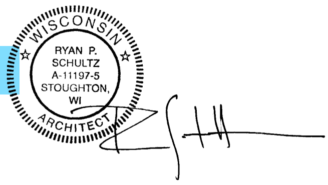
Calculation Report: «1750 OX Residences - 1750 N Oxford Ave. - Eau Claire, WI»

XC structural engineering

June 30, 2020

changes highlighted in blue



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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 Introduction

The calculations shown below correspond to the dimensioning of the floor structure carried out during the design of the building. These calculations have been superseded by those provided by the truss manufacturer.

8.2.2 Trusses A and B. Roof

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

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

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

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

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

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

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

8.2.3 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.

[1] The load combinations are listed in 9.3.3.

Figure 1: Trusses key plan.

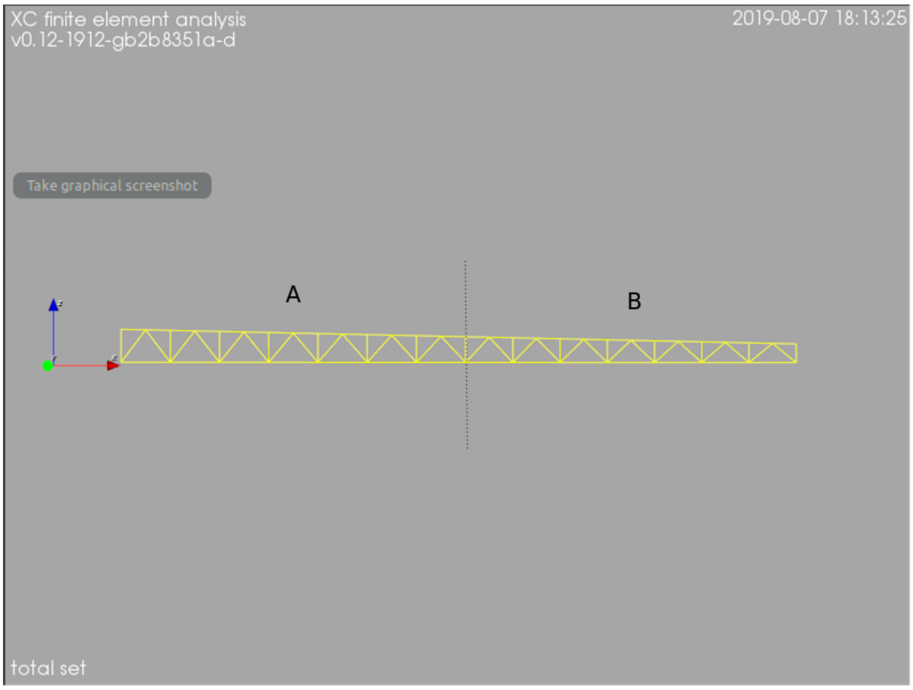


Figure 2: Roof trusses at zones A and B (see key plan in figure 1).

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

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

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

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

8.2.4 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.

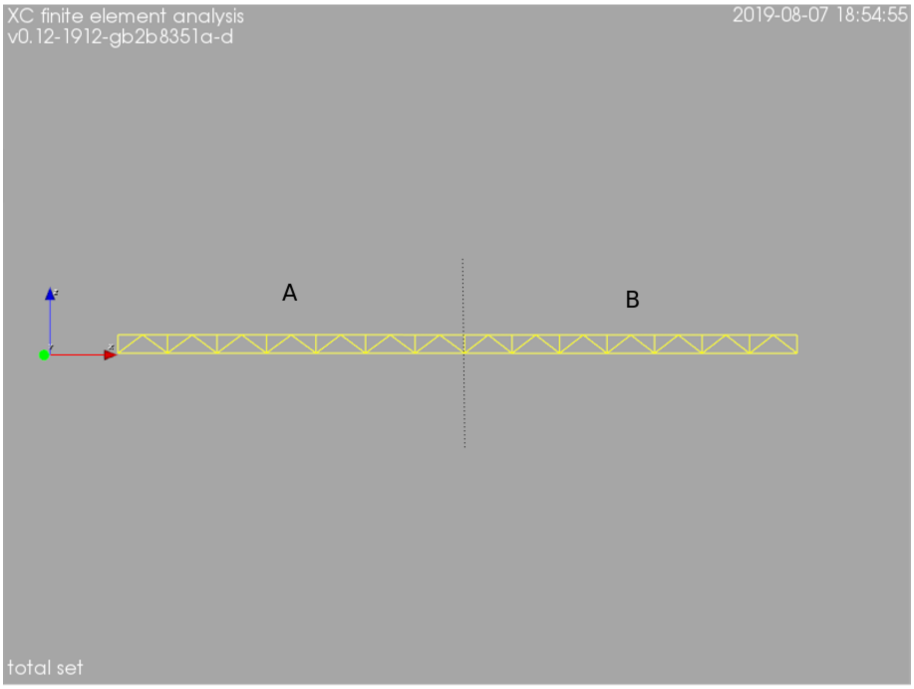


Figure 3: Third floor trusses at zones A and B (see key plan in figure 1).

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

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

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

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

8.2.5 Trusses C and D. Roof

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

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.

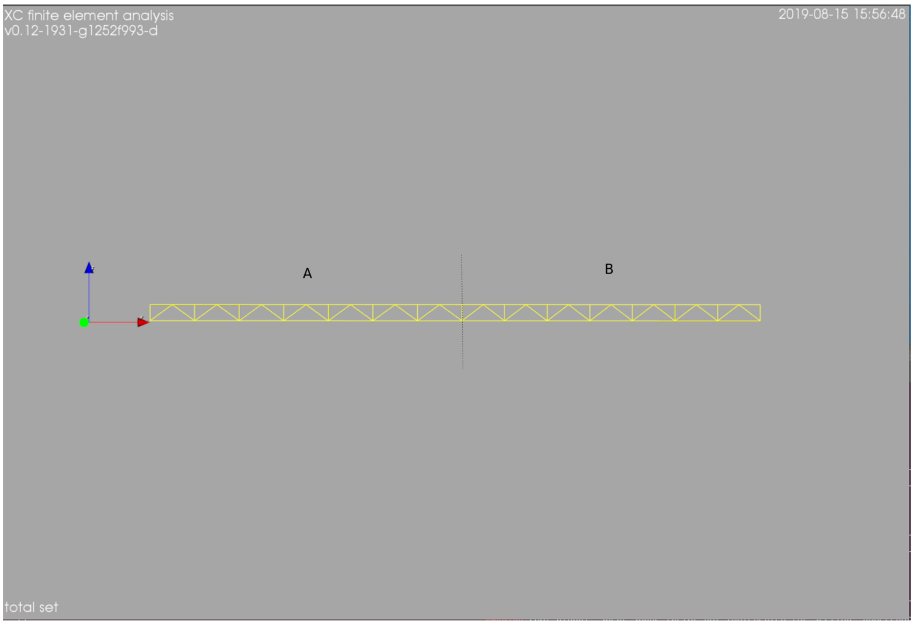


Figure 4: Second floor trusses at zones A and B (see key plan in figure 1).

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

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

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

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

8.2.6 Trusses C and D. Third floor

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

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.

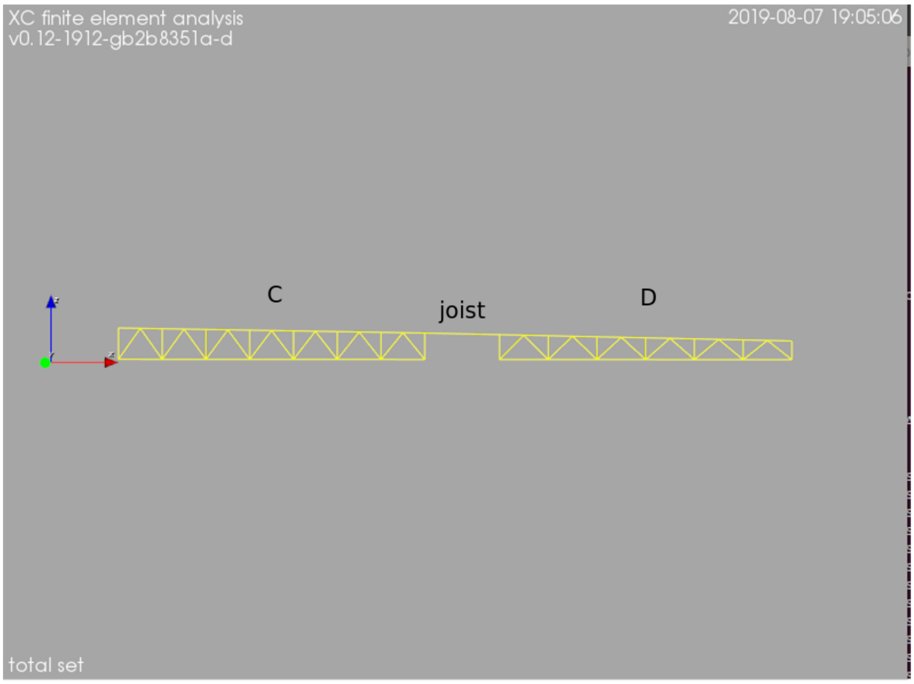


Figure 5: Roof trusses at zones C and D (see key plan in figure 1).

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

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

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

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

8.2.7 Trusses C and D. Second floor

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

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.

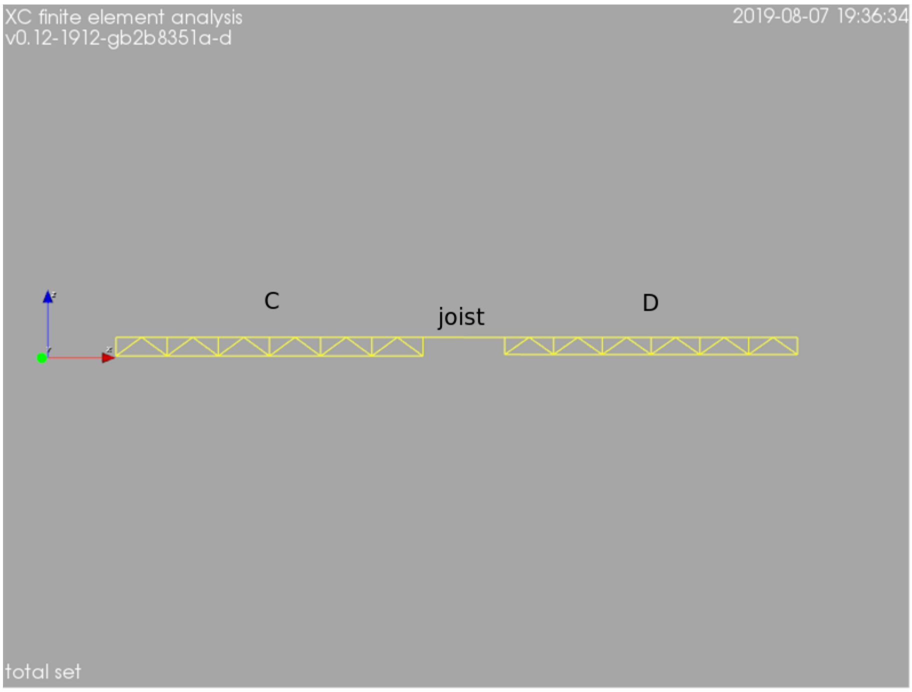


Figure 6: Third floor trusses at zones C and D (see key plan in figure 1).

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

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

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

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

8.2.8 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)

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.

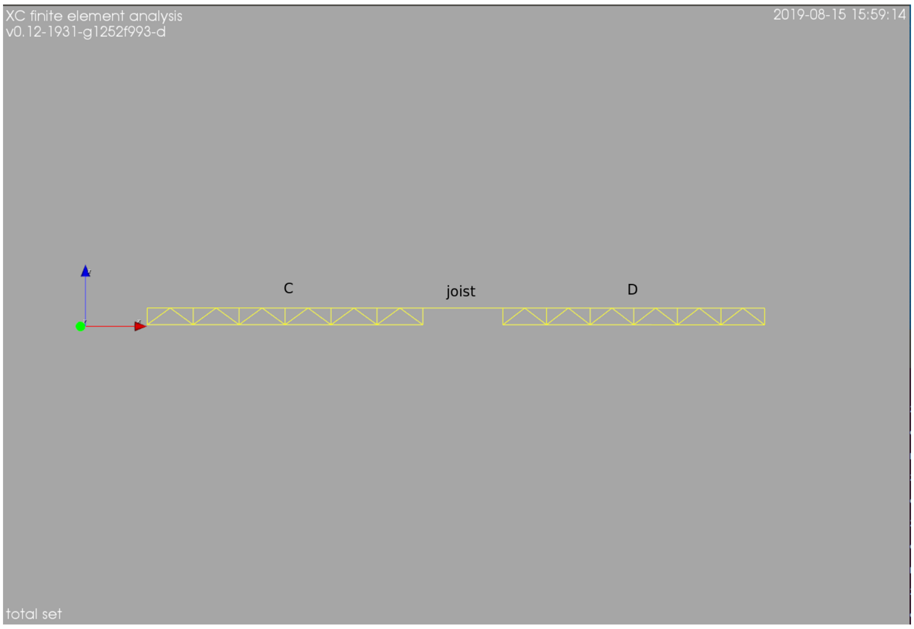


Figure 7: Second floor trusses at zones C and D (see key plan in figure 1).

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

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

(27)

8.2.9 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.

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

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

(30)

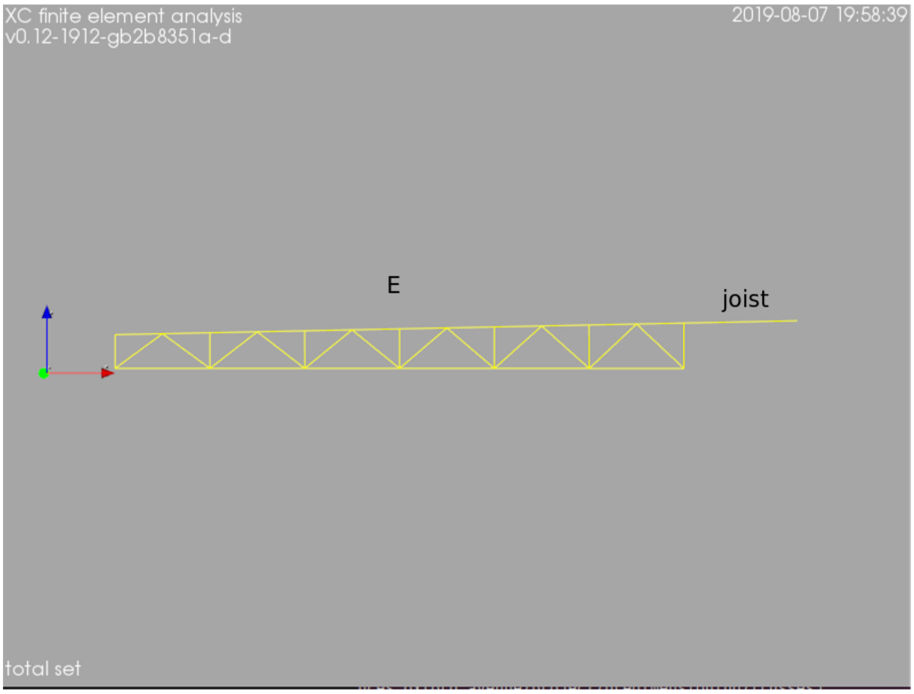


Figure 8: Roof truss at zone E (see key plan in figure 1).

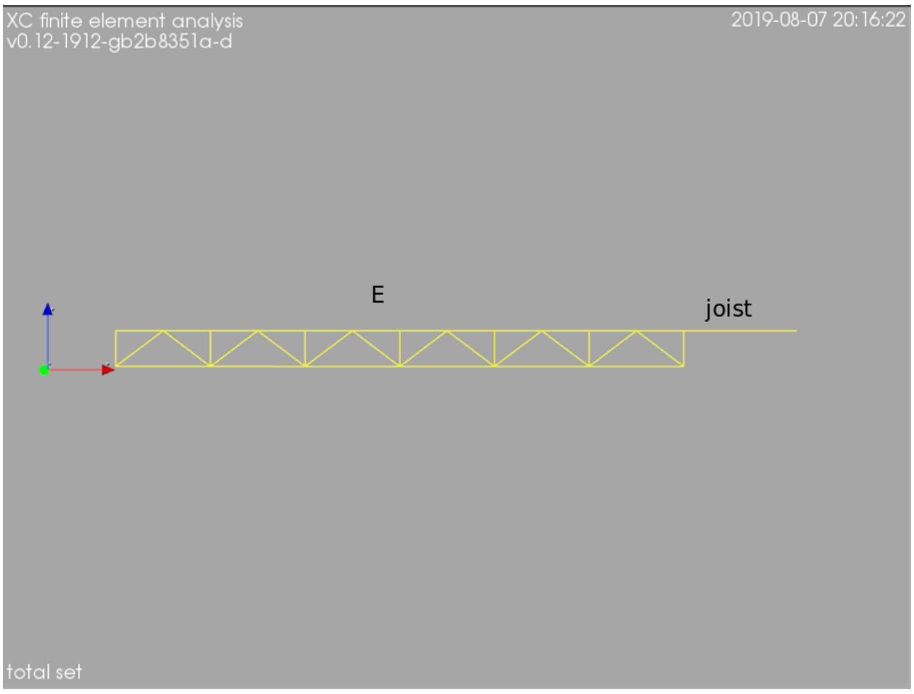


Figure 9: Third floor truss at zone E (see key plan in figure 1).

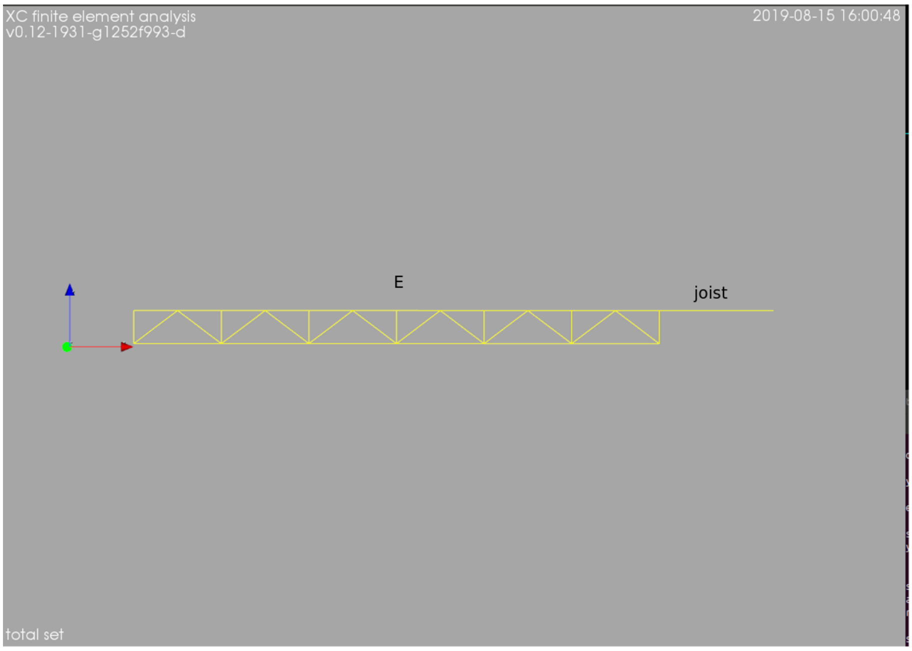


Figure 10: Second floor truss at zone E (see key plan in figure 1).

8.2.10 Truss E. Second floor

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

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

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

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

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

$$(33)$$

8.3 Joists

8.3.1 Corridor floor sheathing

Three layers of 4x8 foot, 19/32 OSB panels are installed as corridor floor sheathing over corridors joists (nominal 4 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:

Table A
Wood Structural Panel Design Capacities Based on Span Ratings[a]

Span Rating	Strength							Planar Shear	Stiffness and Rigidity				
	Bending $F_b S$ (lb-in/ft of width)		Axial Tension $F_t A$ (lb/ft of width)		Axial Compression $F_c A$ (lb/ft of width)		Shear through the thickness [b] $F_v t_v$ (lb/in of shear-resisting panel length)	Planar Shear F_s (Ib/Q) (lb/ft of width)	Bending EI ($lb\text{-}in^2$/ft of width)		Axial[a1] EA (lb/ft of width x 10^6)		Rigidity through the thickness $G_v t_v$ (lb/in of panel depth)
	Capacities relative to strength axis [c]												
	0°	90°	0°	90°	0°	90°	0° / 90°	0° / 90°	0°	90°	0°	90°	0° / 90°
Sheathing Span®													
24/0	300	97	2,300	780	2,850	2,500	155	130	60,000	11,000	3.35	2.50	77,500
24/16	385	115	2,600	1,300	3,250	2,500	165	150	78,000	16,000	3.80	2.70	83,500
32/16	445	165	2,800	1,650	3,550	3,100	180	165	115,000	25,000	4.15	2.70	83,500
40/20	750	270	2,900	2,100	4,200	4,000	195	205	225,000	56,000	5.00	2.90	88,500
48/24	1,000	405	4,000	2,550	5,000	4,300	220	250	400,000	91,500	5.85	3.30	96,000
Floor Span®													
16 oc	500	180	2,600	1,900	4,000	3,600	170	205	150,000	34,000	4.50	2.70	83,500
20 oc	575	250	2,900	2,100	4,200	4,000	195	205	210,000	40,500	5.00	2.90	87,000
24 oc	770	385	3,350	2,550	5,000	4,300	215	250	300,000	80,500	5.85	3.30	93,000
32 oc	1,050	685	4,000	3,250	6,300	6,200	230	300	650,000	235,000	7.50	4.20	110,000
48 oc	1,900	1,200	5,600	4,750	8,100	6,750	305	385	1,150,000	495,000	8.20	4.60	155,000

(a) The design values in this table correspond with those published in the 2005 edition of the AF&PA American Wood Council's *Allowable Stress Deign (ASD)/LRFD Manual for Engineered Wood Construction* Tables M9.2.1- M9.2.4, which are available from the AF&PA American Wood Council.
(a1) In late January 2008, revised Axial EA 90° (perpendicular) values were submitted for modification to AF&PA based on an industry-wide consensus. The appropriate panel grade and construction adjustment factor, C_G, has already been incorporated into these design values—do not apply the C_G factor a second time. These values do not apply to Structural I panels. See Tables M9.2.1 – M9.2.4 for the appropriate multipliers for Structural I panels.
(b) Shear through the thickness design capacities are limited to sections two feet or less in width; wider sections may require further reductions.
(c) Strength axis is defined as the axis parallel to the face and back orientation of the flakes, which is generally the long panel direction, unless otherwise marked.

Figure 11: Wood structural panel design capacities based on span ratings.

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

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 its stiffness $EI = 1977\ MPa$ 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:

$$\Delta_{LL} = 1.24\ mm = \frac{span}{629} < \frac{span}{540} \implies OK \tag{36}$$

and the deflection under total load is:

$$\Delta_{TL} = 1.68\ mm = \frac{span}{484} < \frac{span}{360} \implies OK \tag{37}$$

Bending strength. The allowable bending stress for the OSB panel is $F_b = 5.62\ MPa$ (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 $F_b' = 8.98\ MPa$.

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

$$\sigma_{max} = 2.41\ MPa < 6.94\ MPa = F_b' \implies OK \tag{38}$$

Table B
Relationship Between Span Rating and Nominal Thickness for OSB[(a)]

Span Rating	Nominal Thickness[(b)] (in.) 3/8	7/16	15/32	1/2	19/32	5/8	23/32	3/4	7/8	1	1-1/8
	Sheathing Span®										
24/0	**0.375**	0.437	0.469	0.500							
24/16		**0.437**	0.469	0.500							
32/16			**0.469**	0.500	0.594	0.625					
40/20					**0.594**	0.625	0.719	0.750			
48/24							**0.719**	0.750	0.875		
	Floor Span®										
16 oc					**0.594**	0.625					
20 oc					**0.594**	0.625					
24 oc							**0.719**	0.750			
32 oc									**0.875**	1.000	
48 oc											**1.125**

(a) The values in this table correspond with those published in the 2005 edition of the AF&PA American Wood Council's *Commentary National Design Specification (NDS) for Wood Construction ASD/LRFD* Table C9.2.3, which is available from the AF&PA American Wood Council.
(b) The predominant thickness for each span rating is highlighted in **bold**.

Figure 12: Relationshib between span rating and nominal thickness for OSB.

Shear strength. The allowable shear stress of the panel is $F_v = 0.19\ MPa$ and the adjusted allowable shear stress is (under the same conditions that we used for the bending stress) $F'_v = 0.30\ MPa$.

The maximum shear stress obtained under total load is:

$$\tau_{max} = 0.05\ MPa < 0.30\ MPa = F'_v \implies OK \tag{39}$$

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:

$$\sigma_{max} = 4.59\ MPa < 8.98\ MPa = F'_b \implies OK \tag{40}$$

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

$$\tau_{max} = 0.08\ MPa < 0.30\ MPa = F'_v \implies OK \tag{41}$$

8.3.2 Corridor Joists

Simply supported 4x6 southern pine 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. $C_L = 1.0$).

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

Structural design of the joist.

Loads.

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

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:

$$S_s = 290.73 \times 10^{-6}\ m^3 \tag{47}$$

Tabulated bending stress:

$$F_b = 9.3\ MPa \tag{48}$$

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

$$F_b' = 9.3\ MPa \tag{49}$$

Tabulated shear stress:

$$F_v = 1.2\ MPa \tag{50}$$

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

$$F_v' = 1.2\ MPa \tag{51}$$

Structural bending check. Design resisting moment:

$$M_s' = 2.70\ kNm \tag{52}$$

Structural bending check: $M_s' = 2.70 > 1.54 = M_{max} \implies OK$

Structural shear check. Design resisting shear:

$$V_s' = 9.97\ kN \tag{53}$$

Structural shear check: $V_s' = 9.97 > 2.57 = V_{max} \implies OK$

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:

$$A_f = 36.96\ cm^2 \tag{56}$$

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

$$F'_{b,f} = 26.51\ MPa \tag{57}$$

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

$$F'_{v,f} = 3.30\ MPa \tag{58}$$

Structural bending check. Design resisting moment:

$$M'_f = 1.83\ kNm \tag{59}$$

Structural bending check: $M'_s = 1.83 > 1.54 = M_{max} \implies OK$

Structural shear check. Design resisting shear:

$$V'_f = 8.13\ kN \tag{60}$$

Structural shear check: $V'_s = 8.13 > 2.57 = V_{max} \implies OK$

8.3.3 Joists under storage/HVAC floor

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

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

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

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

Structural design of the joist.

Loads.

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

Internal forces. Maximum induced moment:

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

Maximum induced shear:

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

Joist mechanical properties. Joist section modulus:

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

Tabulated bending stress:

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

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

$$F_b' = 20.58\ MPa \tag{68}$$

Tabulated shear stress:

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

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

$$F_v' = 1.97\ MPa \tag{70}$$

Structural bending check. Design resisting moment:

$$M_s' = 16.83\ kNm \tag{71}$$

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

Structural shear check. Design resisting shear:

$$V_s' = 27.36\ kN \tag{72}$$

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

Bending stiffness. The deflection obtained under live load is:

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

and the deflection under total load is:

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

8.4 Headers

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

Simply supported 1.75x14 LSL 1.55E headers.

Structural design of the header.

Design load.

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

Internal forces. Maximum induced moment:

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

Maximum induced shear:

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

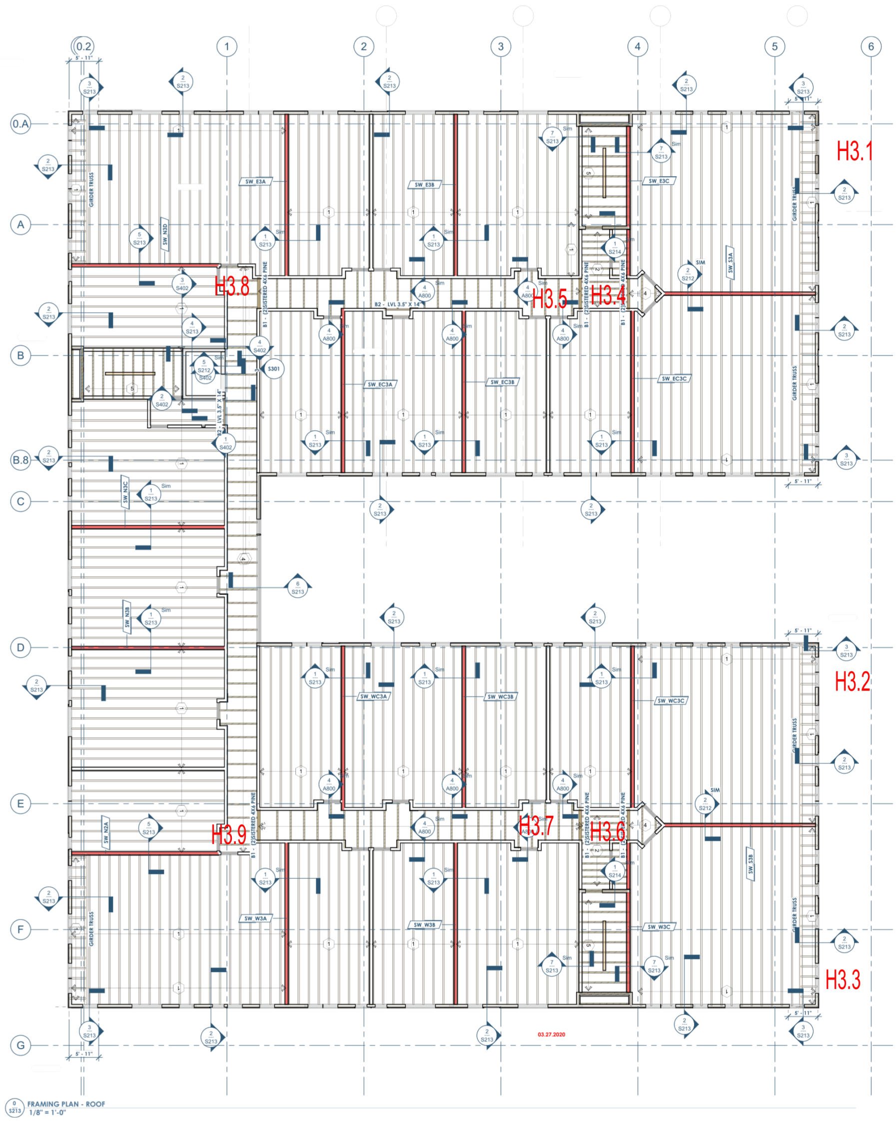


Figure 13: Headers key plan. Roof

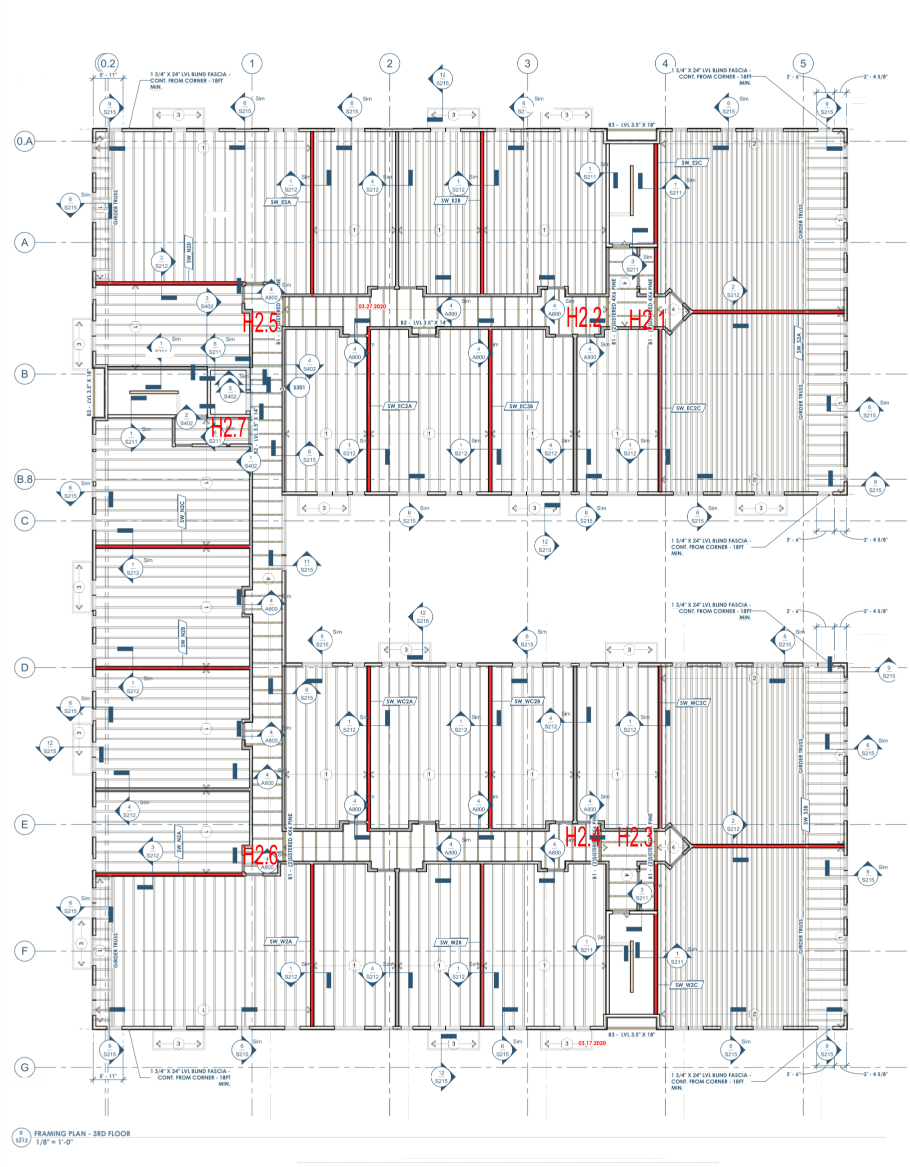


Figure 14: Headers key plan. Third floor

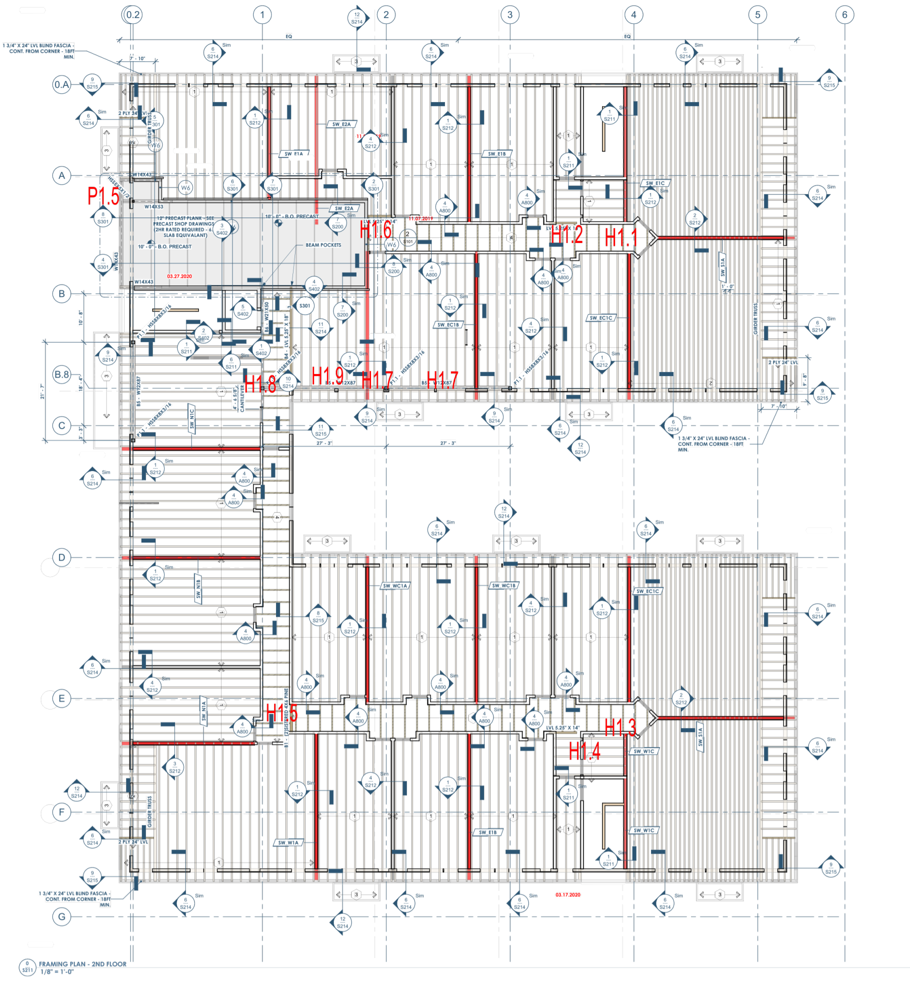


Figure 15: Headers key plan. Second floor

Structural bending check. Design resisting moment:

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

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

Structural shear check. Design resisting shear:

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

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

Bending stiffness. The deflection obtained is:

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

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

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

Structural design of the header.

Design load.

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

Internal forces. Maximum induced moment:

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

Maximum induced shear:

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

Structural bending check. Design resisting moment:

$$M'_s = 10.63\ kNm \tag{84}$$

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

Structural shear check. Design resisting shear:

$$V'_s = 21.44\ kN \tag{85}$$

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

Bending stiffness. The deflection obtained is:

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

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

Mechanical properties of the burned section. Effective char depth:

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

section modulus for a joist exposed on three sides:

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

shear area for a beam exposed on three sides:

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

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

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

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

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

Structural bending check. Design resisting moment:

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

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

Structural shear check. Design resisting shear:

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

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

8.4.3 Headers H3.10 and H2.7

Simply supported 3.5x14" LVL header.

Structural design of the header.

Design load.

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

Internal forces. Maximum induced moment:

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

Maximum induced shear:

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

Structural bending check. Design resisting moment:

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

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

Structural shear check. Design resisting shear:

$$V_s' = 41.41\ kN \tag{98}$$

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

Bending stiffness. The deflection obtained is:

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

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

Mechanical properties of the burned section. Effective char depth:

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

section modulus for a joist exposed on three sides:

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

shear area for a beam exposed on three sides:

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

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

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

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

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

Structural bending check. Design resisting moment:

$$M_f' = 32.8\ kNm \tag{105}$$

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

Structural shear check. Design resisting shear:

$$V_f' = 38.82\ kN \tag{106}$$

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

8.4.4 Header H1.9

Simply supported 5.25x18" LVL 1.55E beam.

Structural design of the header.

Design load.

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

Internal forces. Maximum induced moment:

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

Maximum induced shear:

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

Structural bending check. Design resisting moment:

$$M'_s = 87.66\ kNm \tag{110}$$

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

Structural shear check. Design resisting shear:

$$V'_s = 79.87\ kN \tag{111}$$

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

Bending stiffness. The deflection obtained is:

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

8.4.5 Facade headers

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

Structural design of the header.

Design load.

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

Internal forces. Maximum induced moment:

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

Maximum induced shear:

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

Structural bending check. Design resisting moment:

$$M'_s = 21.96\ kNm \tag{116}$$

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

Structural shear check. Design resisting shear:

$$V'_s = 50.53\ kN \tag{117}$$

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

Bending stiffness. The deflection obtained is:

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

8.4.6 Corridor headers

Simply supported 3.5x16 LSL 1.55E header.

Structural design of the header.

Design load.

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

Internal forces. Maximum induced moment:

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

Maximum induced shear:

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

Structural bending check. Design resisting moment:

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

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

Structural shear check. Design resisting shear:

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

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

Bending stiffness. The deflection obtained is:

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

8.5 Steel beams

8.5.1 Steel beam at courtyard facade

Continuous beam supporting the second floor trusses between the axes 1, 2 and 3 (see figure 16). The beam has two equal spans; L= 8.31 m $(27' - 3'')$.

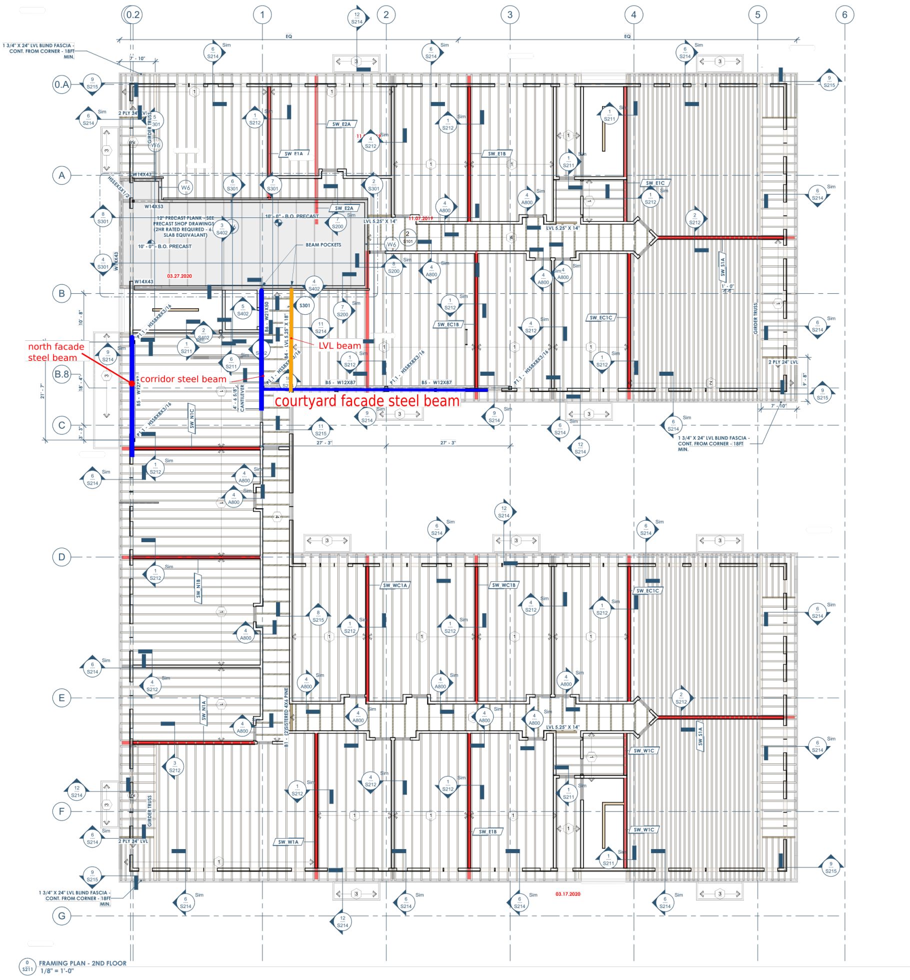


Figure 16: Second floor beams key plan.

Design loads. The design loads are show in table 10. The beam then carries a the following loads:

- Dead load: 14.88 kN/m.
- Live load: 22.86 kN/m.
- Snow load: 10.05 kN/m.
- Wind load: -6.35 kN/m.

Load combinations

Serviceability limit states

SLS01	$1.0 \times LL$
SLS02	$1.0 \times DL + 1.0 \times LL$
SLS03	$1.0 \times DL + 1.0 \times S$

Ultimate limit states

ULS01	$1.4 \times DL$
ULS02	$1.2 \times DL + 1.6 \times LL + 0.5 \times S$
ULS03	$1.2 \times DL + 1.6 \times S + 0.5 \times LL$
ULS04	$1.2 \times DL + 1.6 \times S + 0.5 \times W$
ULS05	$1.2 \times DL + 1.0 \times W + 0.5 \times LL$
ULS06	$1.2 \times DL + 0.5 \times LL + 0.2 \times S$
ULS07	$0.9 \times DL + 1.0 \times W$

Structural design of the beam.

Internal forces. Maximum induced moment:

$$M_{max} = -513\ kN \cdot m \tag{125}$$

Maximum induced shear:

$$V_{max} = 309\ kN \tag{126}$$

W12X87 shape mechanical properties. Steel: A572
Shear strength:

$$V_u = 862.32\ kN \tag{127}$$

Structural shear check: $V_u = 862.32 > 309.00 = V_{max} \implies OK$
Resisting moment:

$$M_u = 670.68\ kN \cdot m \tag{128}$$

Structural bending check: $M_U = 670.68 > 513.00 = M_{max} \implies OK$

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

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

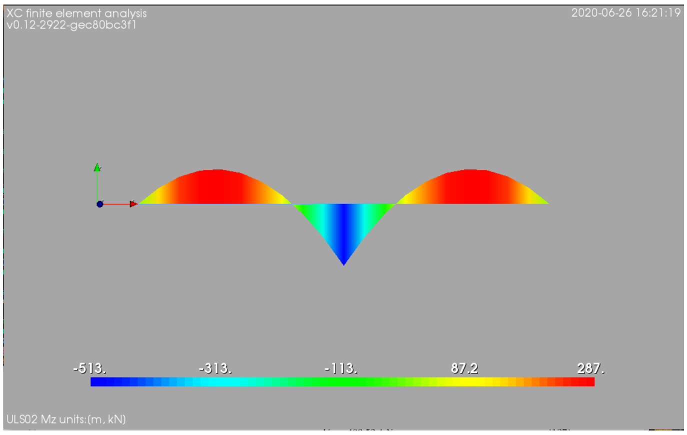


Figure 17: Courtyard facade steel beam. ULS02. M_z

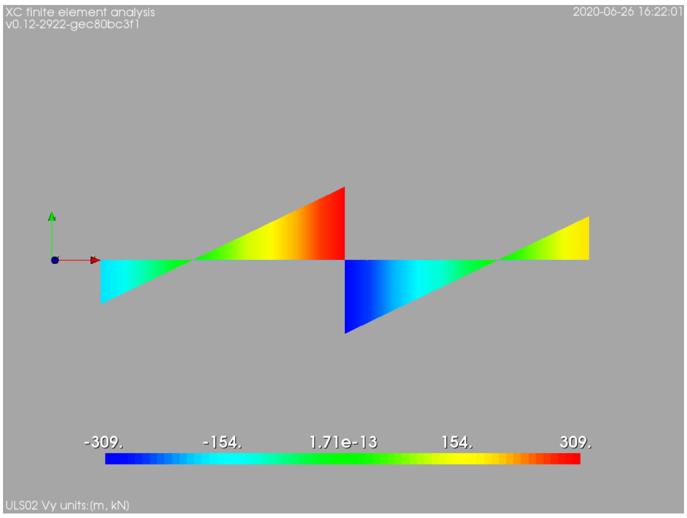


Figure 18: Courtyard facade steel beam. ULS02. V_y

Bending stiffness. The deflections obtained for SLS01, SLS02 and SLS03 are:

$$\Delta_{TL} = 9.54\ mm < 15.38\ mm = \frac{L}{540} \implies OK \quad (129)$$

$$\Delta_{TL} = 10.40\ mm < 23.07\ mm = \frac{L}{360} \implies OK \quad (130)$$

$$\Delta_{TL} = 15.74\ mm < 23.07\ mm = \frac{L}{360} \implies OK \quad (131)$$

8.5.2 Steel beam at corridor

This beam supports the second floor trusses near the elevator well (see figure 16) . It has a main span of 6.91 m long ($22' - 8''$) and a cantilever that spans 1.04 m ($3' - 4''$).

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

Structural design of the beam.

Loads.

$$w_{load} = 51.94\ kN/m \quad (132)$$

Internal forces. Maximum induced moment:

$$M_{max} = 315.06\ kNm \quad (133)$$

Maximum induced shear:

$$V_{max} = 182.52\ kN \quad (134)$$

Structural shape (W21X50) mechanical properties. Steel: ASTM A-572
Shear strength:

$$V_u = 607.72\ kN \quad (135)$$

Structural shear check: $V_u = 607.72 > 182.52 = V_{max} \implies OK$
Resisting moment:

$$M_u = 371.86\ kN \cdot m \quad (136)$$

Structural bending check: $M_U = 371.86 > 316.06 = M_{max} \implies OK$

Bending stiffness. The deflection obtained is:

$$\Delta_{TL} = 16.88\ mm = \frac{L}{409} < \frac{span}{360} \implies OK \quad (137)$$

8.5.3 North steel beam

This beam supports the second floor trusses near the stairs well at north facade (see figure 16) . It spans 7.26 m ($23' - 10''$).

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

Table 11: Steel beam at corridor. Trusses reactions.

Design loads. The design loads are shown in table 11. The beam then carries a the following loads:

- Dead load: 15.52 kN/m.
- Live load: 21.74 kN/m.
- Snow load: 9.52 kN/m.
- Wind load: -6.01 kN/m.

Load combinations

Serviceability limit states

SLS01	$1.0 \times LL$
SLS02	$1.0 \times DL + 1.0 \times LL$
SLS03	$1.0 \times DL + 1.0 \times S$

Ultimate limit states

ULS01	$1.4 \times DL$
ULS02	$1.2 \times DL + 1.6 \times LL + 0.5 \times S$
ULS03	$1.2 \times DL + 1.6 \times S + 0.5 \times LL$
ULS04	$1.2 \times DL + 1.6 \times S + 0.5 \times W$
ULS05	$1.2 \times DL + 1.0 \times W + 0.5 \times LL$
ULS06	$1.2 \times DL + 0.5 \times LL + 0.2 \times S$
ULS07	$0.9 \times DL + 1.0 \times W$

Structural design of the beam.

Internal forces. Maximum induced moment:

$$M_{max} = 383 \ kN \cdot m \tag{138}$$

Maximum induced shear:

$$V_{max} = 211 \ kN \tag{139}$$

W12X87 shape mechanical properties. Steel: A572
Shear strength:

$$V_u = 862.32 \ kN \tag{140}$$

Structural shear check: $V_u = 862.32 > 211.00 = V_{max} \implies OK$
Resisting moment:

$$M_u = 670.68 \ kN \cdot m \tag{141}$$

Structural bending check: $M_U = 670.68 > 383.00 = M_{max} \implies OK$

Bending stiffness. The deflections obtained for SLS01, SLS02 and SLS03 are:

$$\Delta_{TL} = 12.77\ mm < 13.45\ mm = \frac{L}{540} \implies OK \tag{142}$$
$$\Delta_{TL} = 14.71\ mm < 30.26\ mm = \frac{L}{240} \implies OK \tag{143}$$
$$\Delta_{TL} = 21.89\ mm < 30.26\ mm = \frac{L}{240} \implies OK \tag{144}$$

8.6 Columns

8.6.1 Columns P3.1 to P3.6

Loads.

- Load from header: 4.8 kN

Internal forces. Maximum induced axial force:

$$N_{max} = 4.8\ kN \tag{145}$$

Compression check.

Mechanical properties

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

$$N'_s = 42.46\ kN \tag{146}$$

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

8.6.2 Column P3.7

Loads.

- Load from header 3.10: 27.61 kN

Internal forces. Maximum induced axial force:

$$N_{max} = 27.61\ kN \tag{147}$$

Compression check.

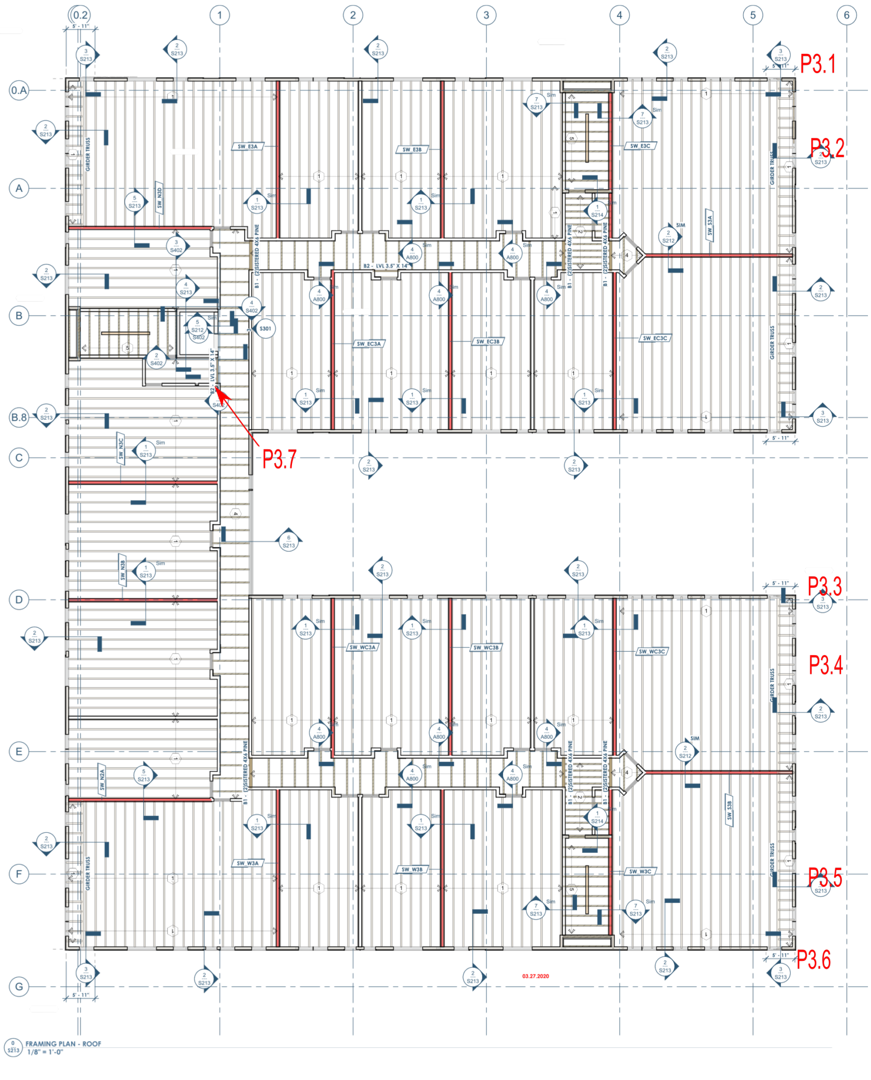


Figure 19: Columns key plan. Roof

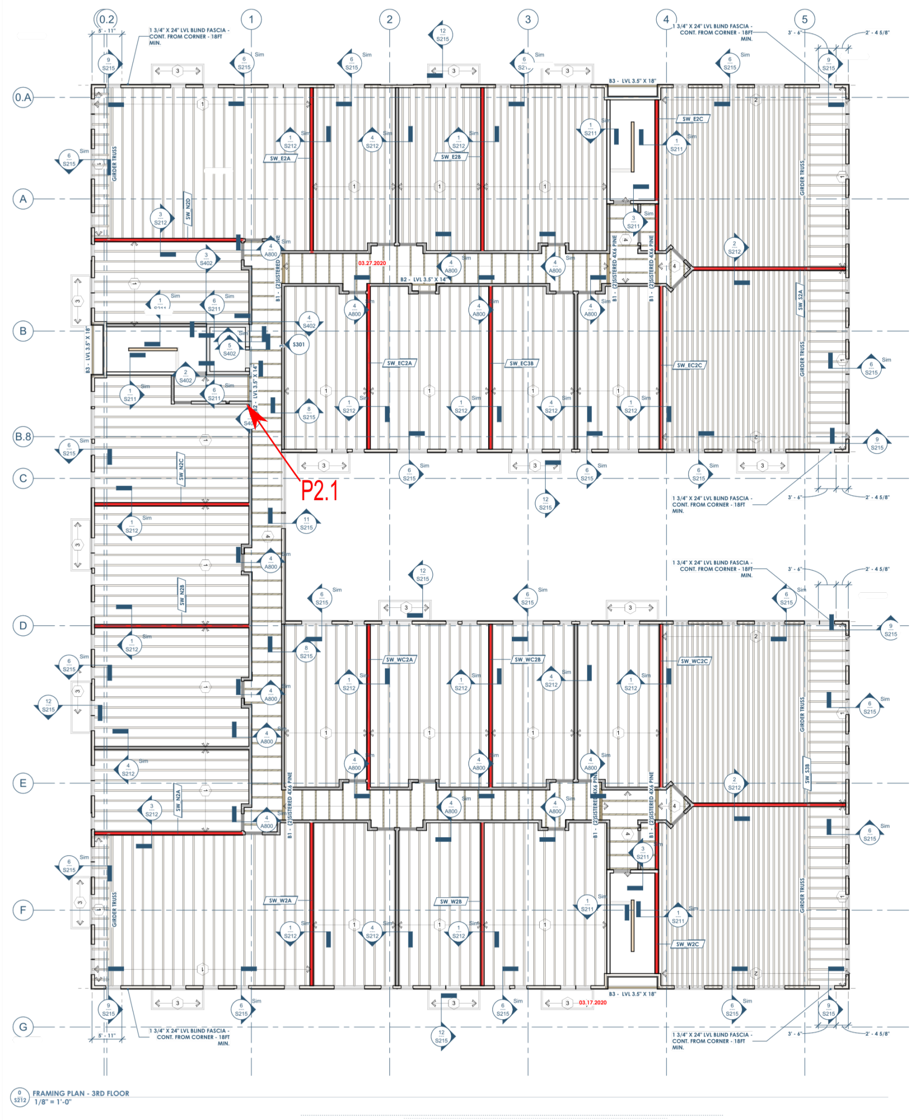


Figure 20: Columns key plan. Third floor

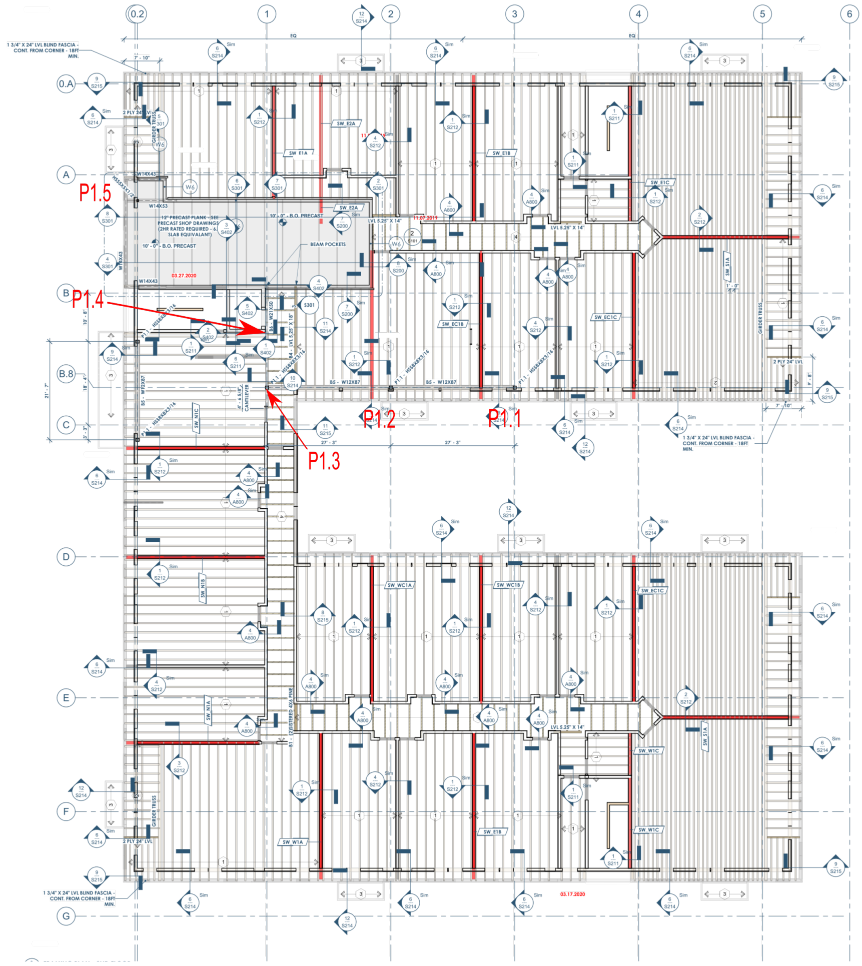


Figure 21: Columns key plan. Second floor

Mechanical properties

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

$$N'_s = 74.34\ kN \tag{148}$$

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

8.6.3 Column P2.1

Loads.

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

Internal forces. Maximum induced axial force:

$$N_{max} = 55.22\ kN \tag{149}$$

Compression check.

Mechanical properties

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

$$N'_s = 74.34\ kN \tag{150}$$

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

8.6.4 2x6 wood stud capacity

Compression capacity.

Mechanical properties.

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

$$N'_s = 19.90\ kN \tag{151}$$

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

Loads.

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

Internal forces. Maximum induced axial force:

$$N_{max} = 258.96\ kN M_{y,max} = 5.18\ kNm M_{z,max} = 12.95\ kNm \tag{152}$$

Mechanical properties.

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

Compression check.

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

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

$$\frac{P_d}{P_n} + \frac{8}{9}(\frac{M_{yd}}{M_{yn}} + \frac{M_{zd}}{M_{zn}}) = 0.97 < 1.0 \implies OK \tag{153}$$

8.6.6 Column P1.4

Loads.

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

Internal forces. Maximum induced axial force:

$$N_{max} = 125.49\ kN M_{y,max} = 7.33\ kNm M_{z,max} = 6.27\ kNm \tag{154}$$

Mechanical properties.

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

Compression check.

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

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

$$\frac{P_d}{P_n} + \frac{8}{9}(\frac{M_{yd}}{M_{yn}} + \frac{M_{zd}}{M_{zn}}) = 0.67 < 1.0 \implies OK \tag{155}$$

8.7 Bearing walls

8.7.1 Facade bearing walls at first floor

Loads.

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

Internal forces. Internal forces:

$$N_{max} = 21.14\ kN \tag{156}$$
$$M_{max} = 0.36\ kN \cdot m \tag{157}$$

Bending and axial compression check.

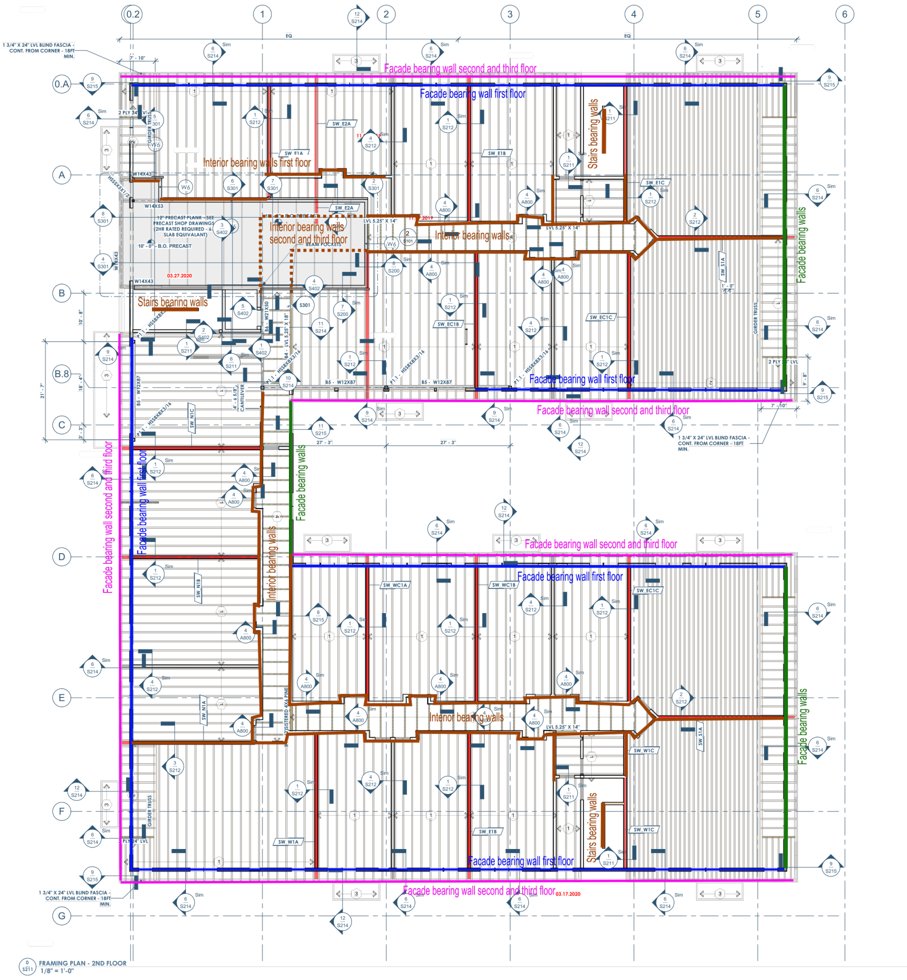


Figure 22: Bearing walls key plan. Roof

Mechanical properties

- Species: Hem-fir stud
- Spacing: 0.49 m
- Stud height: 2.64 m
- Repetitive member factor: $C_r = 1.15$
- Size factor: $C_F = 1.3$
- $E_{min} = 3.03\ GPa$
- $F'_c = 6.09\ MPa$.
- $F'_b = 6.97\ MPa$.
- Sections dimensions: (2x8"), effective (1.5x7.5")= $38.1 \times 190.5\ mm$.
- Unbraced length x axis: 2.64 m
- Unbraced length y axis: 0.3 m
- Stud stability factor: $C_P = 0.85$

$$N'_s = 44.21\ kN \tag{158}$$

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

$$CF = 0.52 < 1 \implies OK \tag{159}$$

8.7.2 Facade bearing walls at second floor

Loads.

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

Internal forces. Internal forces:

$$N_{max} = 14.88\ kN \tag{160}$$
$$M_{max} = 0.36\ kN \cdot m \tag{161}$$

Bending and axial compression check.

Mechanical properties

- Species: Hem-fir stud
- Spacing: 0.49 m
- Stud height: 2.64 m
- Repetitive member factor: $C_r = 1.15$
- Size factor: $C_F = 1.3$
- $E_{min} = 3.03\ GPa$
- $F'_c = 6.09\ MPa.$
- $F'_b = 6.95\ MPa.$
- Sections dimensions: (2x8”), effective (1.5x7.5”)= $38.1 \times 190.5\ mm.$
- Unbraced length x axis: 2.64 m
- Unbraced length y axis: 0.3 m
- Stud stability factor: $C_P = 0.85$

$$N'_s = 44.21\ kN \tag{162}$$

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

$$CF = 0.38 < 1 \implies OK \tag{163}$$

8.7.3 Facade bearing walls at third floor

Loads.

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

Internal forces. Internal forces:

$$N_{max} = 7.81\ kN \tag{164}$$
$$M_{max} = 0.36\ kN \cdot m \tag{165}$$

Bending and axial compression check.

Mechanical properties

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

$$N'_s = 44.21\ kN \tag{166}$$

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

$$CF = 0.28 < 1 \implies OK \tag{167}$$

8.7.4 Top plates

Loads

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

Bending strength checking: Maximum induced moment:

$$M_{max} = 0.42 kN \cdot m \tag{168}$$
$$\sigma_{max} = 9.21\ MPa \tag{169}$$

Bending strength:

$$F'_b = 10.05\ MPa \tag{170}$$

Structural bending check:

$$F'_b = 10.05 > 9.21 = \sigma_{max} \implies OK \tag{171}$$

Perpendicular to grain strength checking: Maximum induced reaction:

$$R_{max} = 9.38\ kN \quad (172)$$
$$\sigma_{max,perp} = 1.29\ MPa \quad (173)$$

$$F'_{c,perp} = 2.79 > 1.29 = \sigma_{max,perp} \implies OK \quad (174)$$

8.7.5 Interior bearing walls at first floor

Loads.

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

Internal forces. Internal forces:

$$N_{max} = 25.9\ kN \quad (175)$$
$$M_{max} = 0.0\ kN \cdot m \quad (176)$$

Bending and axial compression check.

Mechanical properties

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

$$N'_s = 26.00\ kN \quad (177)$$

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

$$CF = 0.99 < 1 \implies OK \quad (178)$$

8.7.6 Interior bearing walls at second floor

Loads.

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

Internal forces. Internal forces:

$$N_{max} = 18.26\ kN \tag{179}$$
$$M_{max} = 0.0\ kN \cdot m \tag{180}$$

Bending and axial compression check.

Mechanical properties

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

$$N'_s = 26.00\ kN \tag{181}$$

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

$$CF = 0.49 < 1 \implies OK \tag{182}$$

8.7.7 Interior bearing walls at third floor

Loads.

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

Internal forces. Internal forces:

$$N_{max} = 20.54\ kN \quad (183)$$
$$M_{max} = 0.00\ kN \cdot m \quad (184)$$

Bending and axial compression check.

Mechanical properties

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

$$N'_s = 26.00\ kN \quad (185)$$

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

$$CF = 0.62 < 1 \implies OK \quad (186)$$

8.7.8 Top plates

Loads

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

Bending strength checking: Maximum induced moment:

$$M_{max} = 0.24kN \cdot m \quad (187)$$
$$\sigma_{max} = 7.15\ MPa \quad (188)$$

Bending strength:

$$F'_b = 10.05\ MPa \quad (189)$$

Structural bending check:

$$F'_b = 10.05 > 7.15 = \sigma_{max} \implies OK \quad (190)$$

Perpendicular to grain strength checking: Maximum induced reaction:

$$R_{max} = 8.56\ kN \tag{191}$$
$$\sigma_{max,perp} = 1.61\ MPa \tag{192}$$

$$F'_{c,perp} = 2.79 > 1.61 = \sigma_{max,perp} \implies OK \tag{193}$$

8.8 Lateral. Diaphragms/Shear walls

8.8.1 East and West facades shear walls

The shear walls of the East facade are those denoted by the letters "E" and "W" in figures 23 to 25. The wind load on each floor per unit length is as follows:

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

The shear values obtained for each wall are as follows:

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

And the cumulated values are:

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

leading to the following dimensions:

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

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

8.8.2 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 23 to 25. The wind load on each floor per unit length is as follows:

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

The shear values obtained for each wall are as follows:

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

And the cumulated values are:

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

leading to the following dimensions:

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

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

8.8.3 South facades shear walls

The shear walls of the South facade are those denoted by the letter "S" in figures 23 to 25. The wind load on each floor per unit length is as follows:

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

The shear values obtained for each wall are as follows:

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

And the cumulated values are:

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

leading to the following dimensions:

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

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

8.8.4 North facade shear walls

The shear walls of the North facade are those denoted by the letter "N" in figures 23 to 25. The wind load on each floor per unit length is as follows:

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

The shear values obtained for each wall are as follows:

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

And the cumulated values are:

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

leading to the following dimensions:

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

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

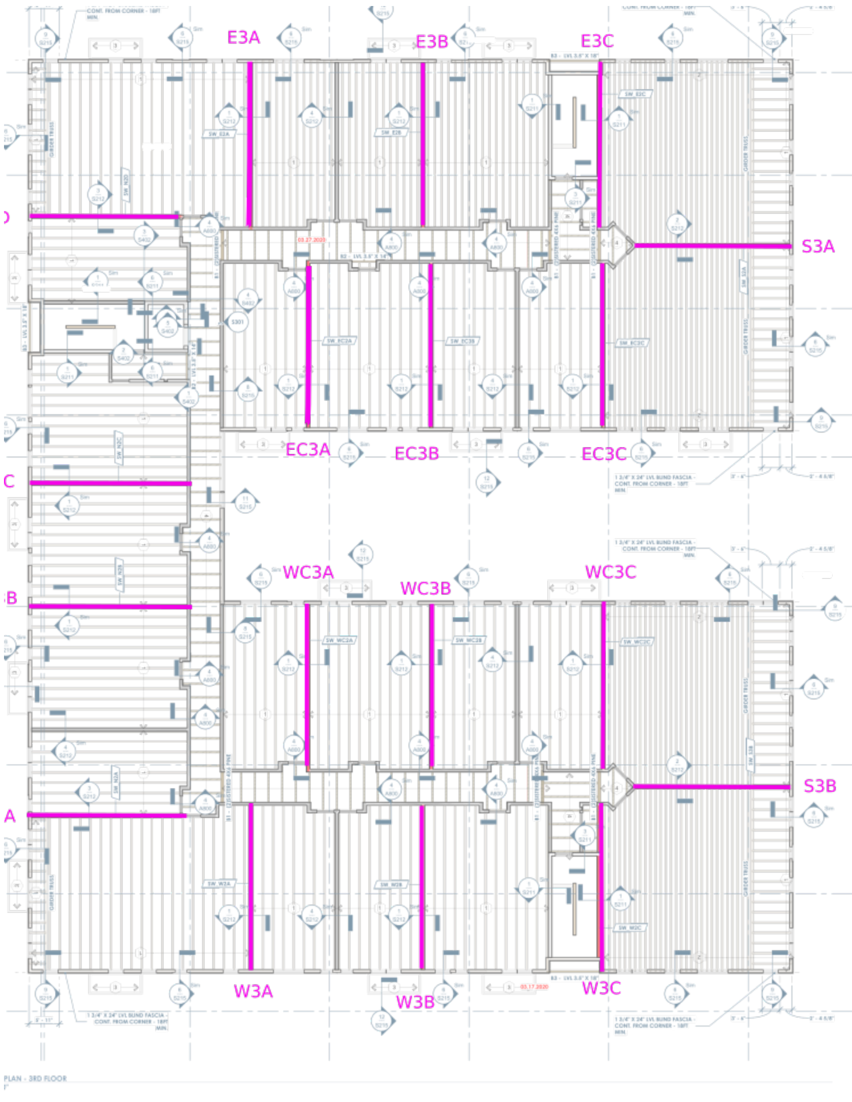


Figure 23: Shear walls on the third floor.

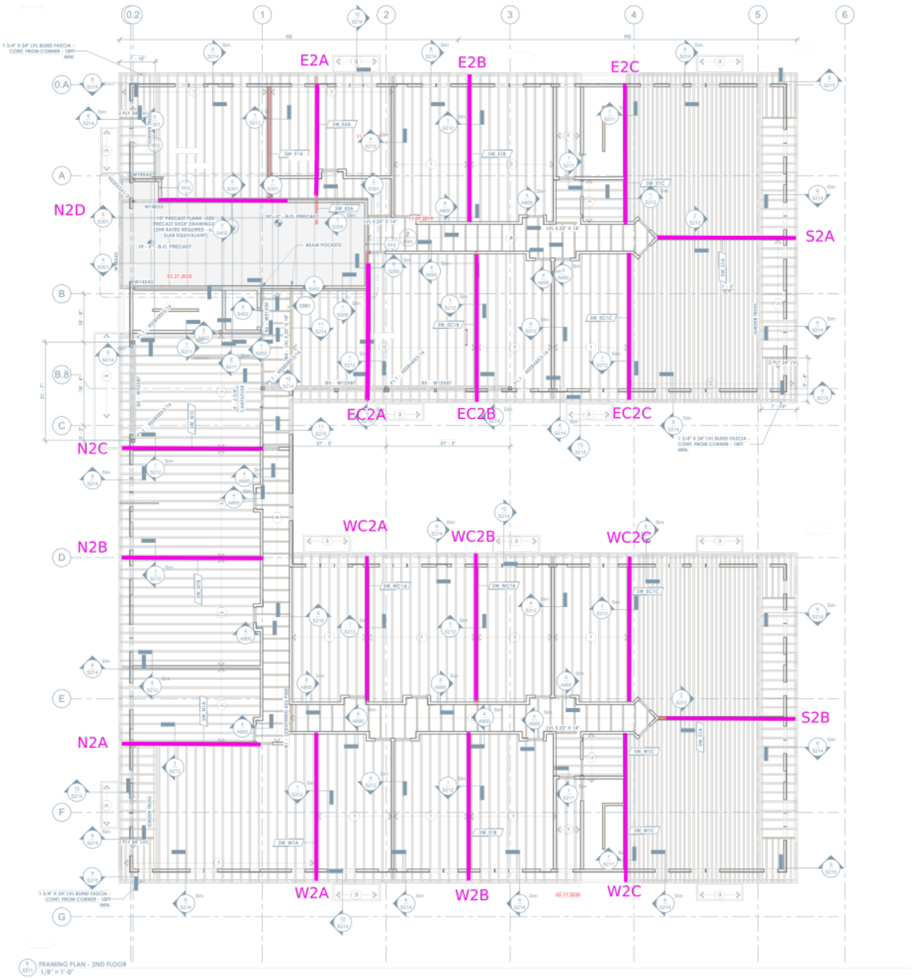


Figure 24: Shear walls on the second floor.

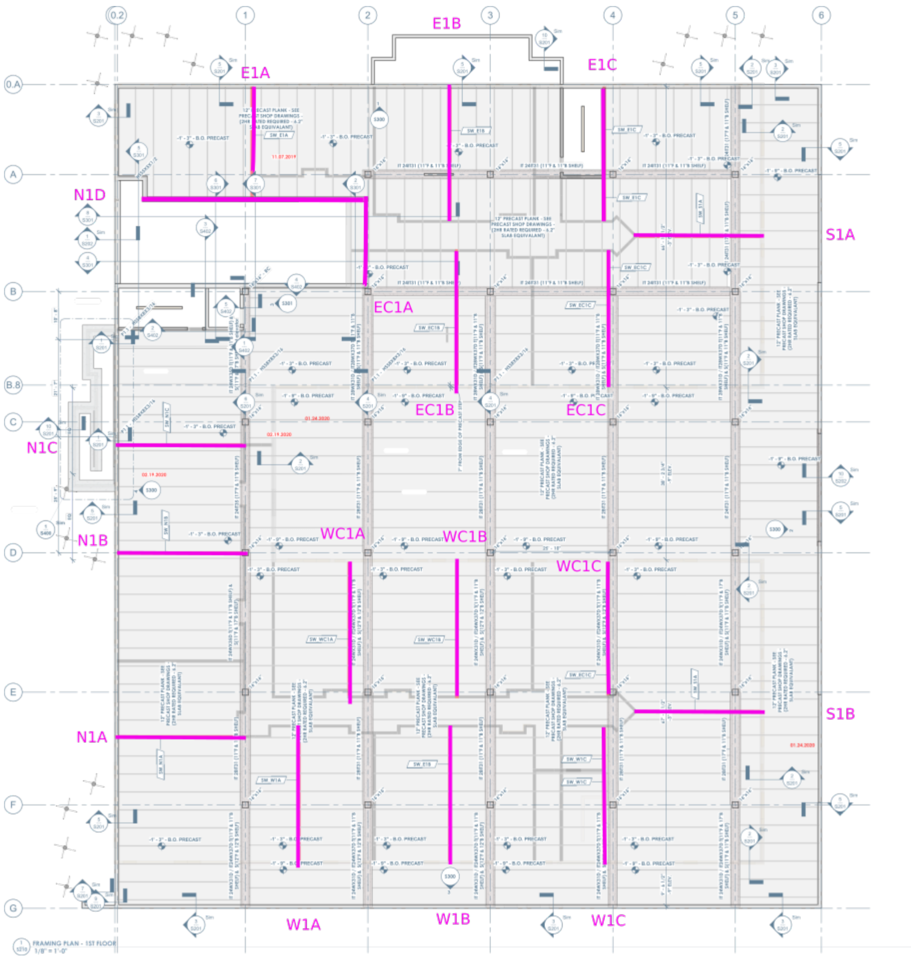


Figure 25: Shear walls on the first floor.

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 26). The hollow core planks are 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 35. See in figures 27 to 34 load distribution for each load case.

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

$$1 \text{ kN/m} = \quad 68.52178 \text{ lb/ft}$$
$$1 \text{ kN/m}^2 = \quad 20.885434 \text{ psf}$$

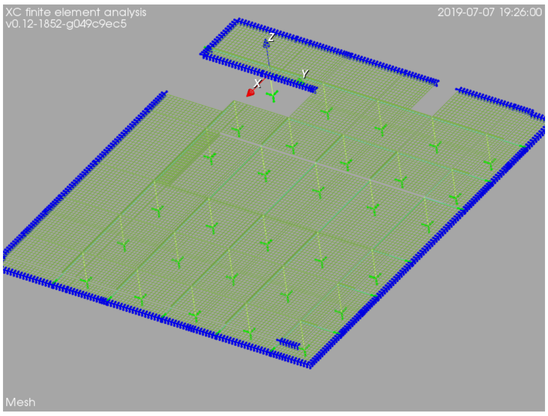


Figure 26: Elastic model, mesh.

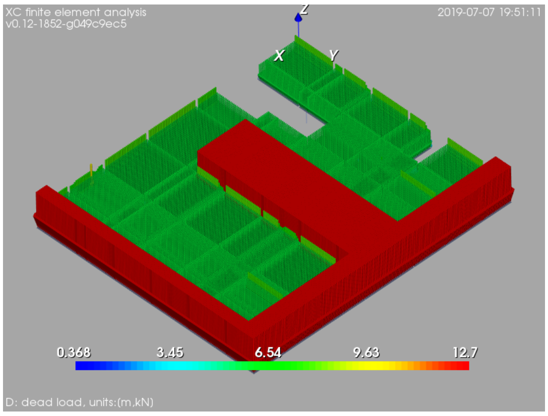


Figure 27: Load case D: dead load (include slab self-weight) [units: kN,m].

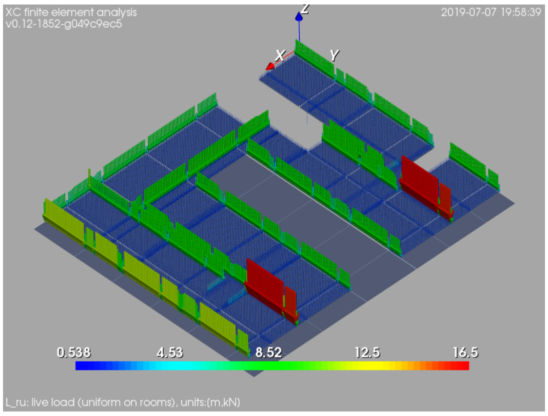


Figure 28: Load case Lru: live load (uniform on rooms) [units: kN,m].

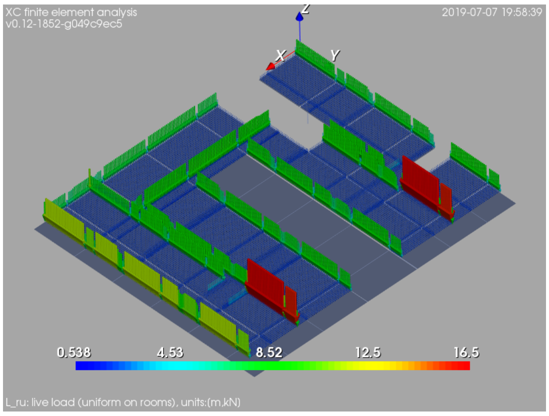


Figure 29: Load case Lrs: live load (staggered pattern on rooms) [units: kN,m].

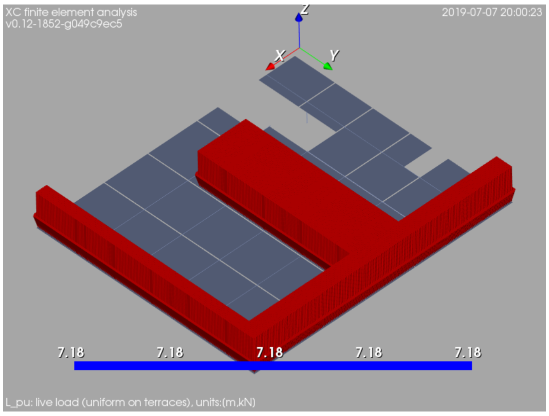


Figure 30: Load case Lpu: live load (uniform on patios) [units: kN,m].

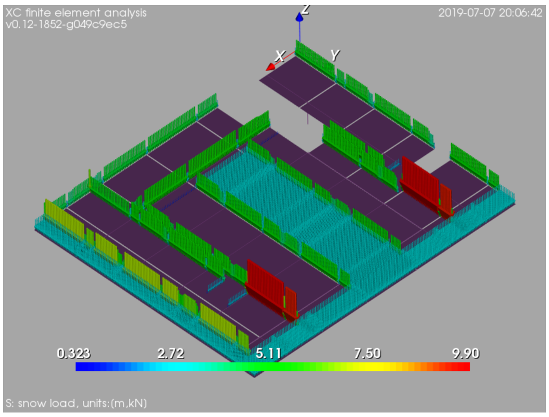


Figure 31: Load case S: snow [units: kN,m].

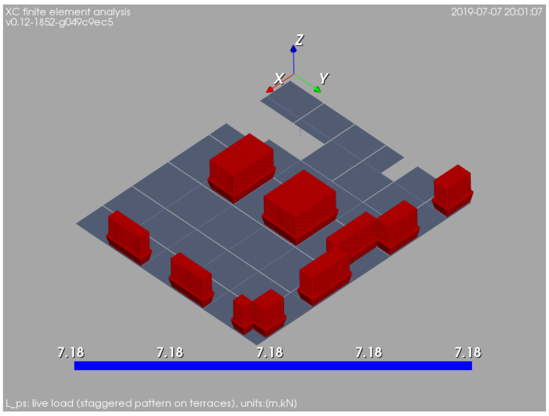


Figure 32: Load case Lps: live load (staggered pattern on patios) [units: kN,m].

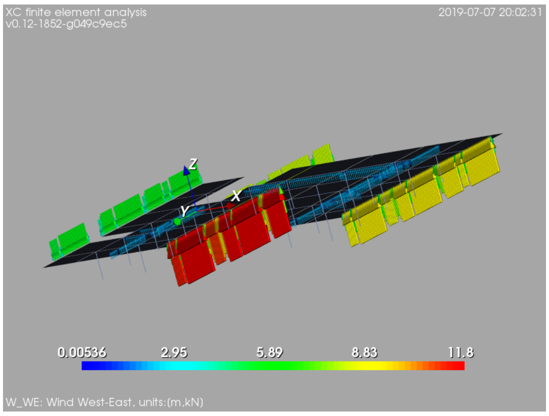


Figure 33: Load case W_WE: wind West-East [units: kN,m].

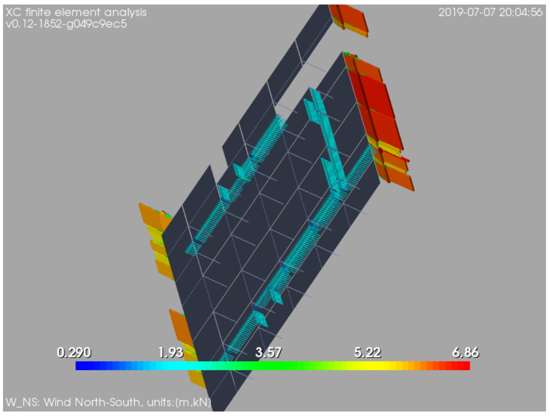
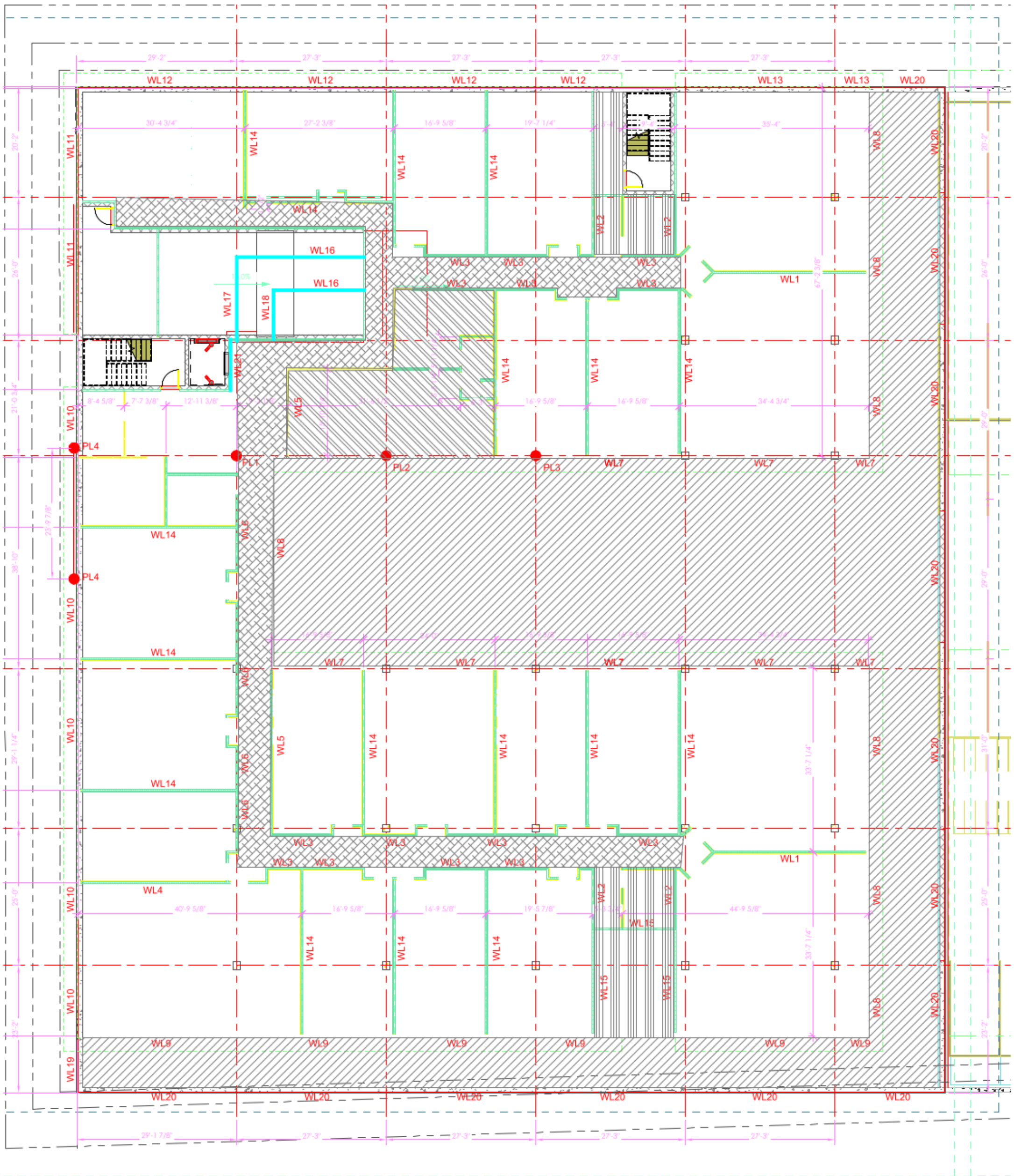


Figure 34: Load case W_NS: wind North-South [units: kN,m].



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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 35: 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 16: 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 16. 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 17.

Flexure design. The capacity factor for the bending in the longitudinal and transverse directions are shown in figures 36 and 37.

Shear design. The results of the shear strength verification are shown in the table 18. The results of the punching shear strength verification are shown in table 19.

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 17: 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 18: 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 19: Two-way shear design. Capacity factors

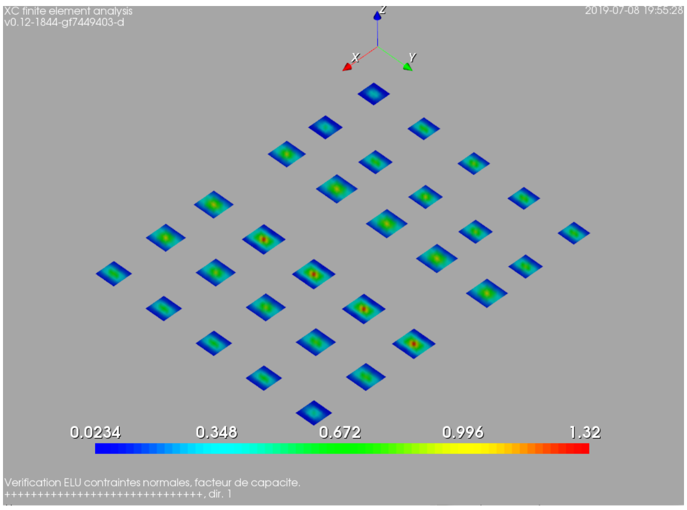


Figure 36: Flexure in the longitudinal direction. Capacity factor.

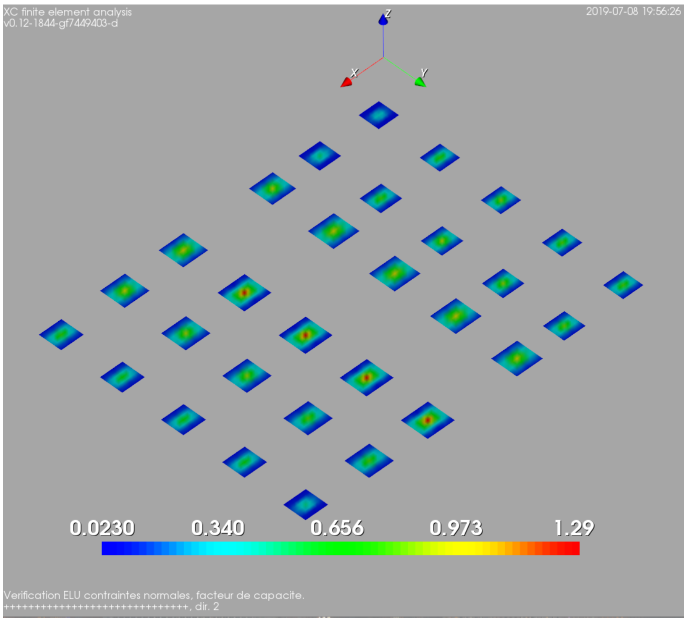


Figure 37: Flexure in the transverse direction. Capacity factor.

- 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 (54.29×10^6 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 20: Concrete walls reinforcing schedule

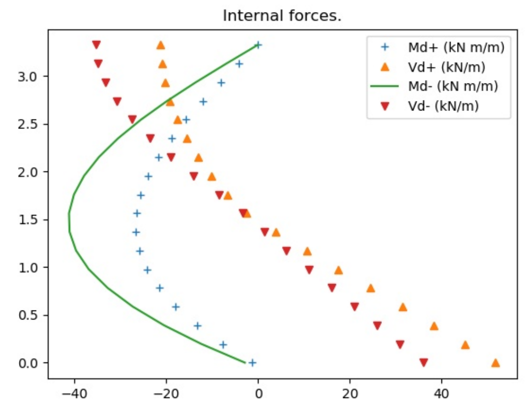
<table>
<tr><th colspan="2">T1</th></tr>
<tr><td>
</td><td>WALL GEOMETRY
Stem top thickness:
$b_{top} = 0.25\ m$
Stem height:
$h_{stem} = 3.15\ m$
Stem bottom thickness:
$b_{bottom} = 0.25\ m$
Footing thickness:
$b_{footing} = 0.36\ m$</td></tr>
<tr><td colspan="2">MATERIALS
Concrete: C4000 Steel: A615G60 Concrete cover: 55 mm</td></tr>
</table>

Table 21: 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 20.

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 21 to 26.

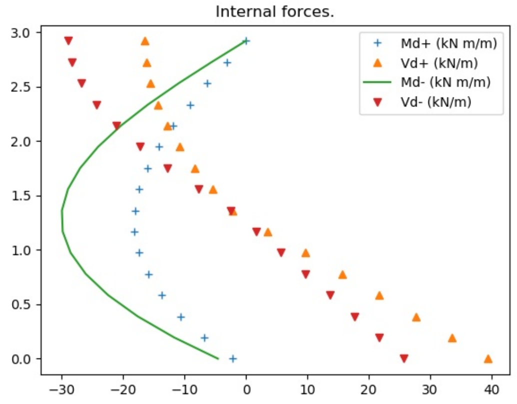
<table>
<tr><th colspan="2">T2</th></tr>
<tr><td>
</td><td>WALL GEOMETRY
Stem top thickness:
$b_{top} = 0.25\ m$
Stem height:
$h_{stem} = 2.74\ m$
Stem bottom thickness:
$b_{bottom} = 0.25\ m$
Footing thickness:
$b_{footing} = 0.36\ m$</td></tr>
<tr><td colspan="2">MATERIALS
Concrete: C4000 Steel: A615G60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

Table 22: Wall materials and dimensions T2

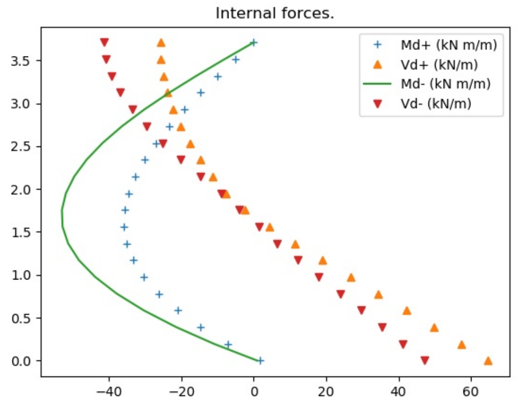
<table>
<tr><th colspan="2">T3</th></tr>
<tr><td>
</td><td>WALL GEOMETRY
Stem top thickness:
$b_{top} = 0.25\ m$
Stem height:
$h_{stem} = 3.53\ m$
Stem bottom thickness:
$b_{bottom} = 0.25\ m$
Footing thickness:
$b_{footing} = 0.36\ m$</td></tr>
<tr><td colspan="2">MATERIALS
Concrete: C4000 Steel: A615G60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

Table 23: Wall materials and dimensions T3

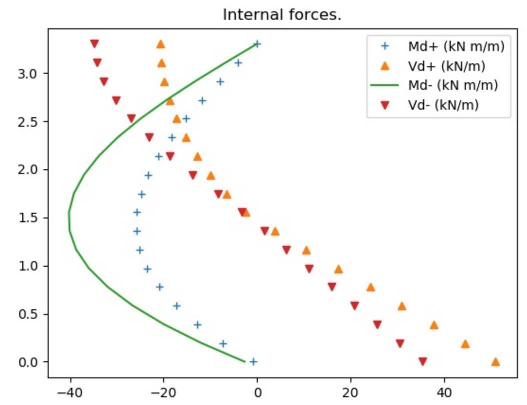
T4	
	WALL GEOMETRY Stem top thickness: $b_{top} = 0.25\ m$ Stem height: $h_{stem} = 3.12\ m$ Stem bottom thickness: $b_{bottom} = 0.25\ m$ Footing thickness: $b_{footing} = 0.36\ m$
MATERIALS Concrete: C4000 Steel: A615G60 Concrete cover: 55 mm	

Table 24: Wall materials and dimensions T4

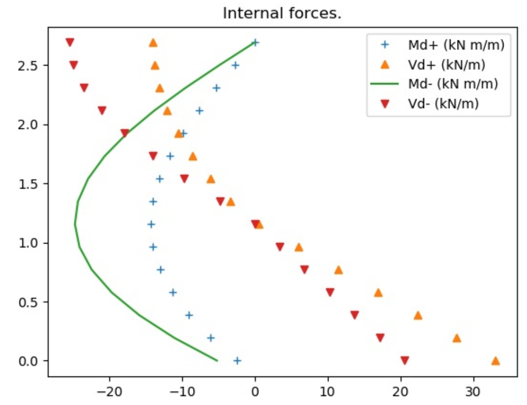
T5	
	WALL GEOMETRY Stem top thickness: $b_{top} = 0.25\ m$ Stem height: $h_{stem} = 2.51\ m$ Stem bottom thickness: $b_{bottom} = 0.25\ m$ Footing thickness: $b_{footing} = 0.36\ m$
MATERIALS Concrete: C4000 Steel: A615G60 Concrete cover: 55 mm	

Table 25: Wall materials and dimensions T5

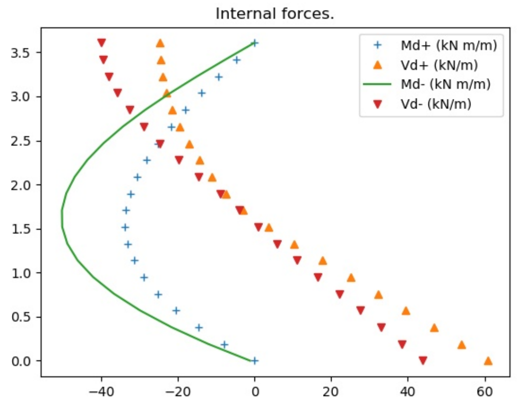
T6	
	**Wall geometry** Stem top thickness: $b_{top} = 0.25\ m$ Stem height: $h_{stem} = 3.43\ m$ Stem bottom thickness: $b_{bottom} = 0.25\ m$ Footing thickness: $b_{footing} = 0.36\ m$
Materials Concrete: C4000 Steel: A615G60 Concrete cover: 55 mm	

Table 26: 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 walls and steel structure bearing hollowcore 2nd floor

9.4.1 Materials

Concrete grade	C4000
Reinforcing steel grade	ASTM A615 Gr60
Structural steel	ASTM A36

9.4.2 Loads

Earth pressure

Internal friction angle (radians)	0.524
Coefficient of earth pressure at rest	0.5
Mass density of the soil (kg/m3)	1000.0
Horizontal force [N/m] due to earth pressure over East wall	20000.0

Dead load

DL West & East walls 2nd floor [N/m]	36620.0
DL East wall 1st floor [N/m]	42850.0
DL steel beam North facade [N/m]	7350.0
DL steel cantilever (West side) [N/m]	44650.0
DL steel cantilever (Central) [N/m]	44650.0
DL steel cantilever (East side) [N/m]	8078.7

Live load

LL West & East walls 2nd floor [N/m]	33030.0
LL East wall 1st floor [N/m]	10770.0
LL steel beam North facade [N/m]	4660.0
LL steel cantilever (West side) [N/m]	39040.0
LL steel cantilever (Central) [N/m]	39040.0
LL steel cantilever (East side) [N/m]	6241.5

Snow load

SL West & East walls 2nd floor [N/m]	13380.0
SL East wall 1st floor [N/m]	-80.0
SL steel beam North facade [N/m]	1750.0
SL steel cantilever (West side) [N/m]	15490.0
SL steel cantilever (Central) [N/m]	15490.0
SL steel cantilever (East side) [N/m]	2716.5

Wind load

WL West & East walls 2nd floor [N/m] (vertical)	8460.0
WL West & East walls 2nd floor [N/m] (horizontal)	1700.0
WL East wall 1st floor [N/m] (vertical)	500.0

9.4.3 Load combination

ELU01	$1.4 \times DL$
ELU02	$1.2 \times DL + 1.6 \times LL + 0.5 \times SL$
ELU03	$1.2 \times DL + 1.6 \times SL + 0.5 \times WL$
ELU04	$1.2 \times DL + 1.0 \times LL + 1.0 \times WL$
ELU05	$0.9 \times DL + 1.0 \times WL$

9.4.4 Structural model

A three-dimensional elastic computer model of the ramp RC walls and cantilever steel structure on 2nd floor is analyzed using XC. The finite element model is depicted in figure 38. See in figures 39 to 46 load distribution for each load case.

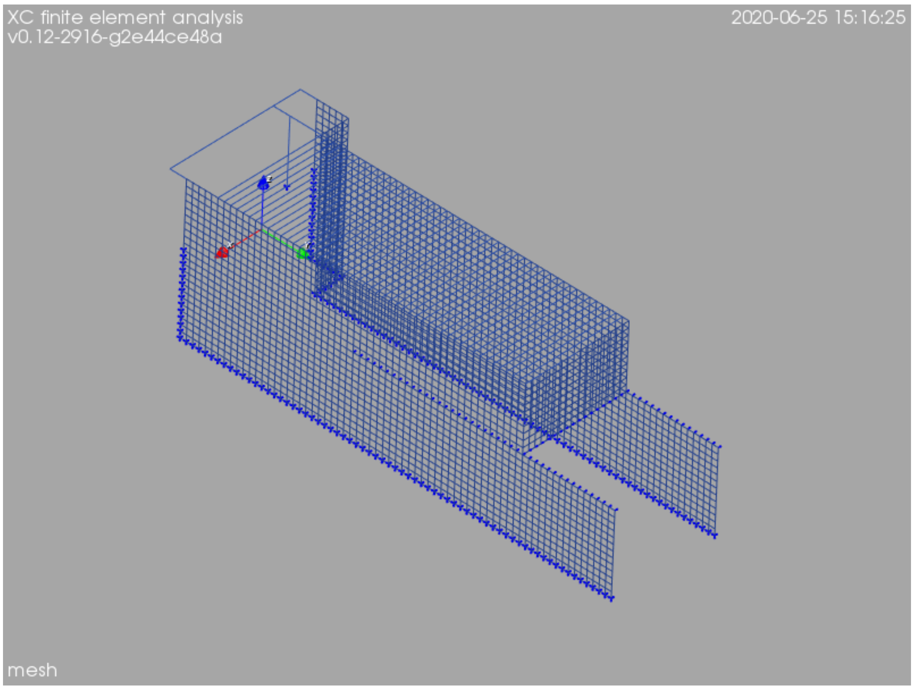


Figure 38: Ramp walls and cantilever, mesh.

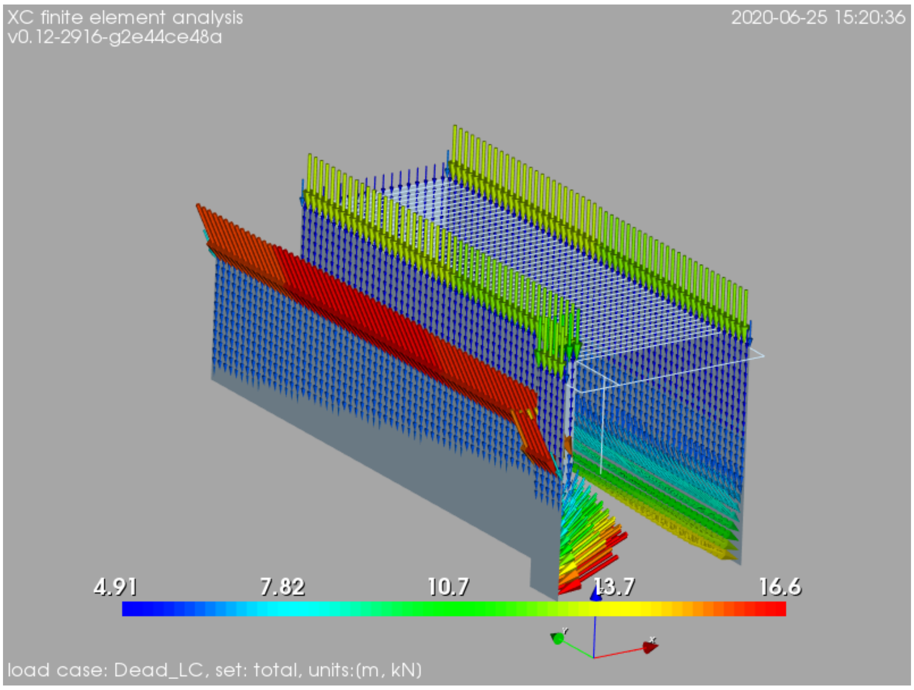


Figure 39: Ramp walls and cantilever, dead load over ramp wall.

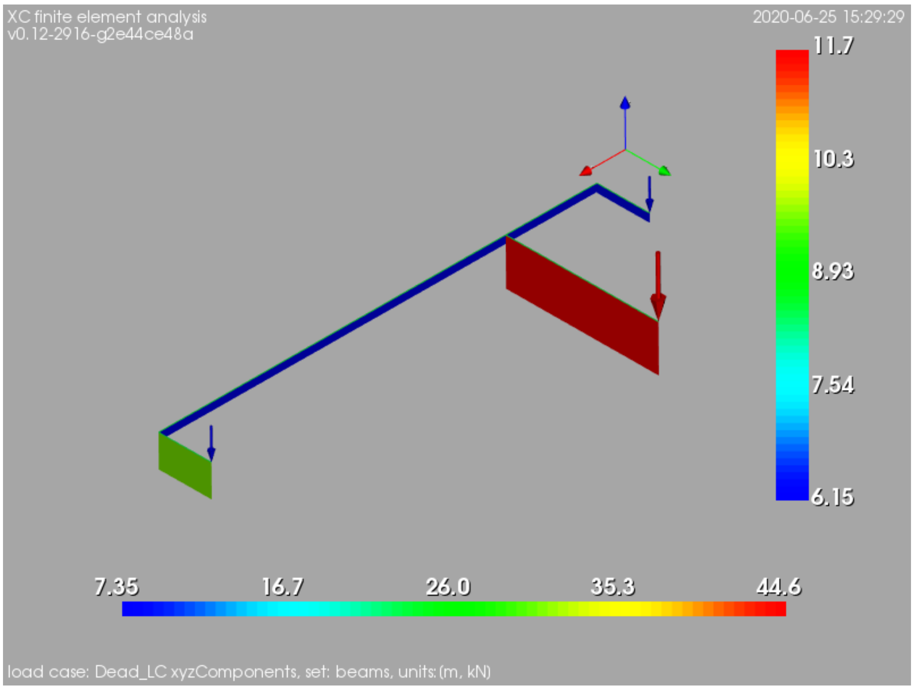


Figure 40: Ramp walls and cantilever, dead load over steel beams.

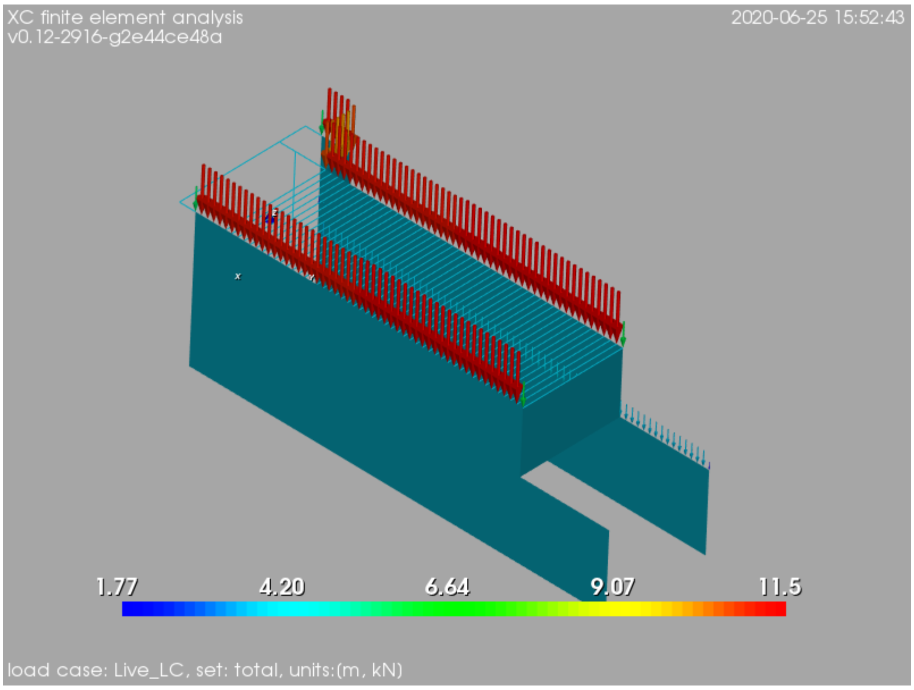


Figure 41: Ramp walls and cantilever, live load over ramp wall.

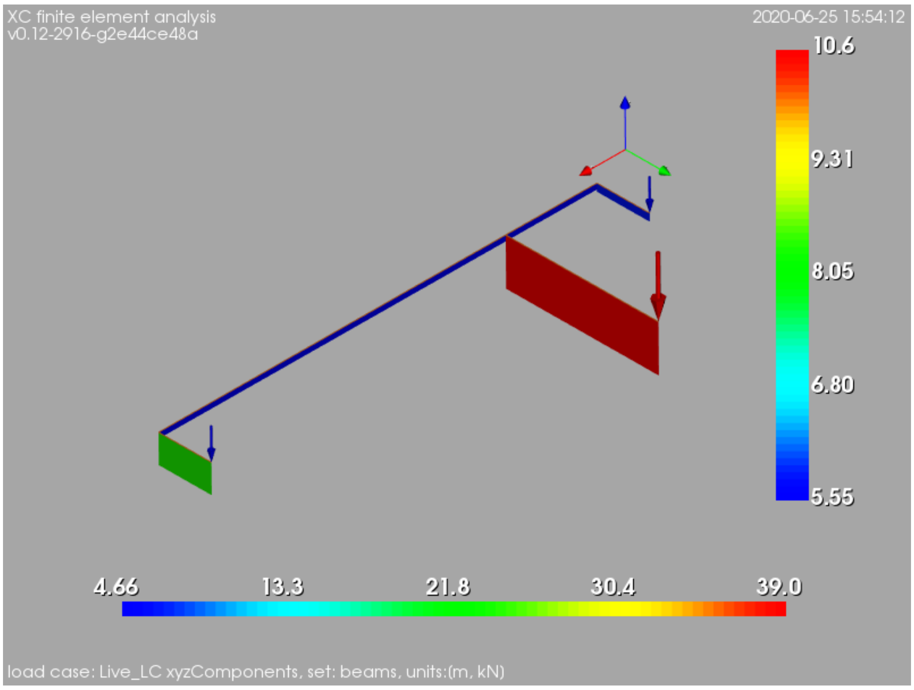


Figure 42: Ramp walls and cantilever, live load over steel beams.

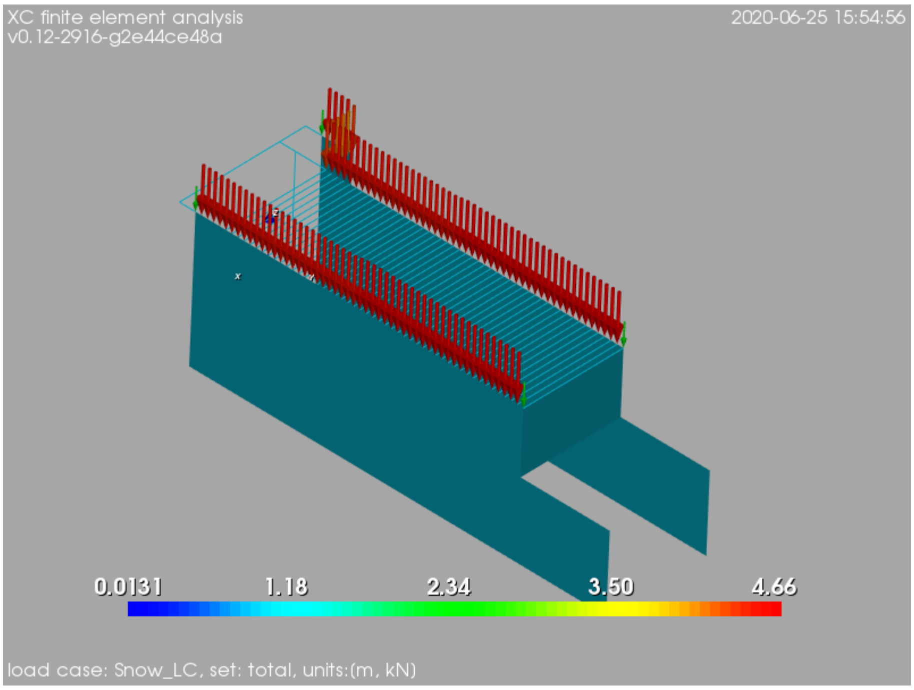


Figure 43: Ramp walls and cantilever, snow load over ramp wall.

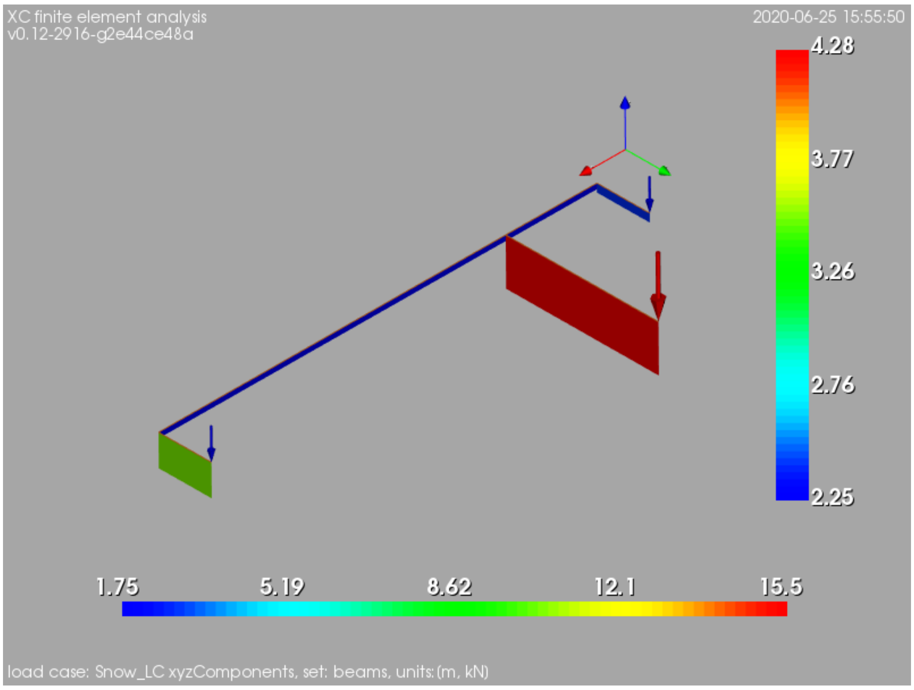


Figure 44: Ramp walls and cantilever, snow load over steel beams.

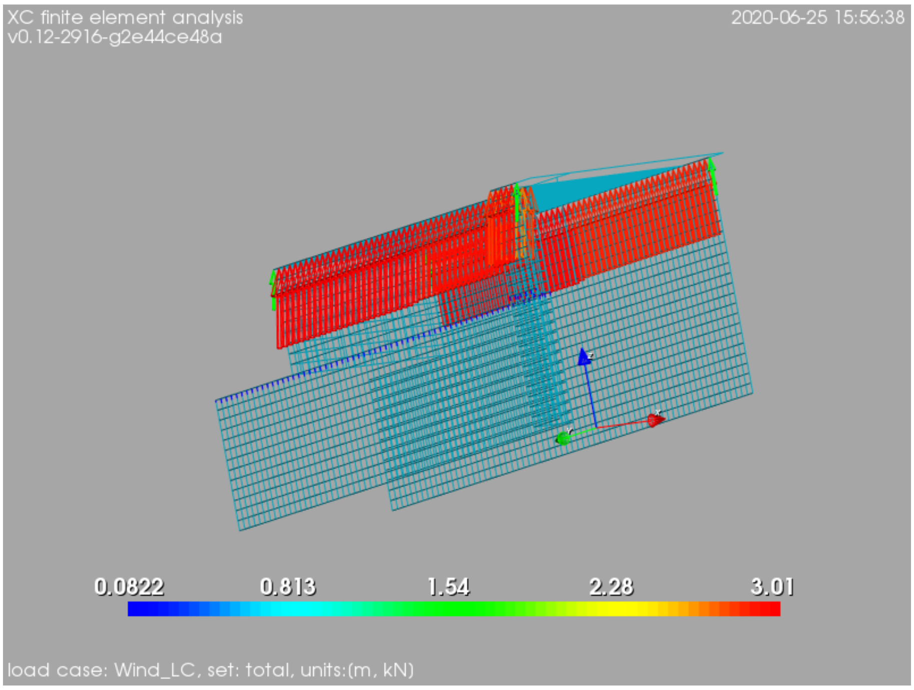


Figure 45: Ramp walls and cantilever, wind load over ramp wall.

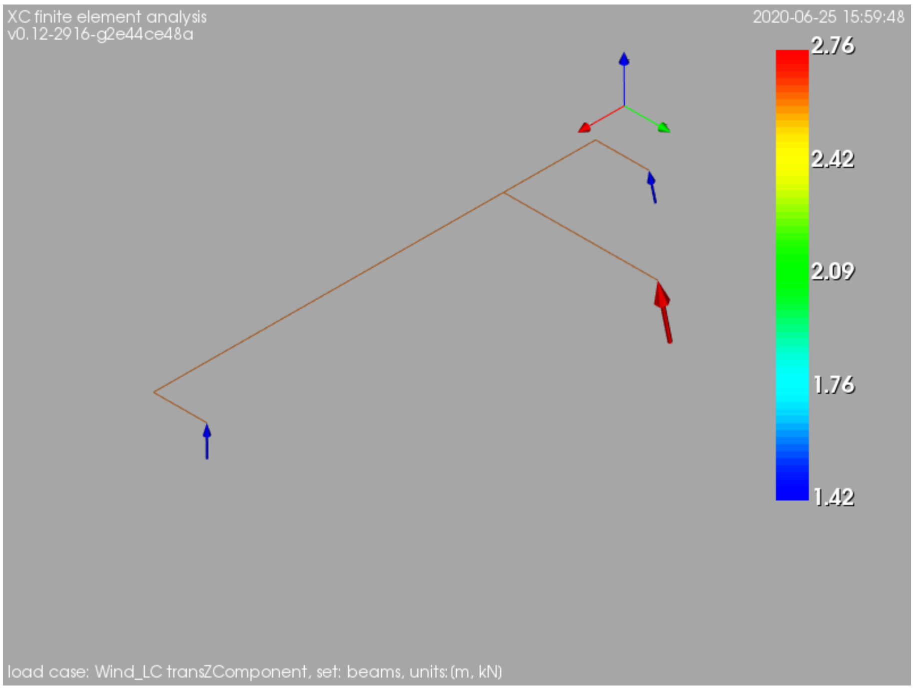
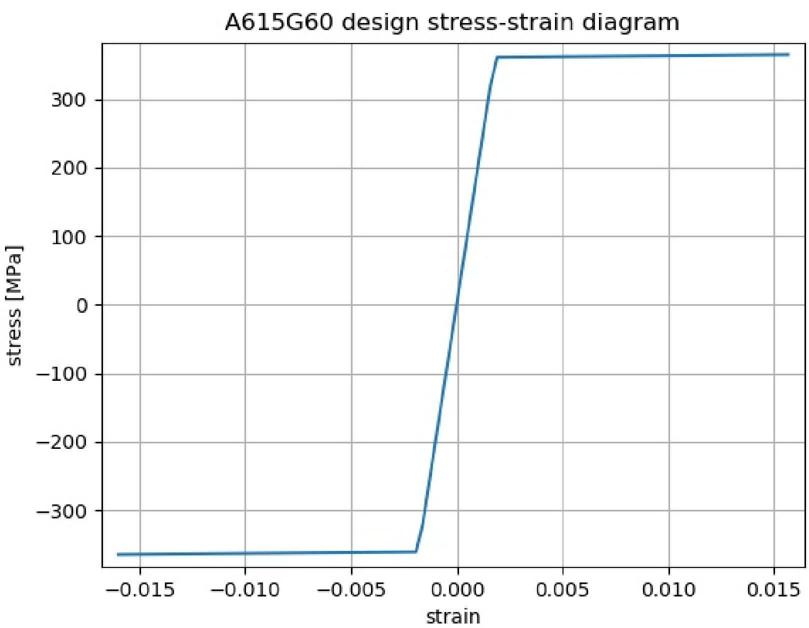
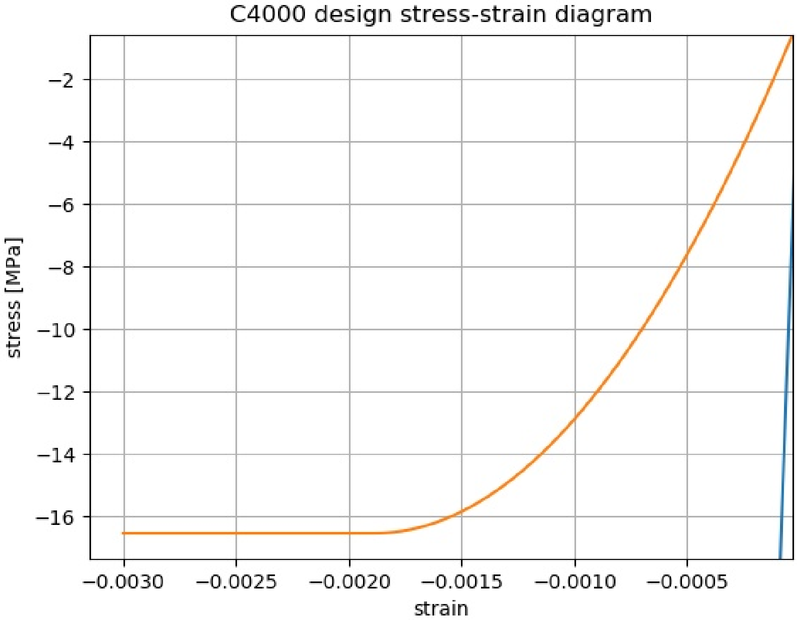


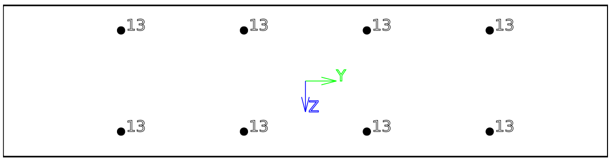
Figure 46: Ramp walls and cantilever, wind load over steel beams.

9.4.5 Material and RC-sections properties

9.4.6 Verification of reinforced-concrete walls




C4000 design stress-strain diagram
stress [MPa]
−2
−4
−6
−8
−10
−12
−14
−16
−0.0030
−0.0025
−0.0020
−0.0015
−0.0010
−0.0005
strain

geomEBwallRCSects1	
East basement wall. 1 direction.	
	width: $b = 1.00\ m$ depth: $h = 0.25\ m$

Materials - mechanical properties:

Concrete: C4000	Modulus of elasticity: $E_c = 24.86\ GPa$
Steel: A615G60	Modulus of elasticity: $E_s = 200.00\ GPa$

Sections - geometric and mechanical characteristics:

Gross section:

$A_{gross} = 0.250\ m^2$

C.O.G.: $(0.000, 0.000)\ m$

Inertia tensor (cm^4): $\begin{pmatrix} 43.91 & 0.00 & 0.00 \\ 0.00 & 13.02 & 0.00 \\ 0.00 & 0.00 & 208.33 \end{pmatrix}$

Homogenized section:

$A_{homog.} = 0.262\ m^2$

C.O.G.: $(-0.000, 0.000)\ m$

Inertia tensor (cm^4): $\begin{pmatrix} 43.91 & 0.00 & 0.00 \\ 0.00 & 13.83 & 0.00 \\ 0.00 & 0.00 & 214.30 \end{pmatrix}$

Passive reinforcement:

Total area $A_s = 10.13\ cm^2$ Geometric quantity $\rho = 4.05‰$

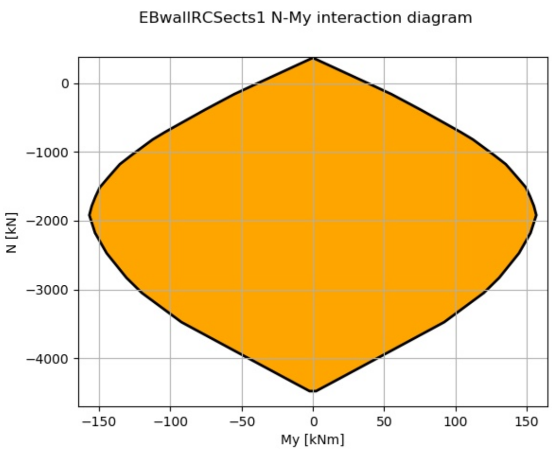
Layers of main reinforcement:

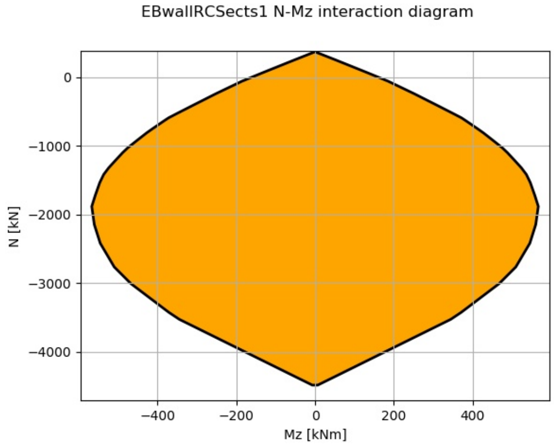
Id	N^o bars	ϕ (mm)	area (cm^2)	c. geom. (‰)	eff. cover (cm)	y_{COG} (m)	z_{COG} (m)
	4	13.0	1.27	0.51	4.1	-0.000	-0.084
	4	13.0	1.27	0.51	4.1	-0.000	0.084

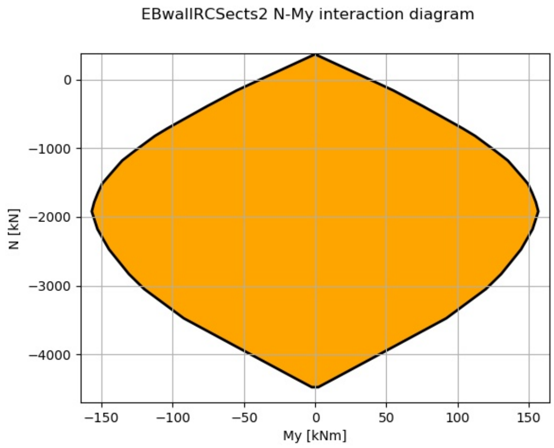
Layers of shear reinforcement:

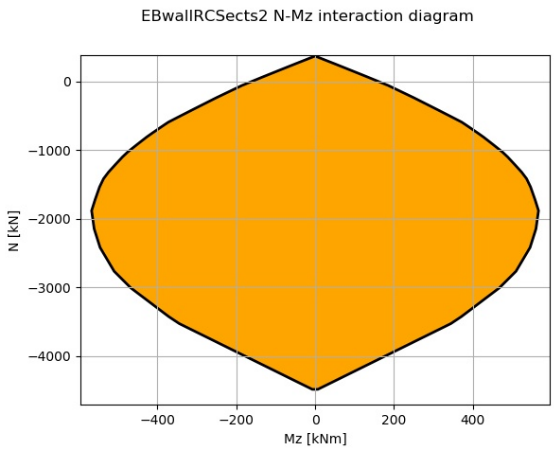
Id	N^o branch	ϕ (mm)	area (cm^2)	spac. (cm)	area/m (cm^2/m)	α $(^\circ)$	β $(^\circ)$
noName	0.0	0.0	0.00	20.0	0.00	90.0	45.0
noName	0.0	0.0	0.00	20.0	0.00	90.0	45.0

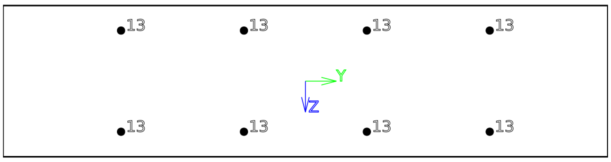
Table 27: East basement wall. 1 direction. (geomEBwallRCSects1).




EBwallRCSects1 N-Mz interaction diagram
N [kN]
Mz [kNm]
0
−1000
−2000
−3000
−4000
−400
−200
0
200
400


EBwallRCSects2 N-My interaction diagram
N [kN]
My [kNm]
0
−1000
−2000
−3000
−4000
−150
−100
−50
0
50
100
150


EBwallRCSects2 N-Mz interaction diagram
N [kN]
Mz [kNm]
0
−1000
−2000
−3000
−4000
−400
−200
0
200
400

geomEBwallRCSects2	
East basement wall. 2 direction.	
	width: $b = 1.00\ m$ depth: $h = 0.25\ m$

Materials - mechanical properties:

Concrete: C4000	Modulus of elasticity: $E_c = 24.86\ GPa$
Steel: A615G60	Modulus of elasticity: $E_s = 200.00\ GPa$

Sections - geometric and mechanical characteristics:

Gross section:

$A_{gross} = 0.250\ m^2$

C.O.G.: $(0.000, 0.000)\ m$

Inertia tensor (cm^4): $\begin{pmatrix} 43.91 & 0.00 & 0.00 \\ 0.00 & 13.02 & 0.00 \\ 0.00 & 0.00 & 208.33 \end{pmatrix}$

Homogenized section:

$A_{homog.} = 0.262\ m^2$

C.O.G.: $(-0.000, 0.000)\ m$

Inertia tensor (cm^4): $\begin{pmatrix} 43.91 & 0.00 & 0.00 \\ 0.00 & 13.83 & 0.00 \\ 0.00 & 0.00 & 214.30 \end{pmatrix}$

Passive reinforcement:

Total area $A_s = 10.13\ cm^2$ Geometric quantity $\rho = 4.05‰$

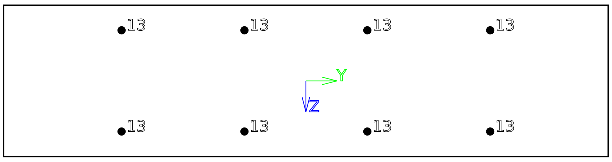
Layers of main reinforcement:

Id	N^o bars	ϕ (mm)	area (cm^2)	c. geom. (‰)	eff. cover (cm)	y_{COG} (m)	z_{COG} (m)
	4	13.0	1.27	0.51	4.1	-0.000	-0.084
	4	13.0	1.27	0.51	4.1	-0.000	0.084

Layers of shear reinforcement:

Id	N^o branch	ϕ (mm)	area (cm^2)	spac. (cm)	area/m (cm^2/m)	α (°)	β (°)
noName	0.0	0.0	0.00	20.0	0.00	90.0	45.0
noName	0.0	0.0	0.00	20.0	0.00	90.0	45.0

Table 28: East basement wall. 2 direction. (geomEBwallRCSects2).

geomWBwallRCSects1	
West basement wall. 1 direction.	
	width: $b = 1.00\ m$ depth: $h = 0.25\ m$

Materials - mechanical properties:

Concrete: C4000	Modulus of elasticity: $E_c = 24.86\ GPa$
Steel: A615G60	Modulus of elasticity: $E_s = 200.00\ GPa$

Sections - geometric and mechanical characteristics:

Gross section:

$A_{gross} = 0.250\ m^2$

C.O.G.: $(0.000, 0.000)\ m$

$$\text{Inertia tensor } (cm^4): \begin{pmatrix} 43.91 & 0.00 & 0.00 \\ 0.00 & 13.02 & 0.00 \\ 0.00 & 0.00 & 208.33 \end{pmatrix}$$

Homogenized section:

$A_{homog.} = 0.262\ m^2$

C.O.G.: $(-0.000, 0.000)\ m$

$$\text{Inertia tensor } (cm^4): \begin{pmatrix} 43.91 & 0.00 & 0.00 \\ 0.00 & 13.83 & 0.00 \\ 0.00 & 0.00 & 214.30 \end{pmatrix}$$

Passive reinforcement:

Total area $A_s = 10.13\ cm^2$ Geometric quantity $\rho = 4.05‰$

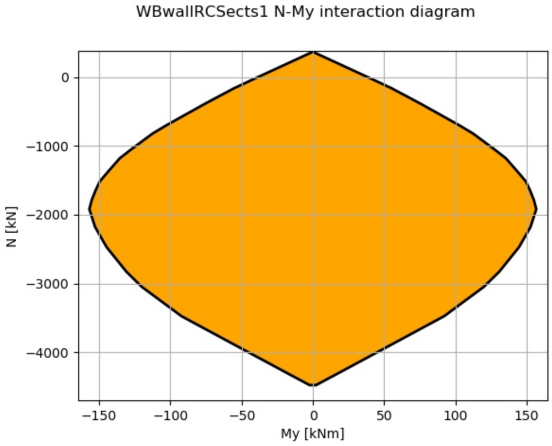
Layers of main reinforcement:

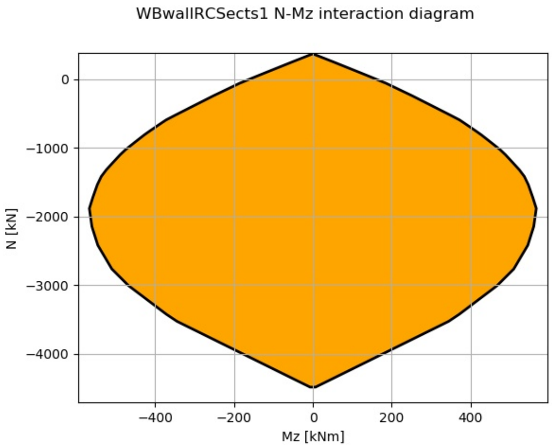
Id	N^o bars	ϕ (mm)	area (cm^2)	c. geom. $(‰)$	eff. cover (cm)	y_{COG} (m)	z_{COG} (m)
	4	13.0	1.27	0.51	4.1	-0.000	-0.084
	4	13.0	1.27	0.51	4.1	-0.000	0.084

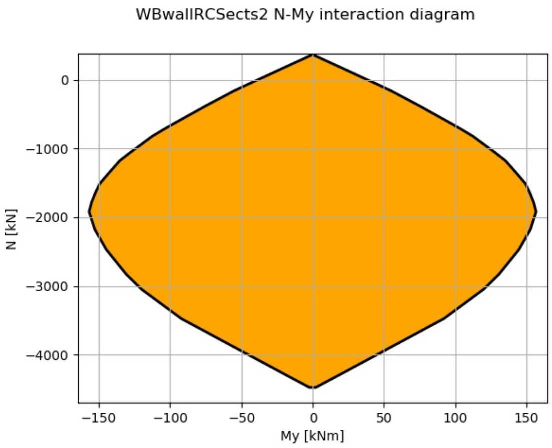
Layers of shear reinforcement:

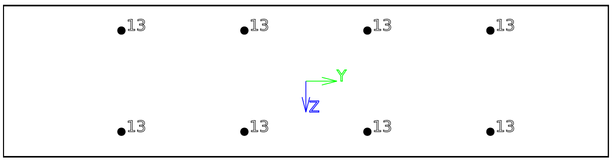
Id	N^o branch	ϕ (mm)	area (cm^2)	spac. (cm)	area/m (cm^2/m)	α $(^\circ)$	β $(^\circ)$
noName	0.0	0.0	0.00	20.0	0.00	90.0	45.0
noName	0.0	0.0	0.00	20.0	0.00	90.0	45.0

Table 29: West basement wall. 1 direction. (geomWBwallRCSects1).


WBwallRCSects1 N-My interaction diagram
0
−1000
−2000
−3000
−4000
N [kN]
−150
−100
−50
0
50
100
150
My [kNm]


WBwallRCSects1 N-Mz interaction diagram
0
−1000
−2000
−3000
−4000
N [kN]
−400
−200
0
200
400
Mz [kNm]


WBwallRCSects2 N-My interaction diagram
0
−1000
−2000
−3000
−4000
N [kN]
−150
−100
−50
0
50
100
150
My [kNm]

geomWBwallRCSects2	
West basement wall. 2 direction.	
	width: $b = 1.00\ m$ depth: $h = 0.25\ m$

Materials - mechanical properties:

Concrete: C4000	Modulus of elasticity: $E_c = 24.86\ GPa$
Steel: A615G60	Modulus of elasticity: $E_s = 200.00\ GPa$

Sections - geometric and mechanical characteristics:

Gross section:

$A_{gross} = 0.250\ m^2$

C.O.G.: $(0.000, 0.000)\ m$

Inertia tensor (cm^4): $\begin{pmatrix} 43.91 & 0.00 & 0.00 \\ 0.00 & 13.02 & 0.00 \\ 0.00 & 0.00 & 208.33 \end{pmatrix}$

Homogenized section:

$A_{homog.} = 0.262\ m^2$

C.O.G.: $(-0.000, 0.000)\ m$

Inertia tensor (cm^4): $\begin{pmatrix} 43.91 & 0.00 & 0.00 \\ 0.00 & 13.83 & 0.00 \\ 0.00 & 0.00 & 214.30 \end{pmatrix}$

Passive reinforcement:

Total area $A_s = 10.13\ cm^2$ Geometric quantity $\rho = 4.05‰$

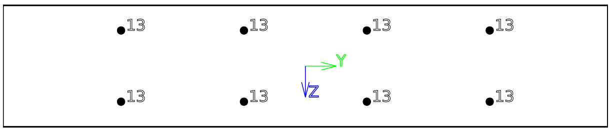
Layers of main reinforcement:

Id	N^o bars	ϕ (mm)	area (cm^2)	c. geom. (‰)	eff. cover (cm)	y_{COG} (m)	z_{COG} (m)
	4	13.0	1.27	0.51	4.1	-0.000	-0.084
	4	13.0	1.27	0.51	4.1	-0.000	0.084

Layers of shear reinforcement:

Id	N^o branch	ϕ (mm)	area (cm^2)	spac. (cm)	area/m (cm^2/m)	α $(^\circ)$	β $(^\circ)$
noName	0.0	0.0	0.00	20.0	0.00	90.0	45.0
noName	0.0	0.0	0.00	20.0	0.00	90.0	45.0

Table 30: West basement wall. 2 direction. (geomWBwallRCSects2).

geomE1FwallRCSects1	
East first floor wall. 1 direction.	
	width: $b = 1.00\ m$ depth: $h = 0.20\ m$

Materials - mechanical properties:

Concrete: C4000	Modulus of elasticity: $E_c = 24.86\ GPa$
Steel: A615G60	Modulus of elasticity: $E_s = 200.00\ GPa$

Sections - geometric and mechanical characteristics:

Gross section:

$A_{gross} = 0.200\ m^2$

C.O.G.: $(0.000, -0.000)\ m$

Inertia tensor (cm^4): $\begin{pmatrix} 23.20 & 0.00 & 0.00 \\ 0.00 & 6.67 & 0.00 \\ 0.00 & 0.00 & 166.67 \end{pmatrix}$

Homogenized section:

$A_{homog.} = 0.212\ m^2$

C.O.G.: $(-0.000, -0.000)\ m$

Inertia tensor (cm^4): $\begin{pmatrix} 23.20 & 0.00 & 0.00 \\ 0.00 & 7.07 & 0.00 \\ 0.00 & 0.00 & 172.63 \end{pmatrix}$

Passive reinforcement:

Total area $A_s = 10.13\ cm^2$ Geometric quantity $\rho = 5.07‰$

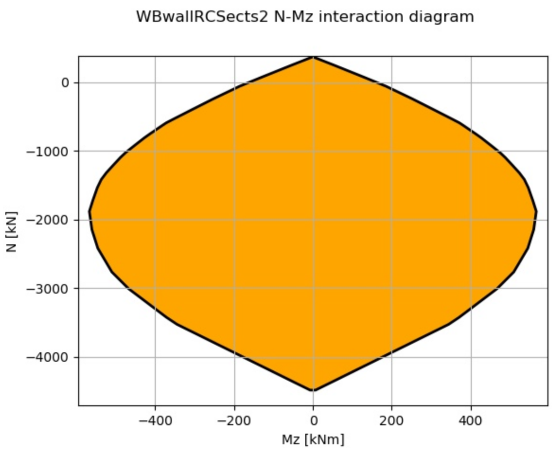
Layers of main reinforcement:

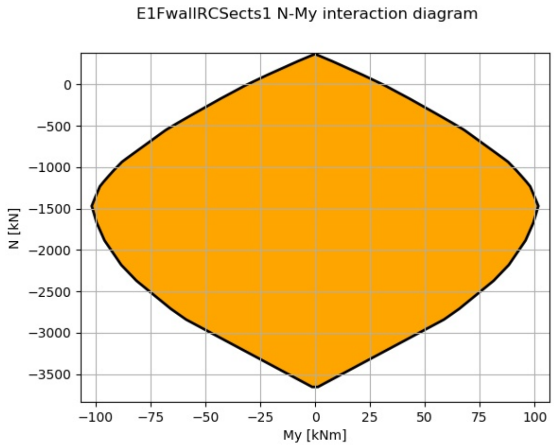
Id	N^o bars	ϕ (mm)	area (cm^2)	c. geom. $(‰)$	eff. cover (cm)	y_{COG} (m)	z_{COG} (m)
	4	13.0	1.27	0.63	4.1	-0.000	-0.059
	4	13.0	1.27	0.63	4.1	-0.000	0.059

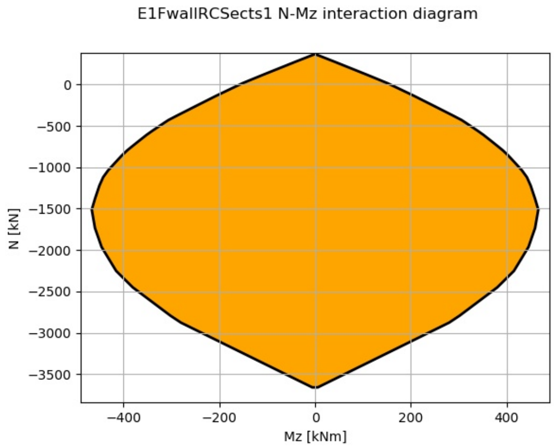
Layers of shear reinforcement:

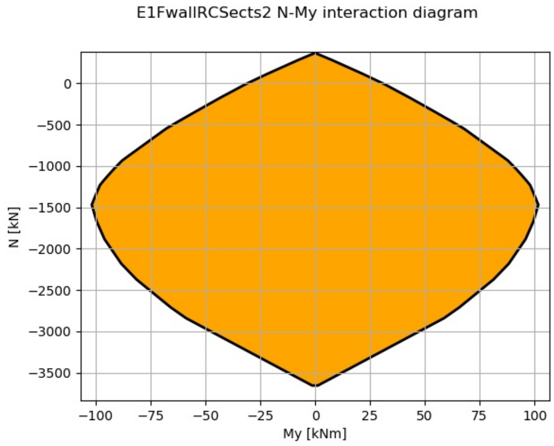
Id	N^o branch	ϕ (mm)	area (cm^2)	spac. (cm)	area/m (cm^2/m)	α $(^\circ)$	β $(^\circ)$
noName	0.0	0.0	0.00	20.0	0.00	90.0	45.0
noName	0.0	0.0	0.00	20.0	0.00	90.0	45.0

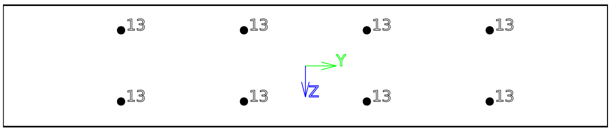
Table 31: East first floor wall. 1 direction. (geomE1FwallRCSects1).




E1FwallRCSects1 N-My interaction diagram
0
−500
−1000
−1500
−2000
−2500
−3000
−3500
N [kN]
−100
−75
−50
−25
0
25
50
75
100
My [kNm]


E1FwallRCSects1 N-Mz interaction diagram
0
−500
−1000
−1500
−2000
−2500
−3000
−3500
N [kN]
−400
−200
0
200
400
Mz [kNm]


E1FwallRCSects2 N-My interaction diagram
0
−500
−1000
−1500
−2000
−2500
−3000
−3500
N [kN]
−100
−75
−50
−25
0
25
50
75
100
My [kNm]

geomE1FwallRCSects2	
East first floor wall. 2 direction.	
	width: $b = 1.00\ m$ depth: $h = 0.20\ m$

Materials - mechanical properties:

Concrete: C4000	Modulus of elasticity: $E_c = 24.86\ GPa$
Steel: A615G60	Modulus of elasticity: $E_s = 200.00\ GPa$

Sections - geometric and mechanical characteristics:

Gross section:

$A_{gross} = 0.200\ m^2$

C.O.G.: $(0.000, -0.000)\ m$

Inertia tensor (cm^4): $\begin{pmatrix} 23.20 & 0.00 & 0.00 \\ 0.00 & 6.67 & 0.00 \\ 0.00 & 0.00 & 166.67 \end{pmatrix}$

Homogenized section:

$A_{homog.} = 0.212\ m^2$

C.O.G.: $(-0.000, -0.000)\ m$

Inertia tensor (cm^4): $\begin{pmatrix} 23.20 & 0.00 & 0.00 \\ 0.00 & 7.07 & 0.00 \\ 0.00 & 0.00 & 172.63 \end{pmatrix}$

Passive reinforcement:

Total area $A_s = 10.13\ cm^2$ Geometric quantity $\rho = 5.07‰$

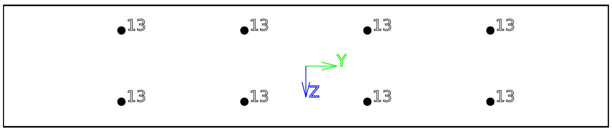
Layers of main reinforcement:

Id	N^o bars	ϕ (mm)	area (cm^2)	c. geom. (‰)	eff. cover (cm)	y_{COG} (m)	z_{COG} (m)
	4	13.0	1.27	0.63	4.1	-0.000	-0.059
	4	13.0	1.27	0.63	4.1	-0.000	0.059

Layers of shear reinforcement:

Id	N^o branch	ϕ (mm)	area (cm^2)	spac. (cm)	area/m (cm^2/m)	α (°)	β (°)
noName	0.0	0.0	0.00	20.0	0.00	90.0	45.0
noName	0.0	0.0	0.00	20.0	0.00	90.0	45.0

Table 32: East first floor wall. 2 direction. (geomE1FwallRCSects2).

geomW1FwallRCSects1	
West first floor wall. 1 direction.	
	width: $b = 1.00\ m$ depth: $h = 0.20\ m$

Materials - mechanical properties:

Concrete: C4000	Modulus of elasticity: $E_c = 24.86\ GPa$
Steel: A615G60	Modulus of elasticity: $E_s = 200.00\ GPa$

Sections - geometric and mechanical characteristics:

Gross section:

$A_{gross} = 0.200\ m^2$

C.O.G.: $(0.000, -0.000)\ m$

Inertia tensor (cm^4): $\begin{pmatrix} 23.20 & 0.00 & 0.00 \\ 0.00 & 6.67 & 0.00 \\ 0.00 & 0.00 & 166.67 \end{pmatrix}$

Homogenized section:

$A_{homog.} = 0.212\ m^2$

C.O.G.: $(-0.000, -0.000)\ m$

Inertia tensor (cm^4): $\begin{pmatrix} 23.20 & 0.00 & 0.00 \\ 0.00 & 7.07 & 0.00 \\ 0.00 & 0.00 & 172.63 \end{pmatrix}$

Passive reinforcement:

Total area $A_s = 10.13\ cm^2$ Geometric quantity $\rho = 5.07‰$

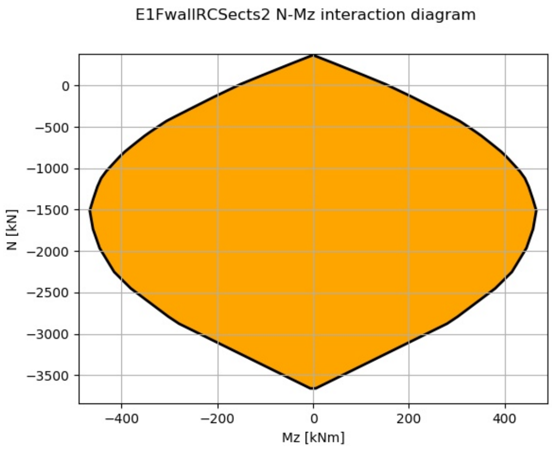
Layers of main reinforcement:

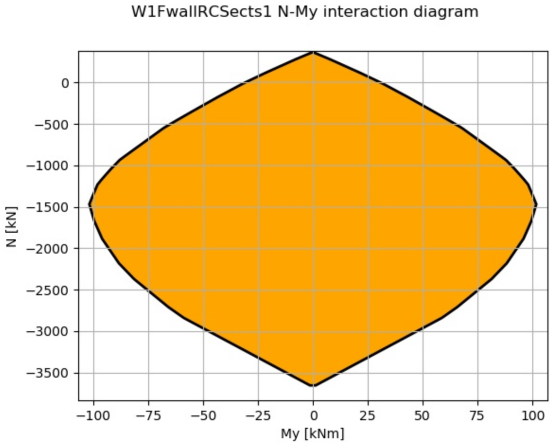
Id	N^o bars	ϕ (mm)	area (cm^2)	c. geom. (‰)	eff. cover (cm)	y_{COG} (m)	z_{COG} (m)
	4	13.0	1.27	0.63	4.1	-0.000	-0.059
	4	13.0	1.27	0.63	4.1	-0.000	0.059

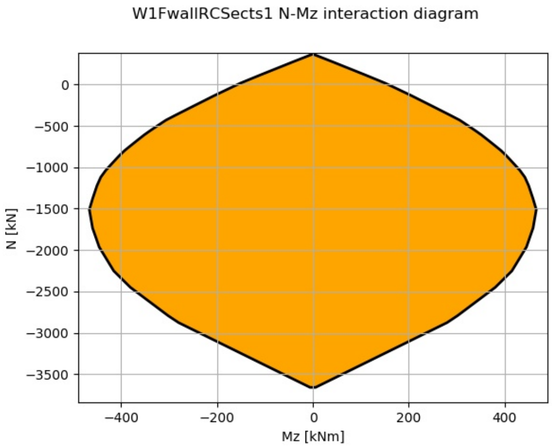
Layers of shear reinforcement:

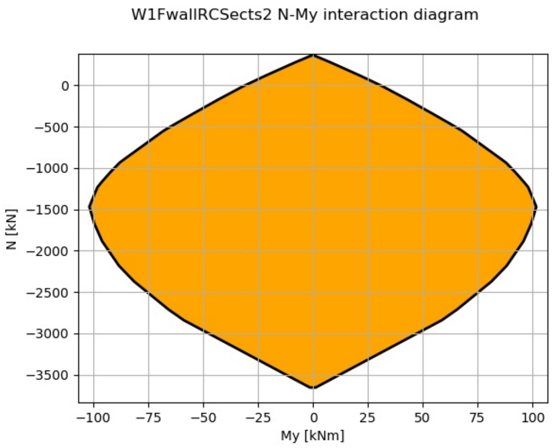
Id	N^o branch	ϕ (mm)	area (cm^2)	spac. (cm)	area/m (cm^2/m)	α (°)	β (°)
noName	0.0	0.0	0.00	20.0	0.00	90.0	45.0
noName	0.0	0.0	0.00	20.0	0.00	90.0	45.0

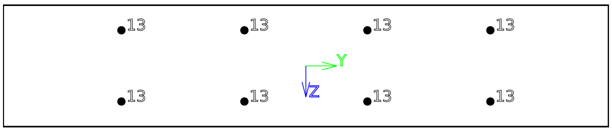
Table 33: West first floor wall. 1 direction. (geomW1FwallRCSects1).




W1FwallRCSects1 N-My interaction diagram
N [kN]
My [kNm]


W1FwallRCSects1 N-Mz interaction diagram
N [kN]
Mz [kNm]


W1FwallRCSects2 N-My interaction diagram
N [kN]
My [kNm]

geomW1FwallRCSects2	
West first floor wall. 2 direction.	
	width: $b = 1.00\ m$ depth: $h = 0.20\ m$

Materials - mechanical properties:

Concrete: C4000	Modulus of elasticity: $E_c = 24.86\ GPa$
Steel: A615G60	Modulus of elasticity: $E_s = 200.00\ GPa$

Sections - geometric and mechanical characteristics:

Gross section:

$A_{gross} = 0.200\ m^2$

C.O.G.: $(0.000, -0.000)\ m$

Inertia tensor (cm^4): $\begin{pmatrix} 23.20 & 0.00 & 0.00 \\ 0.00 & 6.67 & 0.00 \\ 0.00 & 0.00 & 166.67 \end{pmatrix}$

Homogenized section:

$A_{homog.} = 0.212\ m^2$

C.O.G.: $(-0.000, -0.000)\ m$

Inertia tensor (cm^4): $\begin{pmatrix} 23.20 & 0.00 & 0.00 \\ 0.00 & 7.07 & 0.00 \\ 0.00 & 0.00 & 172.63 \end{pmatrix}$

Passive reinforcement:

Total area $A_s = 10.13\ cm^2$ Geometric quantity $\rho = 5.07‰$

Layers of main reinforcement:

Id	N^o bars	ϕ (mm)	area (cm^2)	c. geom. (‰)	eff. cover (cm)	y_{COG} (m)	z_{COG} (m)
	4	13.0	1.27	0.63	4.1	-0.000	-0.059
	4	13.0	1.27	0.63	4.1	-0.000	0.059

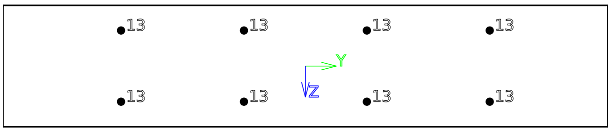
Layers of shear reinforcement:

Id	N^o branch	ϕ (mm)	area (cm^2)	spac. (cm)	area/m (cm^2/m)	α (°)	β (°)
noName	0.0	0.0	0.00	20.0	0.00	90.0	45.0
noName	0.0	0.0	0.00	20.0	0.00	90.0	45.0

Table 34: West first floor wall. 2 direction. (geomW1FwallRCSects2).

geomS1FwallRCSects1

South first floor wall. 1 direction.



width:
$b = 1.00\ m$
depth:
$h = 0.20\ m$

Materials - mechanical properties:

Concrete: C4000	Modulus of elasticity: $E_c = 24.86\ GPa$
Steel: A615G60	Modulus of elasticity: $E_s = 200.00\ GPa$

Sections - geometric and mechanical characteristics:

Gross section:

$A_{gross} = 0.200\ m^2$

C.O.G.: $(0.000, -0.000)\ m$

Inertia tensor (cm^4): $\begin{pmatrix} 23.20 & 0.00 & 0.00 \\ 0.00 & 6.67 & 0.00 \\ 0.00 & 0.00 & 166.67 \end{pmatrix}$

Homogenized section:

$A_{homog.} = 0.212\ m^2$

C.O.G.: $(-0.000, -0.000)\ m$

Inertia tensor (cm^4): $\begin{pmatrix} 23.20 & 0.00 & 0.00 \\ 0.00 & 7.07 & 0.00 \\ 0.00 & 0.00 & 172.63 \end{pmatrix}$

Passive reinforcement:

Total area $A_s = 10.13\ cm^2$ Geometric quantity $\rho = 5.07‰$

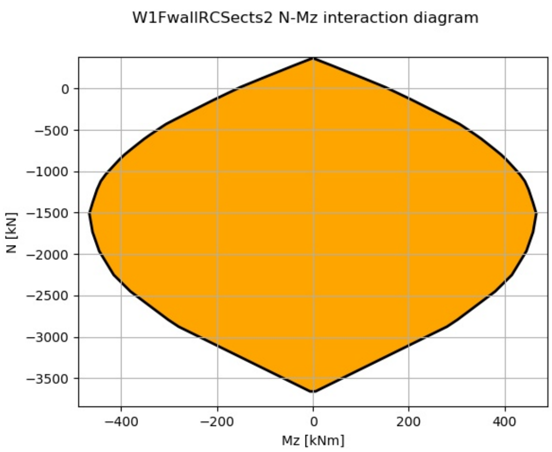
Layers of main reinforcement:

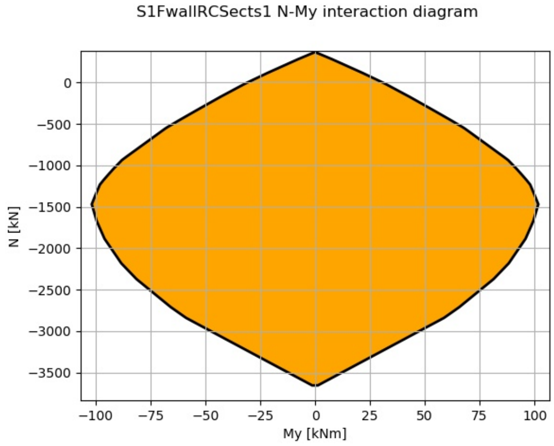
Id	N^o bars	ϕ (mm)	area (cm^2)	c. geom. (‰)	eff. cover (cm)	y_{COG} (m)	z_{COG} (m)
	4	13.0	1.27	0.63	4.1	-0.000	-0.059
	4	13.0	1.27	0.63	4.1	-0.000	0.059

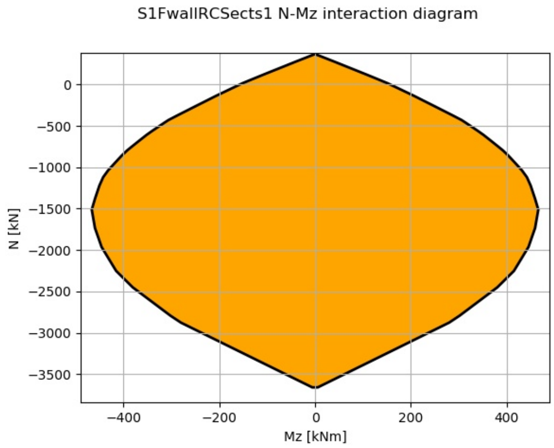
Layers of shear reinforcement:

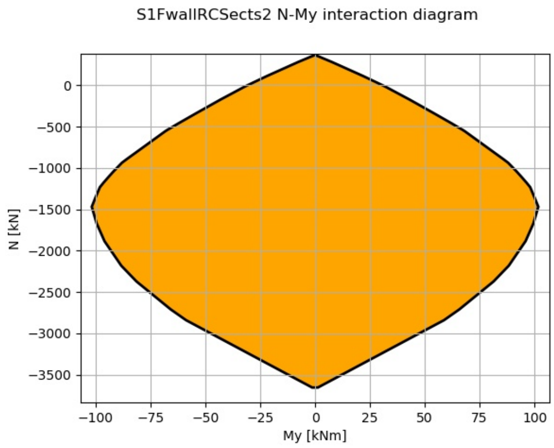
Id	N^o branch	ϕ (mm)	area (cm^2)	spac. (cm)	area/m (cm^2/m)	α ($^{\circ}$)	β ($^{\circ}$)
noName	0.0	0.0	0.00	20.0	0.00	90.0	45.0
noName	0.0	0.0	0.00	20.0	0.00	90.0	45.0

Table 35: South first floor wall. 1 direction. (geomS1FwallRCSects1).



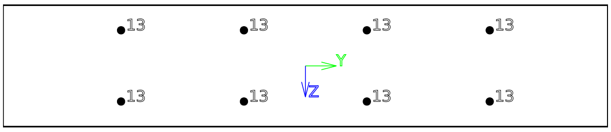

S1FwallRCSects1 N-My interaction diagram
N [kN]
My [kNm]


S1FwallRCSects1 N-Mz interaction diagram
N [kN]
Mz [kNm]


S1FwallRCSects2 N-My interaction diagram
N [kN]
My [kNm]

geomS1FwallRCSects2

South first floor wall. 2 direction.



width:
$b = 1.00\ m$
depth:
$h = 0.20\ m$

Materials - mechanical properties:

Concrete: C4000	Modulus of elasticity: $E_c = 24.86\ GPa$
Steel: A615G60	Modulus of elasticity: $E_s = 200.00\ GPa$

Sections - geometric and mechanical characteristics:

Gross section:

$A_{gross} = 0.200\ m^2$

C.O.G.: $(0.000, -0.000)\ m$

Inertia tensor (cm^4): $\begin{pmatrix} 23.20 & 0.00 & 0.00 \\ 0.00 & 6.67 & 0.00 \\ 0.00 & 0.00 & 166.67 \end{pmatrix}$

Homogenized section:

$A_{homog.} = 0.212\ m^2$

C.O.G.: $(-0.000, -0.000)\ m$

Inertia tensor (cm^4): $\begin{pmatrix} 23.20 & 0.00 & 0.00 \\ 0.00 & 7.07 & 0.00 \\ 0.00 & 0.00 & 172.63 \end{pmatrix}$

Passive reinforcement:

Total area $A_s = 10.13\ cm^2$ Geometric quantity $\rho = 5.07‰$

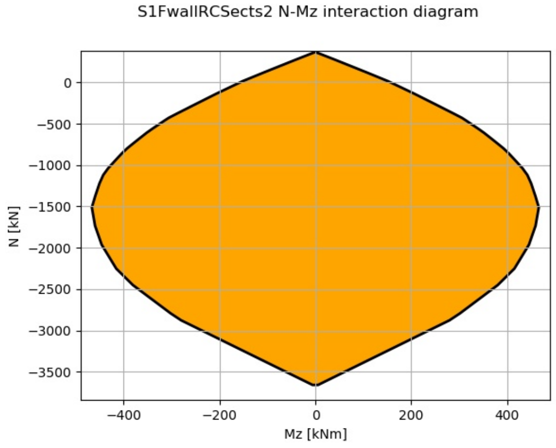
Layers of main reinforcement:

Id	N^o bars	ϕ (mm)	area (cm^2)	c. geom. (‰)	eff. cover (cm)	y_{COG} (m)	z_{COG} (m)
	4	13.0	1.27	0.63	4.1	-0.000	-0.059
	4	13.0	1.27	0.63	4.1	-0.000	0.059

Layers of shear reinforcement:

Id	N^o branch	ϕ (mm)	area (cm^2)	spac. (cm)	area/m (cm^2/m)	α (°)	β (°)
noName	0.0	0.0	0.00	20.0	0.00	90.0	45.0
noName	0.0	0.0	0.00	20.0	0.00	90.0	45.0

Table 36: South first floor wall. 2 direction. (geomS1FwallRCSects2).


S1FwallRCSects2 N-Mz interaction diagram
N [kN]
0
−500
−1000
−1500
−2000
−2500
−3000
−3500
−400
−200
0
200
400
Mz [kNm]

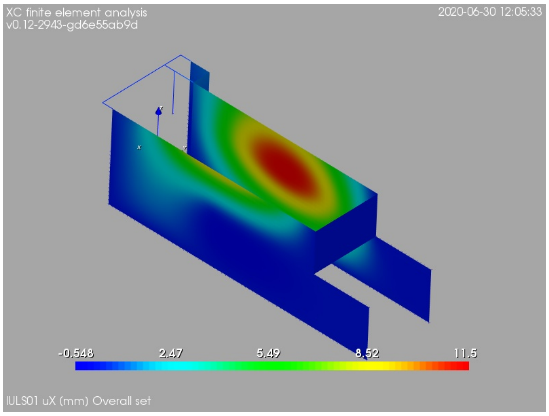


Figure 47: ULS01. Overall set, displacement in global X direction, [mm]

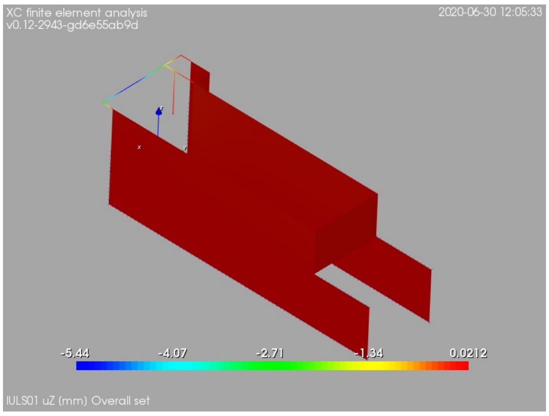


Figure 48: ULS01. Overall set, displacement in global Z direction, [mm]

Displacements and internal forces in ULS

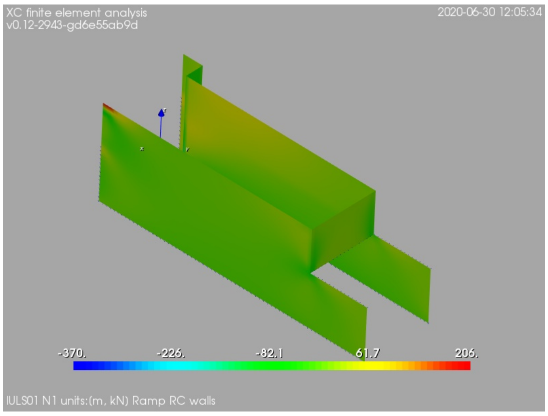


Figure 49: ULS01. Ramp rc walls, internal axial force in local direction 1, units:[m, kN]

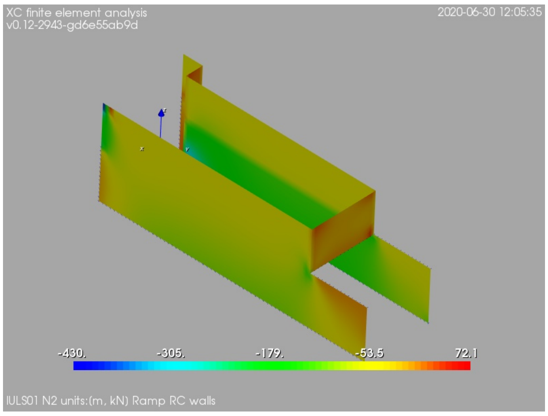


Figure 50: ULS01. Ramp rc walls, internal axial force in local direction 2, units:[m, kN]

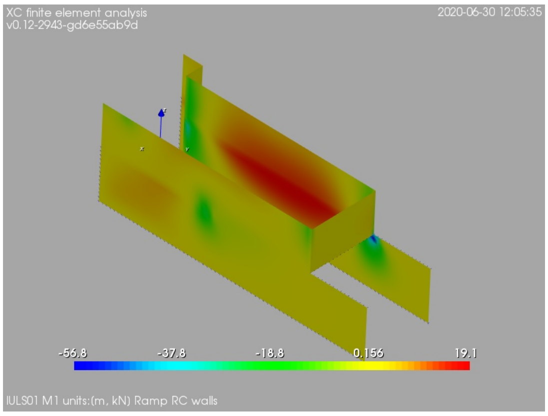


Figure 51: ULS01. Ramp rc walls, bending moment about local axis 1, units:[m, kN]

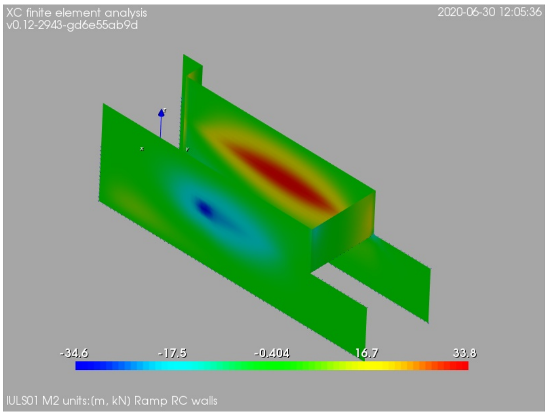


Figure 52: ULS01. Ramp rc walls, bending moment about local axis 2, units:[m, kN]

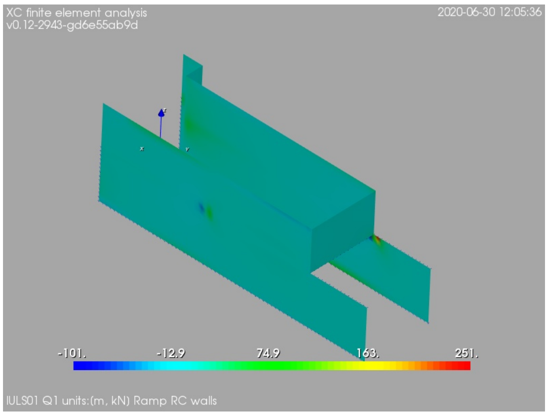


Figure 53: ULS01. Ramp rc walls, internal shear force in local direction 1, units:[m, kN]

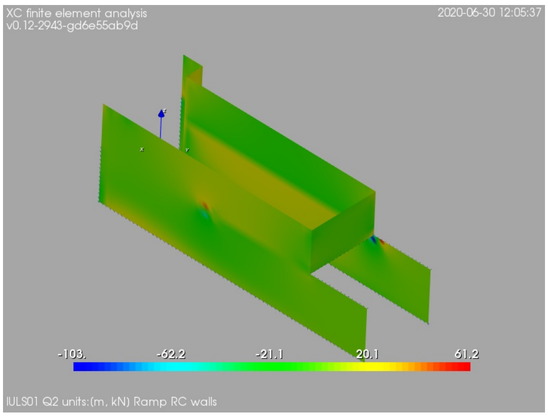


Figure 54: ULS01. Ramp rc walls, internal shear force in local direction 2, units:[m, kN]

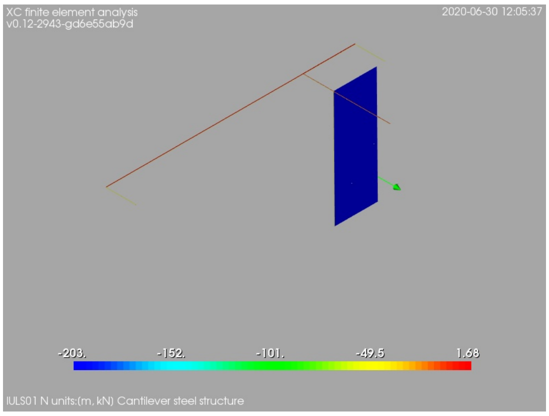


Figure 55: ULS01. Cantilever steel structure, internal axial force, units:[m, kN]

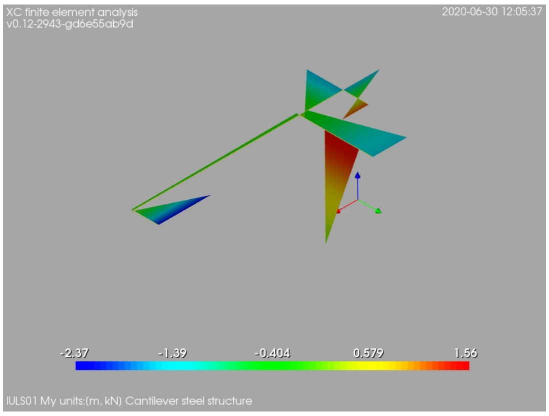


Figure 56: ULS01. Cantilever steel structure, bending moment about local axis y, units:[m, kN]

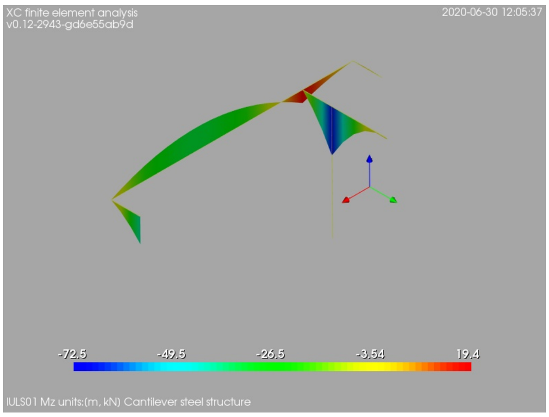


Figure 57: ULS01. Cantilever steel structure, bending moment about local axis z, units:[m, kN]

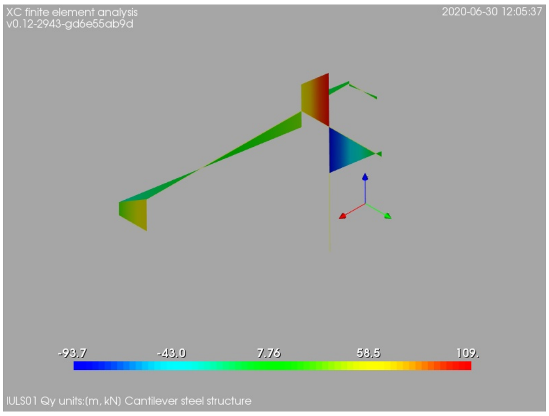


Figure 58: ULS01. Cantilever steel structure, internal shear force in local direction y, units:[m, kN]

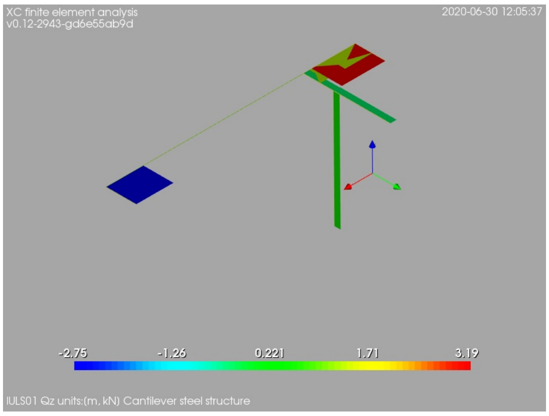


Figure 59: ULS01. Cantilever steel structure, internal shear force in local direction z, units:[m, kN]

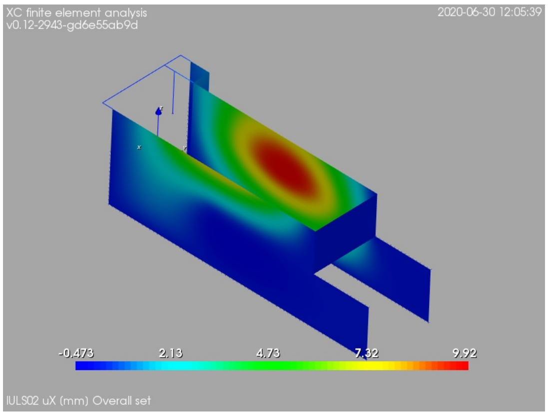


Figure 60: ULS02. Overall set, displacement in global X direction, [mm]

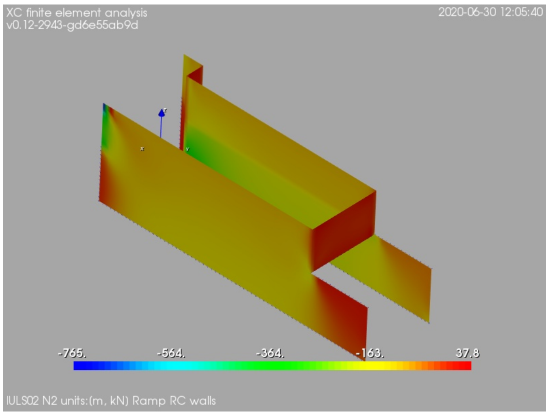


Figure 63: ULS02. Ramp rc walls, internal axial force in local direction 2, units:[m, kN]

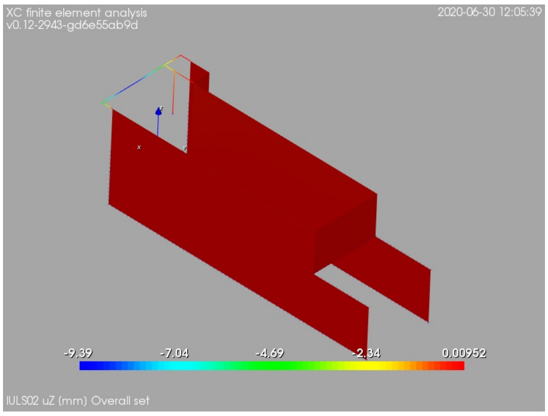


Figure 61: ULS02. Overall set, displacement in global Z direction, [mm]

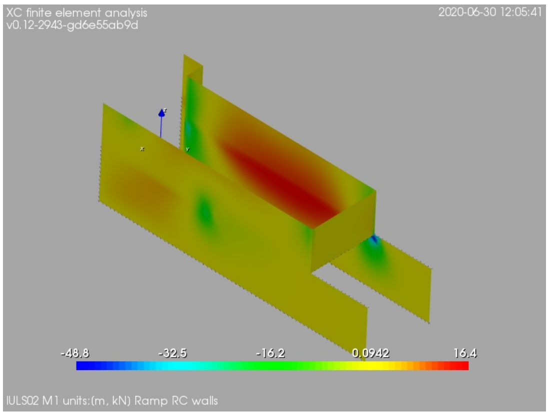


Figure 64: ULS02. Ramp rc walls, bending moment about local axis 1, units:[m, kN]

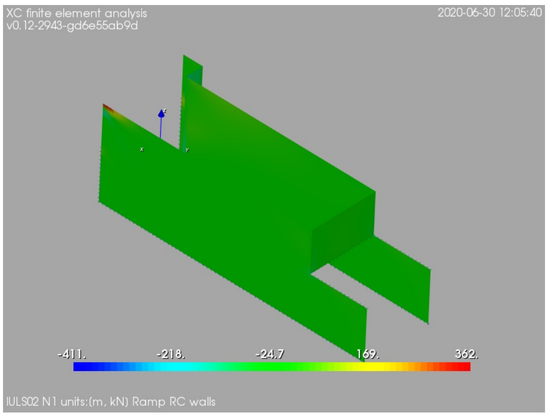


Figure 62: ULS02. Ramp rc walls, internal axial force in local direction 1, units:[m, kN]

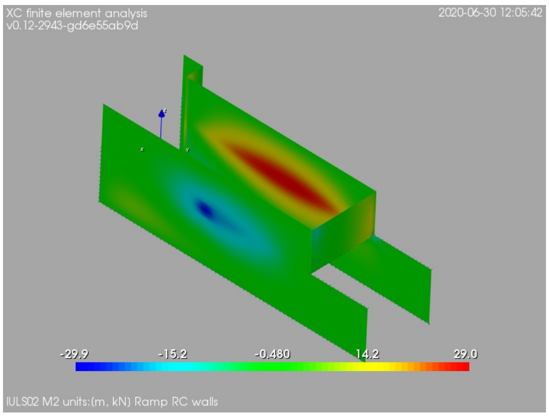


Figure 65: ULS02. Ramp rc walls, bending moment about local axis 2, units:[m, kN]

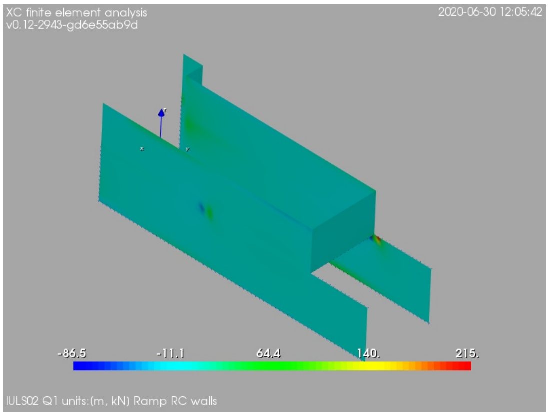


Figure 66: ULS02. Ramp rc walls, internal shear force in local direction 1, units:[m, kN]

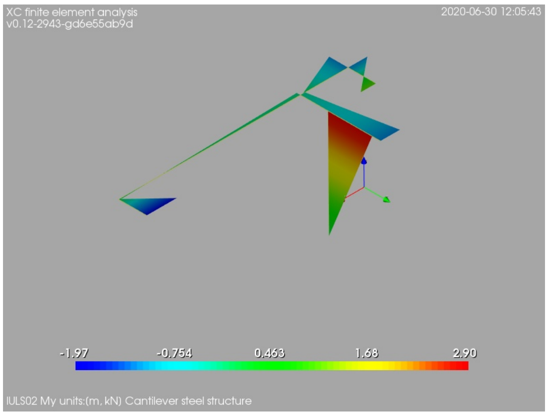


Figure 69: ULS02. Cantilever steel structure, bending moment about local axis y, units:[m, kN]

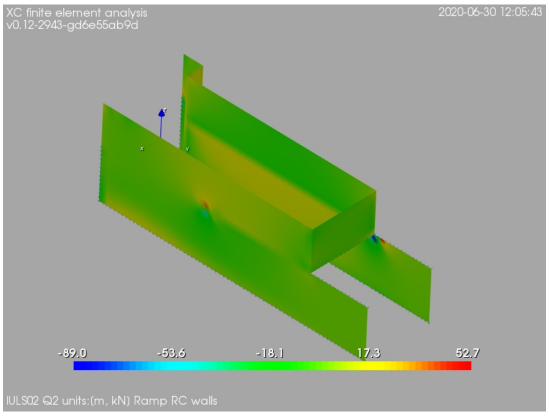


Figure 67: ULS02. Ramp rc walls, internal shear force in local direction 2, units:[m, kN]

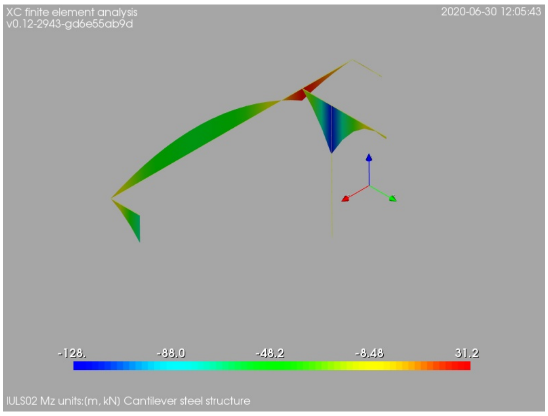


Figure 70: ULS02. Cantilever steel structure, bending moment about local axis z, units:[m, kN]

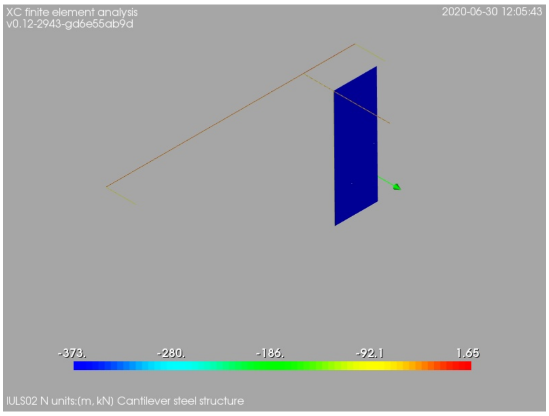


Figure 68: ULS02. Cantilever steel structure, internal axial force, units:[m, kN]

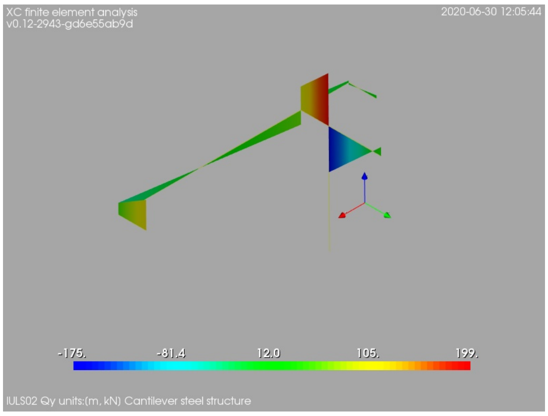


Figure 71: ULS02. Cantilever steel structure, internal shear force in local direction y, units:[m, kN]

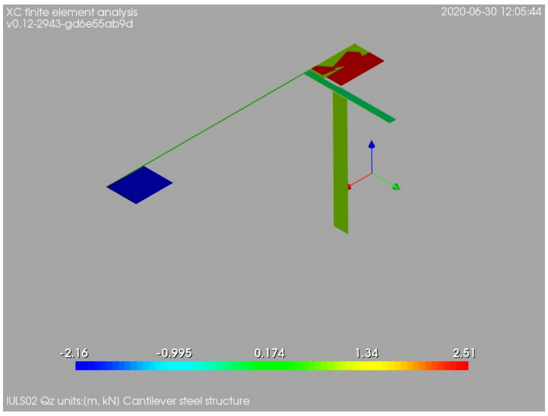


Figure 72: ULS02. Cantilever steel structure, internal shear force in local direction z, units:[m, kN]

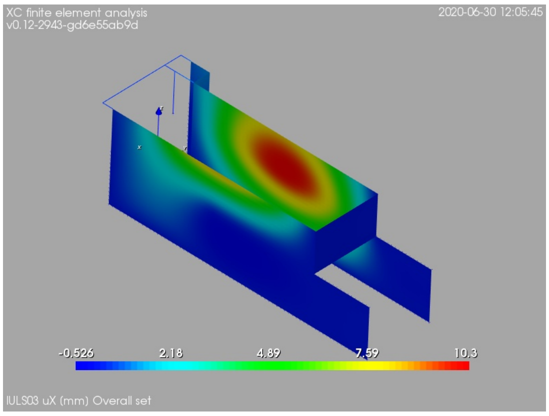


Figure 73: ULS03. Overall set, displacement in global X direction, [mm]

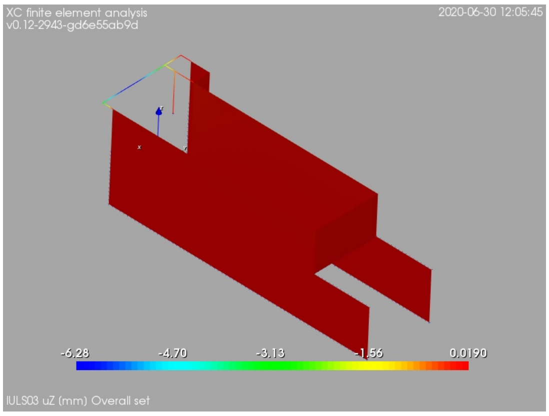


Figure 74: ULS03. Overall set, displacement in global Z direction, [mm]

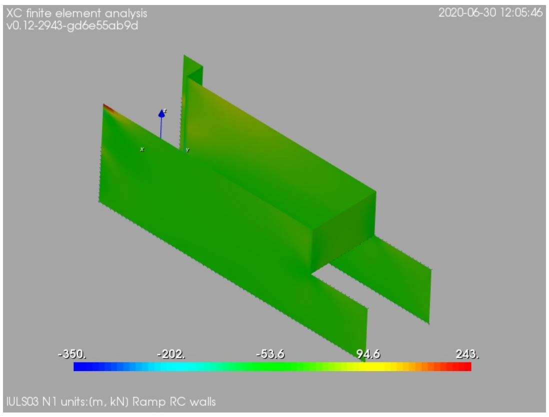


Figure 75: ULS03. Ramp rc walls, internal axial force in local direction 1, units:[m, kN]

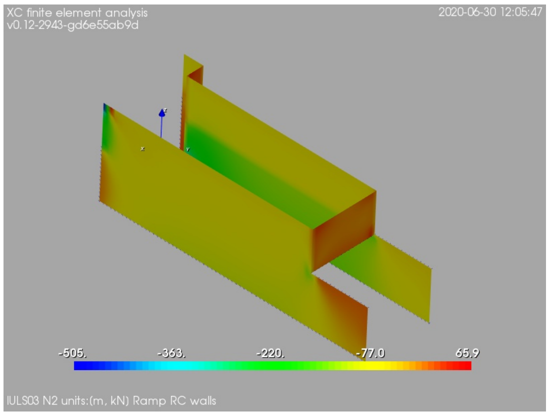


Figure 76: ULS03. Ramp rc walls, internal axial force in local direction 2, units:[m, kN]

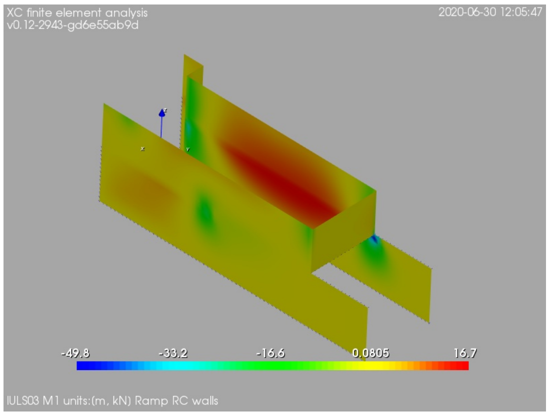


Figure 77: ULS03. Ramp rc walls, bending moment about local axis 1, units:[m, kN]

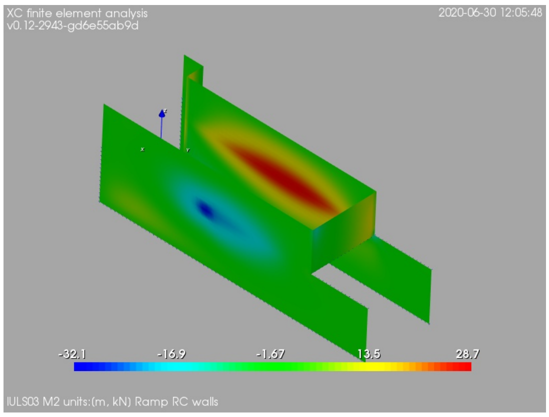


Figure 78: ULS03. Ramp rc walls, bending moment about local axis 2, units:[m, kN]

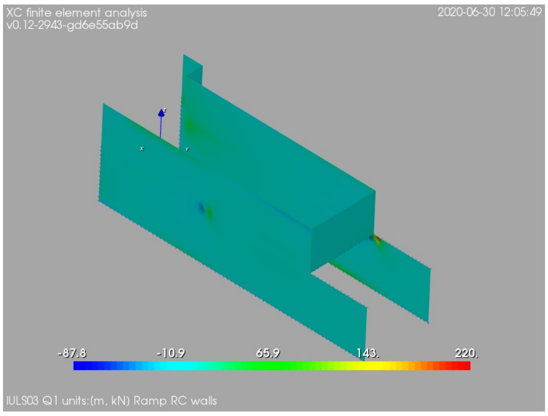


Figure 79: ULS03. Ramp rc walls, internal shear force in local direction 1, units:[m, kN]

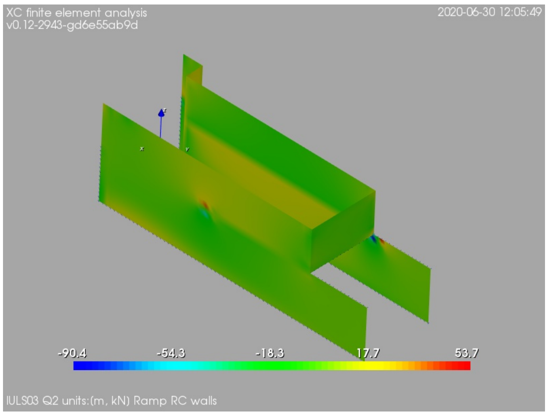


Figure 80: ULS03. Ramp rc walls, internal shear force in local direction 2, units:[m, kN]

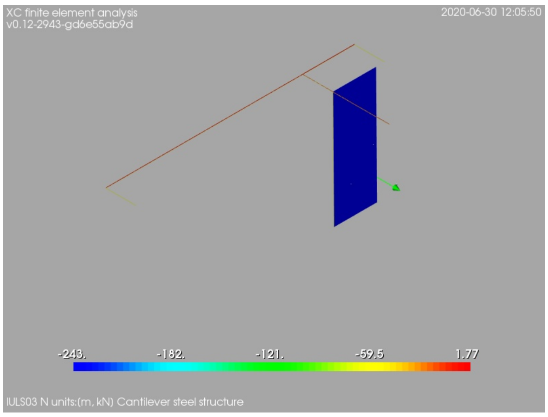


Figure 81: ULS03. Cantilever steel structure, internal axial force, units:[m, kN]

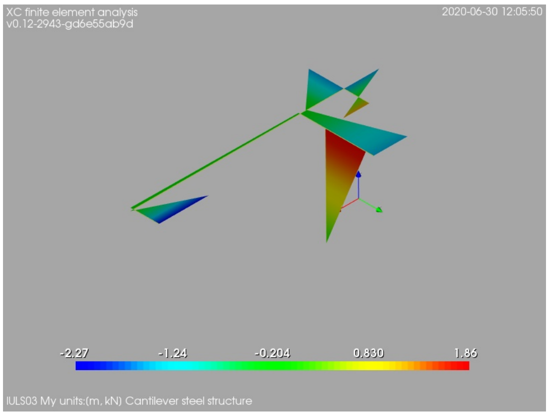


Figure 82: ULS03. Cantilever steel structure, bending moment about local axis y, units:[m, kN]

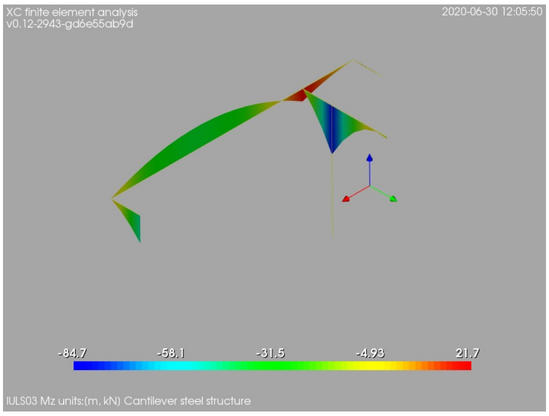


Figure 83: ULS03. Cantilever steel structure, bending moment about local axis z, units:[m, kN]

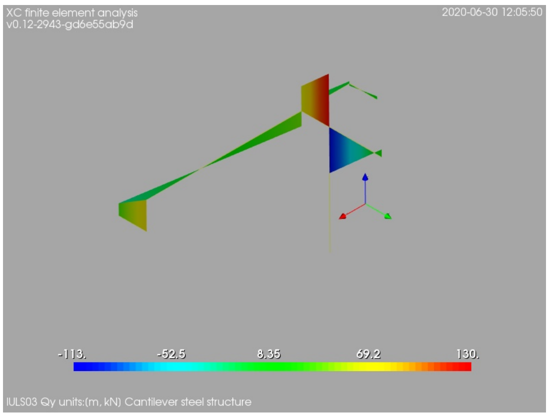


Figure 84: ULS03. Cantilever steel structure, internal shear force in local direction y, units:[m, kN]

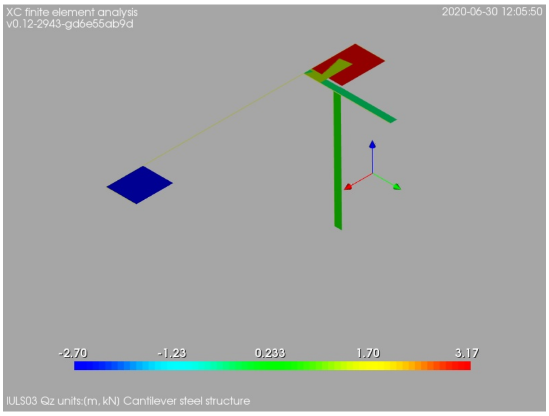


Figure 85: ULS03. Cantilever steel structure, internal shear force in local direction z, units:[m, kN]

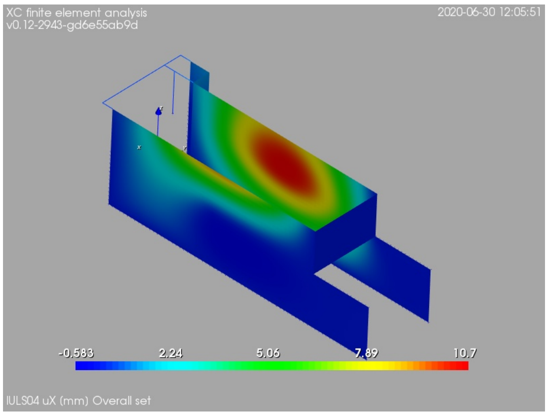


Figure 86: ULS04. Overall set, displacement in global X direction, [mm]

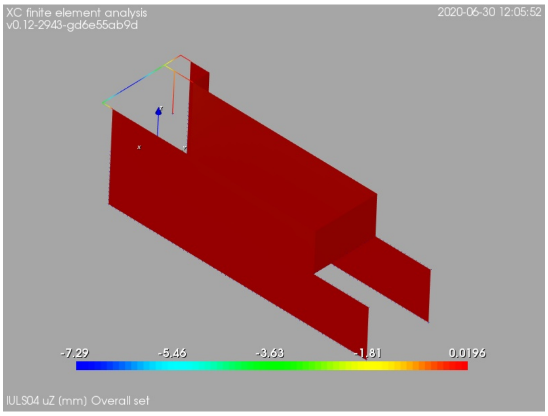


Figure 87: ULS04. Overall set, displacement in global Z direction, [mm]

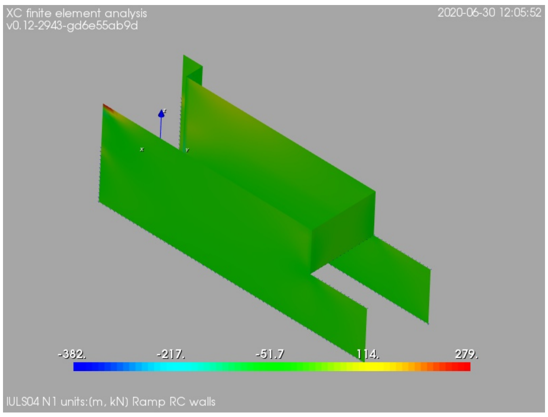


Figure 88: ULS04. Ramp rc walls, internal axial force in local direction 1, units:[m, kN]

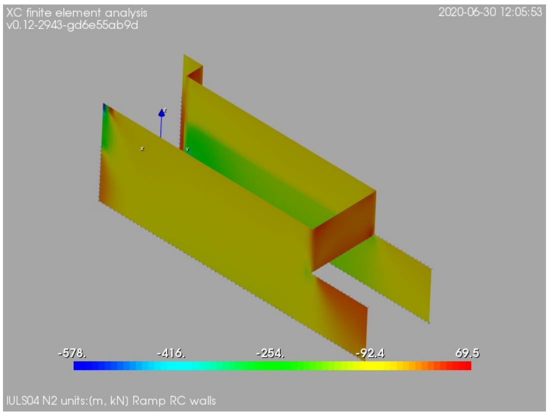


Figure 89: ULS04. Ramp rc walls, internal axial force in local direction 2, units:[m, kN]

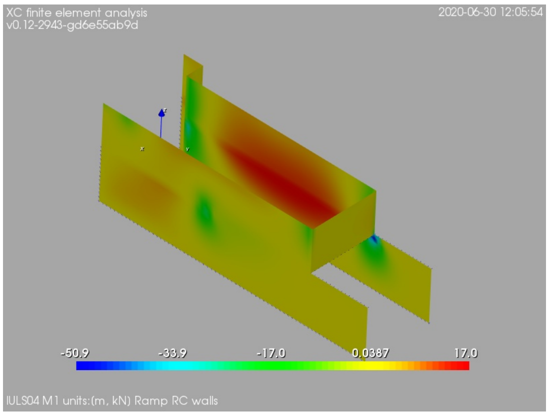


Figure 90: ULS04. Ramp rc walls, bending moment about local axis 1, units:[m, kN]

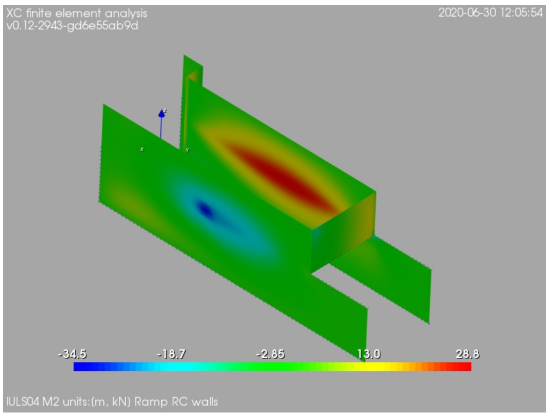


Figure 91: ULS04. Ramp rc walls, bending moment about local axis 2, units:[m, kN]

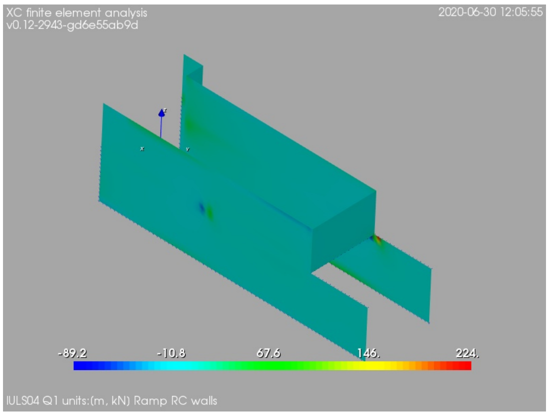


Figure 92: ULS04. Ramp rc walls, internal shear force in local direction 1, units:[m, kN]

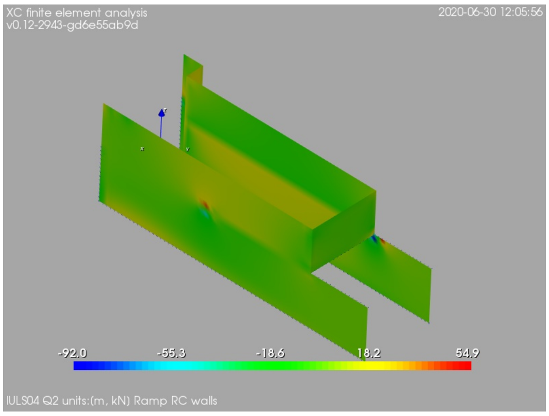


Figure 93: ULS04. Ramp rc walls, internal shear force in local direction 2, units:[m, kN]

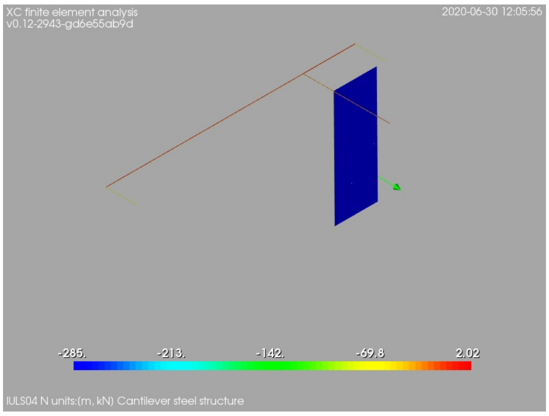


Figure 94: ULS04. Cantilever steel structure, internal axial force, units:[m, kN]

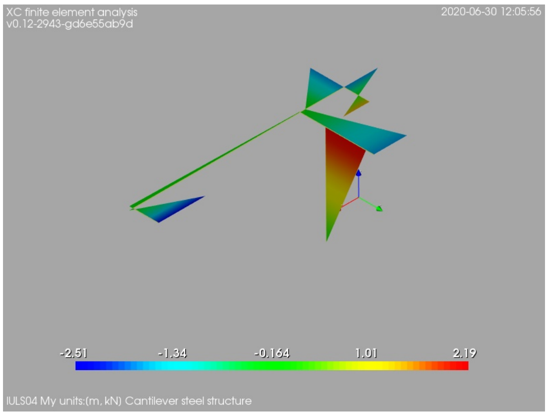


Figure 95: ULS04. Cantilever steel structure, bending moment about local axis y, units:[m, kN]

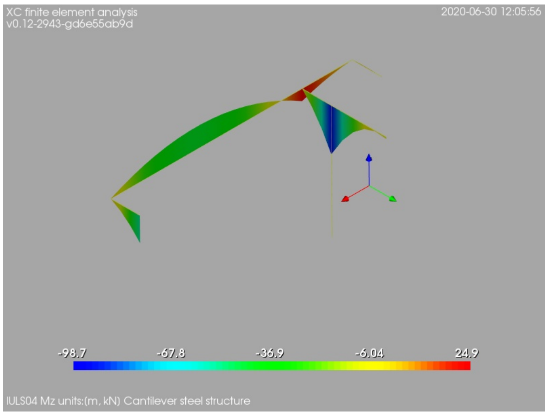


Figure 96: ULS04. Cantilever steel structure, bending moment about local axis z, units:[m, kN]

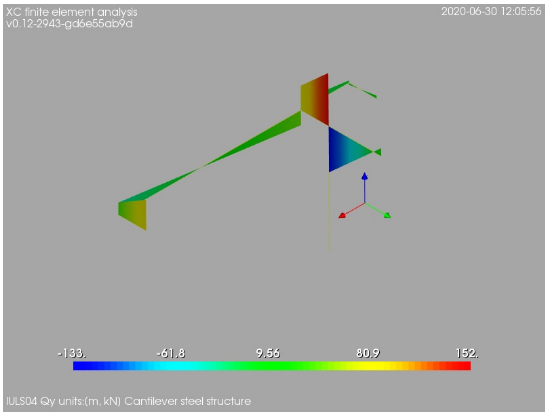


Figure 97: ULS04. Cantilever steel structure, internal shear force in local direction y, units:[m, kN]

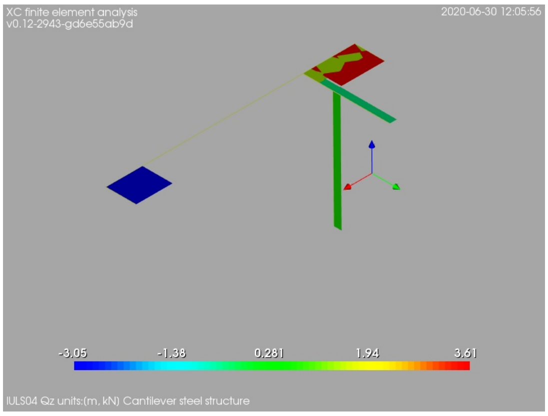


Figure 98: ULS04. Cantilever steel structure, internal shear force in local direction z, units:[m, kN]

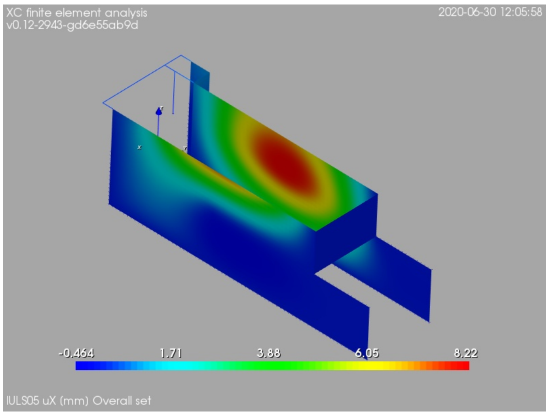


Figure 99: ULS05. Overall set, displacement in global X direction, [mm]

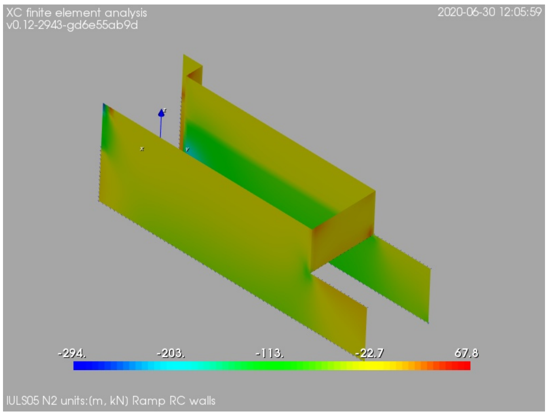


Figure 102: ULS05. Ramp rc walls, internal axial force in local direction 2, units:[m, kN]

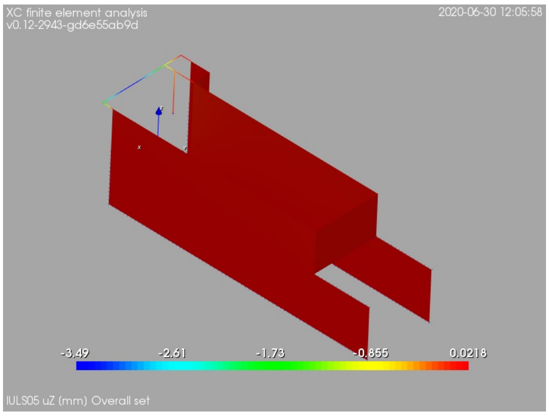


Figure 100: ULS05. Overall set, displacement in global Z direction, [mm]

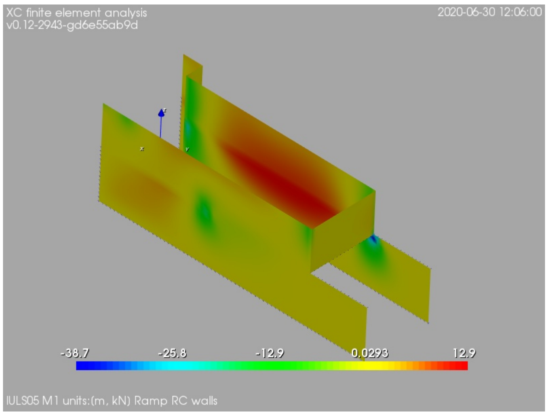


Figure 103: ULS05. Ramp rc walls, bending moment about local axis 1, units:[m, kN]

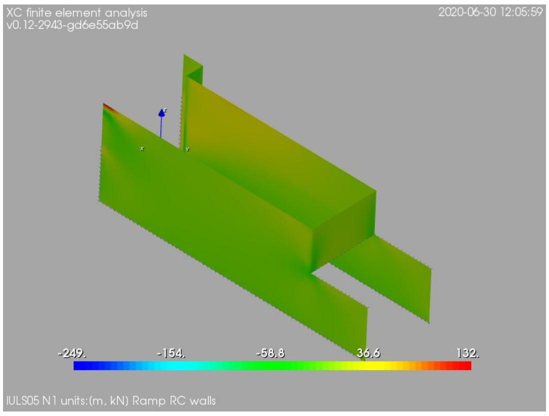


Figure 101: ULS05. Ramp rc walls, internal axial force in local direction 1, units:[m, kN]

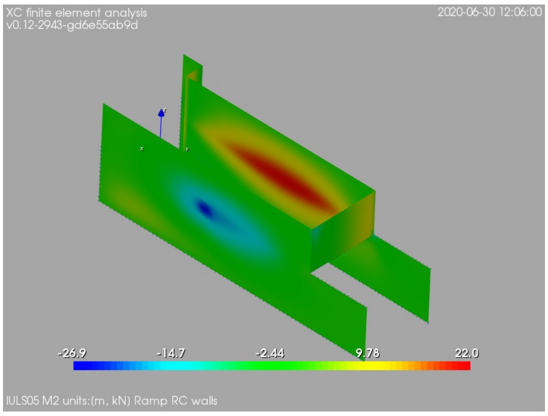


Figure 104: ULS05. Ramp rc walls, bending moment about local axis 2, units:[m, kN]

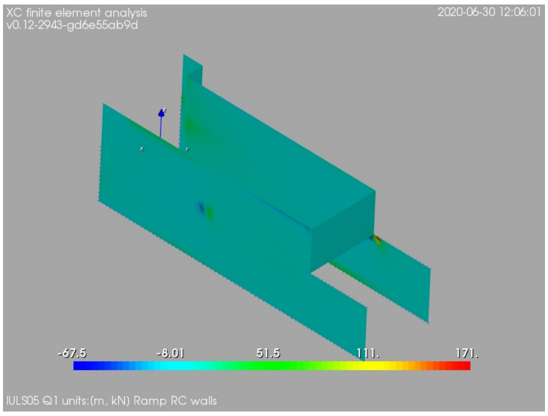


Figure 105: ULS05. Ramp rc walls, internal shear force in local direction 1, units:[m, kN]

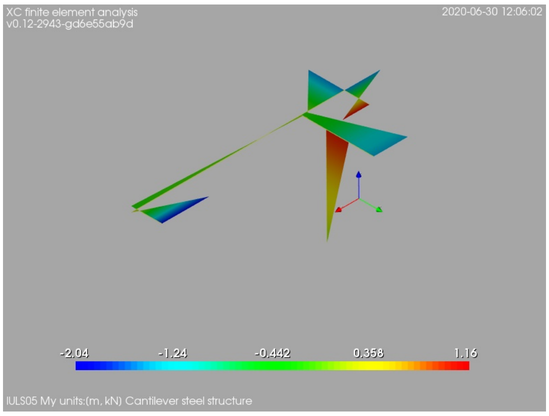


Figure 108: ULS05. Cantilever steel structure, bending moment about local axis y, units:[m, kN]

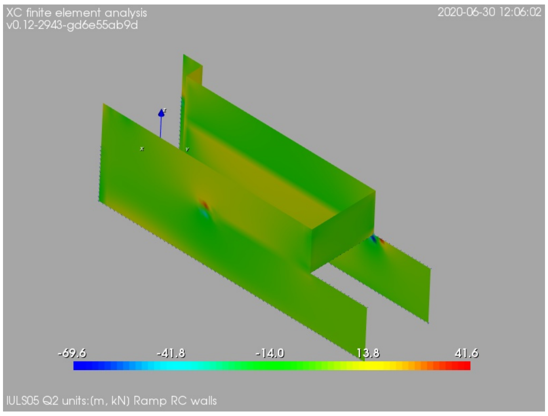


Figure 106: ULS05. Ramp rc walls, internal shear force in local direction 2, units:[m, kN]

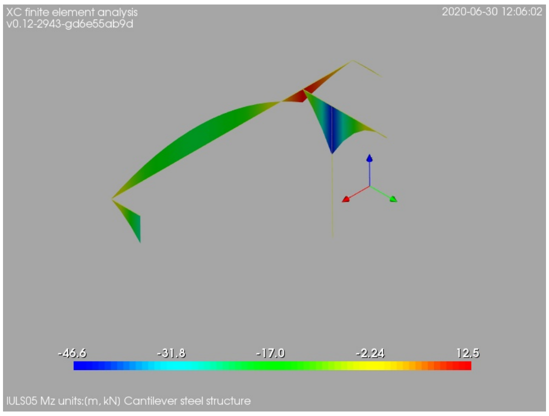


Figure 109: ULS05. Cantilever steel structure, bending moment about local axis z, units:[m, kN]

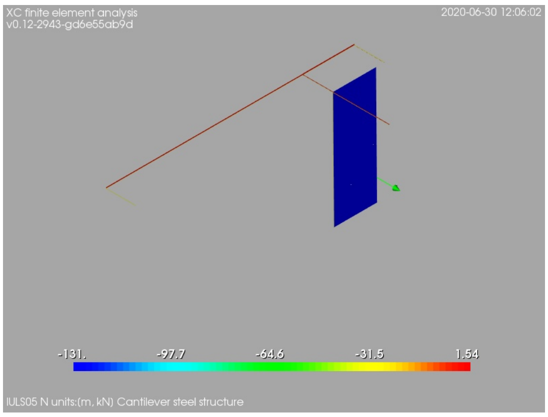


Figure 107: ULS05. Cantilever steel structure, internal axial force, units:[m, kN]

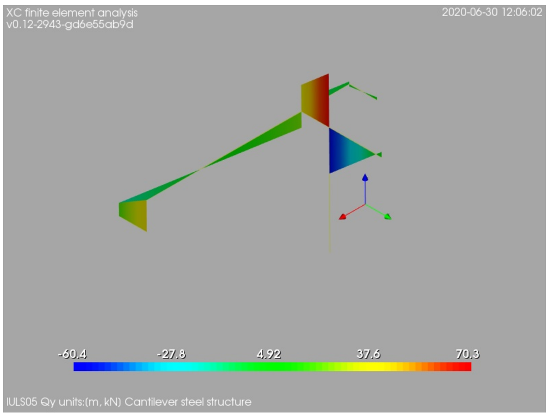


Figure 110: ULS05. Cantilever steel structure, internal shear force in local direction y, units:[m, kN]

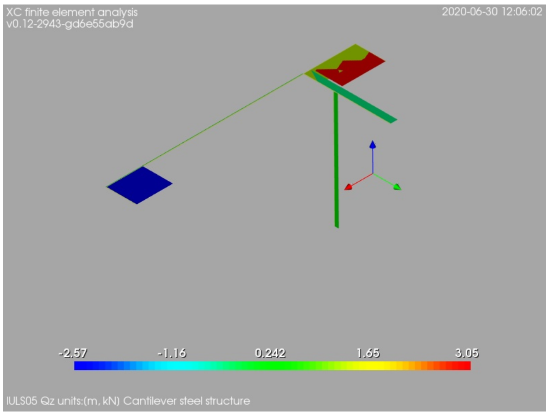


Figure 111: ULS05. Cantilever steel structure, internal shear force in local direction z, units:[m, kN]

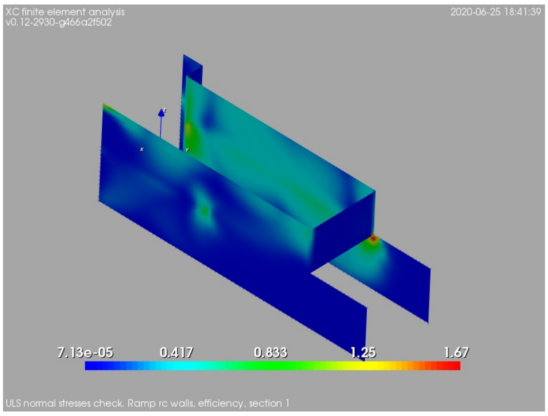


Figure 112: ULS normal stresses check. Ramp rc walls, efficiency, section 1

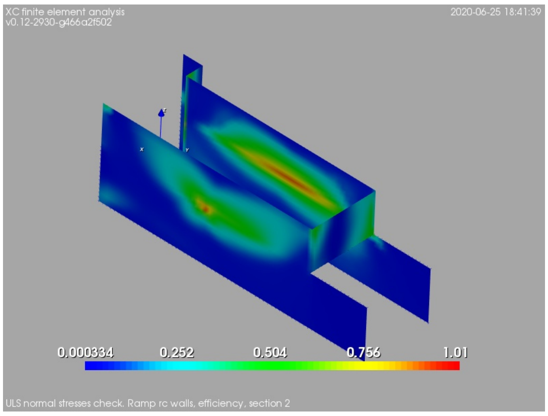


Figure 113: ULS normal stresses check. Ramp rc walls, efficiency, section 2

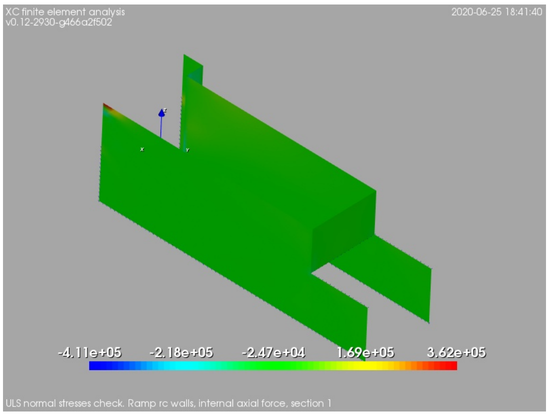


Figure 114: ULS normal stresses check. Ramp rc walls, internal axial force, section 1

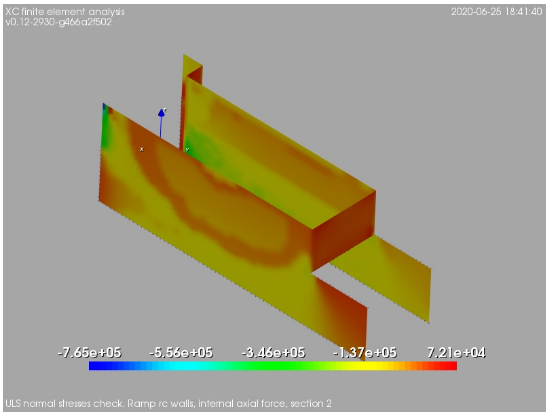


Figure 115: ULS normal stresses check. Ramp rc walls, internal axial force, section 2

Verification of normal stresses

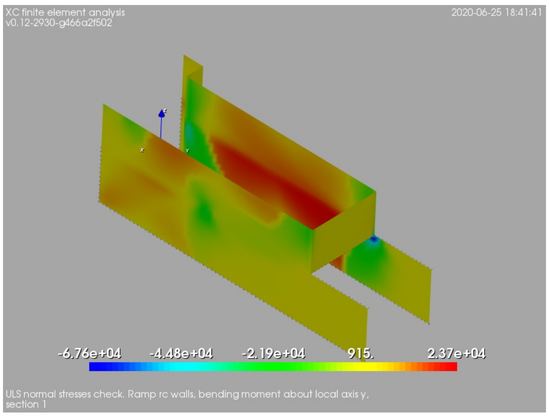


Figure 116: ULS normal stresses check. Ramp rc walls, bending moment about local axis y, section 1

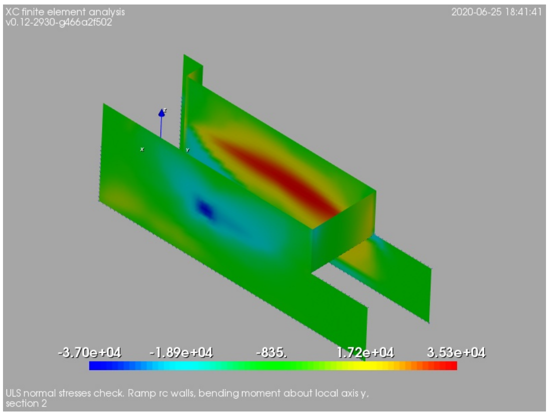


Figure 117: ULS normal stresses check. Ramp rc walls, bending moment about local axis y, section 2

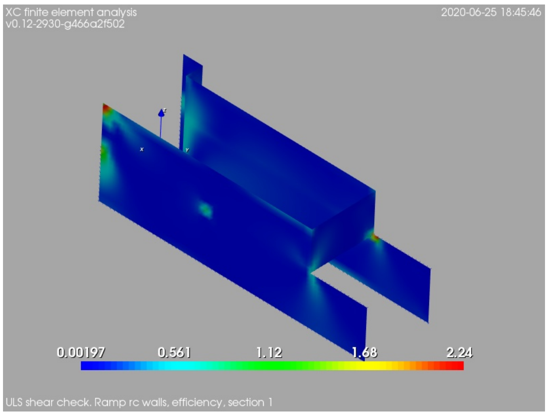


Figure 118: ULS shear check. Ramp rc walls, efficiency, section 1

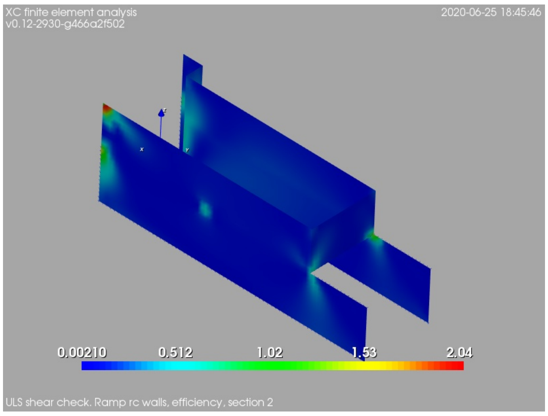


Figure 119: ULS shear check. Ramp rc walls, efficiency, section 2

Verification of shear ULS

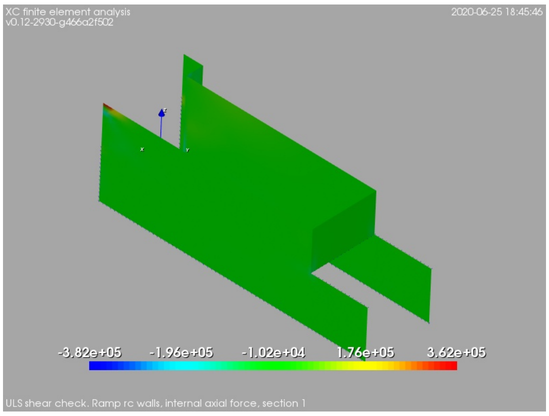


Figure 120: ULS shear check. Ramp rc walls, internal axial force, section 1

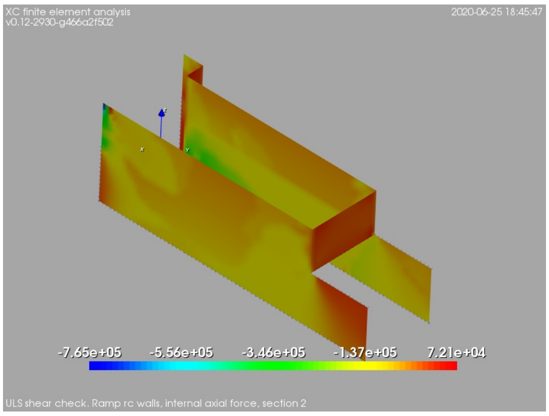


Figure 121: ULS shear check. Ramp rc walls, internal axial force, section 2

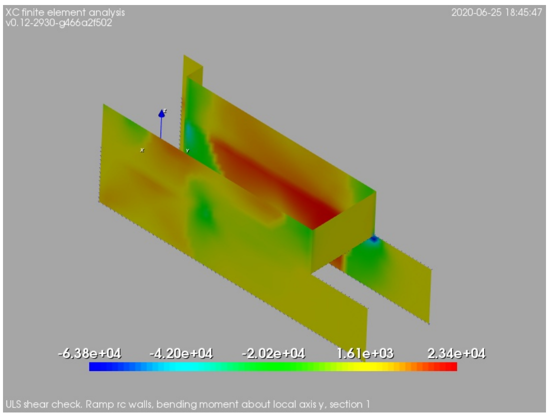


Figure 122: ULS shear check. Ramp rc walls, bending moment about local axis y, section 1

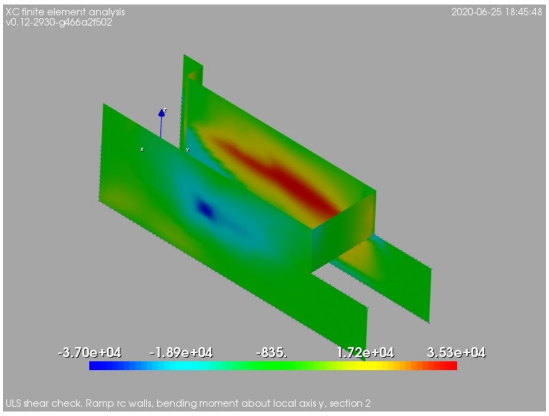


Figure 123: ULS shear check. Ramp rc walls, bending moment about local axis y, section 2

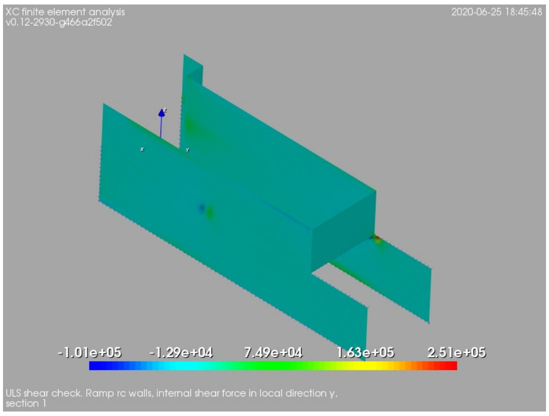


Figure 124: ULS shear check. Ramp rc walls, internal shear force in local direction y, section 1

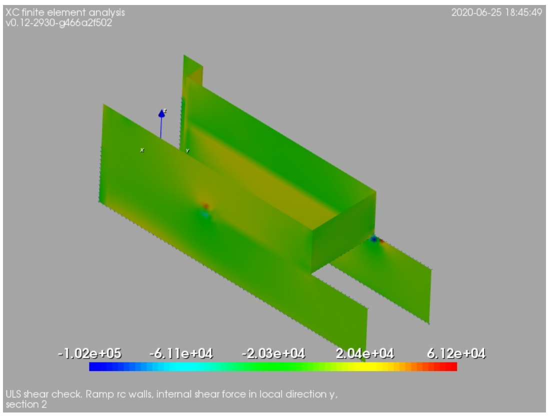


Figure 125: ULS shear check. Ramp rc walls, internal shear force in local direction y, section 2

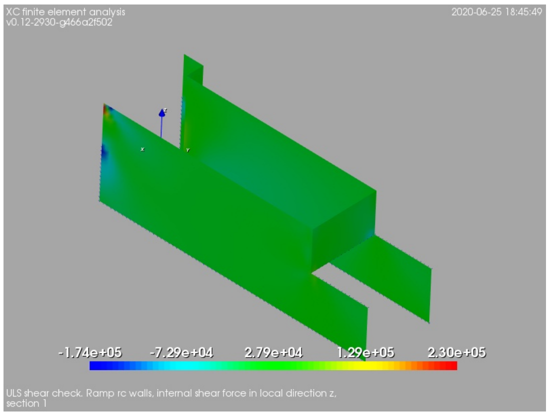


Figure 126: ULS shear check. Ramp rc walls, internal shear force in local direction z, section 1

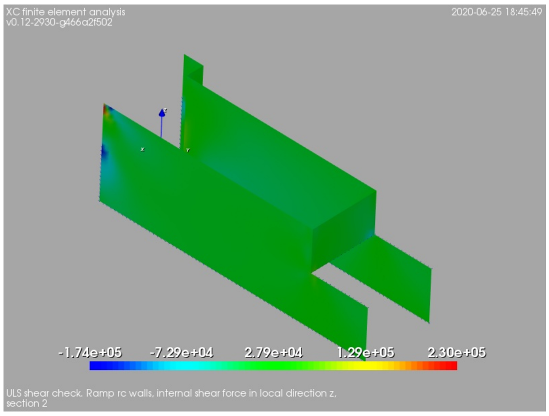


Figure 127: ULS shear check. Ramp rc walls, internal shear force in local direction z, section 2

9.4.7 Verification of steel structure

ULS all loads Beam parallel to facade

shear checking:
$V_{d,max}$= 55.02 kN
$V_{provided}$= 233.1 kN
Capacity factor: F= 0.24
normal stresses checking:
$M_{d,max}$= 56.84 mkN
$M_{provided}$= 170.66 mkN
Capacity factor: F= 0.33
West cantilever
shear checking:
$V_{d,max}$= 115.64 kN
$V_{provided}$= 286.79 kN
Capacity factor: F= 0.4
normal stresses checking:
$M_{d,max}$= 71.16 mkN
$M_{provided}$= 214.07 mkN
Capacity factor: F= 0.33
Central cantilever
shear checking:
$V_{d,max}$= 198.6 kN
$V_{provided}$= 286.79 kN
Capacity factor: F= 0.69
normal stresses checking:
$M_{d,max}$= 127.68 mkN
$M_{provided}$= 214.07 mkN
Capacity factor: F= 0.6
East cantilever
shear checking:
$V_{d,max}$= 11.39 kN
$V_{provided}$= 286.79 kN
Capacity factor: F= 0.04
normal stresses checking:
$M_{d,max}$= 1.86 mkN
$M_{provided}$= 214.07 mkN
Capacity factor: F= 0.01
Column
shear checking:
$V_{d,max}$= 0.05 kN
$V_{provided}$= 130.5 kN
Capacity factor: F= 0.0
normal stresses checking:
$M_{d,max}$= 0.16 mkN
$M_{provided}$= 214.07 mkN
Capacity factor: F= 0.0
shear checking:
$V_{d,max}$= 0.84 kN
$V_{provided}$= 130.5 kN
Capacity factor: F= 0.01
normal stresses checking:
$M_{d,max}$= 2.9 mkN
$M_{provided}$= 32.19 mkN
Capacity factor: F= 0.09

ULS only cantilever loaded Beam parallel to facade

shear checking:
$V_{d,max}$= 55.02 kN
$V_{provided}$= 233.1 kN
Capacity factor: F= 0.24
normal stresses checking:
$M_{d,max}$= 56.84 mkN
$M_{provided}$= 170.66 mkN
Capacity factor: F= 0.33
West cantilever
shear checking:
$V_{d,max}$= 115.64 kN
$V_{provided}$= 286.79 kN
Capacity factor: F= 0.4
normal stresses checking:
$M_{d,max}$= 71.16 mkN
$M_{provided}$= 214.07 mkN
Capacity factor: F= 0.33
Central cantilever
shear checking:
$V_{d,max}$= 198.6 kN
$V_{provided}$= 286.79 kN
Capacity factor: F= 0.69
normal stresses checking:
$M_{d,max}$= 127.68 mkN
$M_{provided}$= 214.07 mkN
Capacity factor: F= 0.6
East cantilever
shear checking:
$V_{d,max}$= 11.39 kN
$V_{provided}$= 286.79 kN
Capacity factor: F= 0.04
normal stresses checking:
$M_{d,max}$= 1.86 mkN
$M_{provided}$= 214.07 mkN
Capacity factor: F= 0.01
Column
shear checking:
$V_{d,max}$= 0.05 kN
$V_{provided}$= 130.5 kN
Capacity factor: F= 0.0
normal stresses checking:
$M_{d,max}$= 0.16 mkN
$M_{provided}$= 214.07 mkN
Capacity factor: F= 0.0
shear checking:
$V_{d,max}$= 0.84 kN

$V_{provided}$= 130.5 kN
Capacity factor: F= 0.01
normal stresses checking:
$M_{d,max}$= 2.9 mkN
$M_{provided}$= 32.19 mkN
Capacity factor: F= 0.09

10 Elevator shaft lintels

10.1 Lintel roof

10.1.1 Geometry data

Clear span= 8 feet 8 inches
Bearing length (at each extremity)= 21/32 inches
Height of masonry above the opening= 8 inches
Mmax= 8841.37 lb-ft = 106096.43 lb-in
Vmax= 3789.16 lb

10.1.2 Load data

Lintel self-weight= 118 $\frac{lb}{ft}$
Wall self weight= 47 $\frac{lb}{ft^2}$
Superimposed dead uniform load= 64.1666666667 $\frac{lb}{ft}$
Live uniform load= 0 $\frac{lb}{ft}$
Snow uniform load= 134.75 $\frac{lb}{ft}$

10.1.3 Check for arching action

Effective span l= 9 feet 4 inches
Height of masonry required above the lintel for arching action to occur= 5 and 11/32 inches
Arching action occurs (not considered conservatively)
Check concrete masonry lintel - strenght design
Maximum bending moment due to lintel self weigth= 1284.89 lb-ft
Maximum bending moment due to wall self weigth= 3427.84 lb-ft
Maximum bending moment due to superimposed dead load= 698.7 lb-ft
Maximum bending moment due to live load= 0.0 lb-ft
Maximum bending moment due to snow load= 1467.28 lb-ft
Maximum bending moment ULS01= 7576.01 lb-ft
Maximum bending moment ULS02= 7227.36 lb-ft
Maximum bending moment ULS03= 8841.37 lb-ft
Maximum bending moment= 30946.00 lb-ft
Bending moment provided= 80862 lb-ft (see fig 128)
Maximum shear force due to lintel self weigth= 550.67 lb
Maximum shear force due to wall self weigth= 1469.08 lb
Maximum shear force due to superimposed dead load= 299.44 lb
Maximum shear force due to live load= 0.0 lb
Maximum shear force due to snow load= 628.83 lb
Maximum shear force ULS01= 3246.86 lb
Maximum shear force ULS02= 3097.44 lb
Maximum shear force ULS03= 3789.16 lb
Maximum shear force= 3789.16 lb
Shear force provided= 10722.00 lb (see fig 128)

10.1.4 Check lintel deflection

Modulus of elasticity E= 194400000.0 psf
Moment of inertia I=0.116894397225 ft^4 $\Delta_{max,dead} = 0.02593$ in
$\Delta_{max,live} = 0.0$ in

Steel Size (No.)	No. of Bars	Bottom Cover (in.)							
		1.5		2		2.5		3	
		ϕV_n (lb)	ϕM_n (in.-lb)	ϕV_n (lb)	ϕM_n (in.-lb)	ϕV_n (lb)	ϕM_n (in.-lb)	ϕV_n (lb)	ϕM_n (in.-lb)
4	1	10,722	145,600	10,722	140,200	10,722	134,800	10,722	129,400
5	1	10,722	221,012	10,722	212,642	10,722	204,272	10,722	195,902
6	1	10,722[c]	306,133[c]	10,722	294,253	10,722	282,373	10,722	270,493
4	2	10,722	282,703	10,722	271,903	10,722	261,103	10,722	250,303
5	2	10,722	421,607	10,722	404,867	10,722	388,127	10,722	371,387
6	2	10,722[c]	571,135[c]	10,722	547,375	10,722	523,615	10,722	499,855

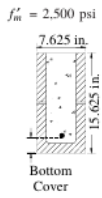


Figure 128: Design shear and moment capacity for nominal 8 x 8 in. concrete masonry lintels

$\Delta_{max,snow} = 0.00703$ in
$\Delta_{max} = 0.03296$ in
$\Delta_{max,allowed} = 0.18667$ in
$\Delta_{max} = 0.03296 < \Delta_{max,allowed} = 0.18667$ ok

10.2 Lintel 3rd and 2nd floor

10.2.1 Geometry data

Clear span= 4 feet 10 inches
Bearing length (at each extremity)= 21/32 inches
Height of masonry above the opening= 3 and 11/32 inches
Mmax= 1910.65 lb-ft = 22927.80 lb-in
Vmax= 1389.56 lb

10.2.2 Load data

Lintel self-weight= 59 $\frac{lb}{ft}$
Wall self weight= 47 $\frac{lb}{ft^2}$
Superimposed dead uniform load= 64.17 $\frac{lb}{ft}$
Live uniform load= 128.33 $\frac{lb}{ft}$
Snow uniform load= 0 $\frac{lb}{ft}$

10.2.3 Check for arching action

Effective span l= 5 feet 5 and 1 inches
Height of masonry required above the lintel for arching action to occur= 3 and 13/32 inches
Arching action occurs (not considered conservatively)

10.2.4 Check concrete masonry lintel - strenght design

Maximum bending moment due to lintel self weigth= 223.09 lb-ft
Maximum bending moment due to wall self weigth= 479.47 lb-ft
Maximum bending moment due to superimposed dead load= 242.63 lb-ft
Maximum bending moment due to live load= 485.26 lb-ft
Maximum bending moment due to snow load= 0.0 lb-ft
Maximum bending moment ULS01= 1323.27 lb-ft
Maximum bending moment ULS02= 1910.65 lb-ft

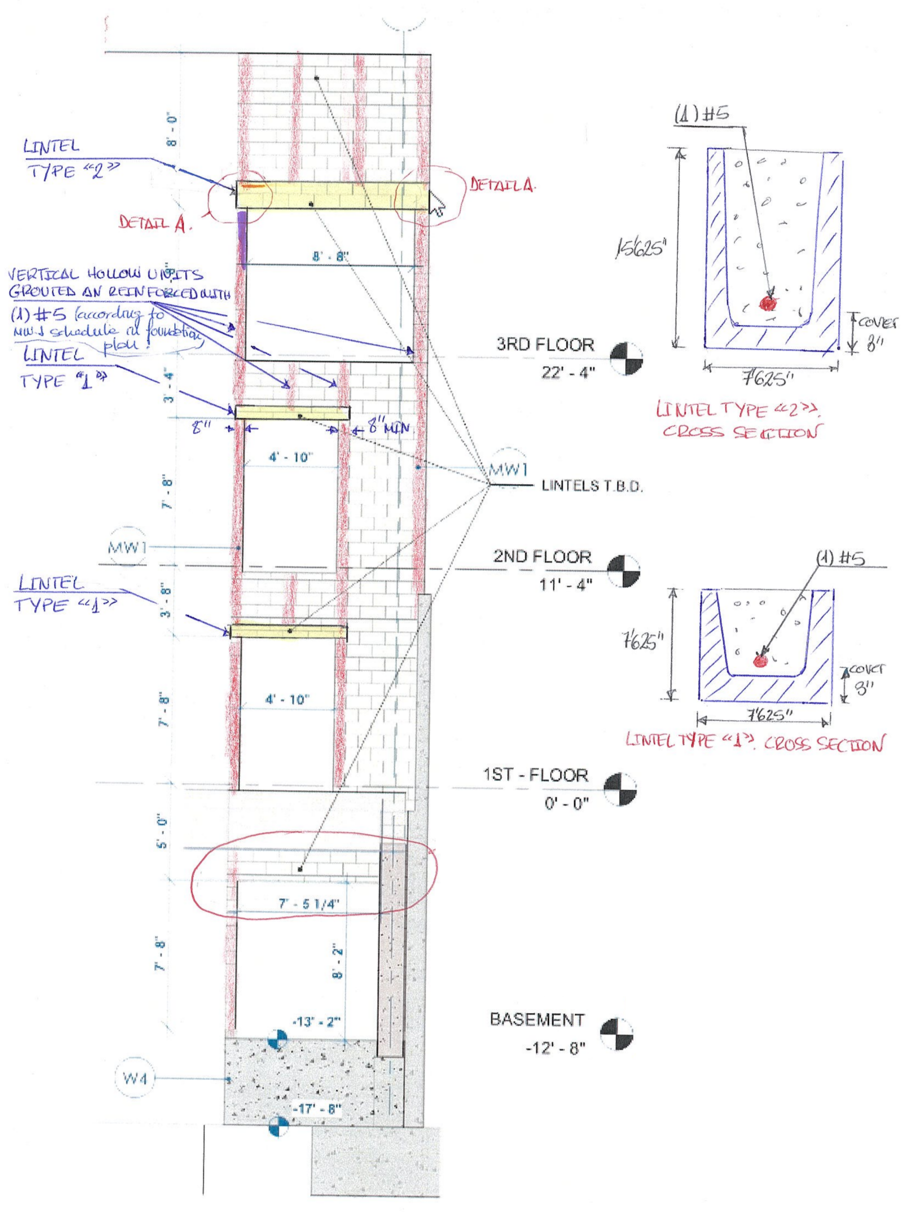


Figure 129: Elevator lintels

Steel Size (No.)	No. of Bars	Bottom Cover (in.) 1.5		2		2.5		3	
		ϕV_n (lb)	ϕM_n (in.-lb)	ϕV_n (lb)	ϕM_n (in.-lb)	ϕV_n (lb)	ϕM_n (in.-lb)	ϕV_n (lb)	ϕM_n (in.-lb)
4	1	5,232	59,200	5,232	53,800	5,232	48,400	5,232	43,000
5	1	5,232	87,092	5,232	78,722	5,232	70,352	5,232	61,982
6	1	5,232[c]	116,053[c]	5,232	104,173	5,232	92,293	5,232	80,413
4	2	5,232	109,903	5,232	99,103	5,232	88,303	5,232	77,503
5	2	5,232	153,767	5,232	137,027	5,232	120,287	5,232	103,547
6	2	5,232[c]	190,975[c]	5,232	167,215	5,232	143,455	5,232	119,695

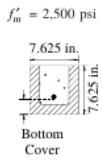


Figure 130: Design shear and moment capacity for nominal 8 x 8 in. concrete masonry lintels

Maximum bending moment ULS03= 1376.86 lb-ft
Maximum bending moment= 1910.65 lb-ft
Bending moment provided= 5165.00 lb-ft (see fig 130)
Maximum shear force due to lintel self weigth= 162.25 lb
Maximum shear force due to wall self weigth= 348.71 lb
Maximum shear force due to superimposed dead load= 176.46 lb
Maximum shear force due to live load= 352.92 lb
Maximum shear force due to snow load= 0.0 lb
Maximum shear force ULS01= 962.38 lb
Maximum shear force ULS02= 1389.56 lb
Maximum shear force ULS03= 1001.36 lb
Maximum shear force= 1389.56 lb
Shear force provided= 5232.00 lb (see fig 130)

10.2.5 Check lintel deflection

Modulus of elasticity E= 194400000.0 psf
Moment of inertia I=0.0135847972744 ft^4 $\Delta_{max,dead} = 0.01353$ in
$\Delta_{max,live} = 0.00695$ in
$\Delta_{max,snow} = 0.0$ in
$\Delta_{max} = 0.02048$ in
$\Delta_{max,allowed} = 0.11$ in
$\Delta_{max} = 0.02048 < \Delta_{max,allowed} = 0.11$ ok

10.3 Lintel 1st floor

10.3.1 Geometry data

Clear span= 2 feet 8 inches
Bearing length (at each extremity)= 0 feet 0 inches
Height of masonry above the opening= 1/2 inches
Mmax= 29849.23 lb-ft = 358190.71 lb-in
Vmax= 44768.24 lb

10.3.2 Load data

Lintel self-weight= 2362.5 $\frac{lb}{ft}$
Wall self weight= 20590.47 $\frac{lb}{ft^2}$
Superimposed dead uniform load= 9216.95 $\frac{lb}{ft}$

Live uniform load= 12289.27 $\frac{lb}{ft}$
Snow uniform load= 6451.86 $\frac{lb}{ft}$

10.3.3 Check for arching action

Effective span l= 2 feet 8 inches
Height of masonry required above the lintel for arching action to occur= 2 and 11/32 inches
Arching action occurs (not considered conservatively)

10.3.4 Check concrete masonry lintel - strenght design

Maximum bending moment due to lintel self weigth= 2100.53 lb-ft
Maximum bending moment due to wall self weigth= -2379.94 lb-ft
Maximum bending moment due to superimposed dead load= 8194.89 lb-ft
Maximum bending moment due to live load= 10926.52 lb-ft
Maximum bending moment due to snow load= 5736.42 lb-ft
Maximum bending moment ULS01= 11081.67 lb-ft
Maximum bending moment ULS02= 29849.23 lb-ft
Maximum bending moment ULS03= 24140.12 lb-ft
Maximum bending moment= 29849.23 lb-ft
Bending moment provided= 40316.08 lb-ft (see fig. 132)
Maximum shear force due to lintel self weigth= 3150.39 lb
Maximum shear force due to wall self weigth= -3569.46 lb
Maximum shear force due to superimposed dead load= 12290.8 lb
Maximum shear force due to live load= 16387.74 lb
Maximum shear force due to snow load= 8603.56 lb
Maximum shear force ULS01= 16620.43 lb
Maximum shear force ULS02= 44768.24 lb
Maximum shear force ULS03= 36205.65 lb
Maximum shear force= 44768.24 lb → #3 at 16 in.

10.3.5 Check lintel deflection

Modulus of elasticity E= 30000000000.0 psf
Moment of inertia I=0.0031255875 ft^4 $\Delta_{max,dead} = 0.00075$ in
$\Delta_{max,live} = 0.00104$ in
$\Delta_{max,snow} = 0.00054$ in
$\Delta_{max} = 0.00233$ in
$\Delta_{max,allowed} = 0.05334$ in
$\Delta_{max} = 0.00233 < \Delta_{max,allowed} = 0.05334$ ok

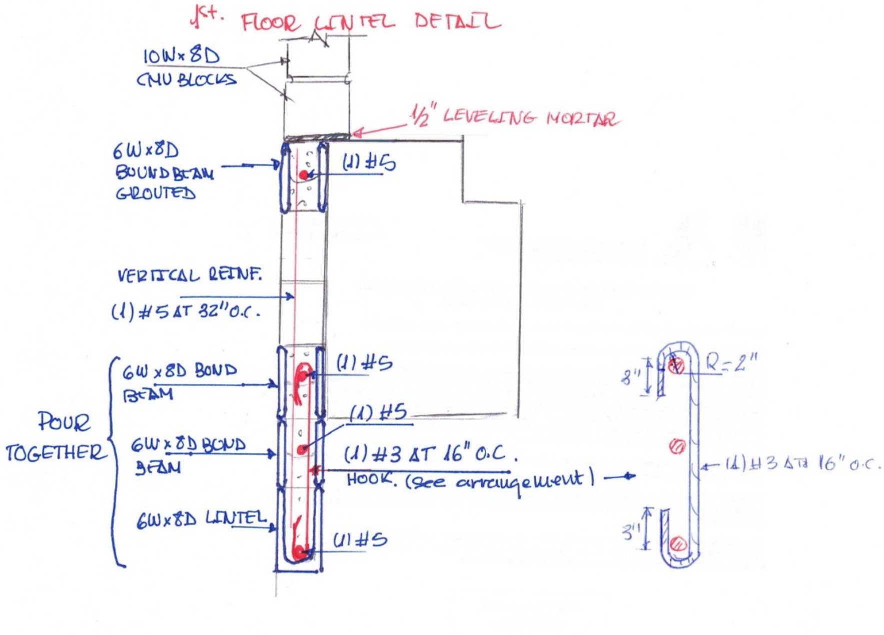


Figure 131: 1st floor lintel detail

Steel Size (No.)	No. of Bars	Bottom Cover (in.)							
		1.5		2		2.5		3	
		ϕV_n (lb)	ϕM_n (in.-lb)	ϕV_n (lb)	ϕM_n (in.-lb)	ϕV_n (lb)	ϕM_n (in.-lb)	ϕV_n (lb)	ϕM_n (in.-lb)
4	1	16,898	242,800	16,898	237,400	16,898	232,000	16,898	226,600
5	1	16,898	371,672	16,898	363,302	16,898	354,932	16,898	346,562
6	1	16,898[c]	519,973[c]	16,898	508,093	16,898	496,213	16,898	484,333
4	2	16,898	477,103	16,898	466,303	16,898	455,503	16,898	444,703
5	2	16,898	722,927	16,898	706,187	16,898	689,447	16,898	672,707
6	2	16,898[c]	998,815[c]	16,898	975,055	16,898	951,295	16,898	927,535

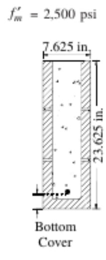


Figure 132: Design shear and moment capacity for nominal 8 x 24 in. concrete masonry lintels

11 Balconies connection

11.1 Loads on the connection

The ties of the balconies are attached to the wood structure by means of the connection represented in figure 133.

According to our calculations the maximum load to be resisted by this connection is:

$$F = 4.83\ kN \tag{194}$$

the angle with respect to the vertical of this force is 23.42 degrees.

11.2 Loads on fasteners

To calculate the loads on the connection fasteners we have used a the finite element model shown in figure 135.

11.3 Fastener strength

The strength of the 3/8 lag screws are as follows:

- Screw design withdrawal: $W_d = 4.6\ kN$.
- Screw design lateral strength: $Z_d = 0.89\ kN$.

11.4 Checking of fastener capacities

The capacities of the fasteners are as follows:

Screw ID	**Z (kN)**	**R (kN)**	CF -
466	0.13	0.98	0.14
638	0.96	1.03	0.93
129	0.15	1.19	0.12
59	0.76	1.38	0.55
94	0.12	4.25	0.03
50	0.64	2.66	0.24
26	0.13	1.40	0.10
393	0.70	1.21	0.58
218	0.13	0.98	0.14
242	0.96	1.03	0.93
543	0.15	1.19	0.12
522	0.76	1.38	0.55
539	0.12	4.25	0.03
490	0.64	2.66	0.24
322	0.13	1.40	0.10
297	0.70	1.21	0.58

where

Screw ID. Screw identifier.

Z: Screw strength.

R: Design load on screw.

$CF = Z/R$: Capacity factor

We can see that the maximum capacity factor is $0.93 < 1$ so the fasteners are strong enough.

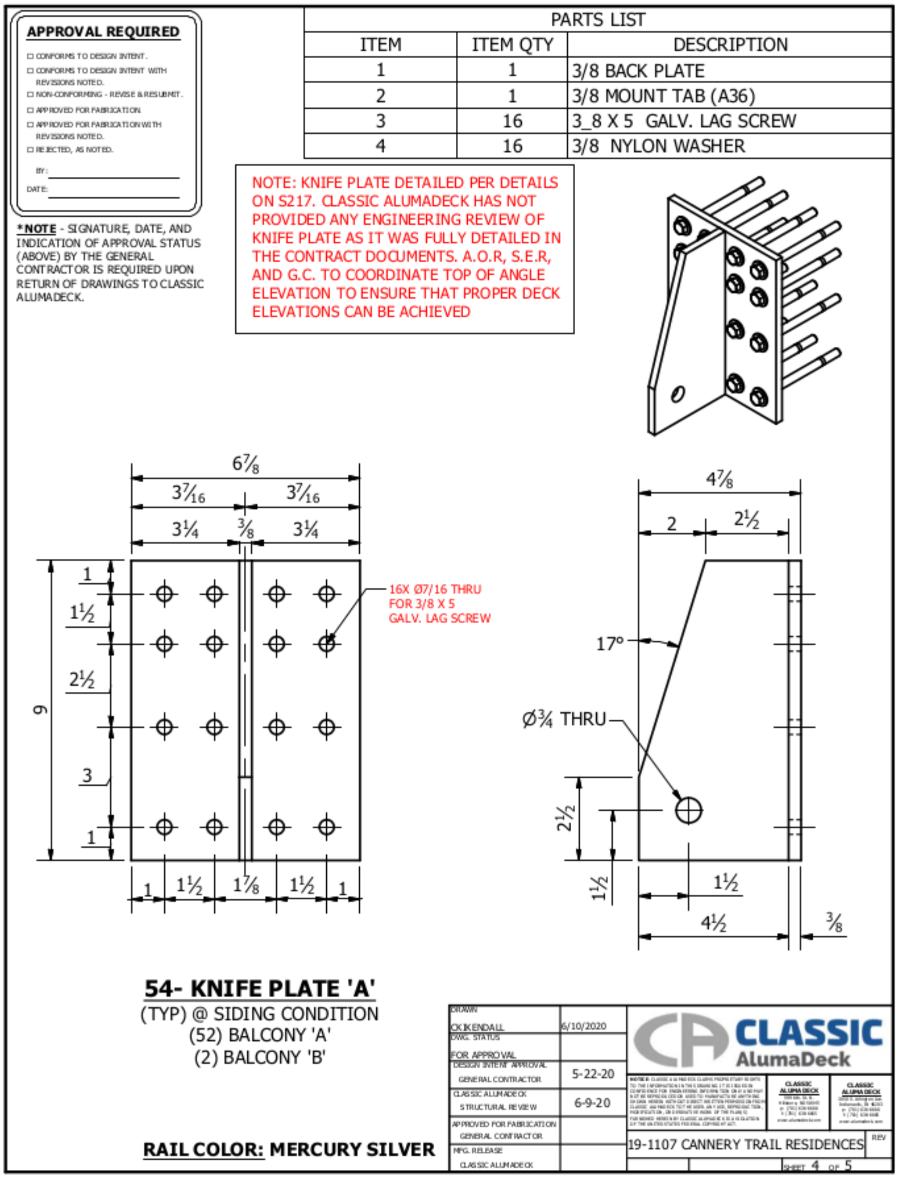


Figure 133: Balconies connection.

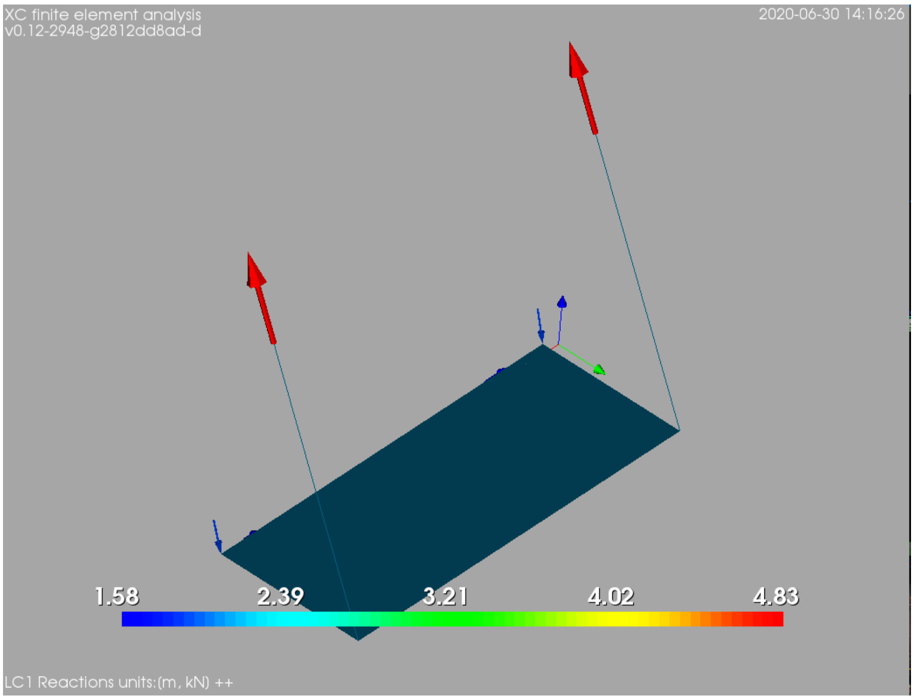


Figure 134: Balconies. Maximum load on connections.

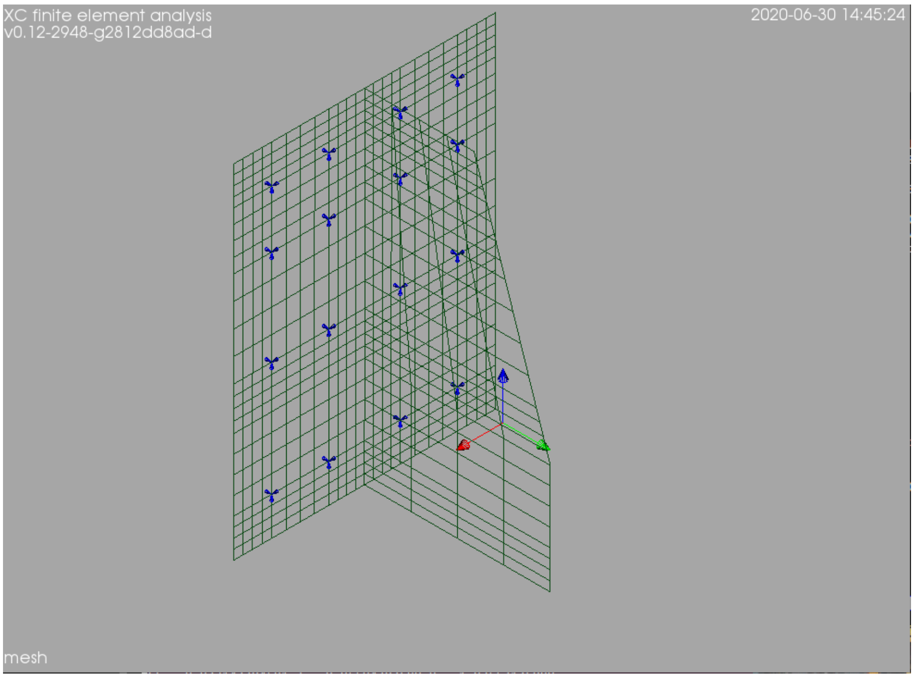


Figure 135: Balconies connection. Finite element model.

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*

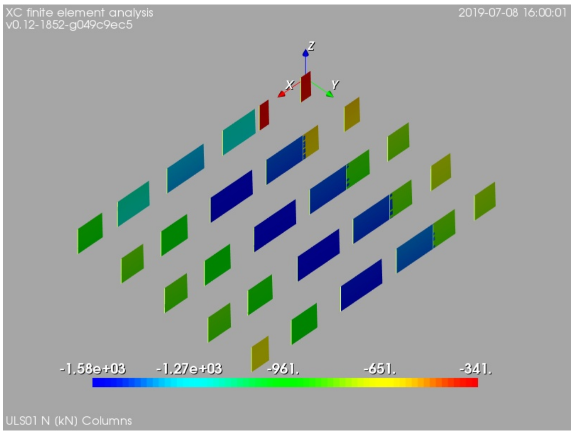


Figure 136: ULS01: 1.4*D. Columns, internal axial force [kN]

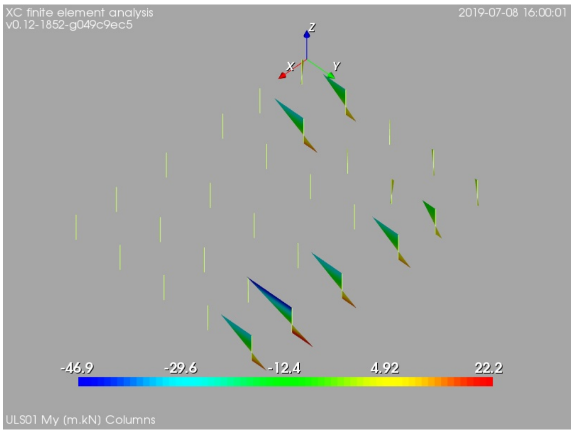


Figure 137: ULS01: 1.4*D. Columns, bending moment about local axis y [m.kN]

B Calculation results. Internal forces on columns

B.1 Ultimate limit states

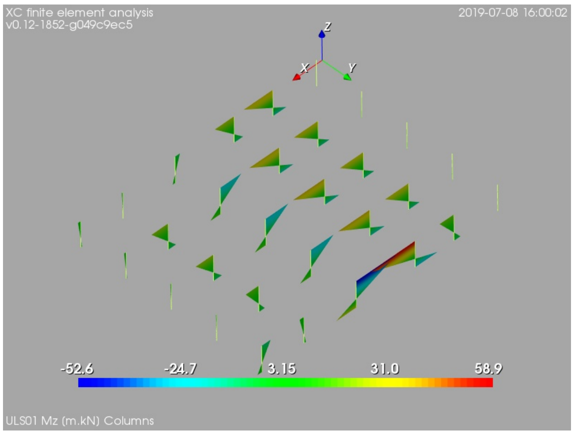


Figure 138: ULS01: 1.4*D. Columns, bending moment about local axis z [m.kN]

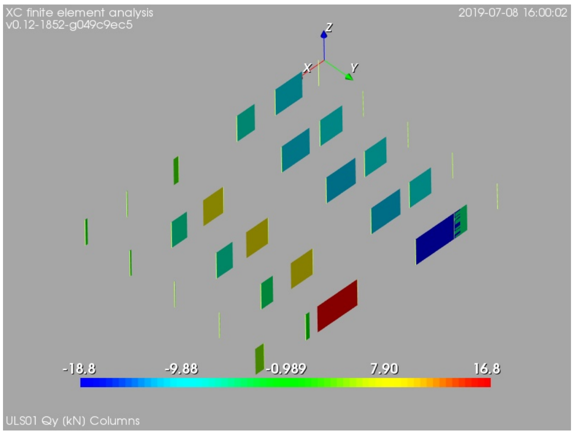


Figure 139: ULS01: 1.4*D. Columns, internal shear force in local direction y [kN]

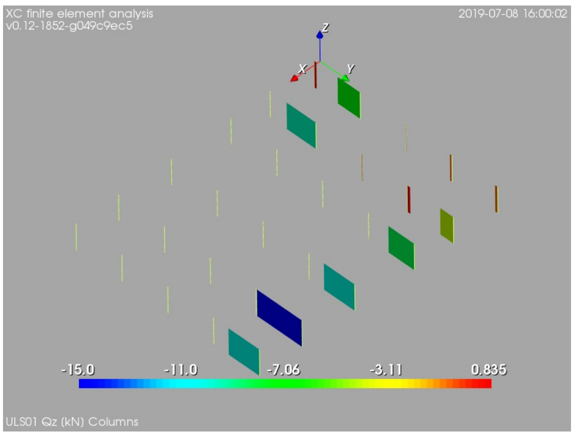


Figure 140: ULS01: 1.4*D. Columns, internal shear force in local direction z [kN]

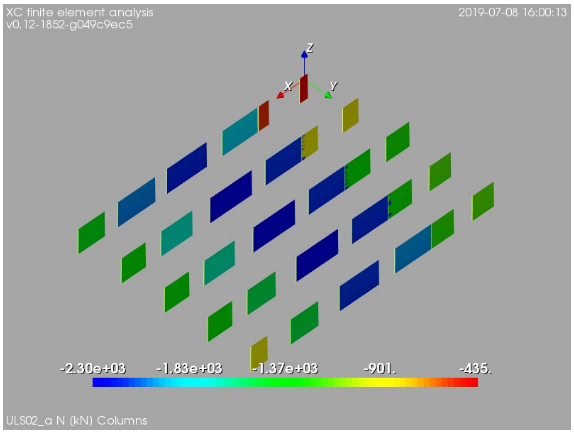


Figure 141: ULS02_a: 1.2*D + 1.6*Lru + Lpu + 0.5*S. Columns, internal axial force [kN]

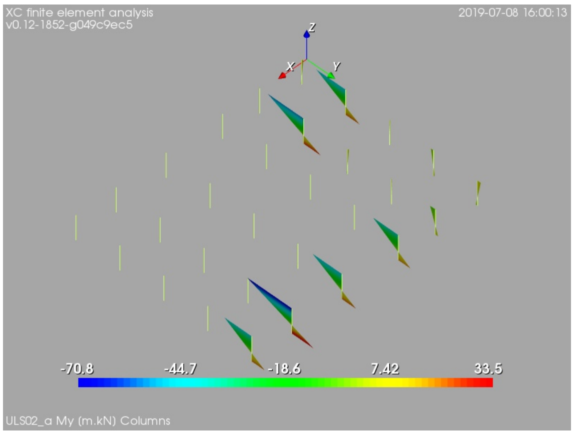


Figure 142: ULS02_a: 1.2*D + 1.6*Lru + Lpu + 0.5*S. Columns, bending moment about local axis y [m.kN]

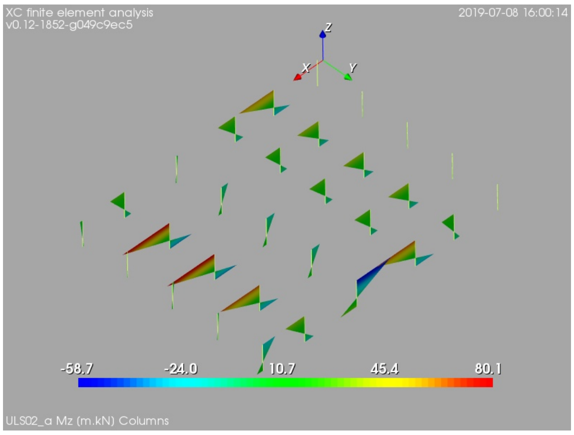


Figure 143: ULS02_a: 1.2*D + 1.6*Lru + Lpu + 0.5*S. Columns, bending moment about local axis z [m.kN]

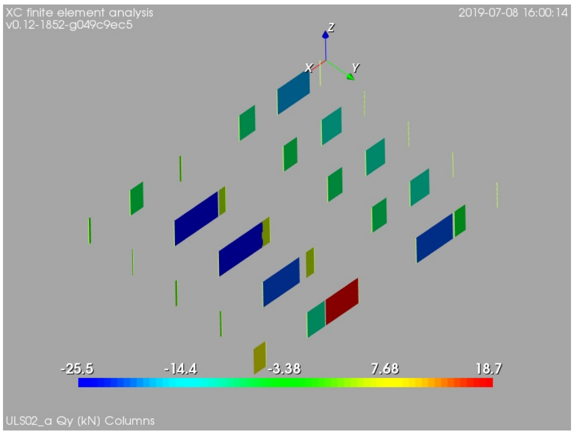


Figure 144: ULS02_a: 1.2*D + 1.6*Lru + Lpu + 0.5*S. Columns, internal shear force in local direction y [kN]

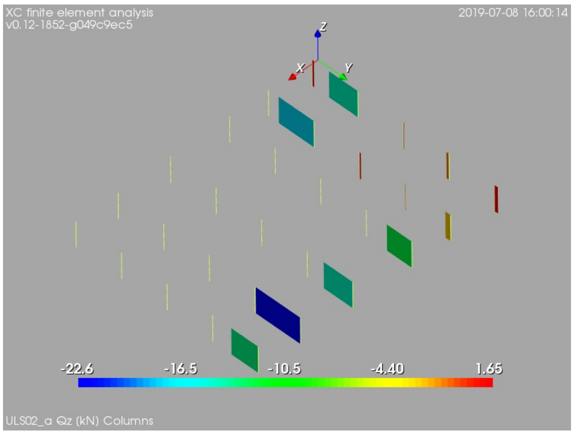


Figure 145: ULS02_a: 1.2*D + 1.6*Lru + Lpu + 0.5*S. Columns, internal shear force in local direction z [kN]

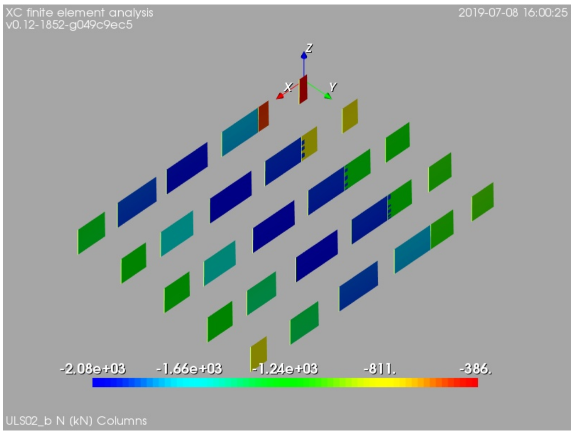


Figure 146: ULS02_b: 1.2*D + 1.6*Lrs + Lps + 0.5*S. Columns, internal axial force [kN]

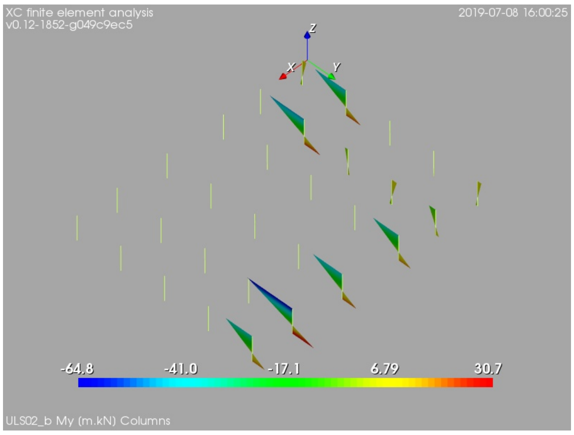


Figure 147: ULS02_b: 1.2*D + 1.6*Lrs + Lps + 0.5*S. Columns, bending moment about local axis y [m.kN]

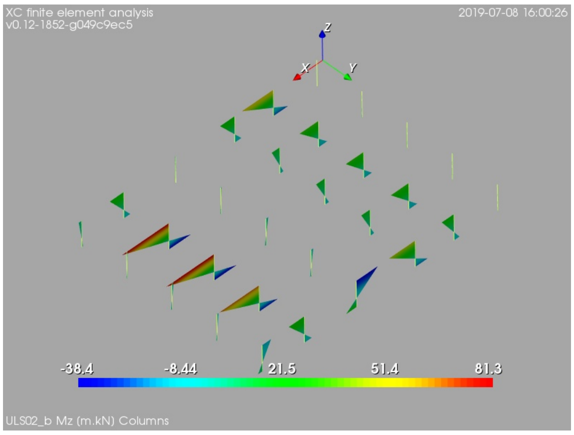


Figure 148: ULS02_b: 1.2*D + 1.6*Lrs + Lps + 0.5*S. Columns, bending moment about local axis z [m.kN]

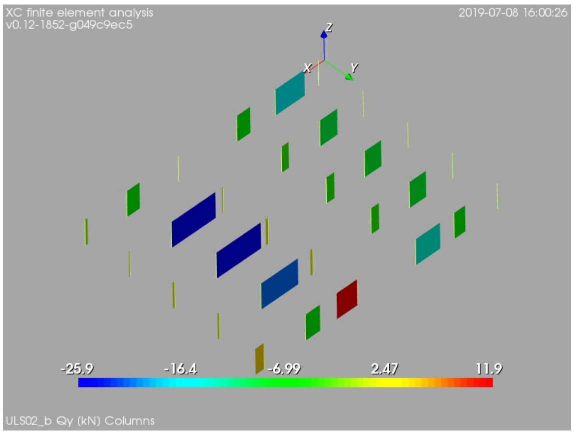


Figure 149: ULS02_b: 1.2*D + 1.6*Lrs + Lps + 0.5*S. Columns, internal shear force in local direction y [kN]

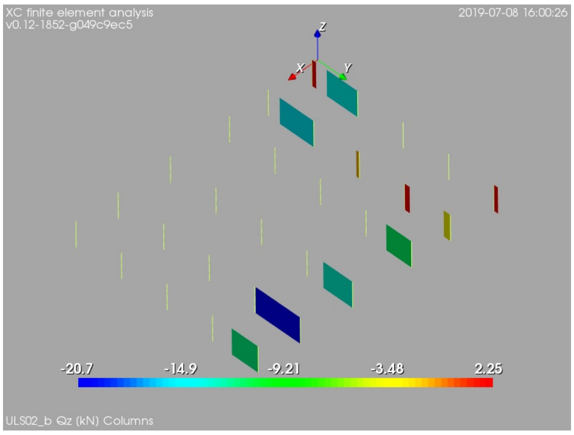


Figure 150: ULS02_b: 1.2*D + 1.6*Lrs + Lps + 0.5*S. Columns, internal shear force in local direction z [kN]

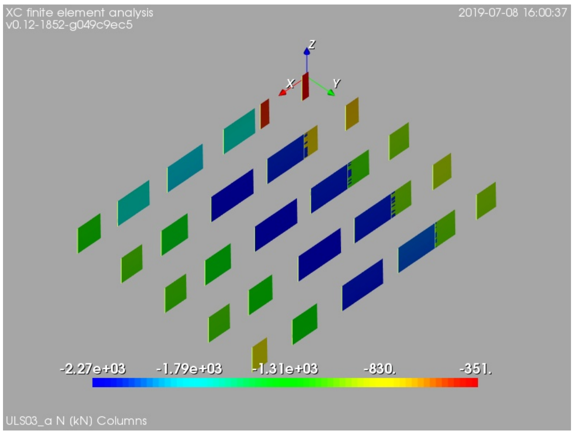


Figure 151: ULS03_a: 1.2*D + 1.6*S + 0.5*Lru + Lpu. Columns, internal axial force [kN]

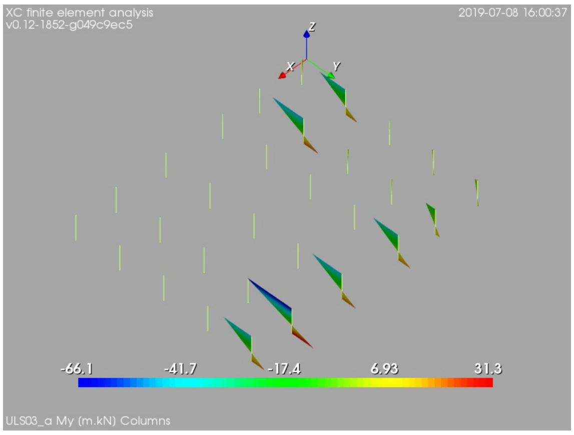


Figure 152: ULS03_a: 1.2*D + 1.6*S + 0.5*Lru + Lpu. Columns, bending moment about local axis y [m.kN]

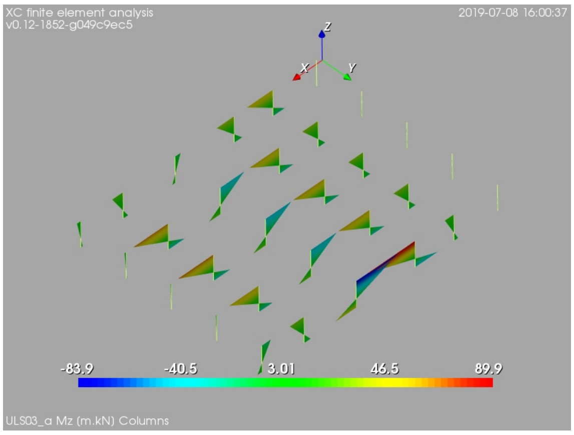


Figure 153: ULS03_a: 1.2*D + 1.6*S + 0.5*Lru + Lpu. Columns, bending moment about local axis z [m.kN]

Figure 154: ULS03_a: 1.2*D + 1.6*S + 0.5*Lru + Lpu. Columns, internal shear force in local direction y [kN]

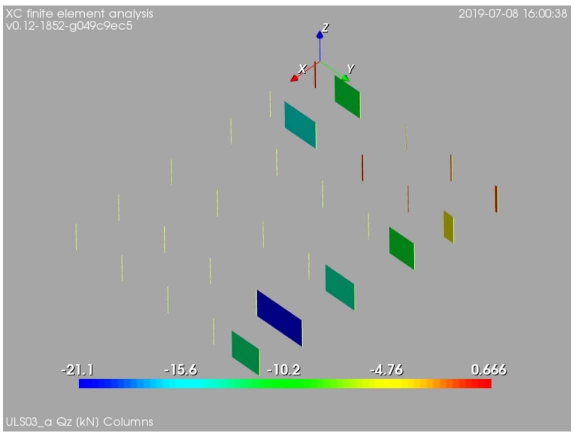


Figure 155: ULS03_a: 1.2*D + 1.6*S + 0.5*Lru + Lpu. Columns, internal shear force in local direction z [kN]

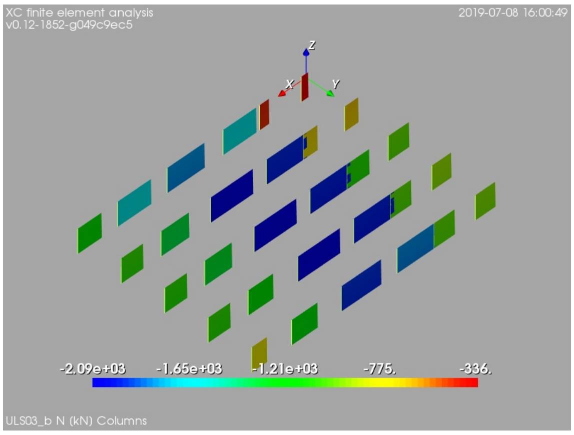


Figure 156: ULS03_b: 1.2*D + 1.6*S + 0.5*Lrs + Lps. Columns, internal axial force [kN]

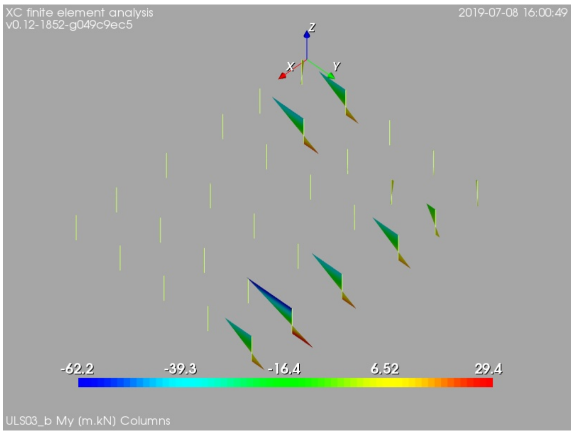


Figure 157: ULS03_b: 1.2*D + 1.6*S + 0.5*Lrs + Lps. Columns, bending moment about local axis y [m.kN]

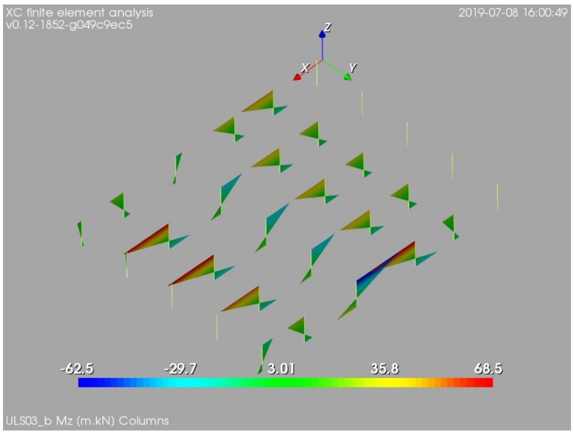


Figure 158: ULS03_b: 1.2*D + 1.6*S + 0.5*Lrs + Lps. Columns, bending moment about local axis z [m.kN]

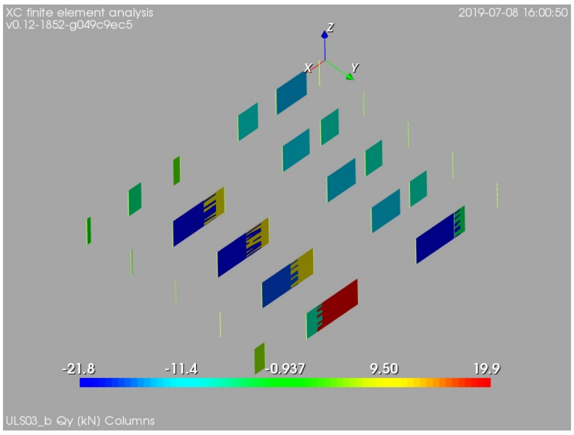


Figure 159: ULS03_b: 1.2*D + 1.6*S + 0.5*Lrs + Lps. Columns, internal shear force in local direction y [kN]

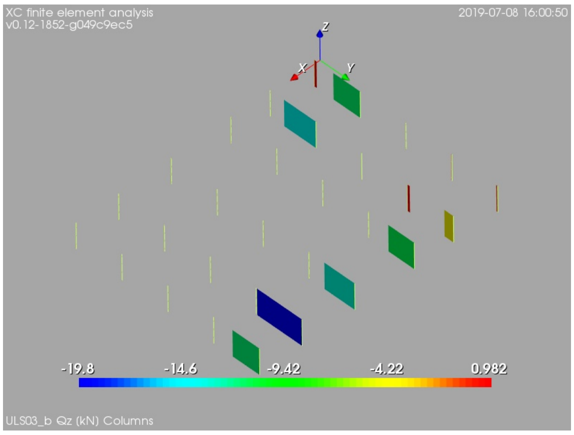


Figure 160: ULS03_b: 1.2*D + 1.6*S + 0.5*Lrs + Lps. Columns, internal shear force in local direction z [kN]

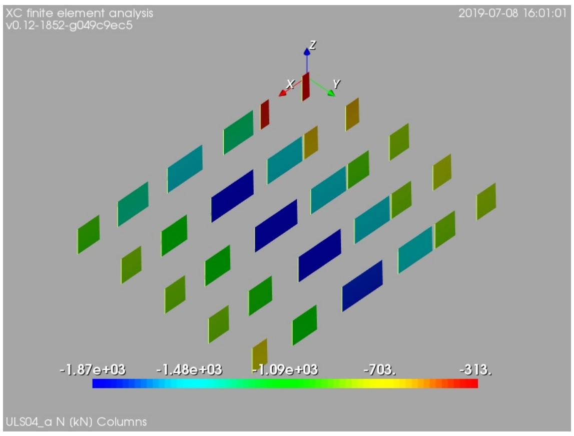


Figure 161: ULS04_a: 1.2*D + 1.6*S + 0.5*W_WE. Columns, internal axial force [kN]

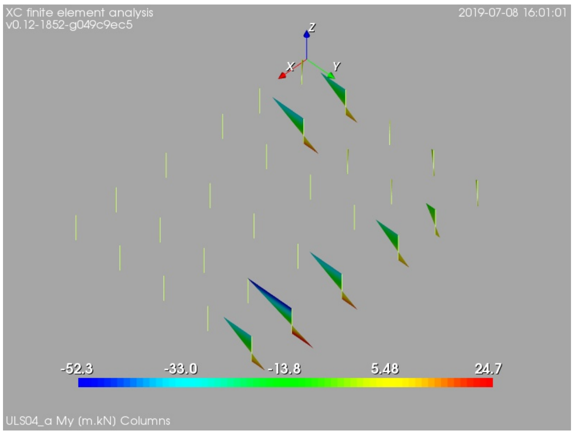


Figure 162: ULS04_a: 1.2*D + 1.6*S + 0.5*W_WE. Columns, bending moment about local axis y [m.kN]

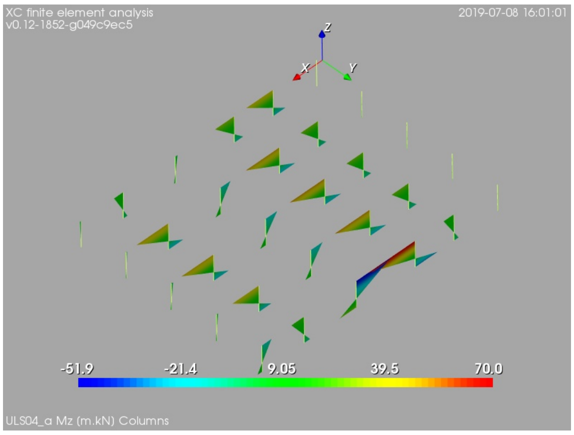


Figure 163: ULS04_a: 1.2*D + 1.6*S + 0.5*W_WE. Columns, bending moment about local axis z [m.kN]

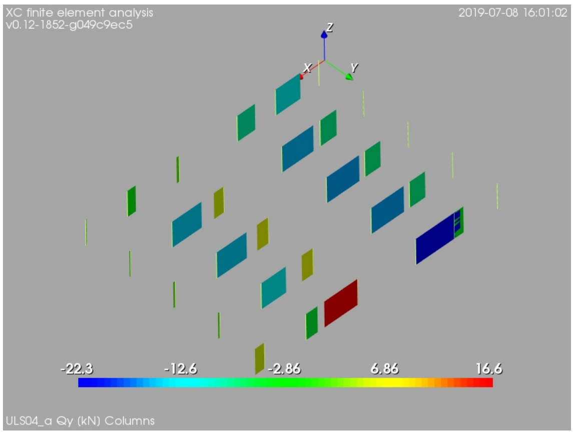


Figure 164: ULS04_a: 1.2*D + 1.6*S + 0.5*W_WE. Columns, internal shear force in local direction y [kN]

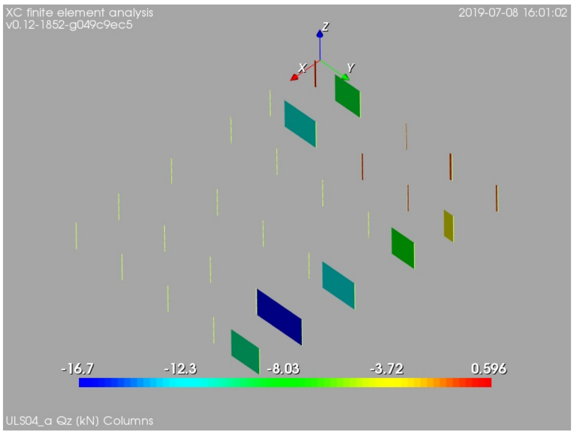


Figure 165: ULS04_a: 1.2*D + 1.6*S + 0.5*W_WE. Columns, internal shear force in local direction z [kN]

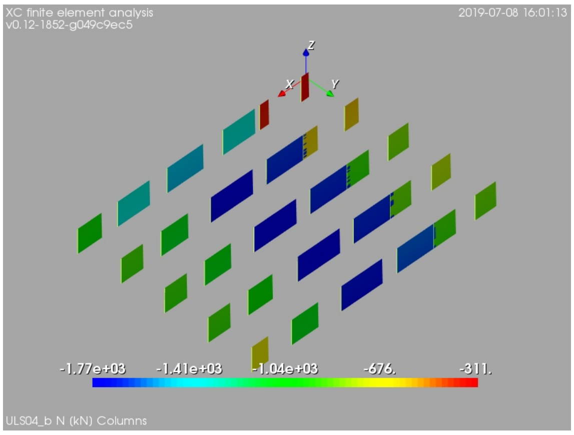


Figure 166: ULS04_b: 1.2*D + 1.6*S + 0.5*W_NS. Columns, internal axial force [kN]

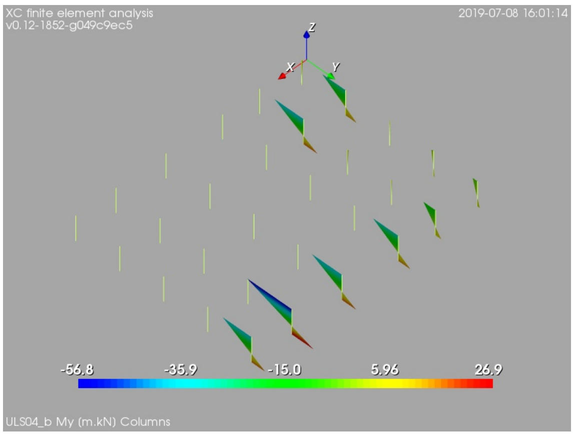


Figure 167: ULS04_b: 1.2*D + 1.6*S + 0.5*W_NS. Columns, bending moment about local axis y [m.kN]

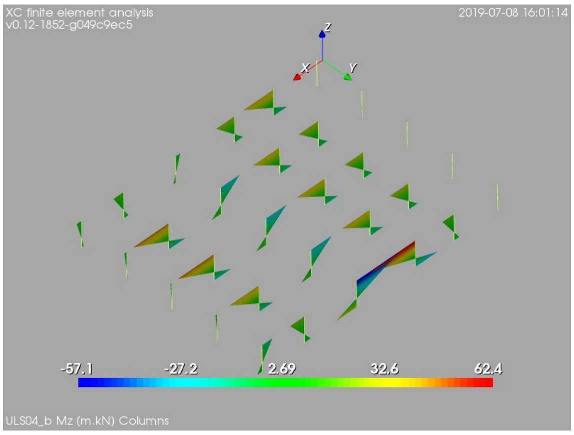


Figure 168: ULS04_b: 1.2*D + 1.6*S + 0.5*W_NS. Columns, bending moment about local axis z [m.kN]

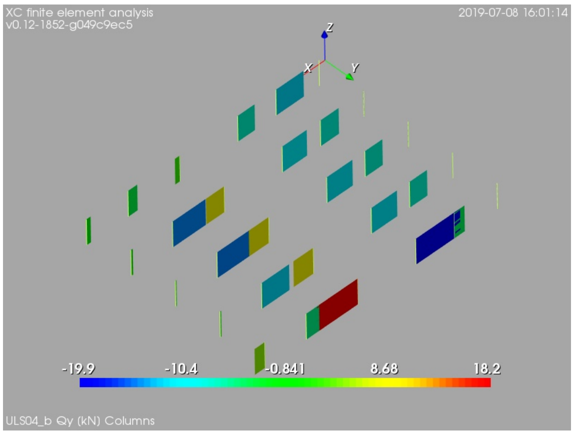


Figure 169: ULS04_b: 1.2*D + 1.6*S + 0.5*W_NS. Columns, internal shear force in local direction y [kN]

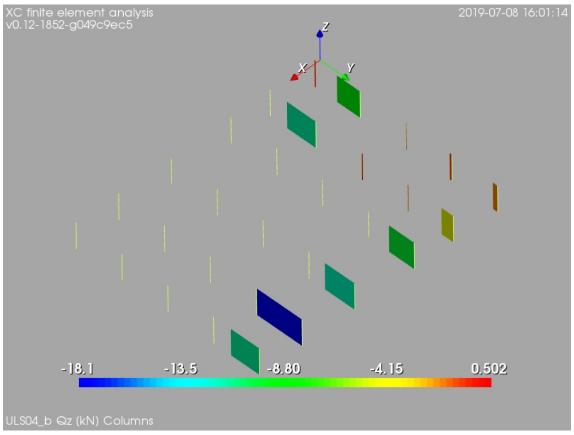


Figure 170: ULS04_b: 1.2*D + 1.6*S + 0.5*W_NS. Columns, internal shear force in local direction z [kN]

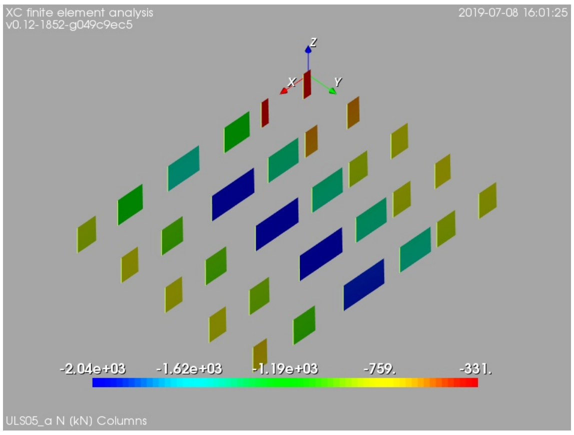


Figure 171: ULS05_a: 1.2*D + W_WE + 0.5*Lru + Lpu. Columns, internal axial force [kN]

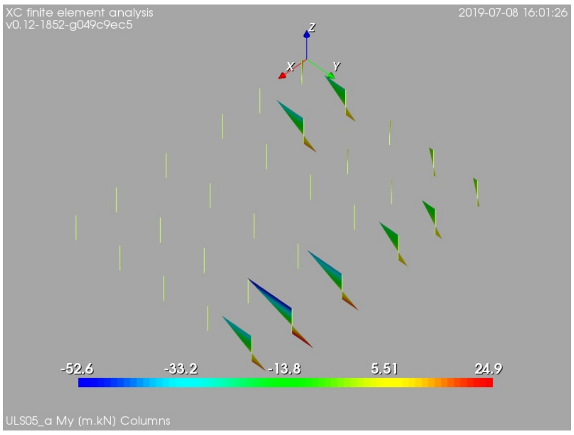


Figure 172: ULS05_a: 1.2*D + W_WE + 0.5*Lru + Lpu. Columns, bending moment about local axis y [m.kN]

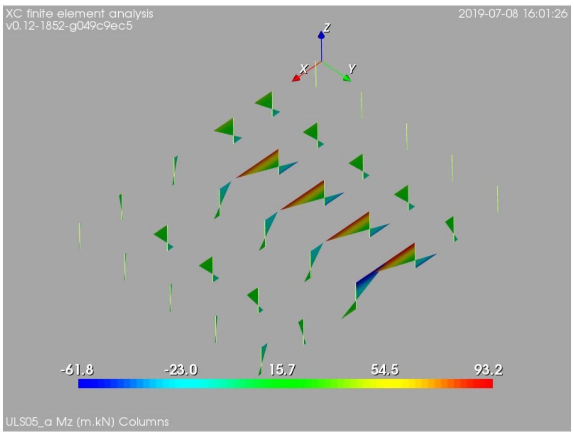


Figure 173: ULS05_a: 1.2*D + W_WE + 0.5*Lru + Lpu. Columns, bending moment about local axis z [m.kN]

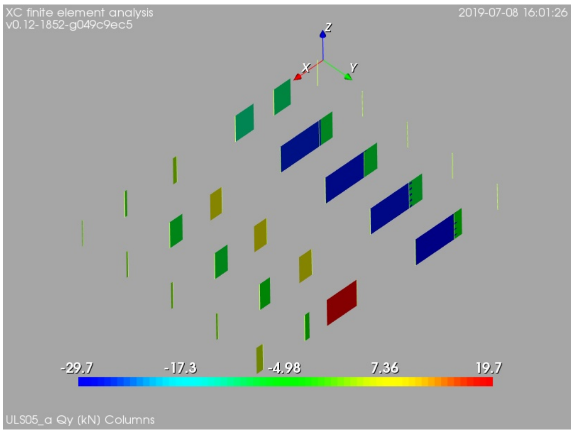


Figure 174: ULS05_a: 1.2*D + W_WE + 0.5*Lru + Lpu. Columns, internal shear force in local direction y [kN]

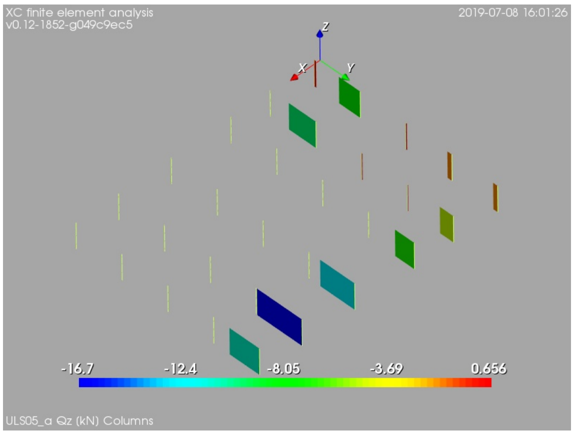


Figure 175: ULS05_a: 1.2*D + W_WE + 0.5*Lru + Lpu. Columns, internal shear force in local direction z [kN]

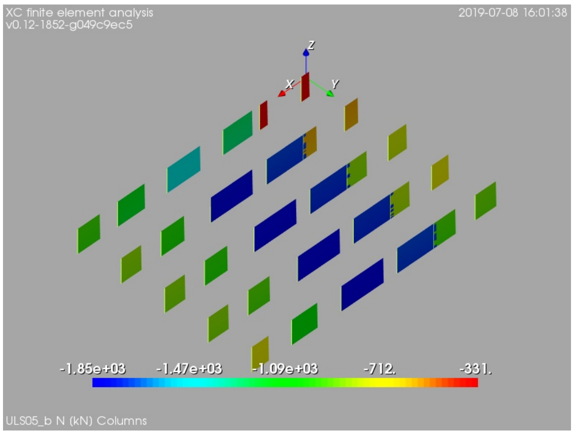


Figure 176: ULS05_b: 1.2*D + W_NS + 0.5*Lru + Lpu. Columns, internal axial force [kN]

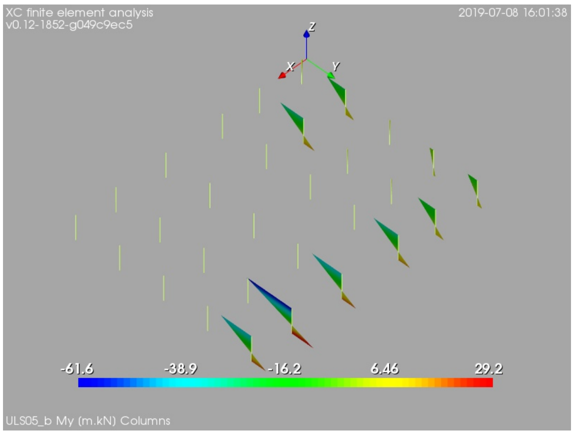


Figure 177: ULS05_b: 1.2*D + W_NS + 0.5*Lru + Lpu. Columns, bending moment about local axis y [m.kN]

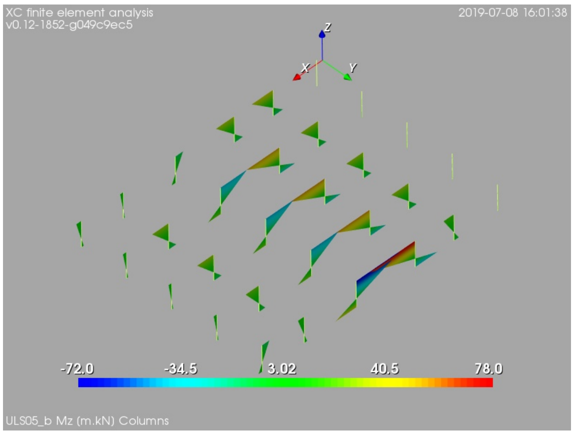


Figure 178: ULS05_b: 1.2*D + W_NS + 0.5*Lru + Lpu. Columns, bending moment about local axis z [m.kN]

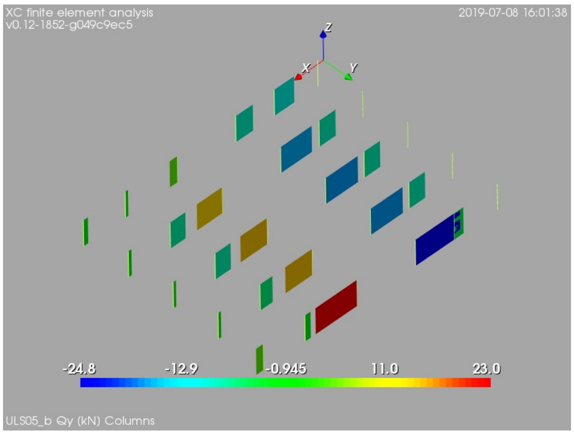


Figure 179: ULS05_b: 1.2*D + W_NS + 0.5*Lru + Lpu. Columns, internal shear force in local direction y [kN]

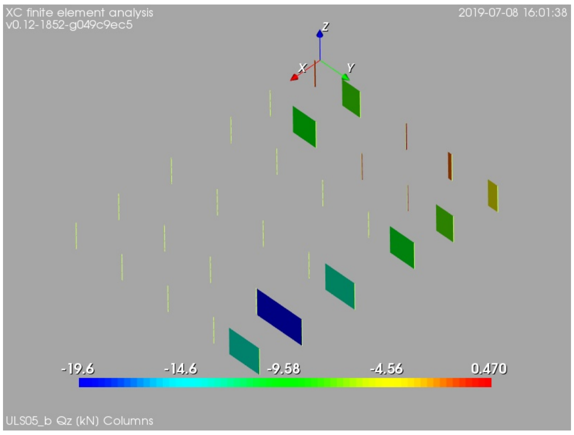


Figure 180: ULS05_b: 1.2*D + W_NS + 0.5*Lru + Lpu. Columns, internal shear force in local direction z [kN]

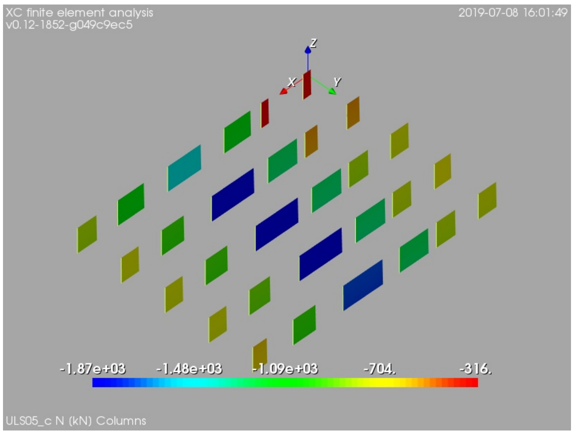


Figure 181: ULS05_c: 1.2*D + W_WE + 0.5*Lrs + Lps. Columns, internal axial force [kN]

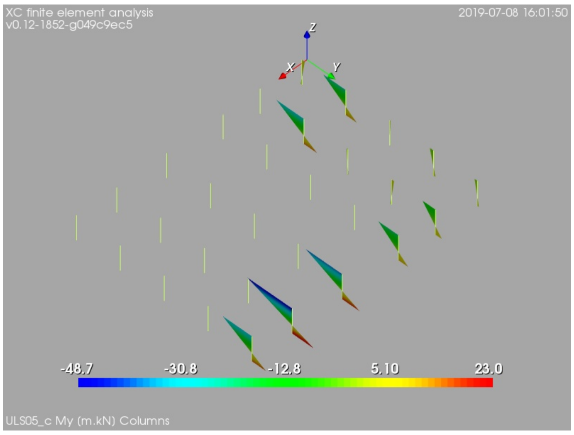


Figure 182: ULS05_c: 1.2*D + W_WE + 0.5*Lrs + Lps. Columns, bending moment about local axis y [m.kN]

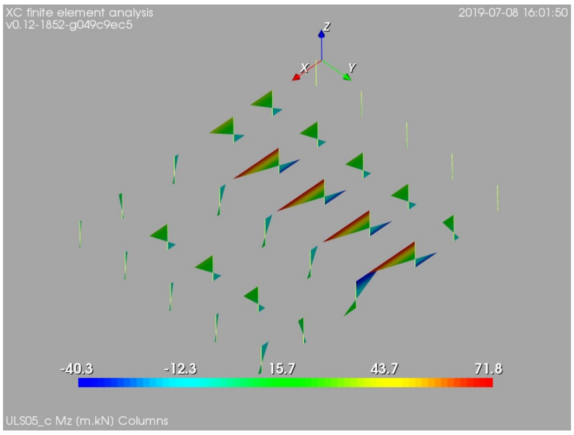


Figure 183: ULS05_c: 1.2*D + W_WE + 0.5*Lrs + Lps. Columns, bending moment about local axis z [m.kN]

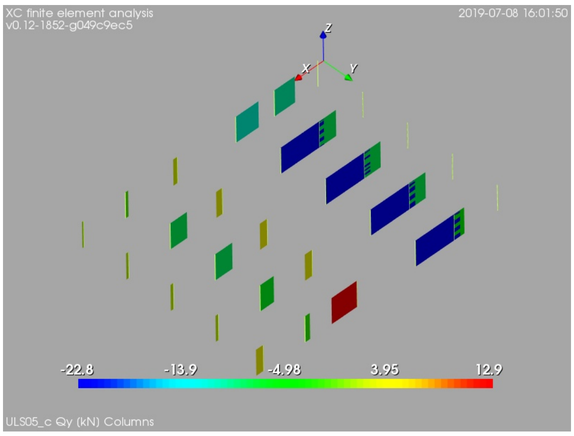


Figure 184: ULS05_c: 1.2*D + W_WE + 0.5*Lrs + Lps. Columns, internal shear force in local direction y [kN]

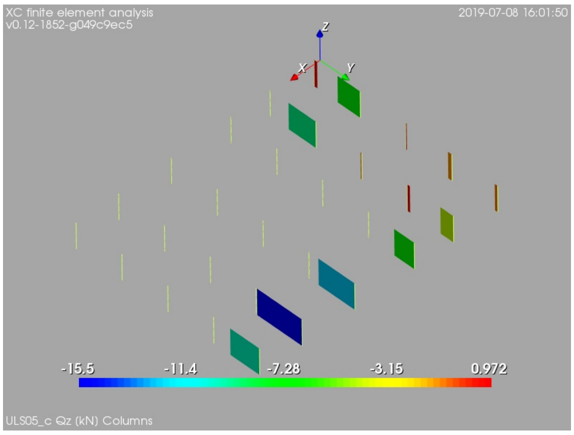


Figure 185: ULS05_c: 1.2*D + W_WE + 0.5*Lrs + Lps. Columns, internal shear force in local direction z [kN]

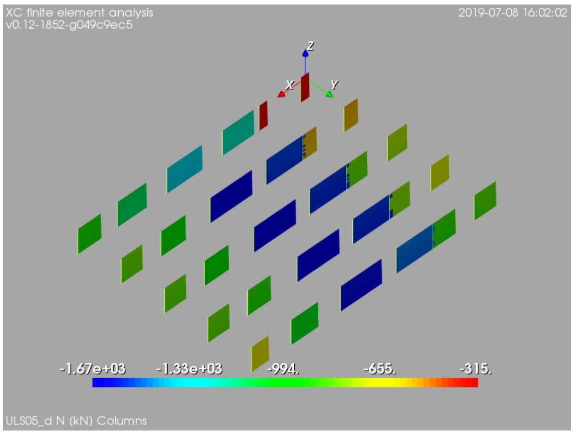


Figure 186: ULS05_d: 1.2*D + W_NS + 0.5*Lrs + Lps. Columns, internal axial force [kN]

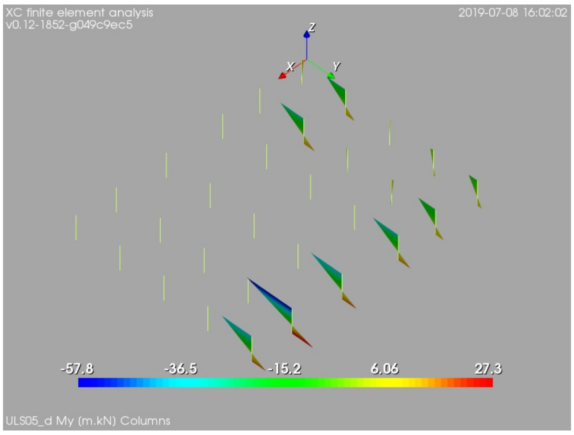


Figure 187: ULS05_d: 1.2*D + W_NS + 0.5*Lrs + Lps. Columns, bending moment about local axis y [m.kN]

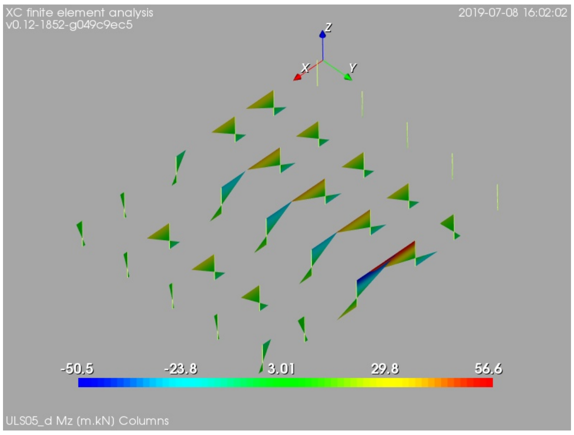


Figure 188: ULS05_d: 1.2*D + W_NS + 0.5*Lrs + Lps. Columns, bending moment about local axis z [m.kN]

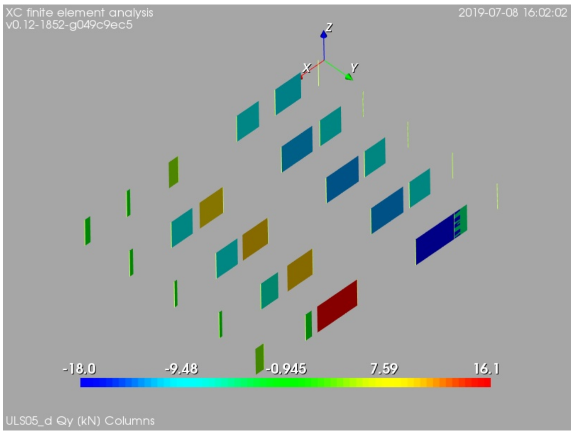


Figure 189: ULS05_d: 1.2*D + W_NS + 0.5*Lrs + Lps. Columns, internal shear force in local direction y [kN]

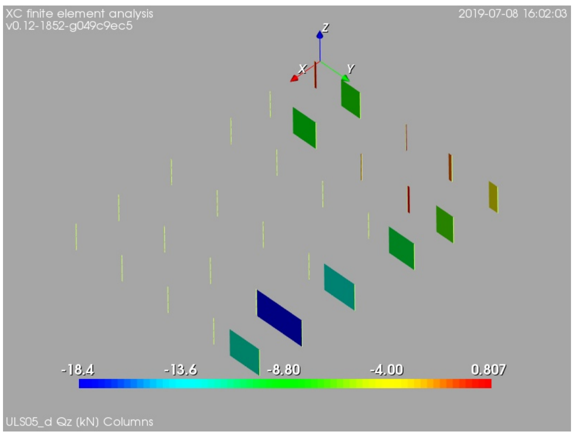


Figure 190: ULS05_d: 1.2*D + W_NS + 0.5*Lrs + Lps. Columns, internal shear force in local direction z [kN]

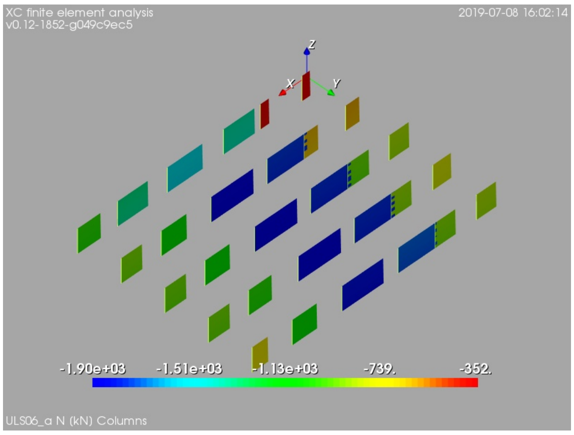


Figure 191: ULS06_a: 1.2*D + 0.5*Lru + Lpu + 0.2*S. Columns, internal axial force [kN]

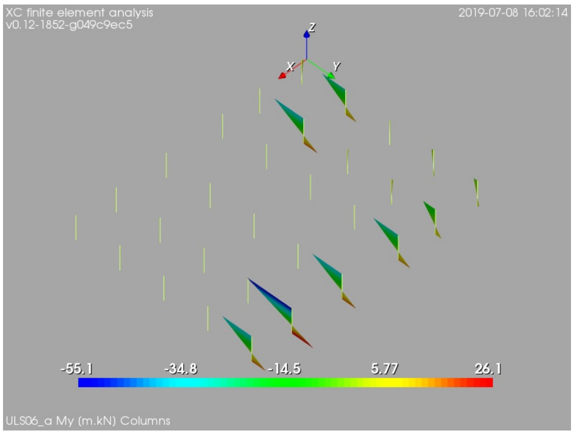


Figure 192: ULS06_a: 1.2*D + 0.5*Lru + Lpu + 0.2*S. Columns, bending moment about local axis y [m.kN]

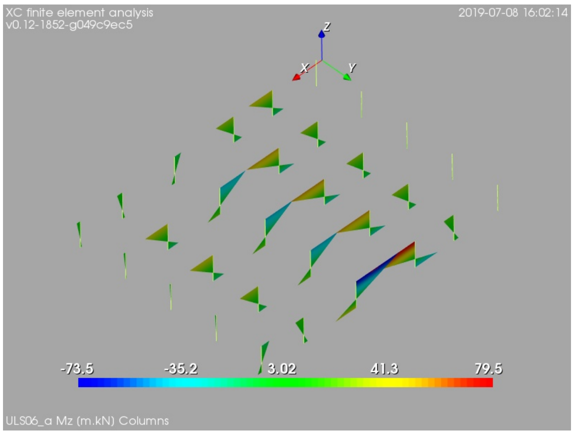


Figure 193: ULS06_a: 1.2*D + 0.5*Lru + Lpu + 0.2*S. Columns, bending moment about local axis z [m.kN]

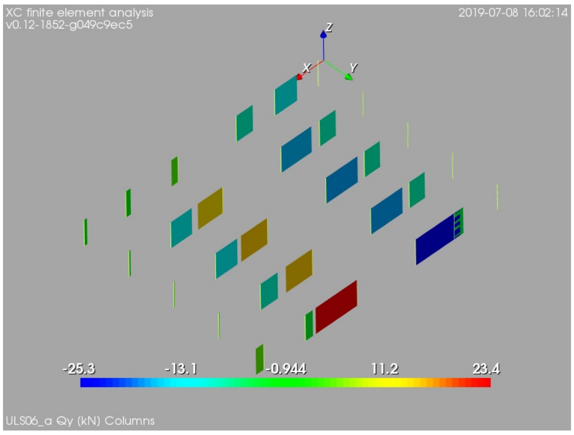


Figure 194: ULS06_a: 1.2*D + 0.5*Lru + Lpu + 0.2*S. Columns, internal shear force in local direction y [kN]

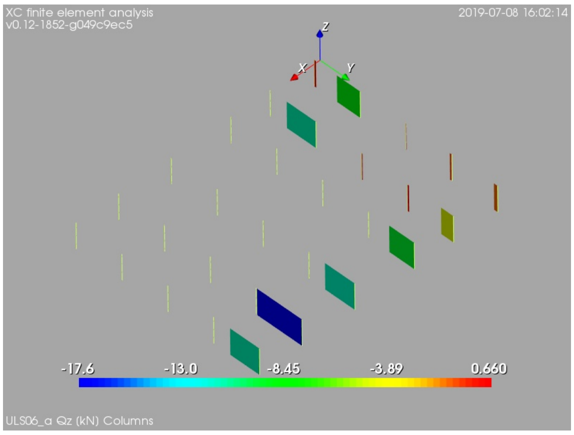


Figure 195: ULS06_a: 1.2*D + 0.5*Lru + Lpu + 0.2*S. Columns, internal shear force in local direction z [kN]

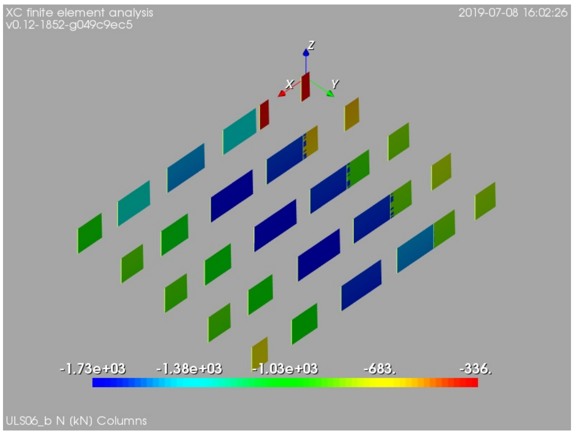


Figure 196: ULS06_b: 1.2*D + 0.5*Lrs + Lps + 0.2*S. Columns, internal axial force [kN]

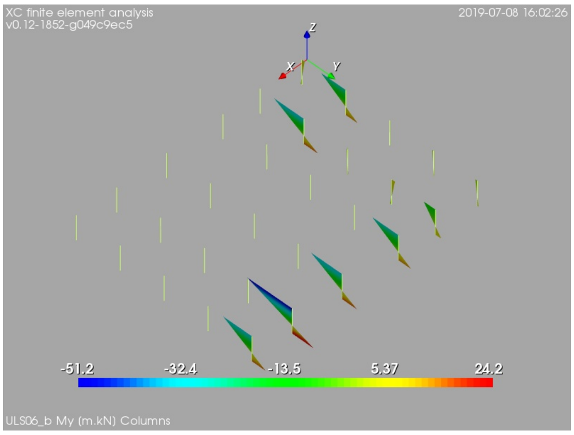


Figure 197: ULS06_b: 1.2*D + 0.5*Lrs + Lps + 0.2*S. Columns, bending moment about local axis y [m.kN]

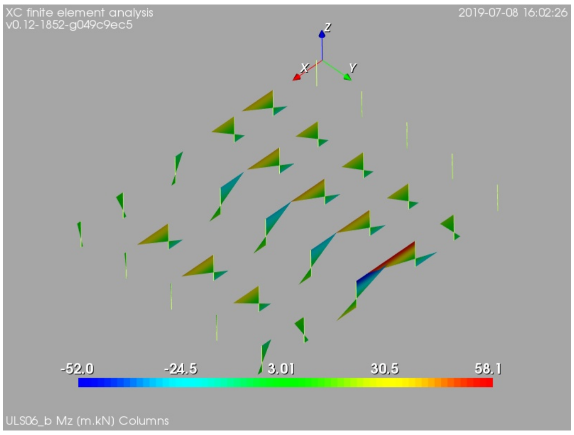


Figure 198: ULS06_b: 1.2*D + 0.5*Lrs + Lps + 0.2*S. Columns, bending moment about local axis z [m.kN]

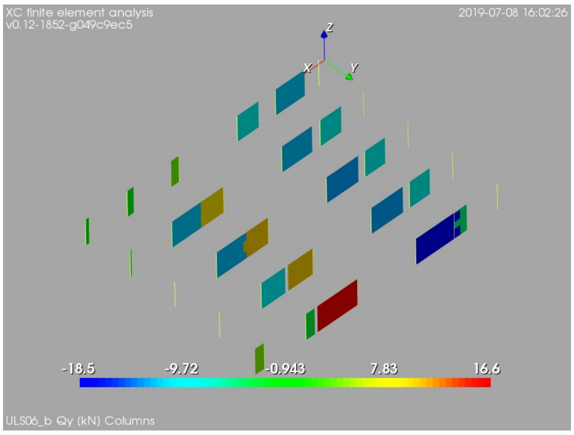


Figure 199: ULS06_b: 1.2*D + 0.5*Lrs + Lps + 0.2*S. Columns, internal shear force in local direction y [kN]

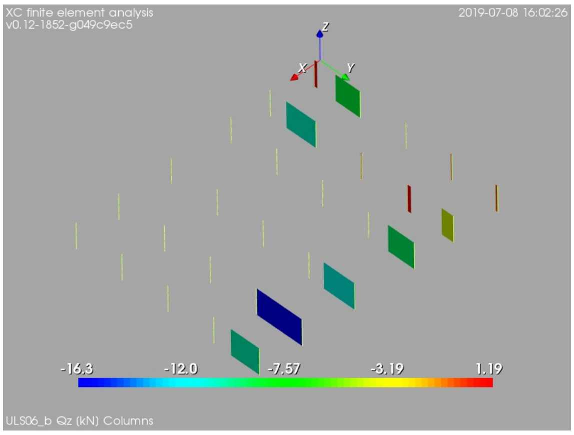


Figure 200: ULS06_b: 1.2*D + 0.5*Lrs + Lps + 0.2*S. Columns, internal shear force in local direction z [kN]

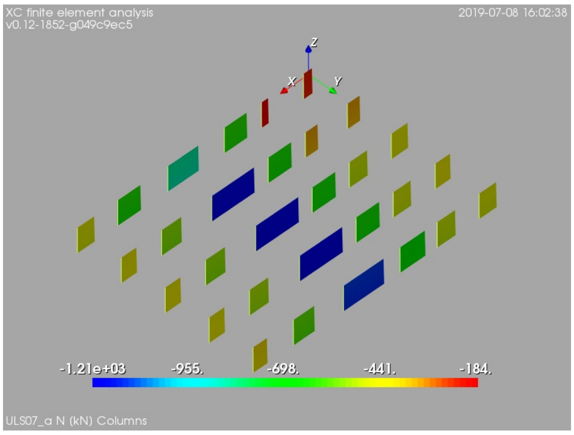


Figure 201: ULS07_a: 0.9*D + W_WE. Columns, internal axial force [kN]

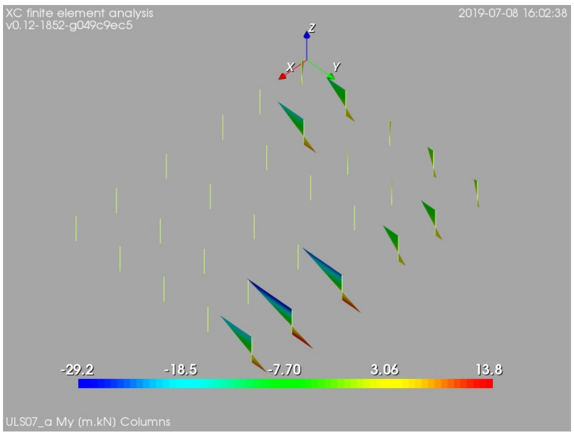


Figure 202: ULS07_a: 0.9*D + W_WE. Columns, bending moment about local axis y [m.kN]

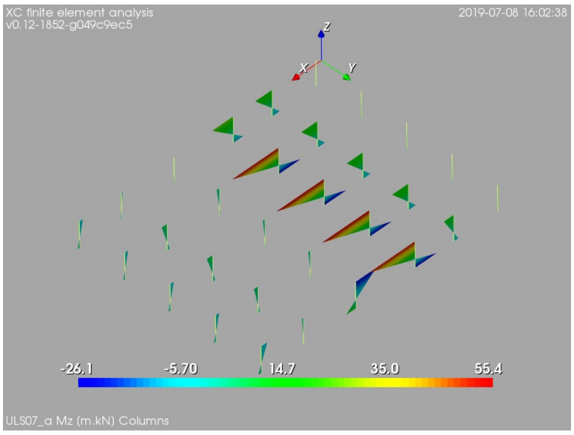


Figure 203: ULS07_a: 0.9*D + W_WE. Columns, bending moment about local axis z [m.kN]

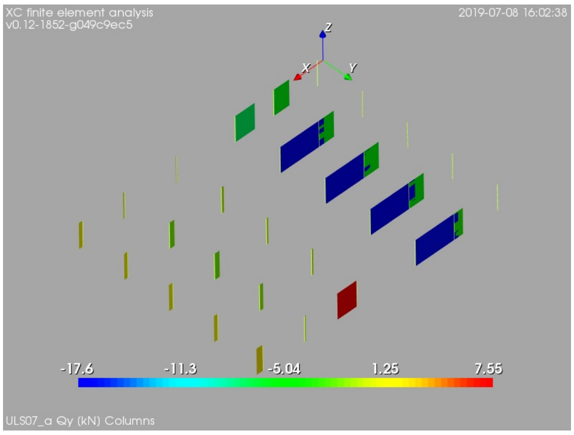


Figure 204: ULS07_a: 0.9*D + W_WE. Columns, internal shear force in local direction y [kN]

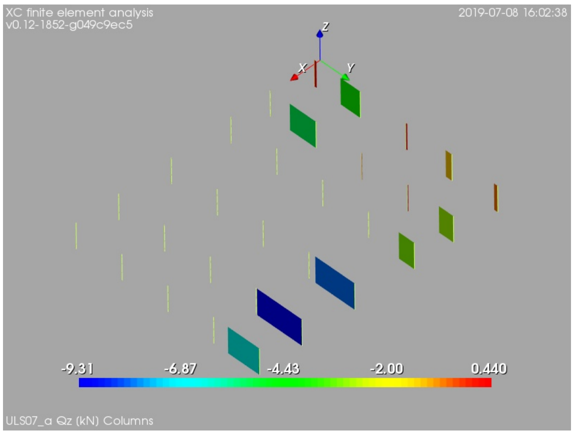


Figure 205: ULS07_a: 0.9*D + W_WE. Columns, internal shear force in local direction z [kN]

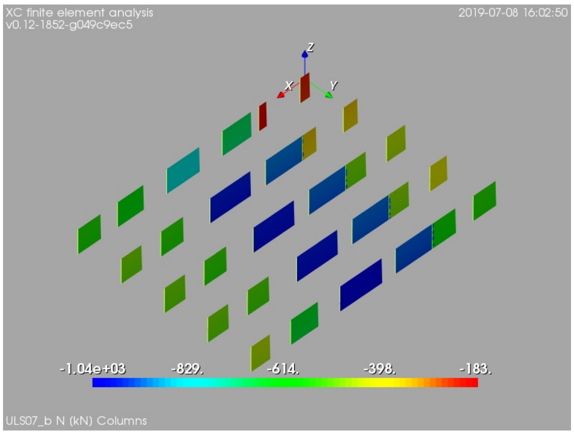


Figure 206: ULS07_b: 0.9*D + W_NS. Columns, internal axial force [kN]

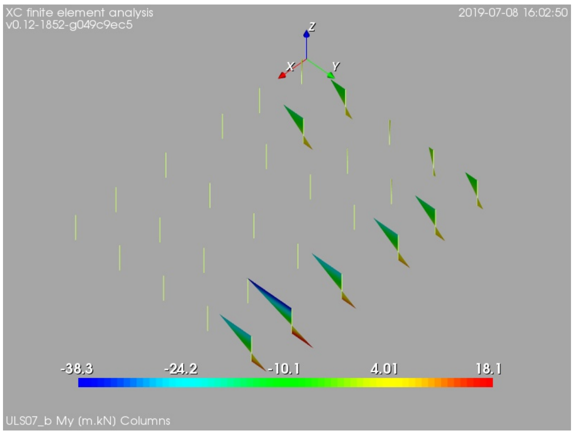


Figure 207: ULS07_b: 0.9*D + W_NS. Columns, bending moment about local axis y [m.kN]

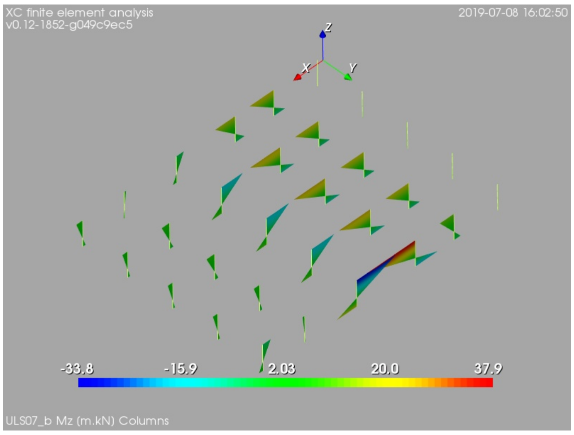


Figure 208: ULS07_b: 0.9*D + W_NS. Columns, bending moment about local axis z [m.kN]

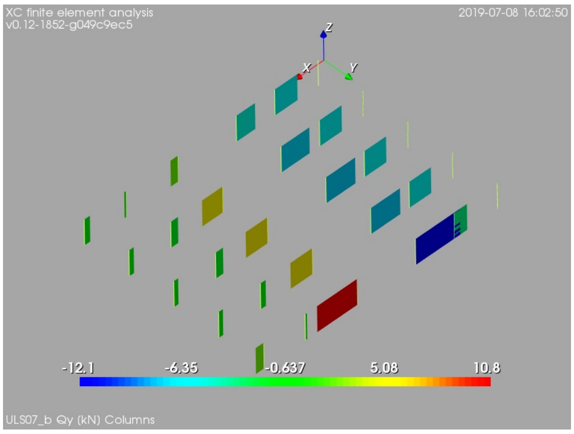


Figure 209: ULS07_b: 0.9*D + W_NS. Columns, internal shear force in local direction y [kN]

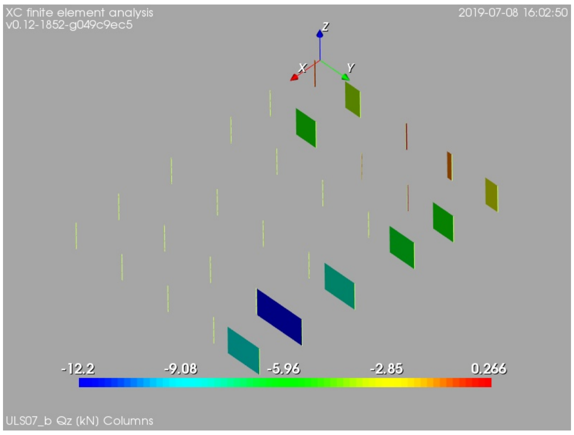


Figure 210: ULS07_b: 0.9*D + W_NS. Columns, internal shear force in local direction z [kN]

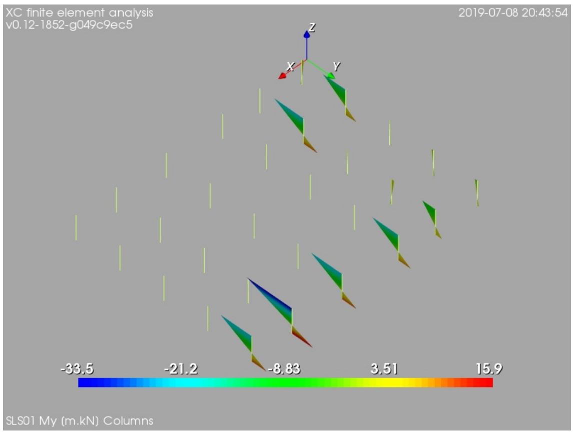


Figure 211: SLS01: 1.0*D. Columns, bending moment about local axis y [m.kN]

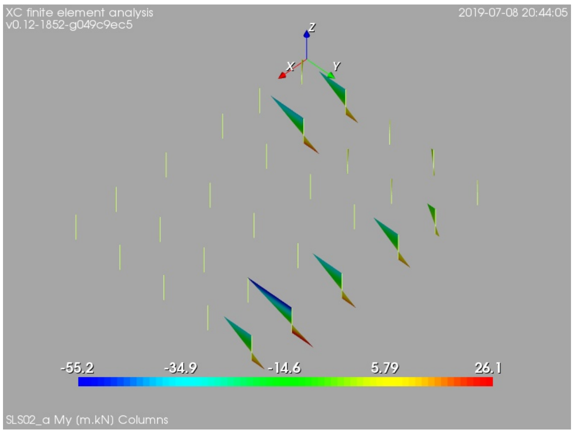


Figure 212: SLS02_a: 1.0*D + 1.0*Lru + Lpu + 0.3*S. Columns, bending moment about local axis y [m.kN]

B.2 Serviceability limit states

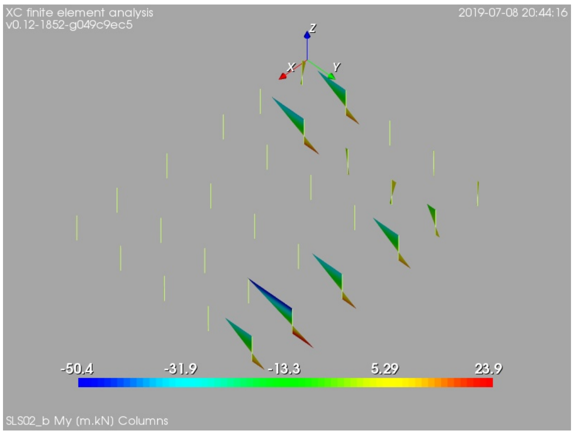


Figure 213: SLS02_b: 1.0*D + 1.0*Lrs + Lps + 0.3*S. Columns, bending moment about local axis y [m.kN]

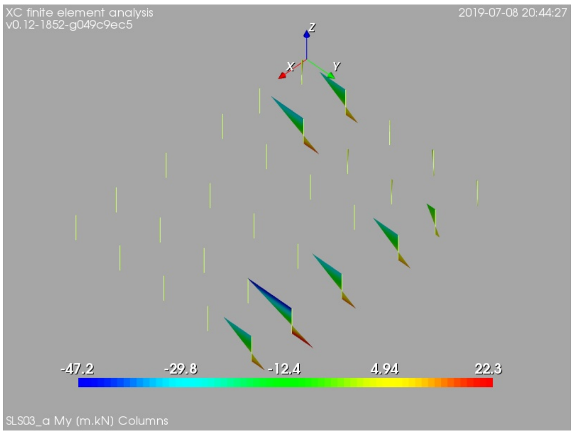


Figure 214: SLS03_a: 1.0*D + 1.0*S + 0.3*Lru + 0.3*Lpu. Columns, bending moment about local axis y [m.kN]

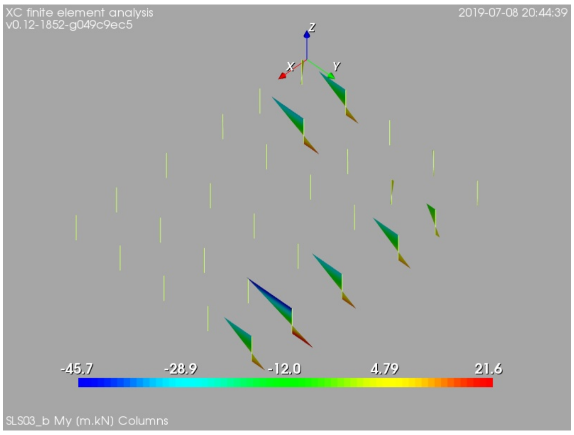


Figure 215: SLS03_b: 1.0*D + 1.0*S + 0.3*Lrs + 0.3*Lps. Columns, bending moment about local axis y [m.kN]

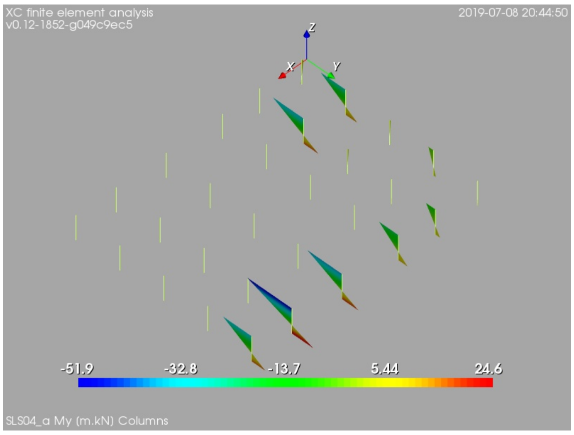


Figure 216: SLS04_a: 1.0*D + W_WE + 1.0*Lru + Lpu. Columns, bending moment about local axis y [m.kN]

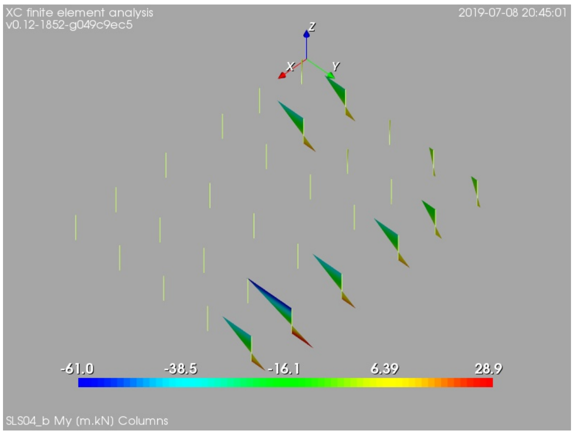


Figure 217: SLS04_b: 1.0*D + W_NS + 1.0*Lru + Lpu. Columns, bending moment about local axis y [m.kN]

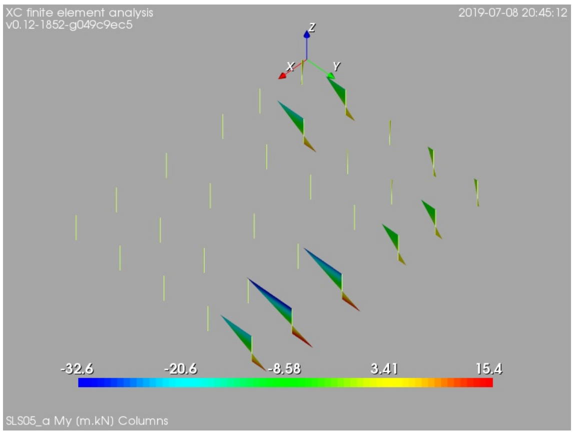


Figure 218: SLS05_a: 1.0*D + W_WE. Columns, bending moment about local axis y [m.kN]

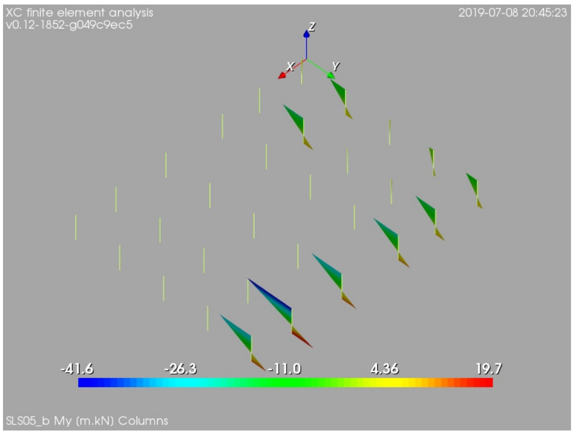


Figure 219: SLS05_b: 1.0*D + W_NS. Columns, bending moment about local axis y [m.kN]