 35 fasteners in 2 rows.
SW_N3D	Wood structural panels – sheathing	3/8	YES	1-3/8	8d	4	840	2	-	-	wood screws 20 (d= 0.32 in) at 25 in. o/c; 30 fasteners in 2 rows.
SW_N2A	Wood structural panels – sheathing	19/32	YES	1-1/2	10d	4	1430	4	-	-	wood screws 20 (d= 0.32 in) at 14 in. o/c; 52 fasteners in 2 rows.
SW_N2B	Wood structural panels – sheathing	19/32	YES	1-1/2	10d	6	950	-	-	-	16d (d= 0.268 in) nails at 13 in. o/c; 28 fasteners in 1 row.
SW_N2C	Wood structural panels – sheathing	19/32	YES	1-1/2	10d	6	950	1	-	-	16d (d= 0.268 in) nails at 12 in. o/c; 59 fasteners in 2 rows.

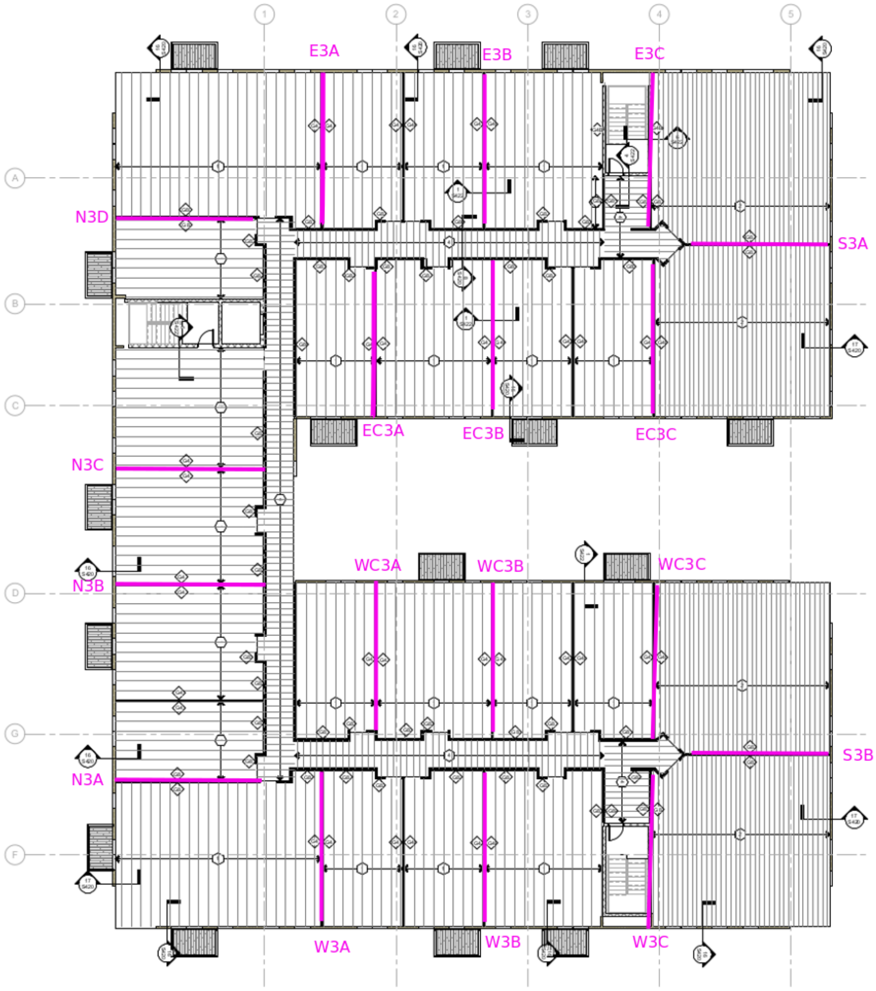


Figure 13: Shear walls on the third floor.

SW_N2D	Wood structural panels – sheathing	19/32	YES	1-1/2	10d	4	1430	4	-	-	wood screws 20 (d= 0.32 in) at 14 in. o/c; 52 fasteners in 2 rows.
SW_N1A	Wood structural panels – sheathing	19/32	YES	1-1/2	10d	3	1860	7	10	36	SDWS log screw (d= 0.197 in) at 12 in. o/c; 58 fasteners in 2 rows.
SW_N1B	Wood structural panels – sheathing	19/32	YES	1-1/2	10d	6	950	-	11	36	16d (d= 0.268 in) nails at 19 in. o/c; 39 fasteners in 2 rows.
SW_N1C	Wood structural panels – sheathing	19/32	YES	1-1/2	10d	6	950	3	11	36	wood screws 20 (d= 0.32 in) at 19 in. o/c; 40 fasteners in 2 rows.
SW_N1D	Wood structural panels – sheathing	19/32	YES	1-1/2	10d	3	1860	7	10	36	SDWS log screw (d= 0.197 in) at 12 in. o/c; 60 fasteners in 2 rows.

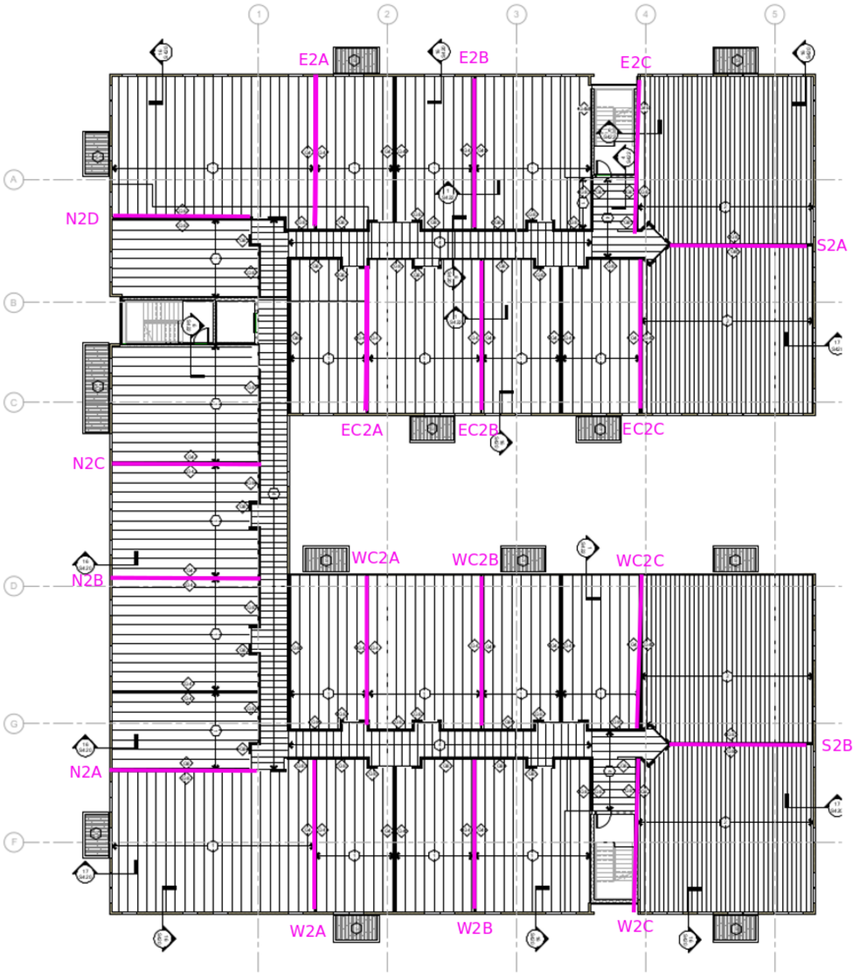


Figure 14: Shear walls on the second floor.

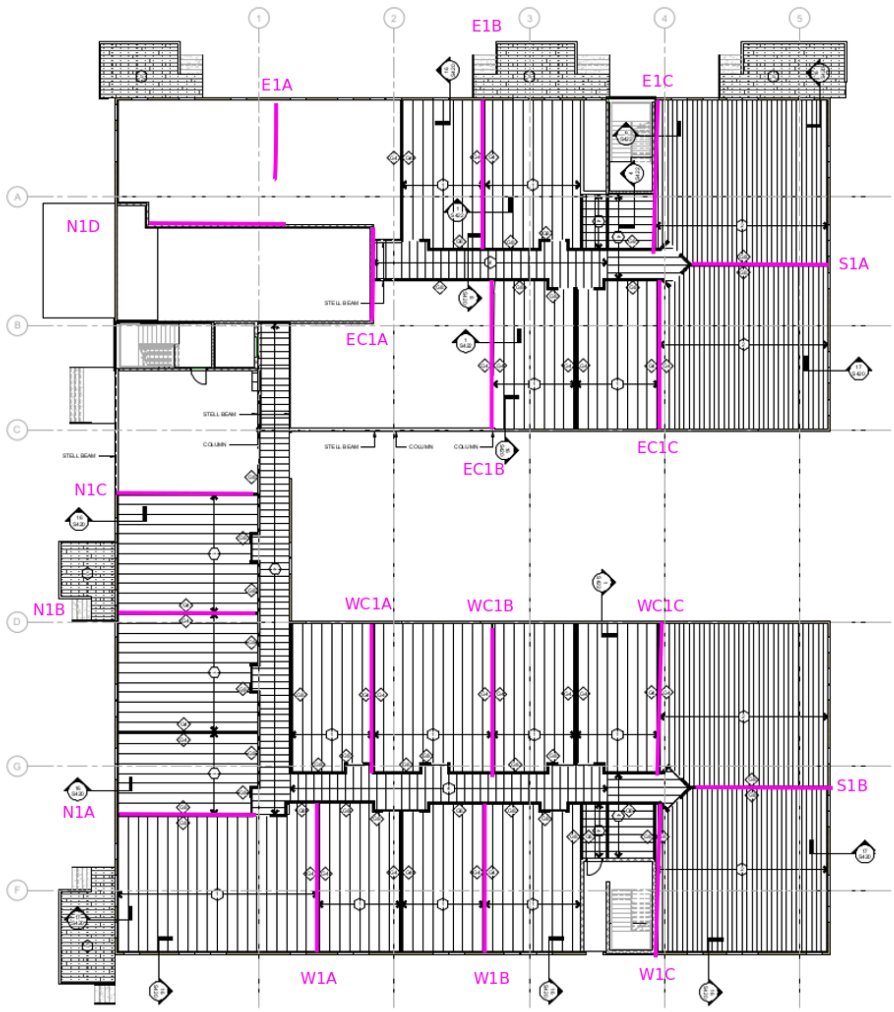


Figure 15: Shear walls on the first floor.

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 16). 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 25. See in figures 17 to 24 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

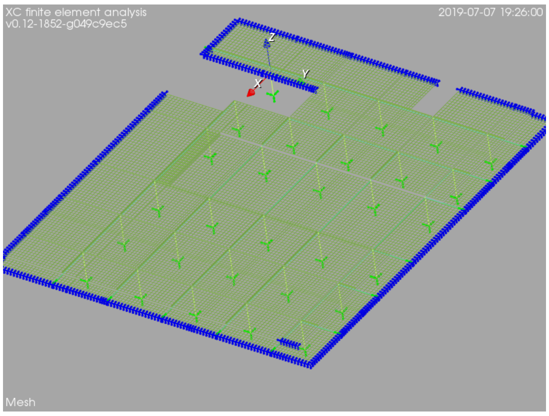


Figure 16: Elastic model, mesh.

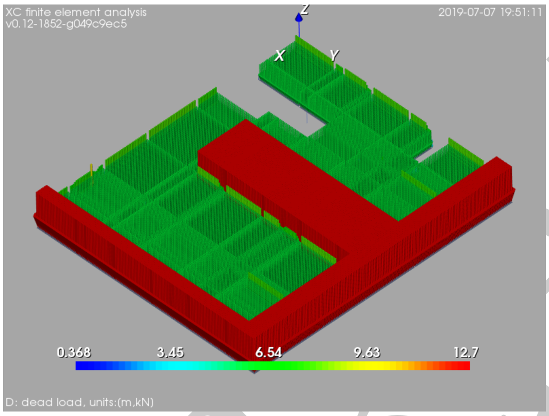


Figure 17: Load case D: dead load (include slab self-weight) [units: kN,m].

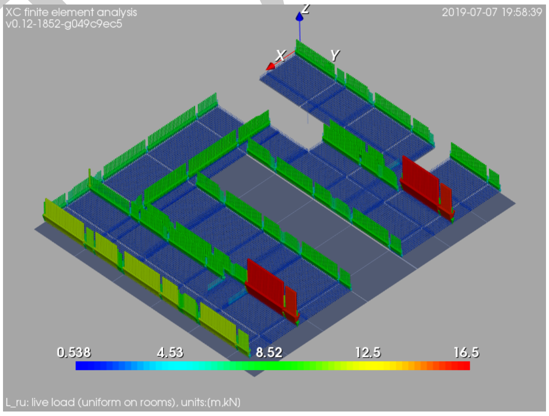


Figure 18: Load case Lru: live load (uniform on rooms) [units: kN,m].

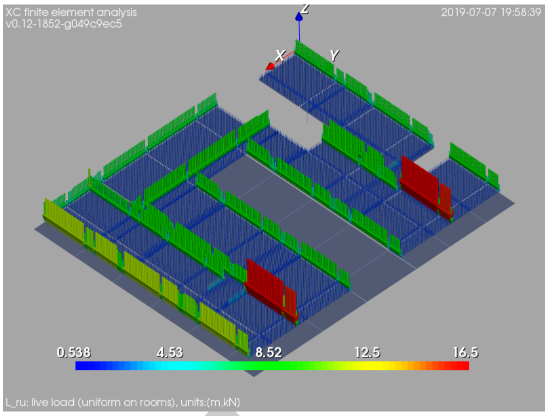


Figure 19: Load case Lrs: live load (staggered pattern on rooms) [units: kN,m].

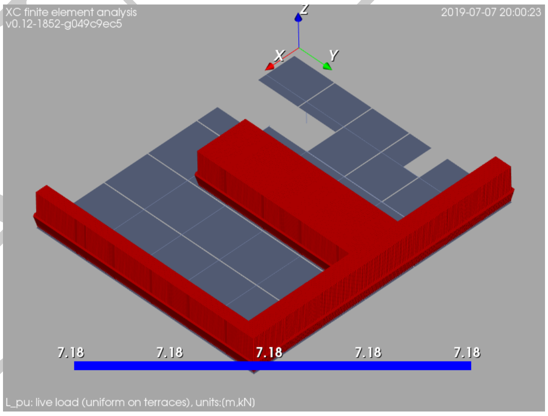


Figure 20: Load case Lpu: live load (uniform on patios) [units: kN,m].

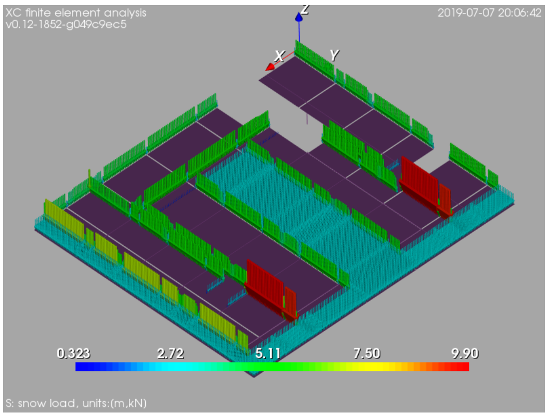


Figure 21: Load case S: snow [units: kN,m].

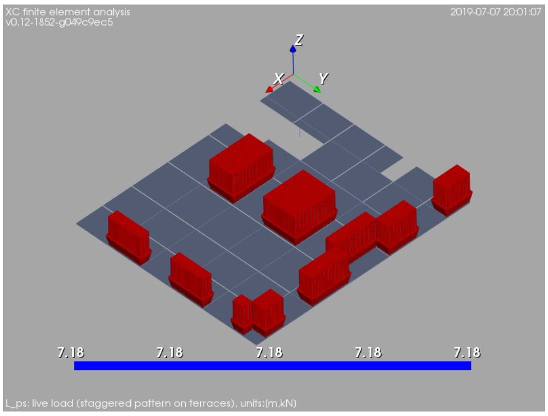


Figure 22: Load case Lps: live load (staggered pattern on patios) [units: kN,m].

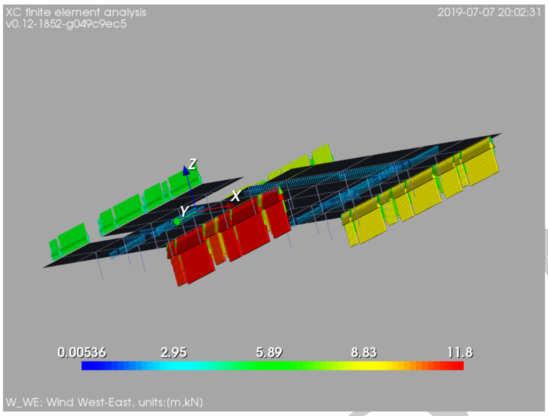


Figure 23: Load case W_WE: wind West-East [units: kN,m].

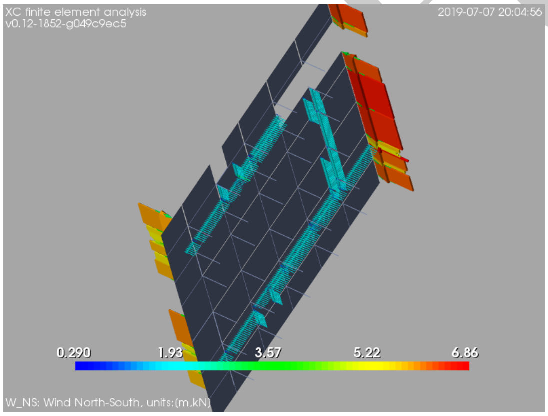
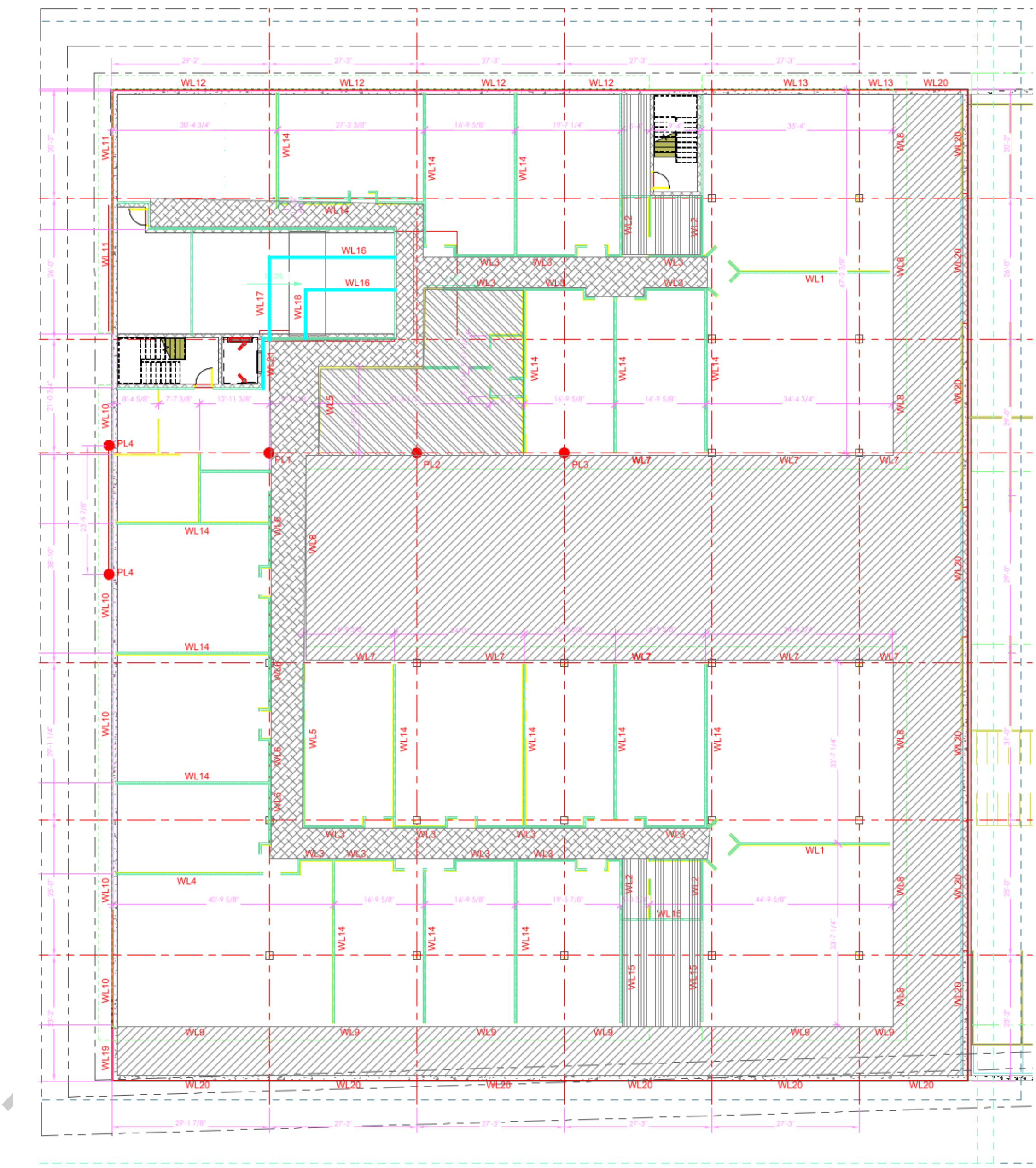


Figure 24: Load case W_NS: wind North-South [units: kN,m].



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 25: 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 26 and 27.

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

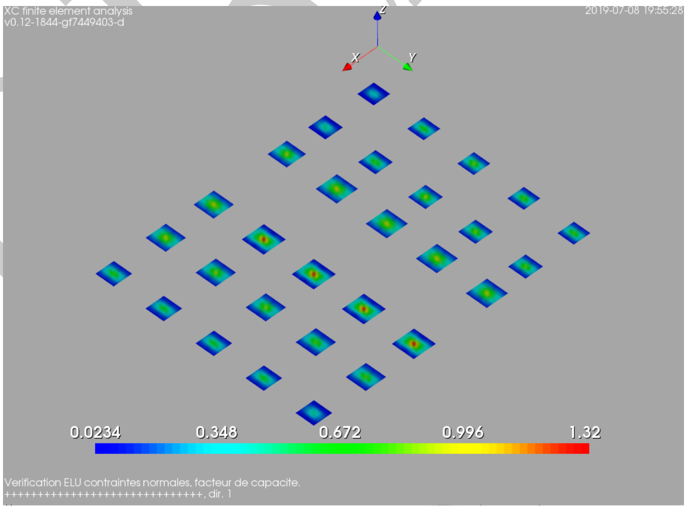


Figure 26: Flexure in the longitudinal direction. Capacity factor.

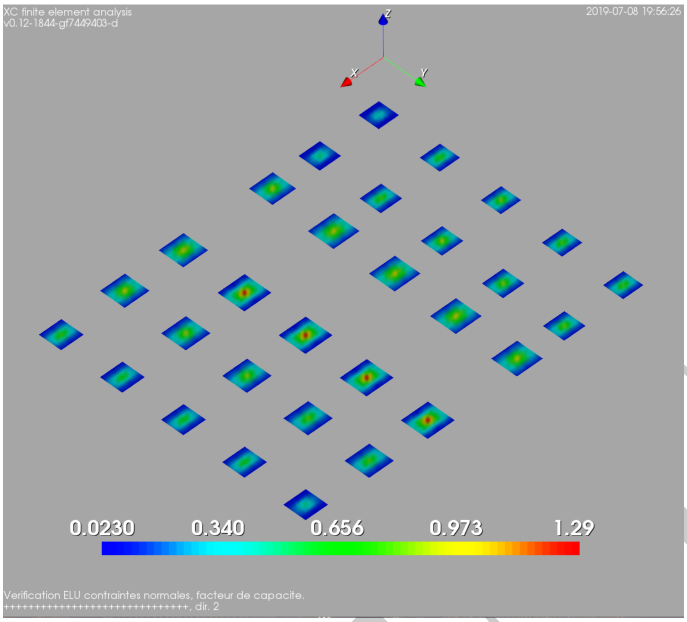


Figure 27: Flexure in the transverse direction. Capacity factor.

- 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	

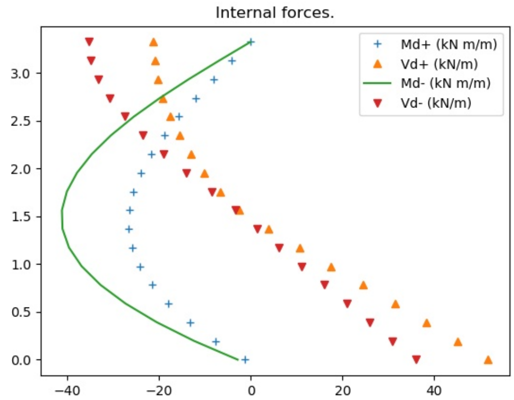
Earth pressure. The soil pressure over the wall has been calculated using the lateral pressure at rest with a coefficient $K_0 = 0.5$.

CONCRETE WALL REINFORCING SCHEDULE					
MARK	TYPE	THICKNESS	REINFORCEMENT		REMARKS
			VERTICAL	HORIZONTAL	
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.

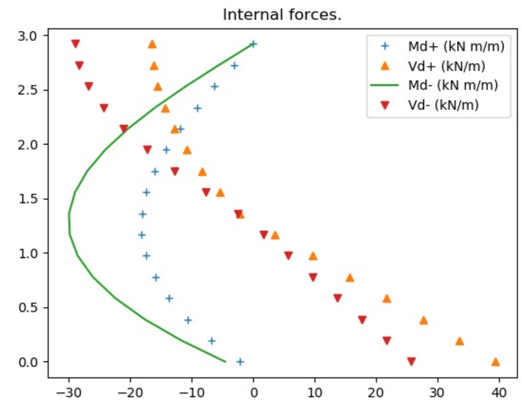
<table>
<tr><th colspan="2">T2</th></tr>
<tr><td>
</td><td>WALL GEOMETRY
Stem top thickness:
$b_{top} = 0.25\ m$
Stem height:
$h_{stem} = 2.74\ m$
Stem bottom thickness:
$b_{bottom} = 0.25\ m$
Footing thickness:
$b_{footing} = 0.36\ m$</td></tr>
<tr><td colspan="2">MATERIALS
Concrete: C4000 Steel: A615G60 Concrete cover: 55 mm</td></tr>
</table>

Table 23: Wall materials and dimensions T2

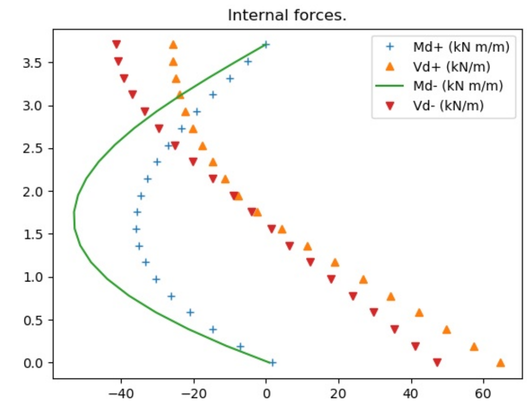
<table>
<tr><th colspan="2">T3</th></tr>
<tr><td>
</td><td>WALL GEOMETRY
Stem top thickness:
$b_{top} = 0.25\ m$
Stem height:
$h_{stem} = 3.53\ m$
Stem bottom thickness:
$b_{bottom} = 0.25\ m$
Footing thickness:
$b_{footing} = 0.36\ m$</td></tr>
<tr><td colspan="2">MATERIALS
Concrete: C4000 Steel: A615G60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

Table 24: Wall materials and dimensions T3

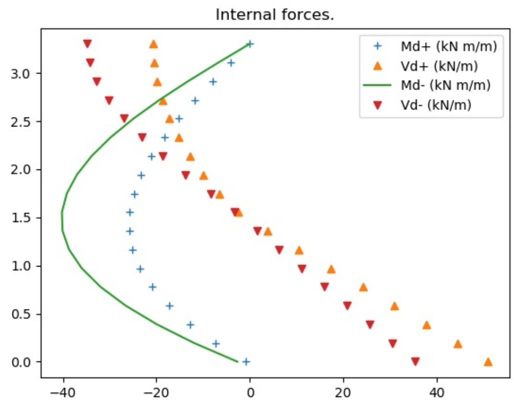
<table>
<tr><th colspan="2">T4</th></tr>
<tr><td>
</td><td>Wall geometry
Stem top thickness:
$b_{top} = 0.25\ m$
Stem height:
$h_{stem} = 3.12\ m$
Stem bottom thickness:
$b_{bottom} = 0.25\ m$
Footing thickness:
$b_{footing} = 0.36\ m$</td></tr>
<tr><td colspan="2">Materials
Concrete: C4000 Steel: A615G60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

Table 25: Wall materials and dimensions T4

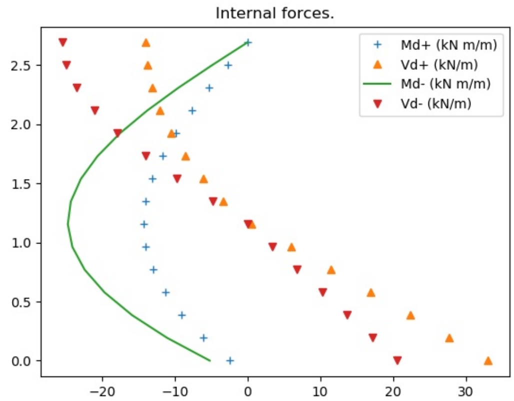
<table>
<tr><th colspan="2">T5</th></tr>
<tr><td>
</td><td>Wall geometry
Stem top thickness:
$b_{top} = 0.25\ m$
Stem height:
$h_{stem} = 2.51\ m$
Stem bottom thickness:
$b_{bottom} = 0.25\ m$
Footing thickness:
$b_{footing} = 0.36\ m$</td></tr>
<tr><td colspan="2">Materials
Concrete: C4000 Steel: A615G60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

Table 26: Wall materials and dimensions T5

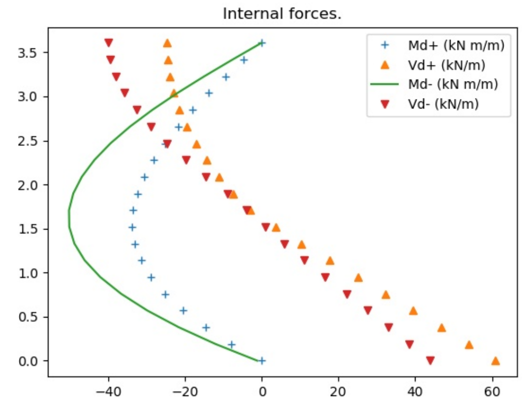
<table>
<tr><th colspan="2">T6</th></tr>
<tr><td>
</td>
<td>WALL GEOMETRY
Stem top thickness:
$b_{top} = 0.25\ m$
Stem height:
$h_{stem} = 3.43\ m$
Stem bottom thickness:
$b_{bottom} = 0.25\ m$
Footing thickness:
$b_{footing} = 0.36\ m$</td></tr>
<tr><td colspan="2">MATERIALS
Concrete: C4000 Steel: A615G60 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. 30)
	Lunif: uniform live load (see fig. 31)
	LconcSpan1: concentrated live load on mid-span 1 (see fig. 32)
	LconcSpan2: concentrated live load on mid-span 2 (see fig. 33)
	LconcSpan3: concentrated live load on mid-span 3 (see fig. 34)
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. 29) analyzed using XC

9.4.2 Acceptance criteria

Figure 28 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. 35 to 37 for the normal stresses check, and figs. 38 to 41 for the shear check. In every case, all the elements have a demand to capacity ratio of 1.0 or less.

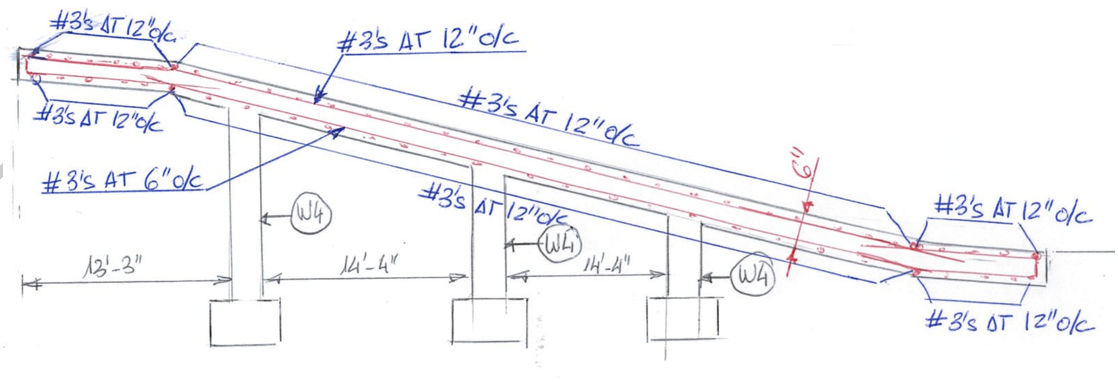


Figure 28: Ramp. Reinforcing layout.

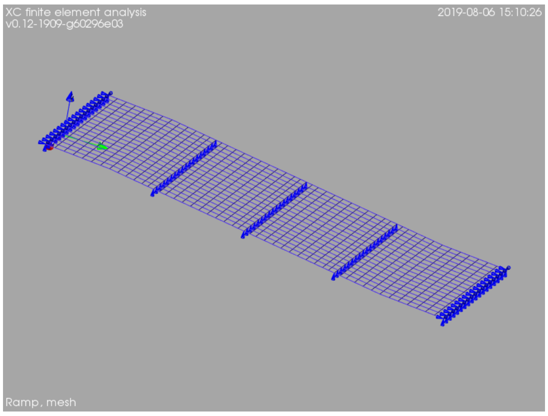


Figure 29: Ramp elastic model, mesh.

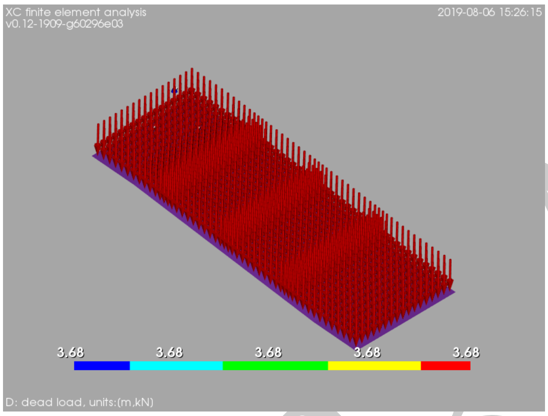


Figure 30: Load case D: dead load (include slab self-weight) [units: kN/m^2].

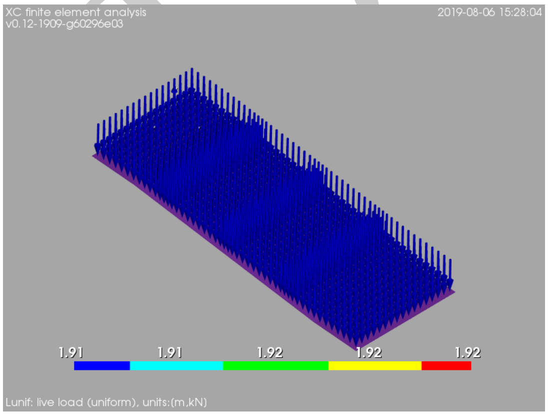


Figure 31: Load case Lv_{unif}: uniform live load (vehicles) [units: kN/m^2].

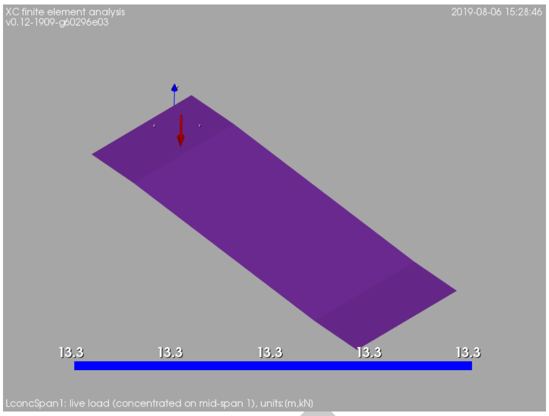


Figure 32: Load case $\text{Lv}_{conc,s1}$: concentrated live load (vehicles) on mid-span 1 [units: kN].

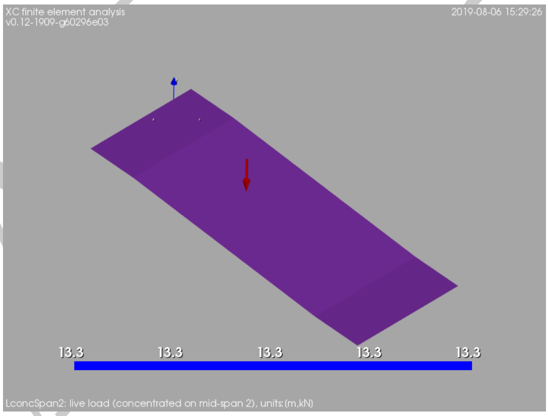


Figure 33: Load case $\text{Lv}_{conc,s2}$: concentrated live load (vehicles) on mid-span 2 [units: kN].

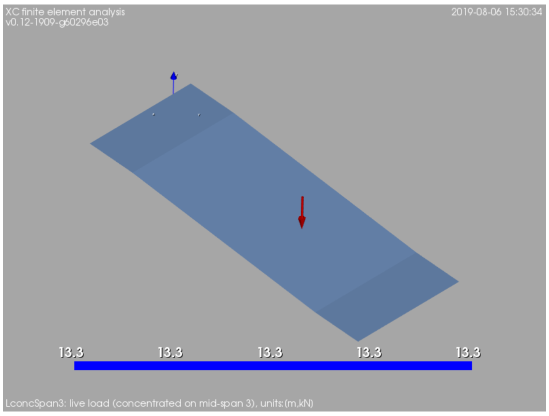


Figure 34: Load case $\text{Lv}_{conc,s3}$: concentrated live load (vehicles) on mid-span 3 [units: kN].

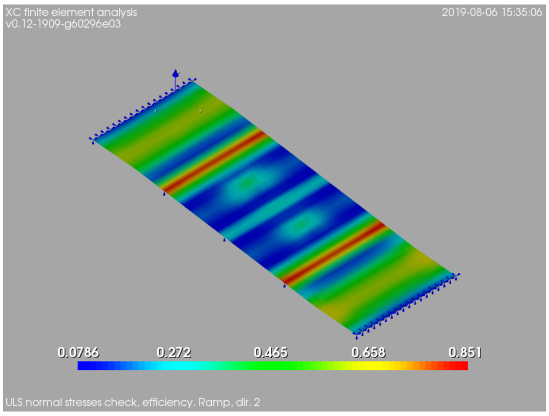


Figure 35: ULS normal stresses check. Efficiency in longitudinal direction

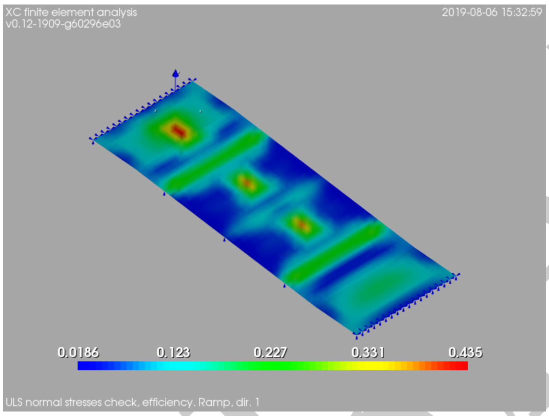


Figure 36: ULS normal stresses check. Efficiency in transversal direction

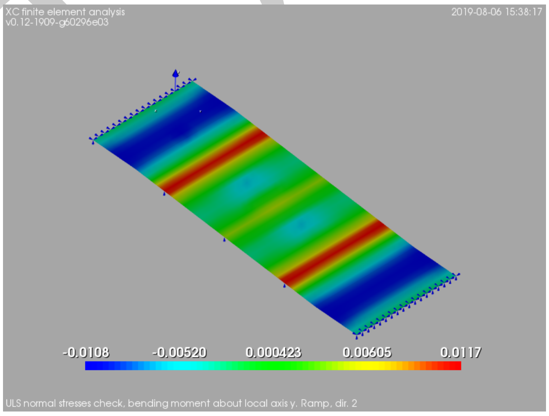


Figure 37: ULS normal stresses check. Bending moment in longitudinal direction [units: kN.m].

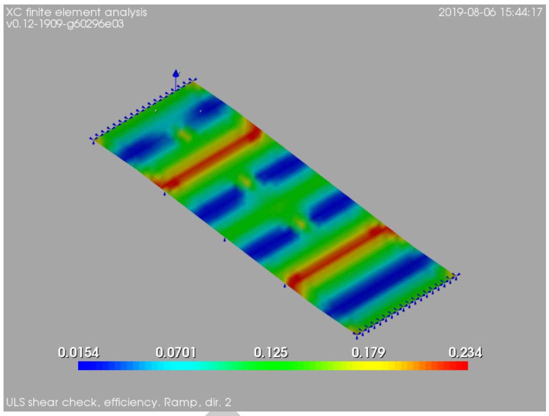


Figure 38: ULS shear check. Efficiency in longitudinal direction

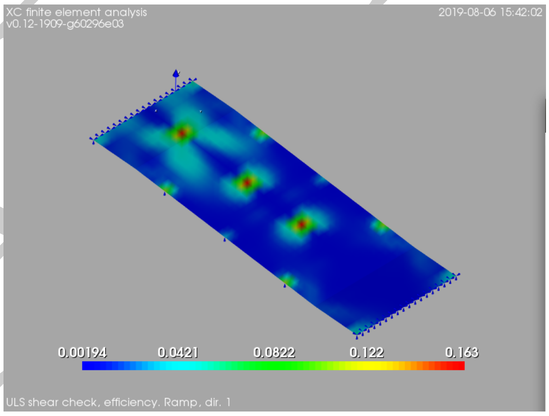


Figure 39: ULS shear check. Efficiency in transversal direction

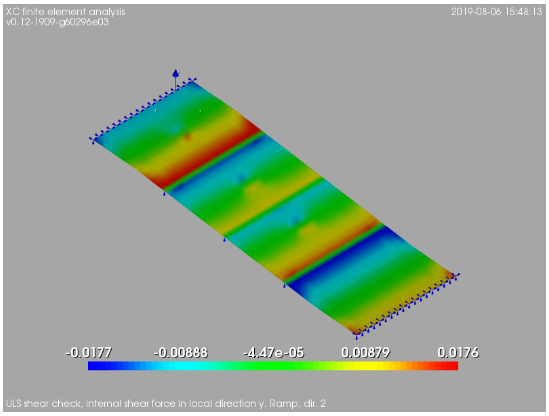


Figure 40: ULS shear check. Internal shear force in longitudinal direction [units: kN].

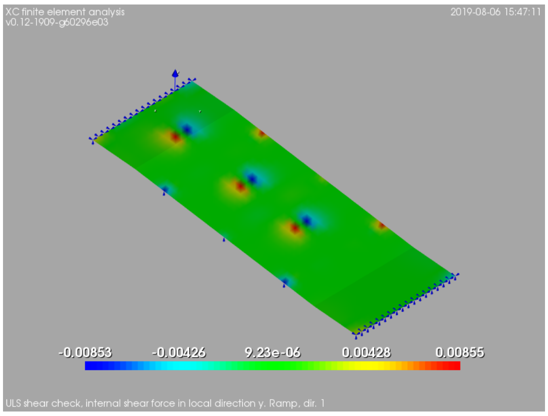


Figure 41: ULS shear check. Internal shear force in transversal direction [units: kN].

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*

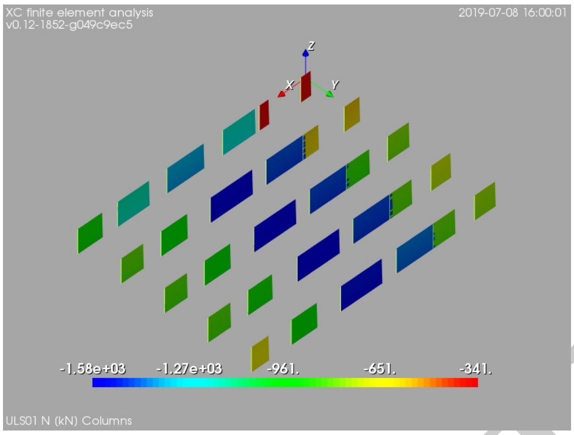


Figure 42: ULS01: 1.4*D. Columns, internal axial force [kN]

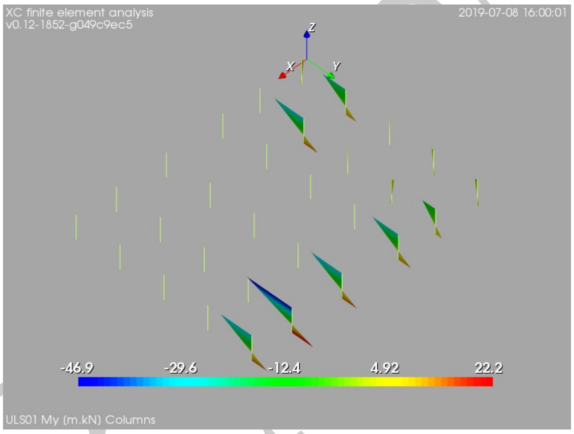


Figure 43: ULS01: 1.4*D. Columns, bending moment about local axis y [m.kN]

B Calculation results. Internal forces on columns

B.1 Ultimate limit states

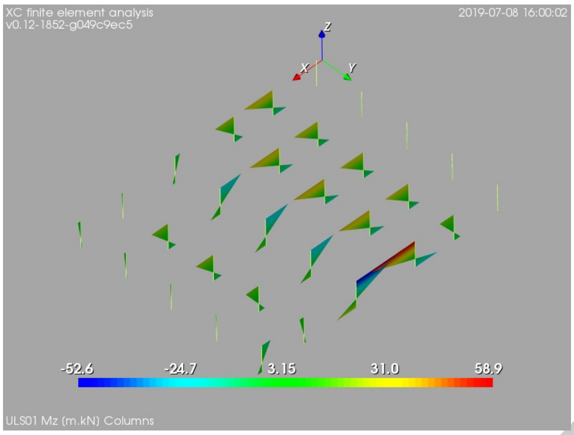


Figure 44: ULS01: 1.4*D. Columns, bending moment about local axis z [m.kN]

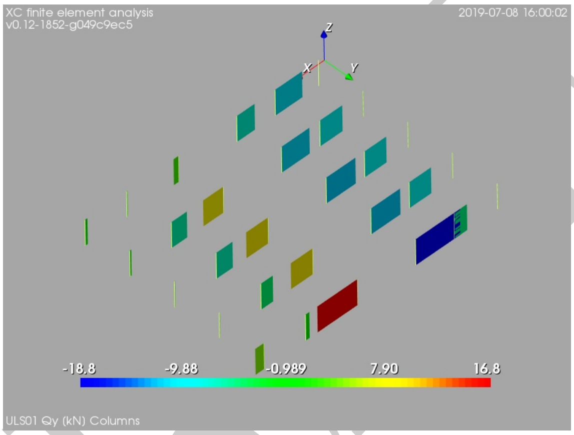


Figure 45: ULS01: 1.4*D. Columns, internal shear force in local direction y [kN]

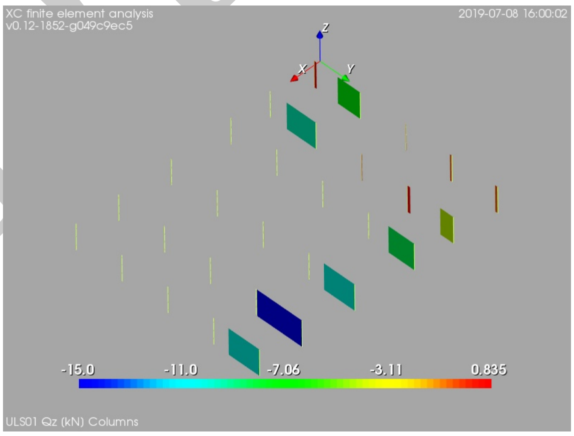


Figure 46: ULS01: 1.4*D. Columns, internal shear force in local direction z [kN]

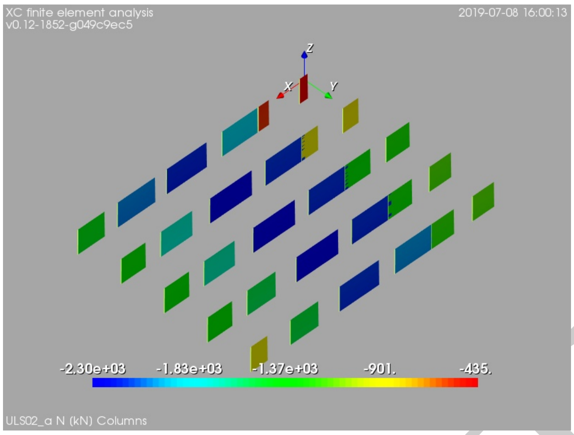


Figure 47: ULS02_a: 1.2*D + 1.6*Lru + Lpu + 0.5*S. Columns, internal axial force [kN]

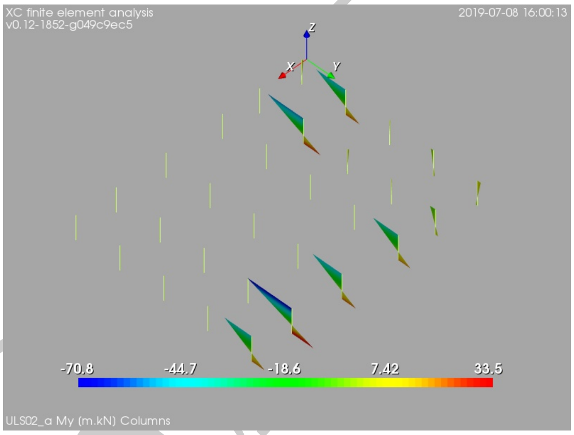


Figure 48: ULS02_a: 1.2*D + 1.6*Lru + Lpu + 0.5*S. Columns, bending moment about local axis y [m.kN]

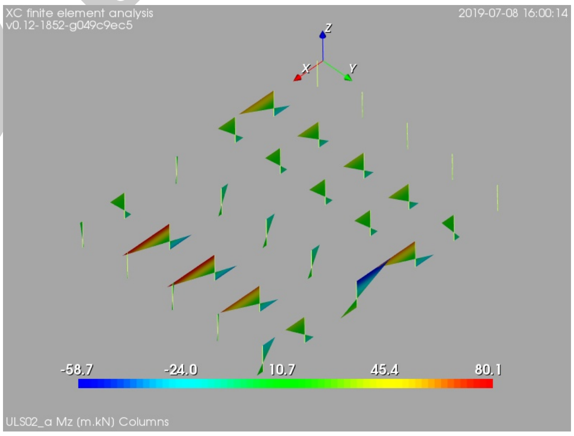


Figure 49: ULS02_a: 1.2*D + 1.6*Lru + Lpu + 0.5*S. Columns, bending moment about local axis z [m.kN]

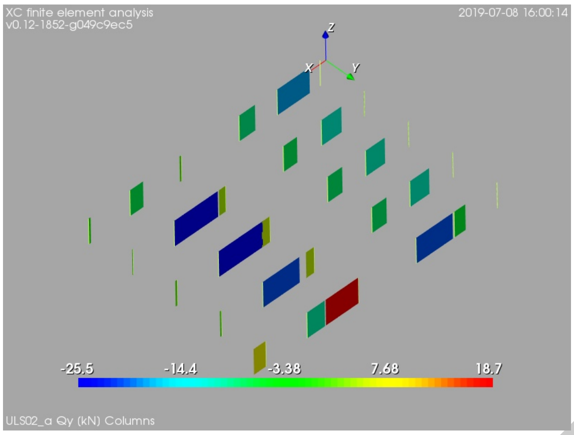


Figure 50: ULS02_a: 1.2*D + 1.6*Lru + Lpu + 0.5*S. Columns, internal shear force in local direction y [kN]

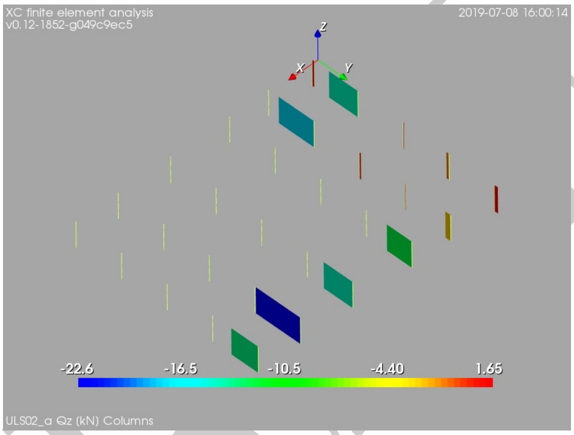


Figure 51: ULS02_a: 1.2*D + 1.6*Lru + Lpu + 0.5*S. Columns, internal shear force in local direction z [kN]

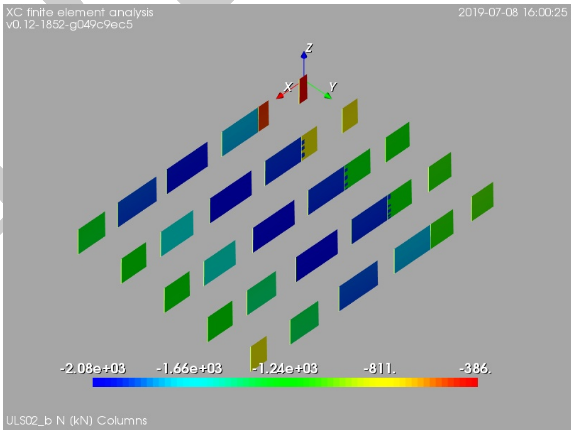


Figure 52: ULS02_b: 1.2*D + 1.6*Lrs + Lps + 0.5*S. Columns, internal axial force [kN]

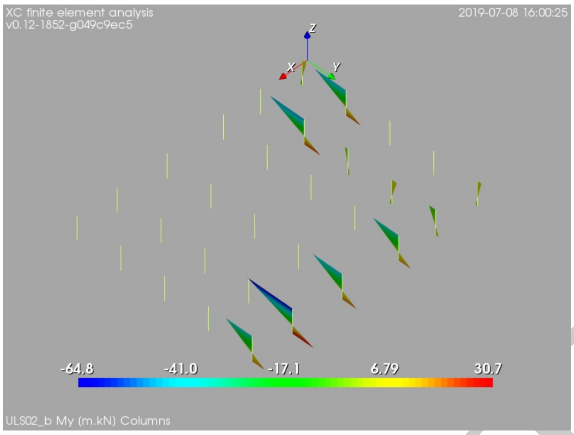


Figure 53: ULS02_b: 1.2*D + 1.6*Lrs + Lps + 0.5*S. Columns, bending moment about local axis y [m.kN]

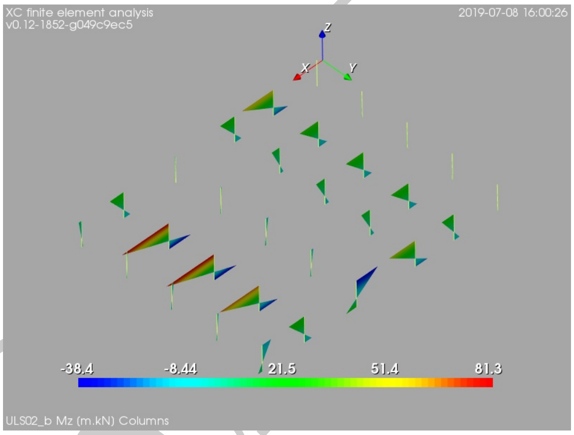


Figure 54: ULS02_b: 1.2*D + 1.6*Lrs + Lps + 0.5*S. Columns, bending moment about local axis z [m.kN]

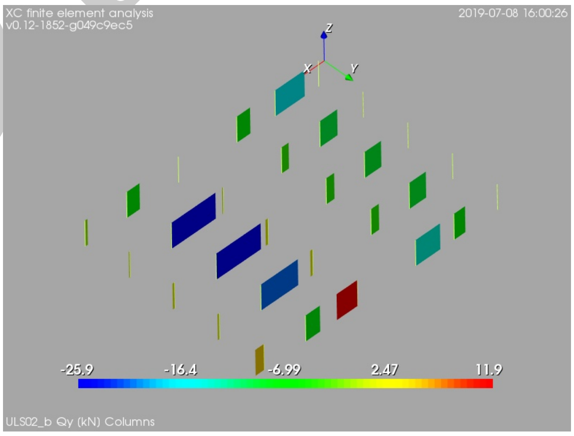


Figure 55: ULS02_b: 1.2*D + 1.6*Lrs + Lps + 0.5*S. Columns, internal shear force in local direction y [kN]

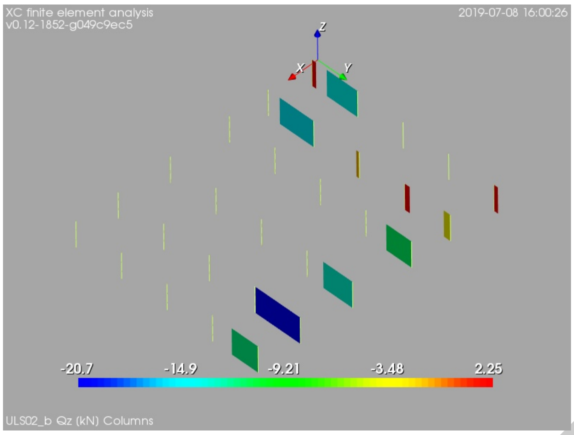


Figure 56: ULS02_b: 1.2*D + 1.6*Lrs + Lps + 0.5*S. Columns, internal shear force in local direction z [kN]

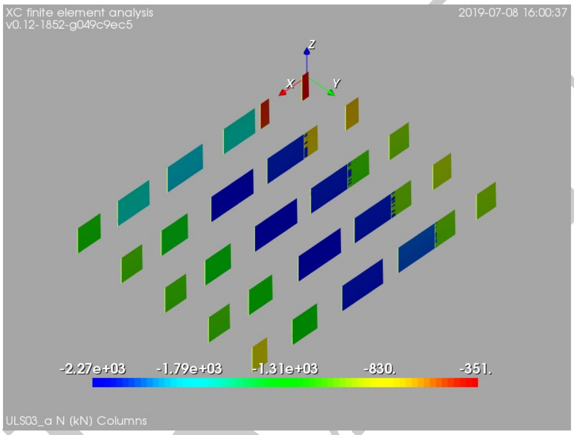


Figure 57: ULS03_a: 1.2*D + 1.6*S + 0.5*Lru + Lpu. Columns, internal axial force [kN]

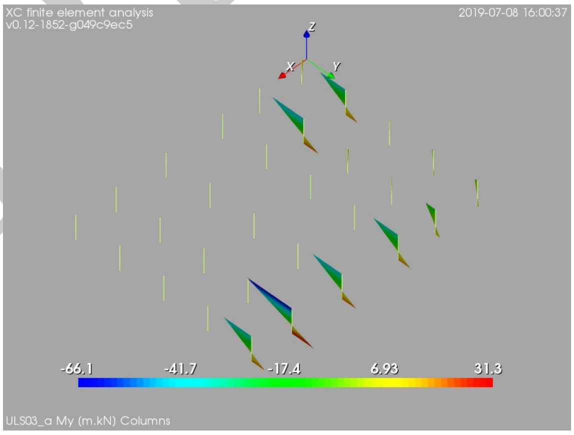


Figure 58: ULS03_a: 1.2*D + 1.6*S + 0.5*Lru + Lpu. Columns, bending moment about local axis y [m.kN]

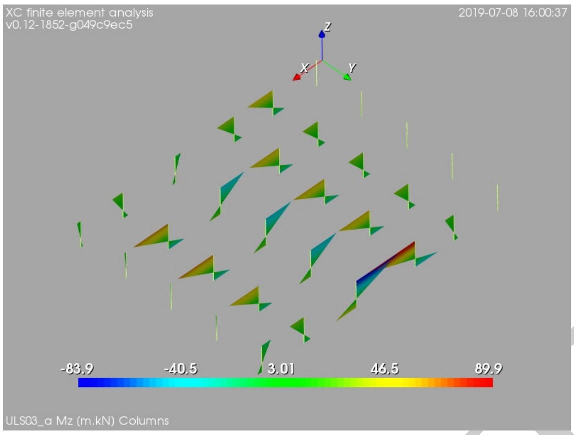


Figure 59: ULS03_a: 1.2*D + 1.6*S + 0.5*Lru + Lpu. Columns, bending moment about local axis z [m.kN]

Figure 60: ULS03_a: 1.2*D + 1.6*S + 0.5*Lru + Lpu. Columns, internal shear force in local direction y [kN]

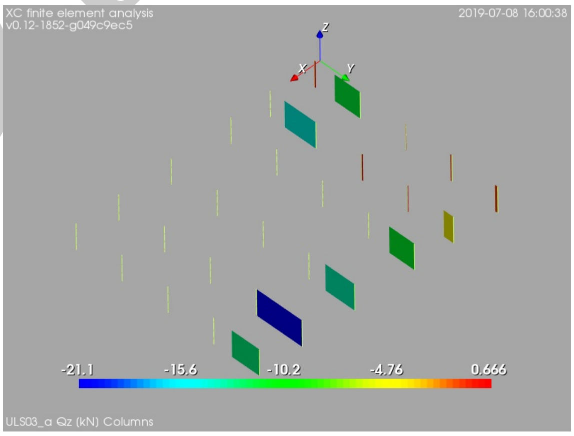


Figure 61: ULS03_a: 1.2*D + 1.6*S + 0.5*Lru + Lpu. Columns, internal shear force in local direction z [kN]

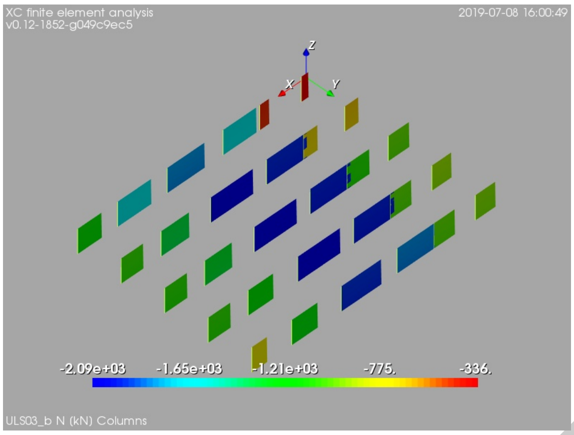


Figure 62: ULS03_b: 1.2*D + 1.6*S + 0.5*Lrs + Lps. Columns, internal axial force [kN]

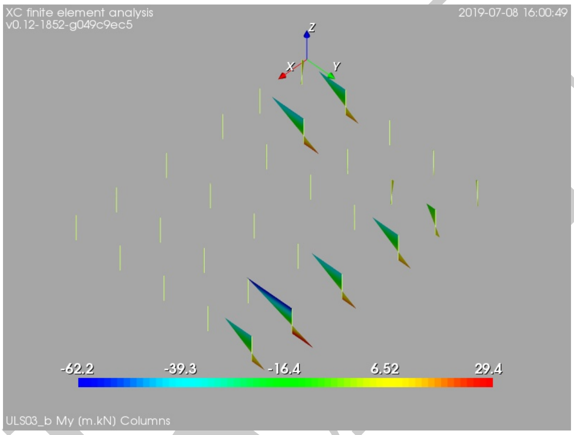


Figure 63: ULS03_b: 1.2*D + 1.6*S + 0.5*Lrs + Lps. Columns, bending moment about local axis y [m.kN]

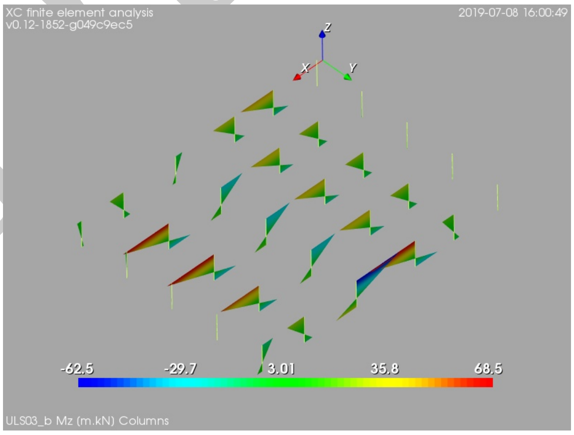


Figure 64: ULS03_b: 1.2*D + 1.6*S + 0.5*Lrs + Lps. Columns, bending moment about local axis z [m.kN]

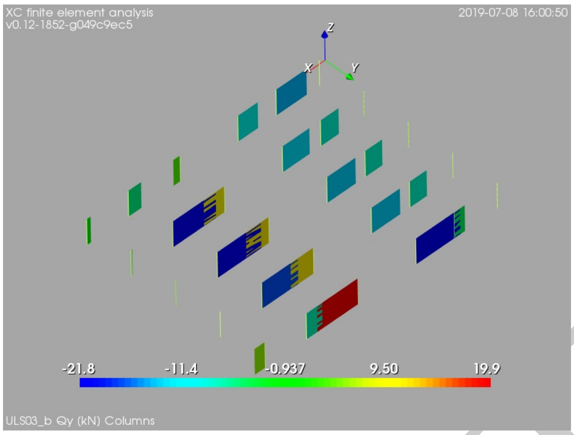


Figure 65: ULS03_b: 1.2*D + 1.6*S + 0.5*Lrs + Lps. Columns, internal shear force in local direction y [kN]

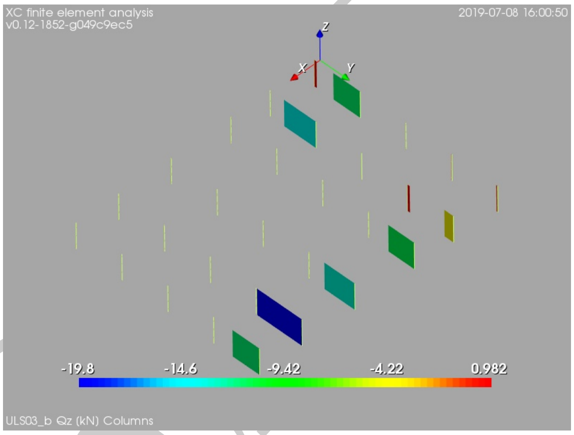


Figure 66: ULS03_b: 1.2*D + 1.6*S + 0.5*Lrs + Lps. Columns, internal shear force in local direction z [kN]

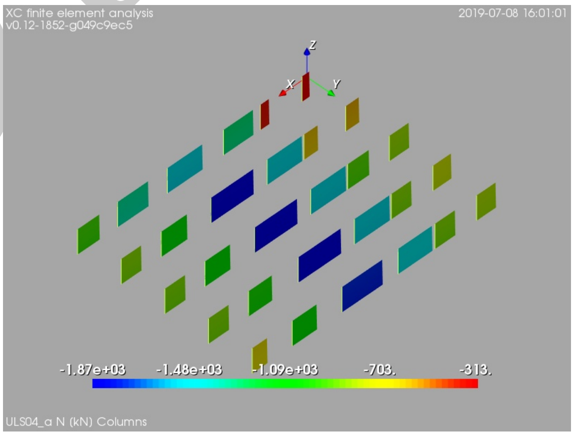


Figure 67: ULS04_a: 1.2*D + 1.6*S + 0.5*W_WE. Columns, internal axial force [kN]

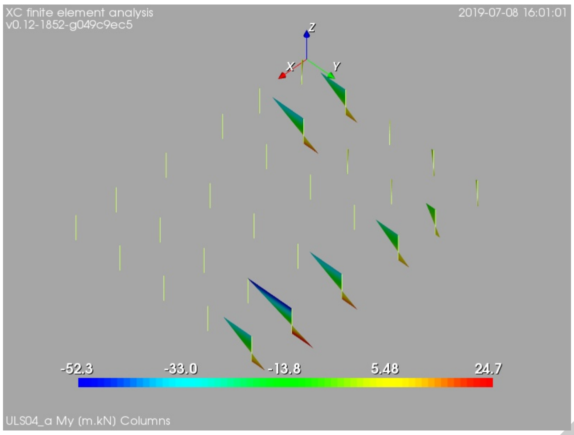


Figure 68: ULS04_a: 1.2*D + 1.6*S + 0.5*W_WE. Columns, bending moment about local axis y [m.kN]

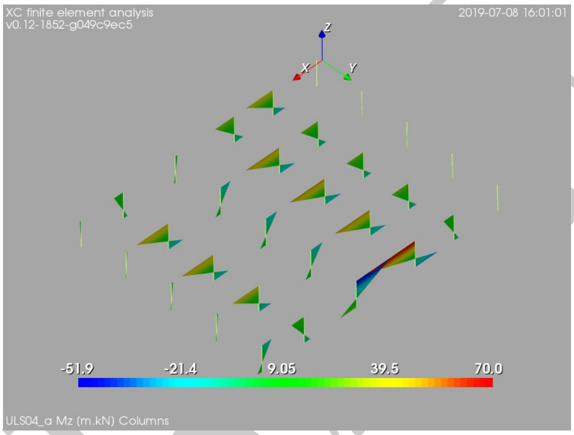


Figure 69: ULS04_a: 1.2*D + 1.6*S + 0.5*W_WE. Columns, bending moment about local axis z [m.kN]

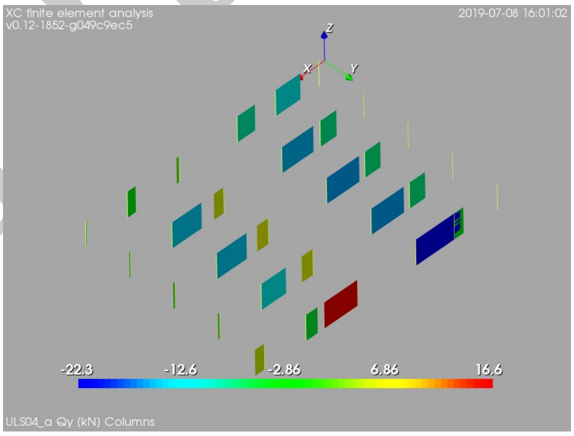


Figure 70: ULS04_a: 1.2*D + 1.6*S + 0.5*W_WE. Columns, internal shear force in local direction y [kN]

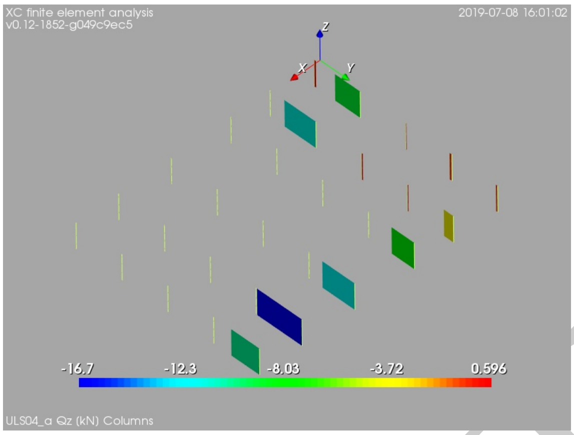


Figure 71: ULS04_a: 1.2*D + 1.6*S + 0.5*W_WE. Columns, internal shear force in local direction z [kN]

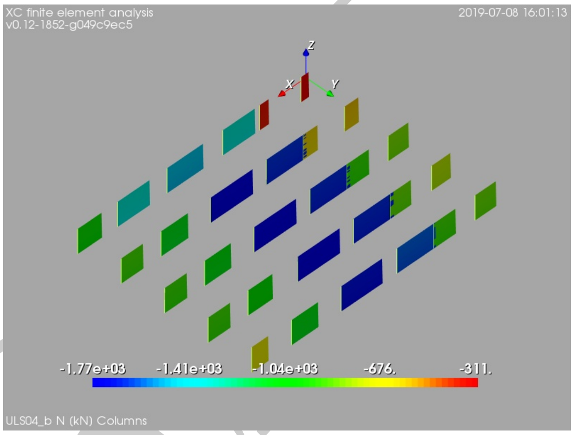


Figure 72: ULS04_b: 1.2*D + 1.6*S + 0.5*W_NS. Columns, internal axial force [kN]

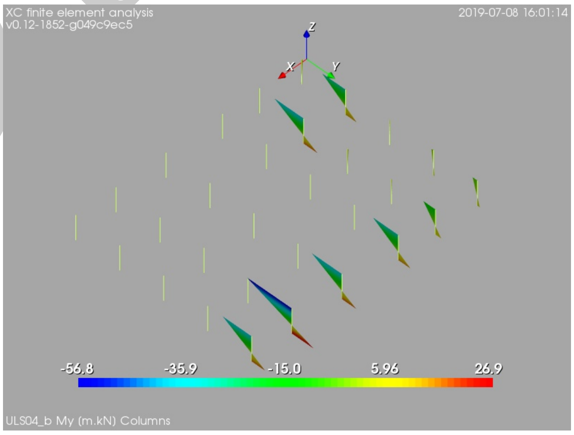


Figure 73: ULS04_b: 1.2*D + 1.6*S + 0.5*W_NS. Columns, bending moment about local axis y [m.kN]

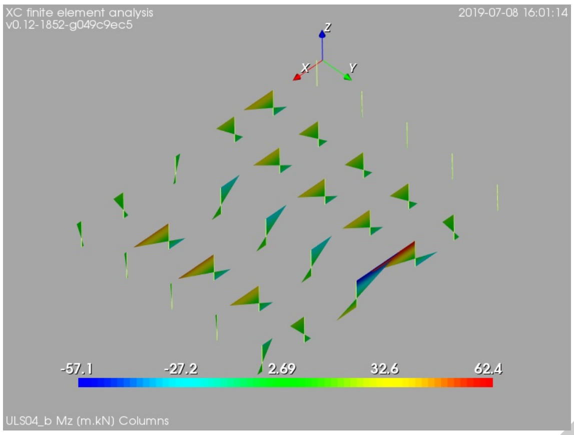


Figure 74: ULS04_b: 1.2*D + 1.6*S + 0.5*W_NS. Columns, bending moment about local axis z [m.kN]

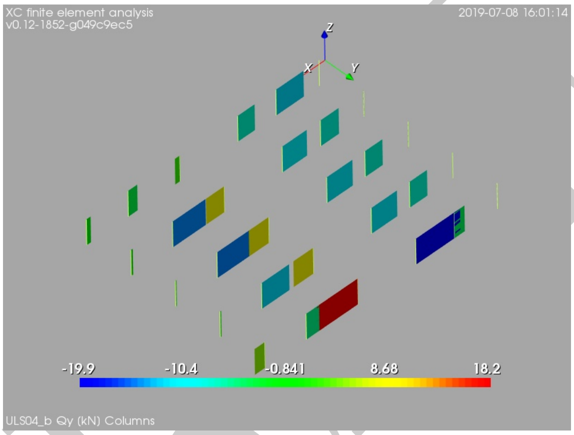


Figure 75: ULS04_b: 1.2*D + 1.6*S + 0.5*W_NS. Columns, internal shear force in local direction y [kN]

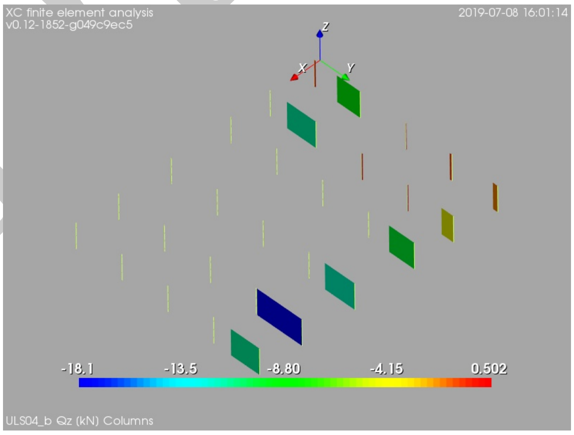


Figure 76: ULS04_b: 1.2*D + 1.6*S + 0.5*W_NS. Columns, internal shear force in local direction z [kN]

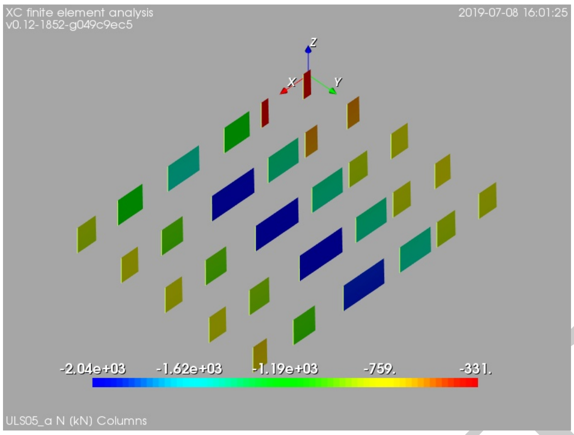


Figure 77: ULS05_a: 1.2*D + W_WE + 0.5*Lru + Lpu. Columns, internal axial force [kN]

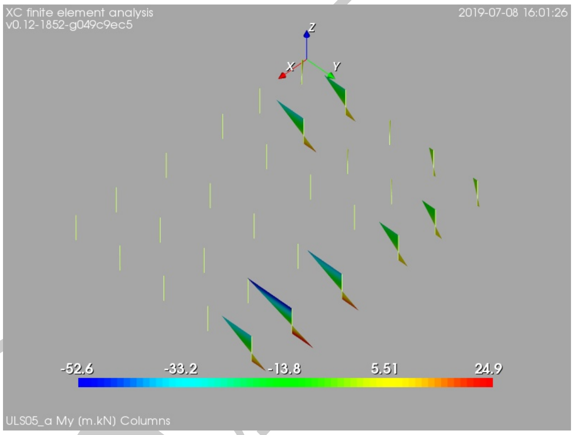


Figure 78: ULS05_a: 1.2*D + W_WE + 0.5*Lru + Lpu. Columns, bending moment about local axis y [m.kN]

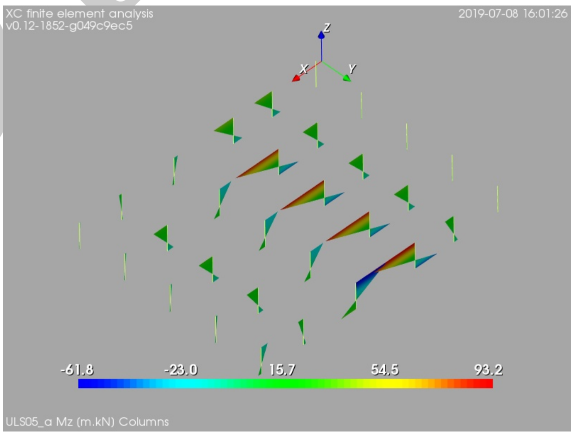


Figure 79: ULS05_a: 1.2*D + W_WE + 0.5*Lru + Lpu. Columns, bending moment about local axis z [m.kN]

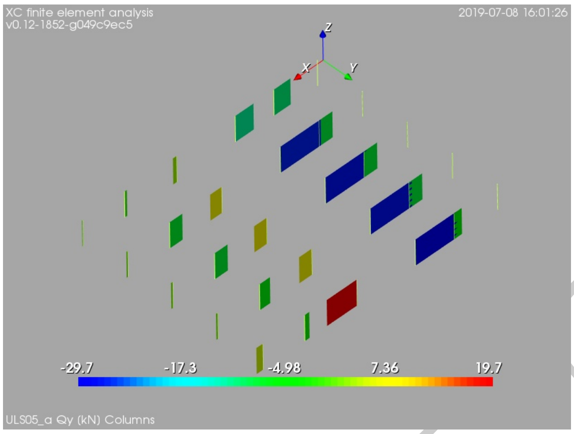


Figure 80: ULS05_a: 1.2*D + W_WE + 0.5*Lru + Lpu. Columns, internal shear force in local direction y [kN]

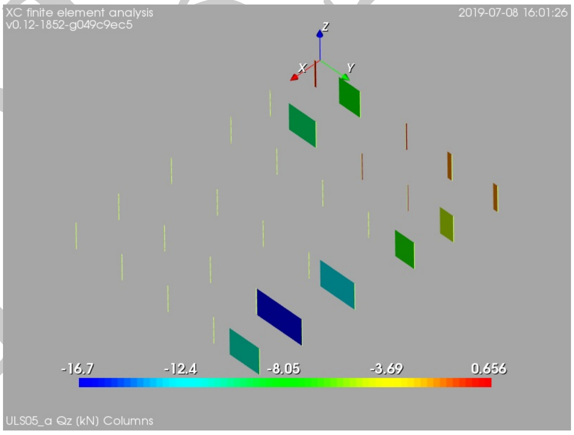


Figure 81: ULS05_a: 1.2*D + W_WE + 0.5*Lru + Lpu. Columns, internal shear force in local direction z [kN]

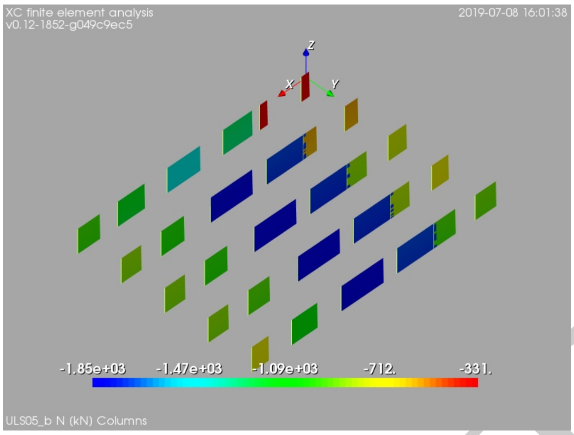


Figure 82: ULS05_b: 1.2*D + W_NS + 0.5*Lru + Lpu. Columns, internal axial force [kN]

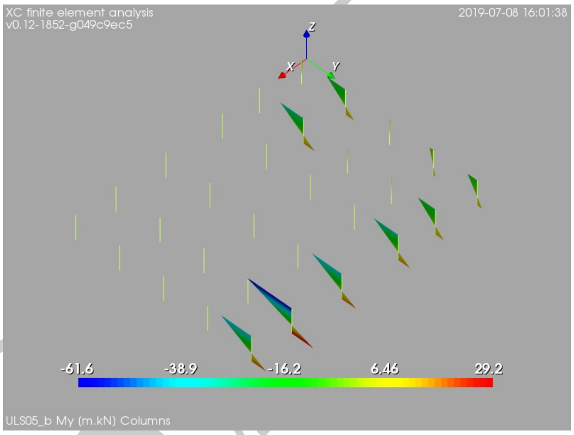


Figure 83: ULS05_b: 1.2*D + W_NS + 0.5*Lru + Lpu. Columns, bending moment about local axis y [m.kN]

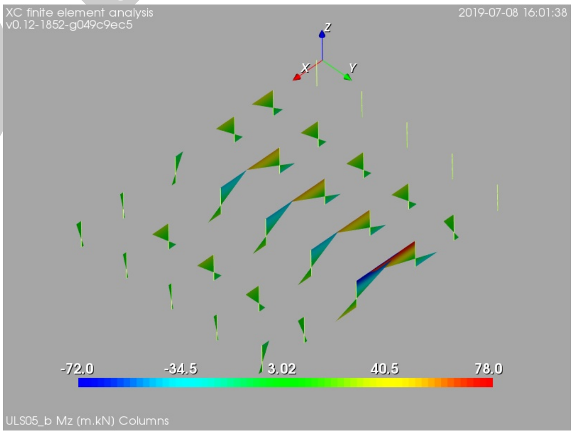


Figure 84: ULS05_b: 1.2*D + W_NS + 0.5*Lru + Lpu. Columns, bending moment about local axis z [m.kN]

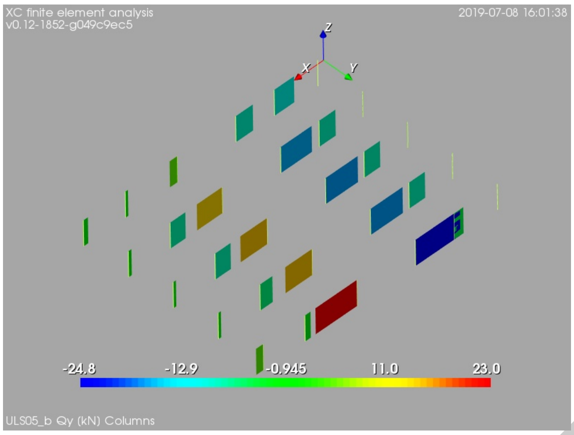


Figure 85: ULS05_b: 1.2*D + W_NS + 0.5*Lru + Lpu. Columns, internal shear force in local direction y [kN]

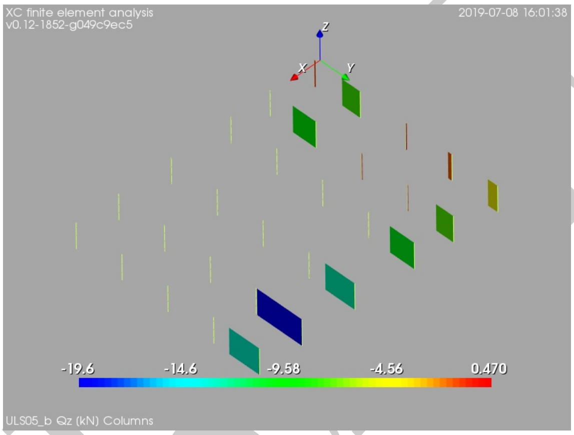


Figure 86: ULS05_b: 1.2*D + W_NS + 0.5*Lru + Lpu. Columns, internal shear force in local direction z [kN]

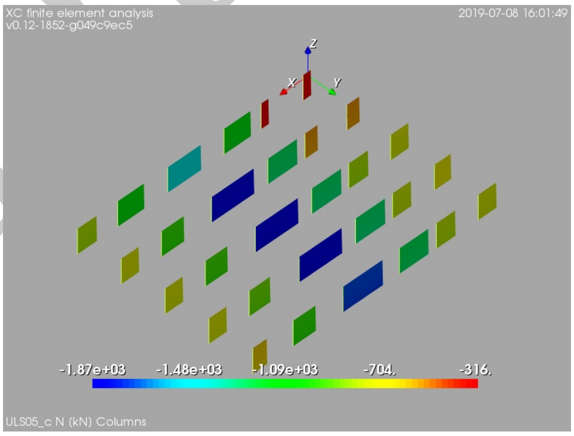


Figure 87: ULS05_c: 1.2*D + W_WE + 0.5*Lrs + Lps. Columns, internal axial force [kN]

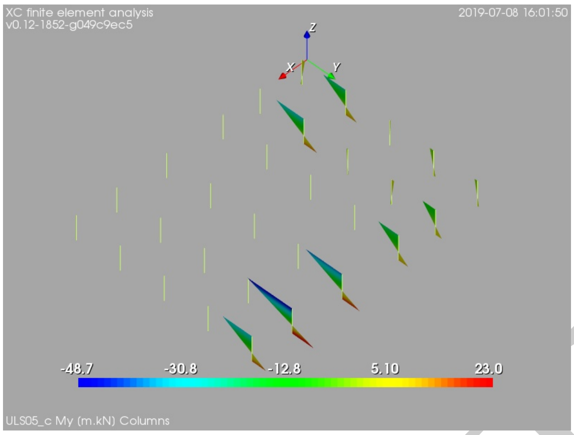


Figure 88: ULS05_c: 1.2*D + W_WE + 0.5*Lrs + Lps. Columns, bending moment about local axis y [m.kN]

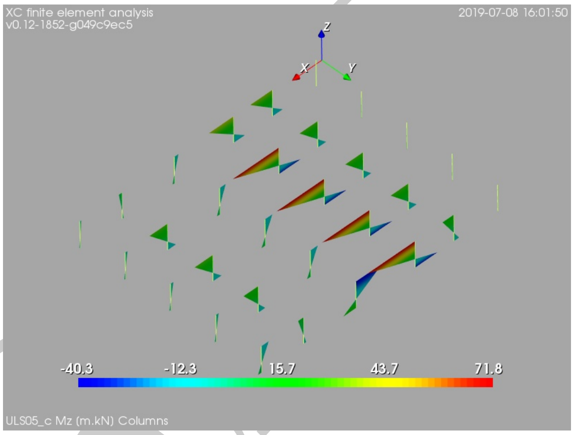


Figure 89: ULS05_c: 1.2*D + W_WE + 0.5*Lrs + Lps. Columns, bending moment about local axis z [m.kN]

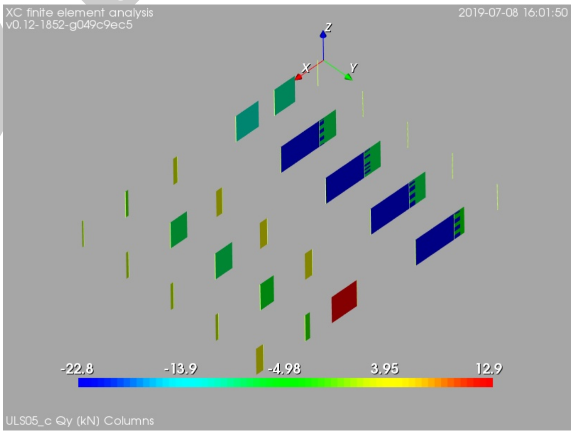


Figure 90: ULS05_c: 1.2*D + W_WE + 0.5*Lrs + Lps. Columns, internal shear force in local direction y [kN]

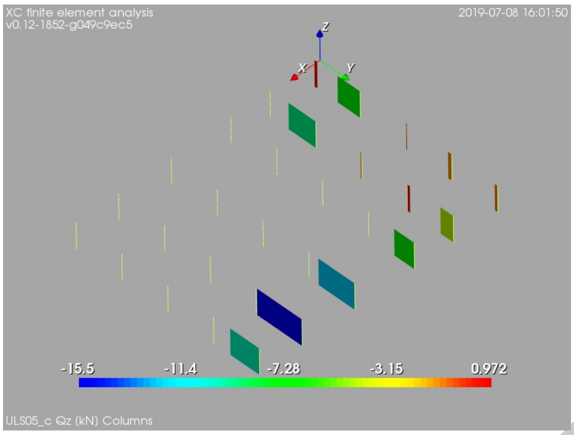


Figure 91: ULS05_c: 1.2*D + W_WE + 0.5*Lrs + Lps. Columns, internal shear force in local direction z [kN]

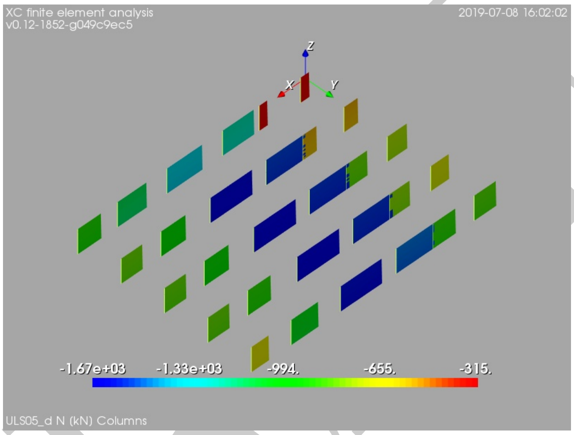


Figure 92: ULS05_d: 1.2*D + W_NS + 0.5*Lrs + Lps. Columns, internal axial force [kN]

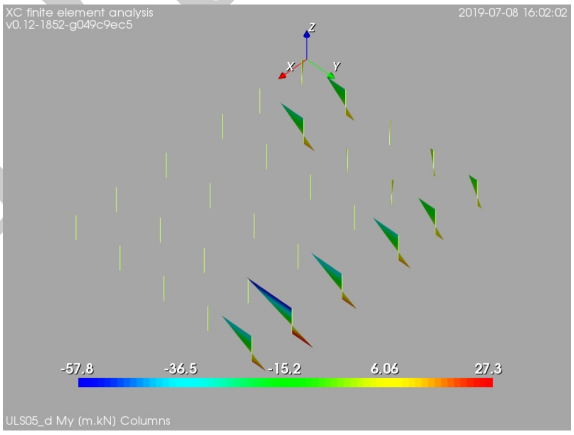


Figure 93: ULS05_d: 1.2*D + W_NS + 0.5*Lrs + Lps. Columns, bending moment about local axis y [m.kN]

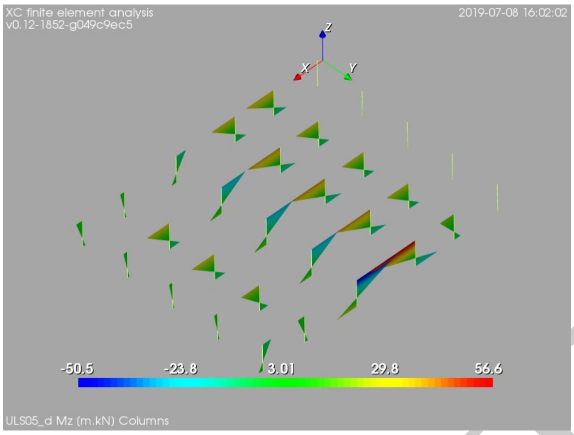


Figure 94: ULS05_d: 1.2*D + W_NS + 0.5*Lrs + Lps. Columns, bending moment about local axis z [m.kN]

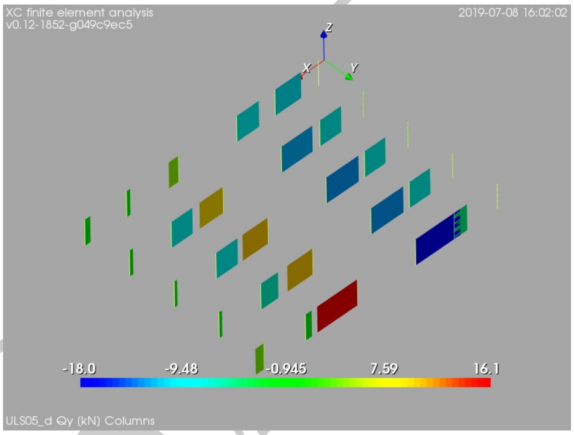


Figure 95: ULS05_d: 1.2*D + W_NS + 0.5*Lrs + Lps. Columns, internal shear force in local direction y [kN]

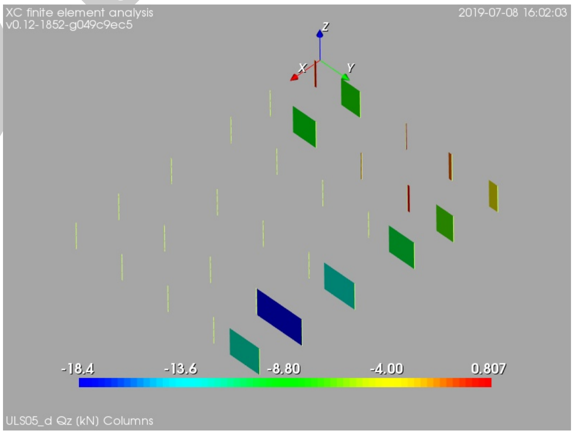


Figure 96: ULS05_d: 1.2*D + W_NS + 0.5*Lrs + Lps. Columns, internal shear force in local direction z [kN]

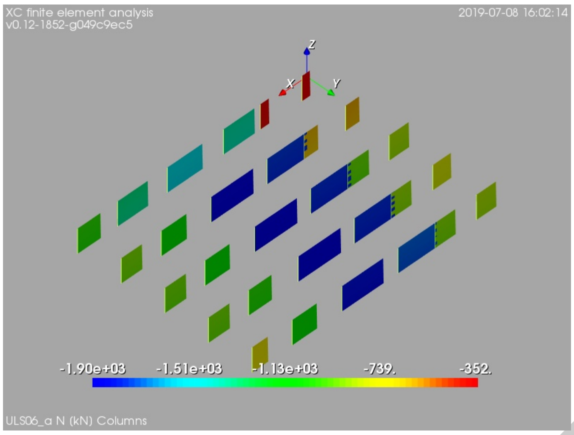


Figure 97: ULS06_a: 1.2*D + 0.5*Lru + Lpu + 0.2*S. Columns, internal axial force [kN]

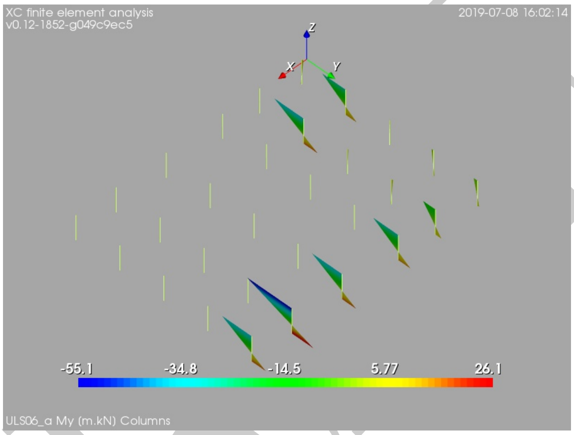


Figure 98: ULS06_a: 1.2*D + 0.5*Lru + Lpu + 0.2*S. Columns, bending moment about local axis y [m.kN]

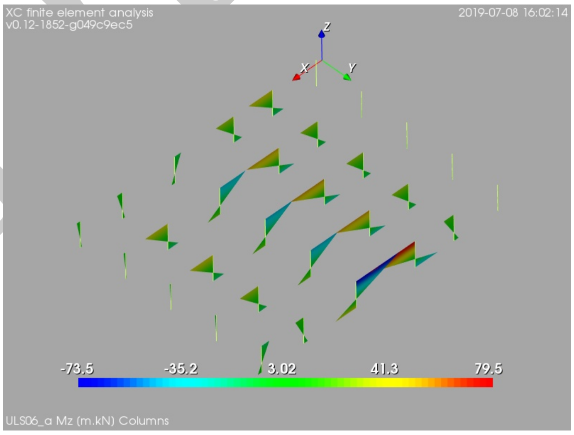


Figure 99: ULS06_a: 1.2*D + 0.5*Lru + Lpu + 0.2*S. Columns, bending moment about local axis z [m.kN]

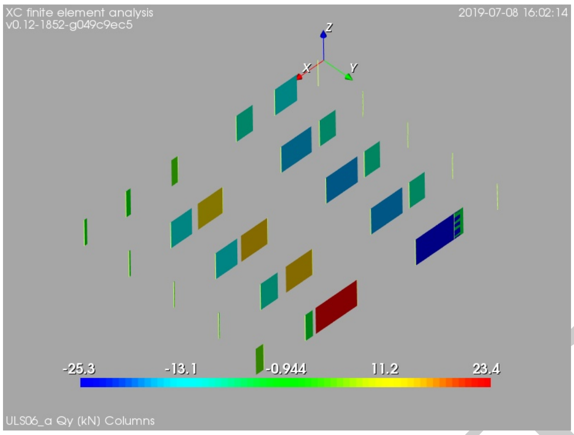


Figure 100: ULS06_a: 1.2*D + 0.5*Lru + Lpu + 0.2*S. Columns, internal shear force in local direction y [kN]

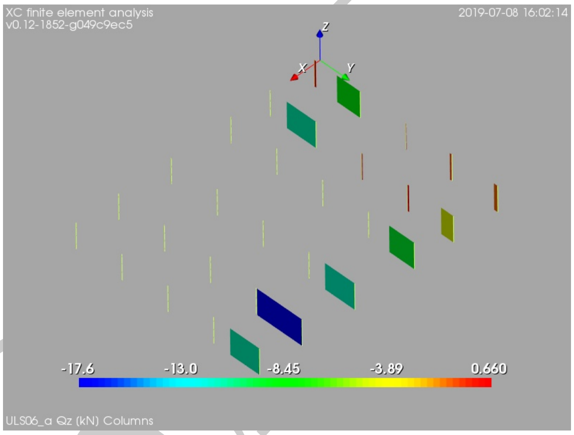


Figure 101: ULS06_a: 1.2*D + 0.5*Lru + Lpu + 0.2*S. Columns, internal shear force in local direction z [kN]

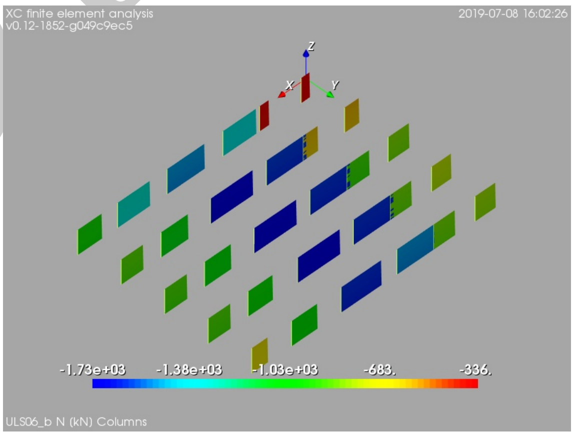


Figure 102: ULS06_b: 1.2*D + 0.5*Lrs + Lps + 0.2*S. Columns, internal axial force [kN]

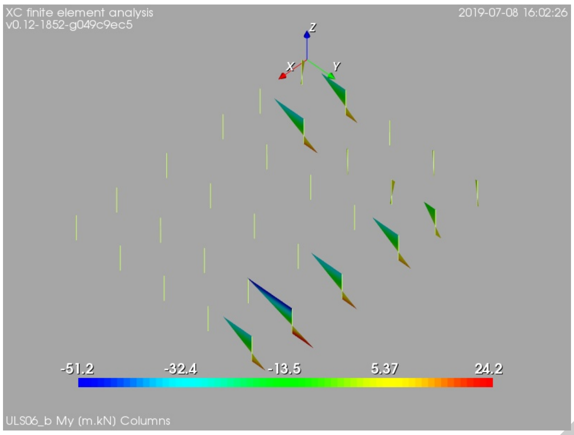


Figure 103: ULS06_b: 1.2*D + 0.5*Lrs + Lps + 0.2*S. Columns, bending moment about local axis y [m.kN]

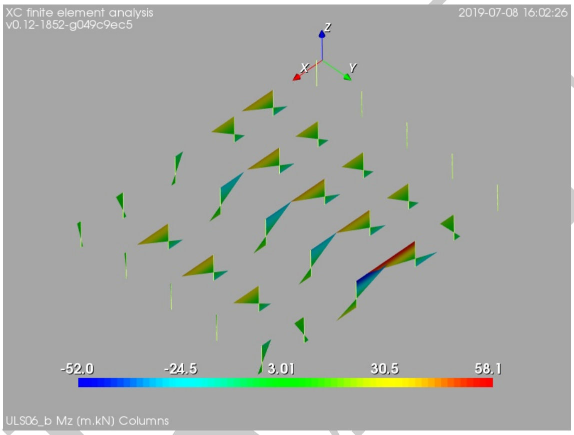


Figure 104: ULS06_b: 1.2*D + 0.5*Lrs + Lps + 0.2*S. Columns, bending moment about local axis z [m.kN]

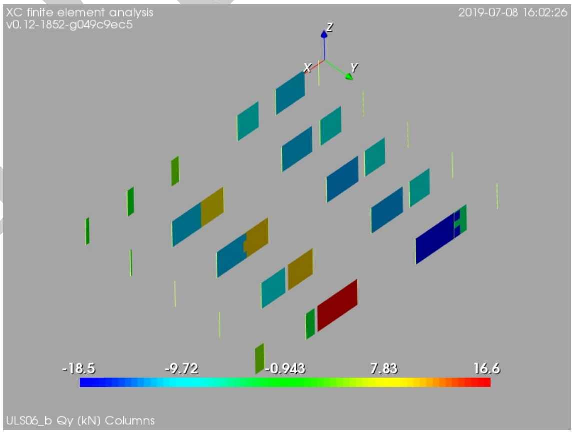


Figure 105: ULS06_b: 1.2*D + 0.5*Lrs + Lps + 0.2*S. Columns, internal shear force in local direction y [kN]

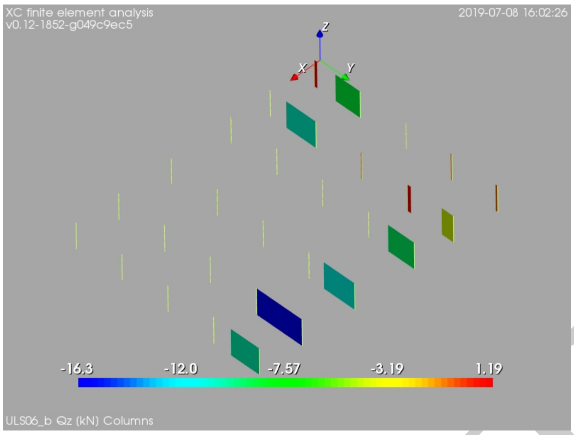


Figure 106: ULS06_b: 1.2*D + 0.5*Lrs + Lps + 0.2*S. Columns, internal shear force in local direction z [kN]

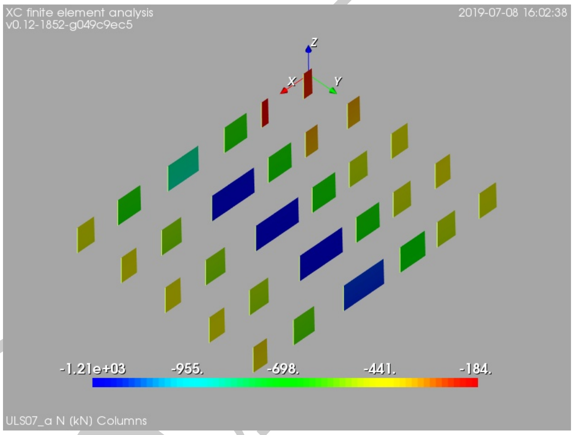


Figure 107: ULS07_a: 0.9*D + W_WE. Columns, internal axial force [kN]

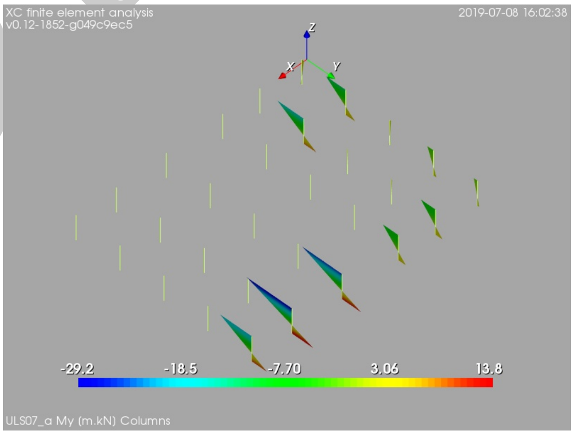


Figure 108: ULS07_a: 0.9*D + W_WE. Columns, bending moment about local axis y [m.kN]

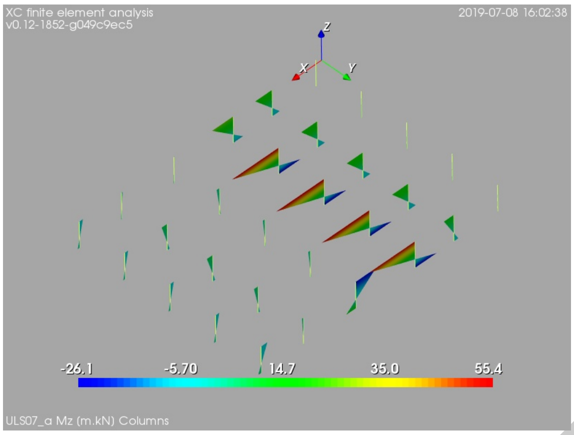


Figure 109: ULS07_a: 0.9*D + W_WE. Columns, bending moment about local axis z [m.kN]

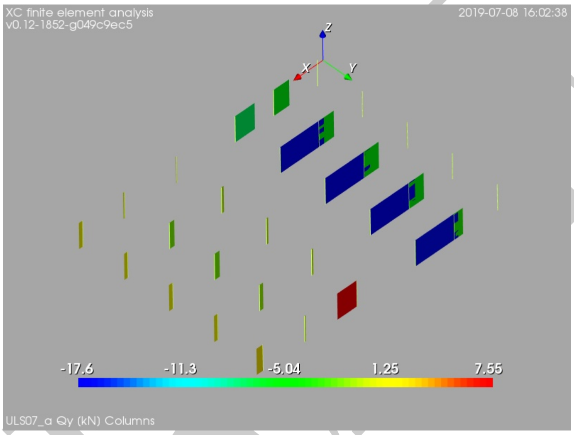


Figure 110: ULS07_a: 0.9*D + W_WE. Columns, internal shear force in local direction y [kN]

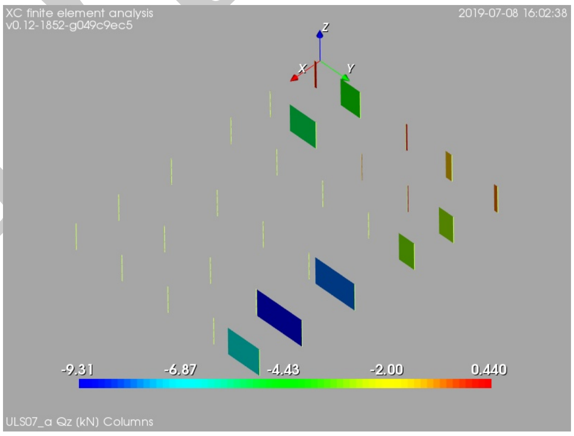


Figure 111: ULS07_a: 0.9*D + W_WE. Columns, internal shear force in local direction z [kN]

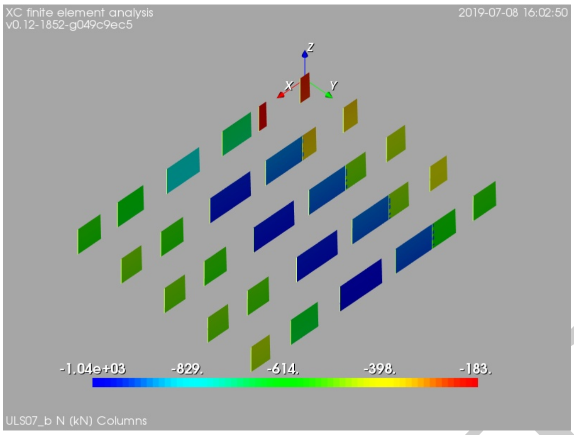


Figure 112: ULS07_b: 0.9*D + W_NS. Columns, internal axial force [kN]

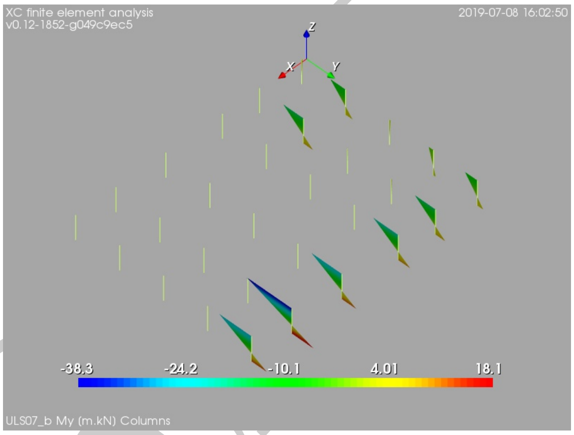


Figure 113: ULS07_b: 0.9*D + W_NS. Columns, bending moment about local axis y [m.kN]

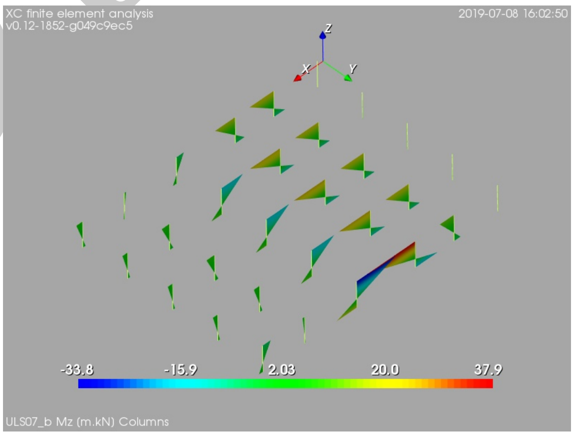


Figure 114: ULS07_b: 0.9*D + W_NS. Columns, bending moment about local axis z [m.kN]

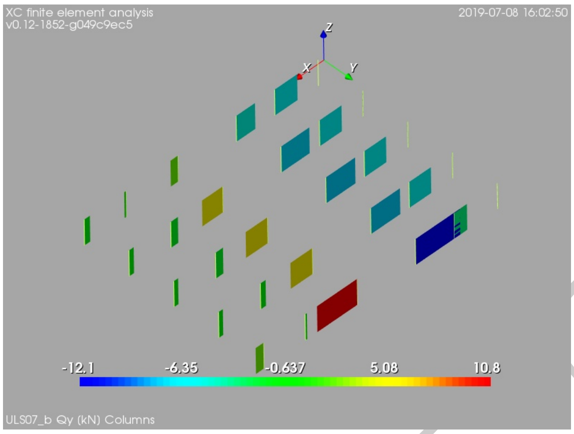


Figure 115: ULS07_b: 0.9*D + W_NS. Columns, internal shear force in local direction y [kN]

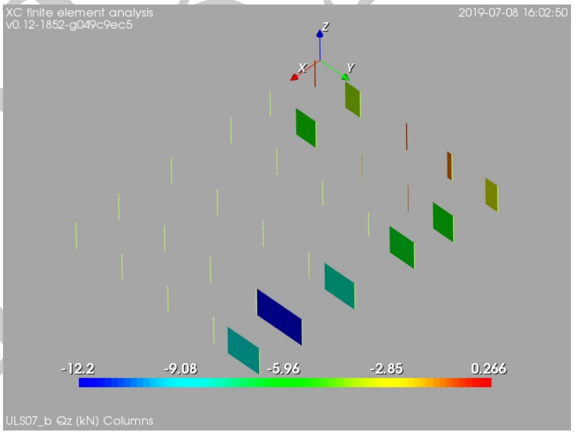


Figure 116: ULS07_b: 0.9*D + W_NS. Columns, internal shear force in local direction z [kN]

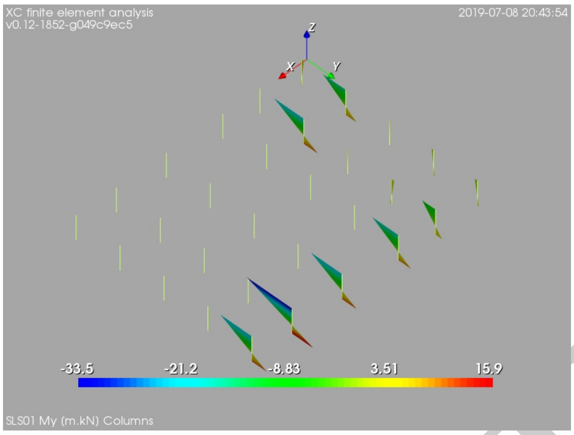


Figure 117: SLS01: 1.0*D. Columns, bending moment about local axis y [m.kN]

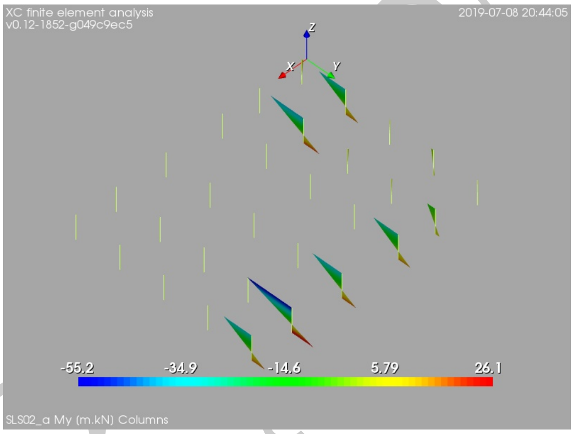


Figure 118: SLS02_a: 1.0*D + 1.0*Lru + Lpu + 0.3*S. Columns, bending moment about local axis y [m.kN]

B.2 Serviceability limit states

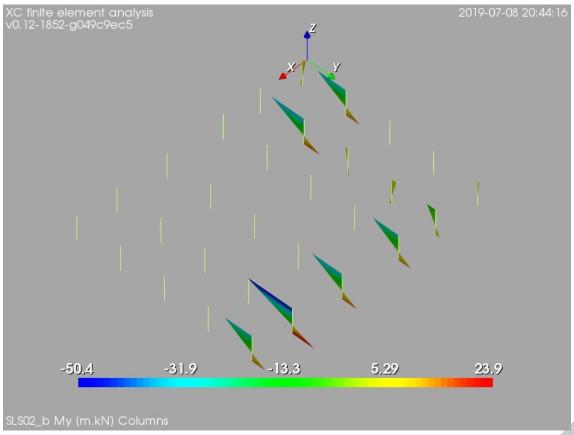


Figure 119: SLS02_b: 1.0*D + 1.0*Lrs + Lps + 0.3*S. Columns, bending moment about local axis y [m.kN]

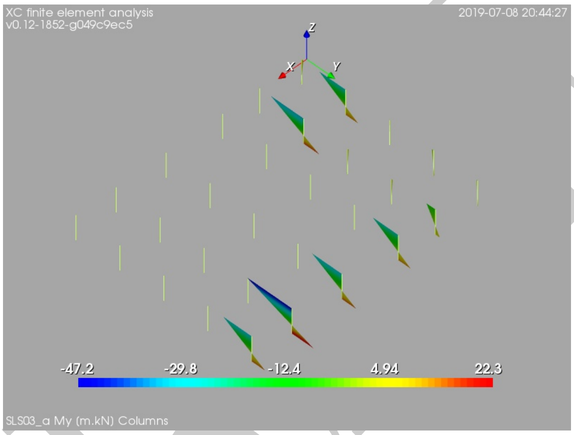


Figure 120: SLS03_a: 1.0*D + 1.0*S + 0.3*Lru + 0.3*Lpu. Columns, bending moment about local axis y [m.kN]

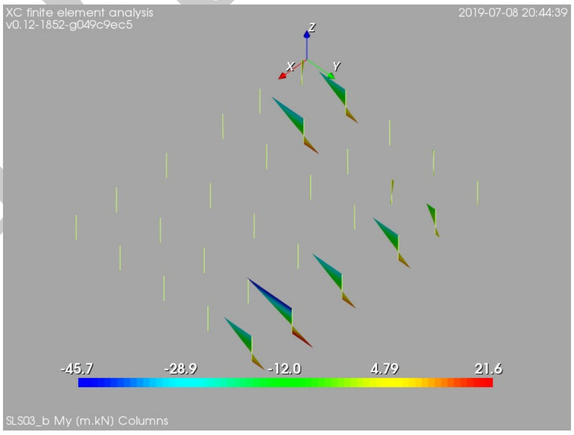


Figure 121: SLS03_b: 1.0*D + 1.0*S + 0.3*Lrs + 0.3*Lps. Columns, bending moment about local axis y [m.kN]

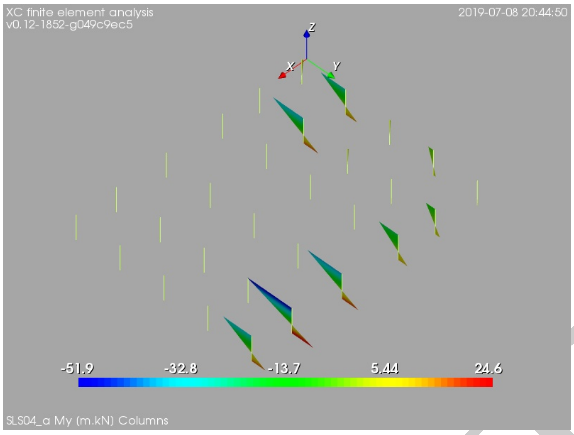


Figure 122: SLS04_a: 1.0*D + W_WE + 1.0*Lru + Lpu. Columns, bending moment about local axis y [m.kN]

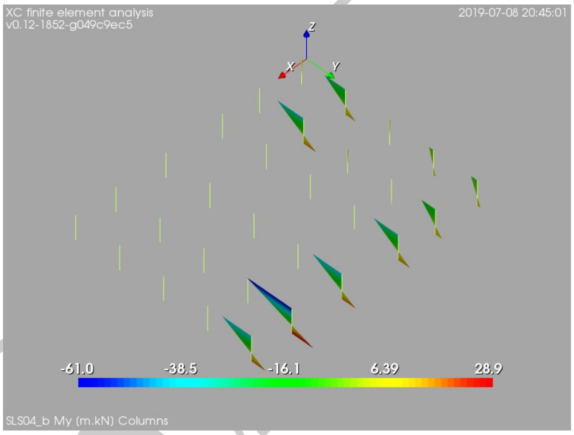


Figure 123: SLS04_b: 1.0*D + W_NS + 1.0*Lru + Lpu. Columns, bending moment about local axis y [m.kN]

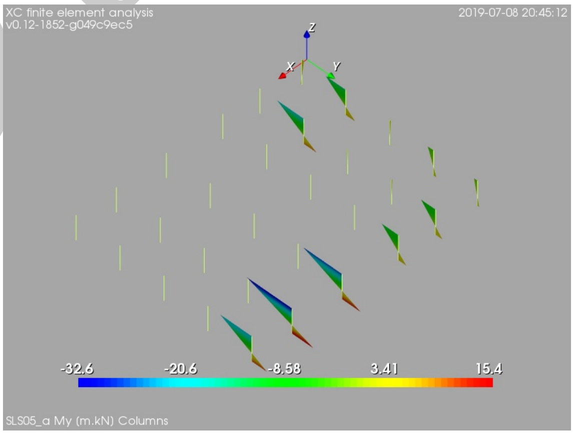


Figure 124: SLS05_a: 1.0*D + W_WE. Columns, bending moment about local axis y [m.kN]

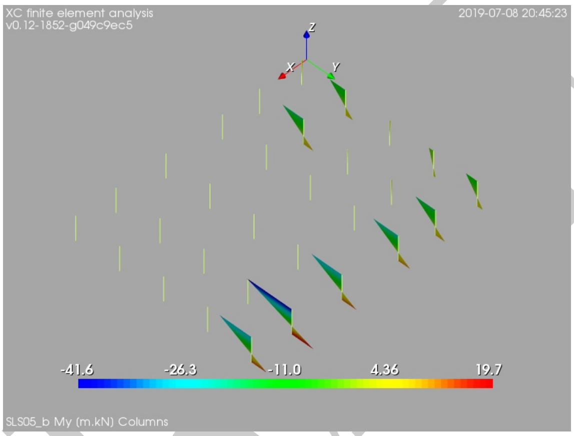


Figure 125: SLS05_b: 1.0*D + W_NS. Columns, bending moment about local axis y [m.kN]