

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

XC structural engineering

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## Contents

<b>1</b>	<b>Introduction and scope</b>	<b>9</b>
<b>2</b>	<b>Building codes</b>	<b>9</b>
<b>3</b>	<b>Loading criteria</b>	<b>9</b>
3.1	Gravity loading . . . . .	9
3.2	Wind design criteria . . . . .	9
3.3	Snow loading . . . . .	9
<b>4</b>	<b>Seismic design criteria</b>	<b>11</b>
<b>5</b>	<b>Materials</b>	<b>11</b>
<b>6</b>	<b>Design and analysis software</b>	<b>11</b>
<b>7</b>	<b>Load combinations</b>	<b>11</b>
<b>8</b>	<b>Wood framing</b>	<b>14</b>
8.1	Gravity . . . . .	14
8.2	Trusses . . . . .	14
8.2.1	Trusses A and B. Roof . . . . .	14
8.2.2	Trusses A and B. Third floor . . . . .	14
8.2.3	Trusses A and B. Second floor . . . . .	17
8.2.4	Trusses C and D. Roof . . . . .	17
8.2.5	Trusses C and D. Third floor . . . . .	18
8.2.6	Trusses C and D. Second floor . . . . .	19
8.2.7	Truss E. Roof . . . . .	20
8.2.8	Truss E. Third floor . . . . .	21
8.2.9	Truss E. Second floor . . . . .	22
8.3	Joists . . . . .	23
8.3.1	Corridor floor sheathing . . . . .	23
8.3.2	Corridor Joists . . . . .	24
8.3.3	Joists under storage/HVAC floor . . . . .	27
8.4	Headers . . . . .	28
8.4.1	Third floor enclosed balconies headers (H3.1 to H3.3) . . . . .	28
8.4.2	Corridor headers (H3.4 to H3.9, H2.1 to H2.6 and H1.1 to H1.6) . . . . .	32
8.4.3	Headers H3.10 and H2.7 . . . . .	33

8.4.4	Header H1.9 . . . . .	34
8.4.5	Facade headers . . . . .	35
8.4.6	Corridor headers . . . . .	35
8.5	Steel beams . . . . .	36
8.5.1	Steel beam at courtyard facade . . . . .	36
8.5.2	Steel beam at corridor . . . . .	38
8.6	Cantilevers . . . . .	40
8.6.1	Cantilevers C1, C3, C4, C6, C11 and C12 . . . . .	40
8.6.2	Cantilevers C2 and C5 . . . . .	42
8.6.3	Cantilevers C7, C8, C9 and C10 . . . . .	42
8.6.4	Cantilever C13 . . . . .	43
8.6.5	Cantilever C14 . . . . .	44
8.7	Columns . . . . .	44
8.7.1	Columns P3.1 to P3.6 . . . . .	44
8.7.2	Column P3.7 . . . . .	48
8.7.3	Column P2.1 . . . . .	48
8.7.4	2x6 wood stud capacity . . . . .	49
8.7.5	Columns P1.1, P1.2 and P1.3 . . . . .	49
8.7.6	Column P1.4 . . . . .	50
8.8	Bearing walls . . . . .	51
8.8.1	Facade bearing walls at first floor . . . . .	51
8.8.2	Facade bearing walls at second floor . . . . .	53
8.8.3	Facade bearing walls at third floor . . . . .	53
8.8.4	Interior bearing walls at first floor . . . . .	54
8.8.5	Interior bearing walls at second floor . . . . .	55
8.8.6	Interior bearing walls at third floor . . . . .	56
8.9	Lateral. Diaphragms/Shear walls . . . . .	57
8.9.1	East and West facades shear walls . . . . .	57
8.9.2	Courtyard facades shear walls . . . . .	59
8.9.3	South facades shear walls . . . . .	60
8.9.4	North facade shear walls . . . . .	61
<b>9</b>	<b>Basement</b>	<b>66</b>
9.1	Structural model . . . . .	66
9.2	Footings . . . . .	70
9.2.1	Loads . . . . .	70
9.2.2	Load combinations . . . . .	70
9.2.3	Footing dimensions and reinforcement . . . . .	70
9.2.4	Limit state checking . . . . .	70
9.3	Basement walls . . . . .	70
9.3.1	Introduction . . . . .	70
9.3.2	Load determination . . . . .	73
9.3.3	Load combinations . . . . .	74
9.3.4	Stem dimensions and reinforcement . . . . .	75
9.4	Ramp . . . . .	81
9.4.1	Design criteria . . . . .	81
9.4.2	Acceptance criteria . . . . .	81

<b>Appendices</b>	<b>85</b>
A Loading criteria . . . . .	85
A.1 Dead loads . . . . .	85
A.2 Live loads . . . . .	86
A.3 Snow loads . . . . .	86
A.4 Wind loads . . . . .	87
A.5 Earthquake loads . . . . .	88
B Calculation results. Internal forces on columns . . . . .	89
B.1 Ultimate limit states . . . . .	89
B.2 Serviceability limit states . . . . .	115

## List of Tables

1 Gravity Loads . . . . .	10
2 Wind Design Criteria . . . . .	10
3 Snow Design Criteria . . . . .	10
4 Seismic Design Criteria . . . . .	11
5 Concrete properties . . . . .	11
6 Reinforcement properties . . . . .	11
7 Combinations Ultimate Limit States . . . . .	12
8 Combinations Serviceability Limit States . . . . .	13
9 Time assigned to wallboard membranes . . . . .	25
10 Steel beam at courtyard facade. Trusses reactions . . . . .	37
11 Steel beam at corridor. Trusses reactions . . . . .	39
12 Cantilever schedule . . . . .	44
17 Column footing schedule . . . . .	70
18 Soil bearing pressures. Capacity factors . . . . .	71
19 Shear design. Capacity factors . . . . .	71
20 Two-way shear design. Capacity factors . . . . .	72
21 Concrete walls reinforcing schedule . . . . .	75
22 Wall materials and dimensions T1 . . . . .	75
23 Wall materials and dimensions T2 . . . . .	76
24 Wall materials and dimensions T3 . . . . .	76
25 Wall materials and dimensions T4 . . . . .	77
26 Wall materials and dimensions T5 . . . . .	77
27 Wall materials and dimensions T6 . . . . .	78

## List of Figures

1 Trusses key plan . . . . .	15
2 Roof trusses at zones A and B (see key plan in figure 1) . . . . .	16
3 Third floor trusses at zones A and B (see key plan in figure 1) . . . . .	16
4 Second floor trusses at zones A and B (see key plan in figure 1) . . . . .	17
5 Roof trusses at zones C and D (see key plan in figure 1) . . . . .	18
6 Third floor trusses at zones C and D (see key plan in figure 1) . . . . .	19
7 Second floor trusses at zones C and D (see key plan in figure 1) . . . . .	20
8 Roof truss at zone E (see key plan in figure 1) . . . . .	21
9 Third floor truss at zone E (see key plan in figure 1) . . . . .	22
10 Second floor truss at zone E (see key plan in figure 1) . . . . .	23
11 Headers key plan. Roof . . . . .	29

12	Headers key plan. Third floor . . . . .	30
13	Headers key plan. Second floor . . . . .	31
14	Second floor beams key plan. . . . .	38
15	Second floor cantilevers key plan. . . . .	41
16	Columns key plan. Roof . . . . .	45
17	Columns key plan. Third floor . . . . .	46
18	Columns key plan. Second floor . . . . .	47
19	Bearing walls key plan. Roof . . . . .	52
20	Shear walls on the third floor. . . . .	63
21	Shear walls on the second floor. . . . .	64
22	Shear walls on the first floor. . . . .	65
23	Elastic model, mesh. . . . .	67
24	Load case D: dead load (include slab selfweight) [units: kN,m]. . . . .	67
25	Load case Lru: live load (uniform on rooms) [units: kN,m]. . . . .	67
26	Load case Lrs: live load (staggered pattern on rooms) [units: kN,m]. . . . .	67
27	Load case Lpu: live load (uniform on patios) [units: kN,m]. . . . .	67
28	Load case S: snow [units: kN,m]. . . . .	67
29	Load case Lps: live load (staggered pattern on patios) [units: kN,m]. . . . .	68
30	Load case W_WE: wind West-East [units: kN,m]. . . . .	68
31	Load case W_NS: wind North-South [units: kN,m]. . . . .	68
32	Load layout on first floor. . . . .	69
33	Flexure in the longitudinal direction. Capacity factor. . . . .	72
34	Flexure in the transverse direction. Capacity factor. . . . .	73
35	Ramp. Reinforcing layout. . . . .	81
36	Ramp elastic model, mesh. . . . .	82
37	Load case D: dead load (include slab selfweight) [units: $kN/m^2$ ]. . . . .	82
38	Load case Lv <sub>unif</sub> : uniform live load (vehicles) [units: $kN/m^2$ ]. . . . .	82
39	Load case Lv <sub>conc,s1</sub> : concentrated live load (vehicles) on mid-span 1 [units: kN]. . . . .	82
40	Load case Lv <sub>conc,s2</sub> : concentrated live load (vehicles) on mid-span 2 [units: kN]. . . . .	82
41	Load case Lv <sub>conc,s3</sub> : concentrated live load (vehicles) on mid-span 3 [units: kN]. . . . .	82
42	ULS normal stresses check. Efficiency in longitudinal direction . . . . .	83
43	ULS normal stresses check. Efficiency in transversal direction . . . . .	83
44	ULS normal stresses check. Bending moment in longitudinal direction [units: kN.m]. . . . .	83
45	ULS shear check. Efficiency in longitudinal direction . . . . .	83
46	ULS shear check. Efficiency in transversal direction . . . . .	83
47	ULS shear check. Internal shear force in longitudinal direction [units: kN]. . . . .	83
48	ULS shear check. Internal shear force in transversal direction [units: kN]. . . . .	84
49	ULS01: 1.4*D. Columns, internal axial force [kN] . . . . .	89
50	ULS01: 1.4*D. Columns, bending moment about local axis y [m.kN] . . . . .	89
51	ULS01: 1.4*D. Columns, bending moment about local axis z [m.kN] . . . . .	90
52	ULS01: 1.4*D. Columns, internal shear force in local direction y [kN] . . . . .	90
53	ULS01: 1.4*D. Columns, internal shear force in local direction z [kN] . . . . .	90
54	ULS02_a: 1.2*D + 1.6*Lru + Lpu + 0.5*S. Columns, internal axial force [kN] . . . . .	91
55	ULS02_a: 1.2*D + 1.6*Lru + Lpu + 0.5*S. Columns, bending moment about local axis y [m.kN] . . . . .	91
56	ULS02_a: 1.2*D + 1.6*Lru + Lpu + 0.5*S. Columns, bending moment about local axis z [m.kN] . . . . .	91
57	ULS02_a: 1.2*D + 1.6*Lru + Lpu + 0.5*S. Columns, internal shear force in local direction y [kN] . . . . .	92
58	ULS02_a: 1.2*D + 1.6*Lru + Lpu + 0.5*S. Columns, internal shear force in local direction z [kN] . . . . .	92
59	ULS02_b: 1.2*D + 1.6*Lrs + Lps + 0.5*S. Columns, internal axial force [kN] . . . . .	92

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LIST OF FIGURES

60	ULS02_b: $1.2*D + 1.6*Lrs + Lps + 0.5*S$ . Columns, bending moment about local axis y [m.kN] . . . . .	93
61	ULS02_b: $1.2*D + 1.6*Lrs + Lps + 0.5*S$ . Columns, bending moment about local axis z [m.kN] . . . . .	93
62	ULS02_b: $1.2*D + 1.6*Lrs + Lps + 0.5*S$ . Columns, internal shear force in local direction y [kN] . . . . .	93
63	ULS02_b: $1.2*D + 1.6*Lrs + Lps + 0.5*S$ . Columns, internal shear force in local direction z [kN] . . . . .	94
64	ULS03_a: $1.2*D + 1.6*S + 0.5*Lru + Lpu$ . Columns, internal axial force [kN] . .	94
65	ULS03_a: $1.2*D + 1.6*S + 0.5*Lru + Lpu$ . Columns, bending moment about local axis y [m.kN] . . . . .	94
66	ULS03_a: $1.2*D + 1.6*S + 0.5*Lru + Lpu$ . Columns, bending moment about local axis z [m.kN] . . . . .	95
67	ULS03_a: $1.2*D + 1.6*S + 0.5*Lru + Lpu$ . Columns, internal shear force in local direction y [kN] . . . . .	95
68	ULS03_a: $1.2*D + 1.6*S + 0.5*Lru + Lpu$ . Columns, internal shear force in local direction z [kN] . . . . .	95
69	ULS03_b: $1.2*D + 1.6*S + 0.5*Lrs + Lps$ . Columns, internal axial force [kN] . .	96
70	ULS03_b: $1.2*D + 1.6*S + 0.5*Lrs + Lps$ . Columns, bending moment about local axis y [m.kN] . . . . .	96
71	ULS03_b: $1.2*D + 1.6*S + 0.5*Lrs + Lps$ . Columns, bending moment about local axis z [m.kN] . . . . .	96
72	ULS03_b: $1.2*D + 1.6*S + 0.5*Lrs + Lps$ . Columns, internal shear force in local direction y [kN] . . . . .	97
73	ULS03_b: $1.2*D + 1.6*S + 0.5*Lrs + Lps$ . Columns, internal shear force in local direction z [kN] . . . . .	97
74	ULS04_a: $1.2*D + 1.6*S + 0.5*W\_WE$ . Columns, internal axial force [kN] . . .	97
75	ULS04_a: $1.2*D + 1.6*S + 0.5*W\_WE$ . Columns, bending moment about local axis y [m.kN] . . . . .	98
76	ULS04_a: $1.2*D + 1.6*S + 0.5*W\_WE$ . Columns, bending moment about local axis z [m.kN] . . . . .	98
77	ULS04_a: $1.2*D + 1.6*S + 0.5*W\_WE$ . Columns, internal shear force in local direction y [kN] . . . . .	98
78	ULS04_a: $1.2*D + 1.6*S + 0.5*W\_WE$ . Columns, internal shear force in local direction z [kN] . . . . .	99
79	ULS04_b: $1.2*D + 1.6*S + 0.5*W\_NS$ . Columns, internal axial force [kN] . . .	99
80	ULS04_b: $1.2*D + 1.6*S + 0.5*W\_NS$ . Columns, bending moment about local axis y [m.kN] . . . . .	99
81	ULS04_b: $1.2*D + 1.6*S + 0.5*W\_NS$ . Columns, bending moment about local axis z [m.kN] . . . . .	100
82	ULS04_b: $1.2*D + 1.6*S + 0.5*W\_NS$ . Columns, internal shear force in local direction y [kN] . . . . .	100
83	ULS04_b: $1.2*D + 1.6*S + 0.5*W\_NS$ . Columns, internal shear force in local direction z [kN] . . . . .	100
84	ULS05_a: $1.2*D + W\_WE + 0.5*Lru + Lpu$ . Columns, internal axial force [kN]	101
85	ULS05_a: $1.2*D + W\_WE + 0.5*Lru + Lpu$ . Columns, bending moment about local axis y [m.kN] . . . . .	101
86	ULS05_a: $1.2*D + W\_WE + 0.5*Lru + Lpu$ . Columns, bending moment about local axis z [m.kN] . . . . .	101
87	ULS05_a: $1.2*D + W\_WE + 0.5*Lru + Lpu$ . Columns, internal shear force in local direction y [kN] . . . . .	102

88	ULS05_a: $1.2*D + W\_WE + 0.5*Lru + Lpu$ . Columns, internal shear force in local direction z [kN] . . . . .	102
89	ULS05_b: $1.2*D + W\_NS + 0.5*Lru + Lpu$ . Columns, internal axial force [kN] . . . . .	103
90	ULS05_b: $1.2*D + W\_NS + 0.5*Lru + Lpu$ . Columns, bending moment about local axis y [m.kN] . . . . .	103
91	ULS05_b: $1.2*D + W\_NS + 0.5*Lru + Lpu$ . Columns, bending moment about local axis z [m.kN] . . . . .	103
92	ULS05_b: $1.2*D + W\_NS + 0.5*Lru + Lpu$ . Columns, internal shear force in local direction y [kN] . . . . .	104
93	ULS05_b: $1.2*D + W\_NS + 0.5*Lru + Lpu$ . Columns, internal shear force in local direction z [kN] . . . . .	104
94	ULS05_c: $1.2*D + W\_WE + 0.5*Lrs + Lps$ . Columns, internal axial force [kN] . . . . .	104
95	ULS05_c: $1.2*D + W\_WE + 0.5*Lrs + Lps$ . Columns, bending moment about local axis y [m.kN] . . . . .	105
96	ULS05_c: $1.2*D + W\_WE + 0.5*Lrs + Lps$ . Columns, bending moment about local axis z [m.kN] . . . . .	105
97	ULS05_c: $1.2*D + W\_WE + 0.5*Lrs + Lps$ . Columns, internal shear force in local direction y [kN] . . . . .	105
98	ULS05_c: $1.2*D + W\_WE + 0.5*Lrs + Lps$ . Columns, internal shear force in local direction z [kN] . . . . .	106
99	ULS05_d: $1.2*D + W\_NS + 0.5*Lrs + Lps$ . Columns, internal axial force [kN] . . . . .	106
100	ULS05_d: $1.2*D + W\_NS + 0.5*Lrs + Lps$ . Columns, bending moment about local axis y [m.kN] . . . . .	106
101	ULS05_d: $1.2*D + W\_NS + 0.5*Lrs + Lps$ . Columns, bending moment about local axis z [m.kN] . . . . .	107
102	ULS05_d: $1.2*D + W\_NS + 0.5*Lrs + Lps$ . Columns, internal shear force in local direction y [kN] . . . . .	107
103	ULS05_d: $1.2*D + W\_NS + 0.5*Lrs + Lps$ . Columns, internal shear force in local direction z [kN] . . . . .	107
104	ULS06_a: $1.2*D + 0.5*Lru + Lpu + 0.2*S$ . Columns, internal axial force [kN] . . . . .	108
105	ULS06_a: $1.2*D + 0.5*Lru + Lpu + 0.2*S$ . Columns, bending moment about local axis y [m.kN] . . . . .	108
106	ULS06_a: $1.2*D + 0.5*Lru + Lpu + 0.2*S$ . Columns, bending moment about local axis z [m.kN] . . . . .	108
107	ULS06_a: $1.2*D + 0.5*Lru + Lpu + 0.2*S$ . Columns, internal shear force in local direction y [kN] . . . . .	109
108	ULS06_a: $1.2*D + 0.5*Lru + Lpu + 0.2*S$ . Columns, internal shear force in local direction z [kN] . . . . .	109
109	ULS06_b: $1.2*D + 0.5*Lrs + Lps + 0.2*S$ . Columns, internal axial force [kN] . . . . .	109
110	ULS06_b: $1.2*D + 0.5*Lrs + Lps + 0.2*S$ . Columns, bending moment about local axis y [m.kN] . . . . .	110
111	ULS06_b: $1.2*D + 0.5*Lrs + Lps + 0.2*S$ . Columns, bending moment about local axis z [m.kN] . . . . .	110
112	ULS06_b: $1.2*D + 0.5*Lrs + Lps + 0.2*S$ . Columns, internal shear force in local direction y [kN] . . . . .	110
113	ULS06_b: $1.2*D + 0.5*Lrs + Lps + 0.2*S$ . Columns, internal shear force in local direction z [kN] . . . . .	111
114	ULS07_a: $0.9*D + W\_WE$ . Columns, internal axial force [kN] . . . . .	111
115	ULS07_a: $0.9*D + W\_WE$ . Columns, bending moment about local axis y [m.kN] . . . . .	111
116	ULS07_a: $0.9*D + W\_WE$ . Columns, bending moment about local axis z [m.kN] . . . . .	112
117	ULS07_a: $0.9*D + W\_WE$ . Columns, internal shear force in local direction y [kN] . . . . .	112
118	ULS07_a: $0.9*D + W\_WE$ . Columns, internal shear force in local direction z [kN] . . . . .	112

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## LIST OF FIGURES

119	ULS07_b: 0.9*D + W_NS. Columns, internal axial force [kN] . . . . .	113
120	ULS07_b: 0.9*D + W_NS. Columns, bending moment about local axis y [m.kN]	113
121	ULS07_b: 0.9*D + W_NS. Columns, bending moment about local axis z [m.kN] .	113
122	ULS07_b: 0.9*D + W_NS. Columns, internal shear force in local direction y [kN]	114
123	ULS07_b: 0.9*D + W_NS. Columns, internal shear force in local direction z [kN]	114
124	SLS01: 1.0*D. Columns, bending moment about local axis y [m.kN] . . . . .	115
125	SLS02_a: 1.0*D + 1.0*Lru + Lpu + 0.3*S. Columns, bending moment about local axis y [m.kN] . . . . .	115
126	SLS02_b: 1.0*D + 1.0*Lrs + Lps + 0.3*S. Columns, bending moment about local axis y [m.kN] . . . . .	116
127	SLS03_a: 1.0*D + 1.0*S + 0.3*Lru + 0.3*Lpu. Columns, bending moment about local axis y [m.kN] . . . . .	116
128	SLS03_b: 1.0*D + 1.0*S + 0.3*Lrs + 0.3*Lps. Columns, bending moment about local axis y [m.kN] . . . . .	116
129	SLS04_a: 1.0*D + W_WE + 1.0*Lru + Lpu. Columns, bending moment about local axis y [m.kN] . . . . .	117
130	SLS04_b: 1.0*D + W_NS + 1.0*Lru + Lpu. Columns, bending moment about local axis y [m.kN] . . . . .	117
131	SLS05_a: 1.0*D + W_WE. Columns, bending moment about local axis y [m.kN]	117
132	SLS05_b: 1.0*D + W_NS. Columns, bending moment about local axis y [m.kN] .	118

CALCULATION REPORT

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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°49'01.8"N 91°30'34.8"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](http://xcengineering.xyz/html_files/software.html)).

## 7 Load combinations

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

Table 7: Combinations Ultimate Limit States

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

Where:

D = dead load

Lru = live load (uniform on rooms)

Lrs = live load (staggered pattern on rooms)

Lpu = live load (uniform on patios)

Lps = live load (staggered pattern on patios)

S = snow load

W\_WE = Wind West-East

W\_NS = Wind North-South

Table 8: Combinations Serviceability Limit States

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

Where:

D = dead load

Lru = live load (uniform on rooms)

Lrs = live load (staggered pattern on rooms)

Lpu = live load (uniform on patios)

Lps = live load (staggered pattern on patios)

S = snow load

W\_WE = Wind West-East

W\_NS = Wind North-South

## 8 Wood framing

### 8.1 Gravity

#### 8.2 Trusses

##### 8.2.1 Trusses A and B. Roof

The deflection results for those trusses (see figure 2) are as follows <sup>1</sup>:

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

The truss depth is allways greater than 24 inches due to the geometry of the roof. The spacing of the trusses is 12 inches.

$$\Delta_{LL,A} = 3.69 \text{ mm} = \frac{L}{3044} < \frac{L}{540} \implies OK \quad (1)$$

$$\Delta_{LL,B} = 2.69 \text{ mm} = \frac{L}{3995} < \frac{L}{540} \implies OK \quad (2)$$

$$\Delta_{TL,A} = 10.49 \text{ mm} = \frac{L}{1069} < \frac{L}{360} \implies OK \quad (3)$$

$$\Delta_{TL,B} = 7.38 \text{ mm} = \frac{L}{1459} < \frac{L}{360} \implies OK \quad (4)$$

##### 8.2.2 Trusses A and B. Third floor

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

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

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

$$\Delta_{LL,A} = 17.32 \text{ mm} = \frac{L}{648} < \frac{L}{540} \implies OK \quad (5)$$

$$\Delta_{LL,B} = 14.71 \text{ mm} = \frac{L}{731} < \frac{L}{540} \implies OK \quad (6)$$

$$\Delta_{TL,A} = 26.45 \text{ mm} = \frac{L}{424} < \frac{L}{360} \implies OK \quad (7)$$

$$\Delta_{TL,B} = 22.46 \text{ mm} = \frac{L}{479} < \frac{L}{360} \implies OK \quad (8)$$

<sup>1</sup>The load combinations are listed in 9.3.3.



Figure 1: Trusses key plan.



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

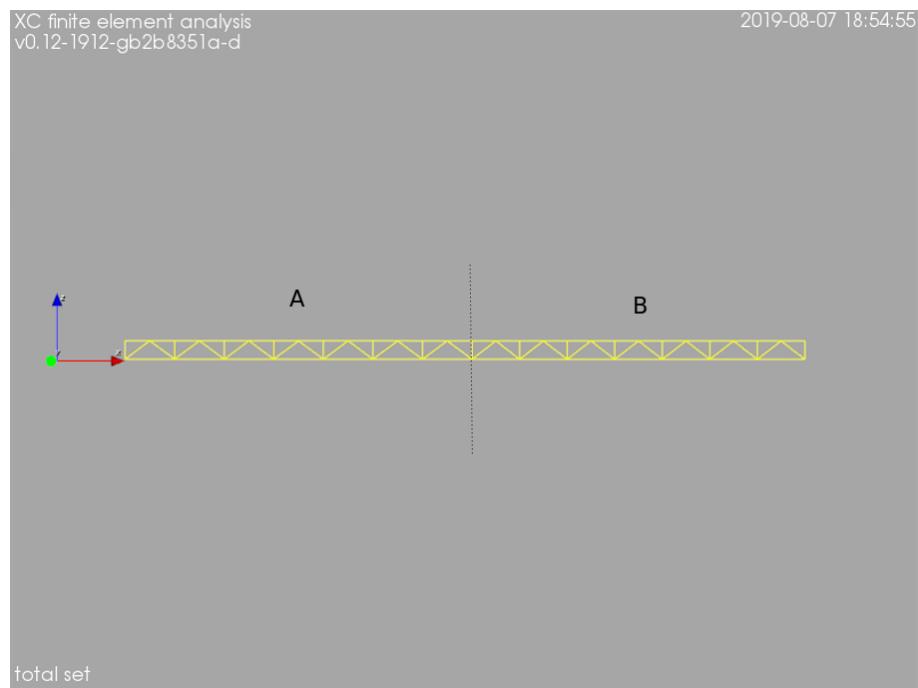


Figure 3: Third floor trusses at zones A and B (see key plan in figure 1).

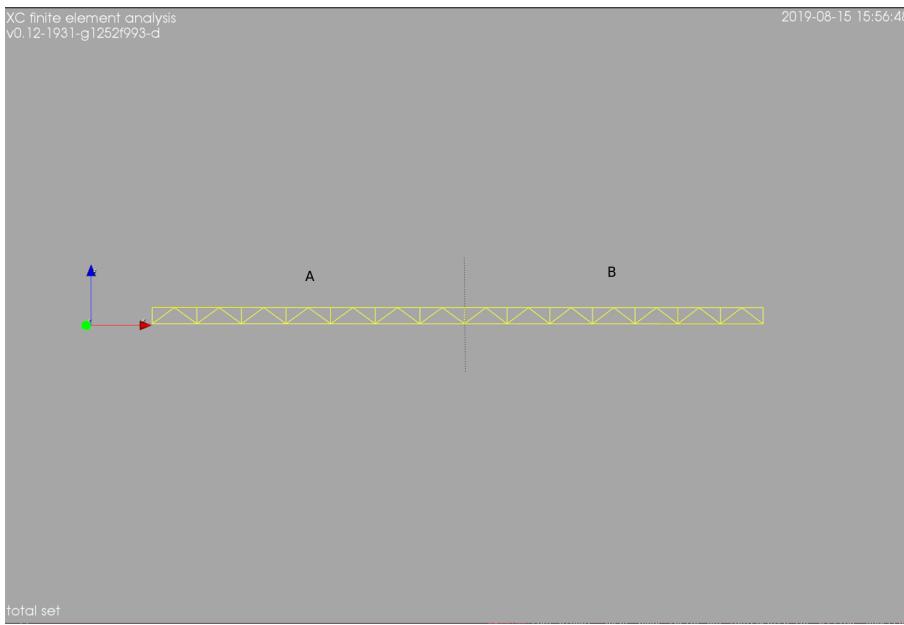


Figure 4: Second floor trusses at zones A and B (see key plan in figure 1).

### 8.2.3 Trusses A and B. Second floor

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

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

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

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

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

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

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

### 8.2.4 Trusses C and D. Roof

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

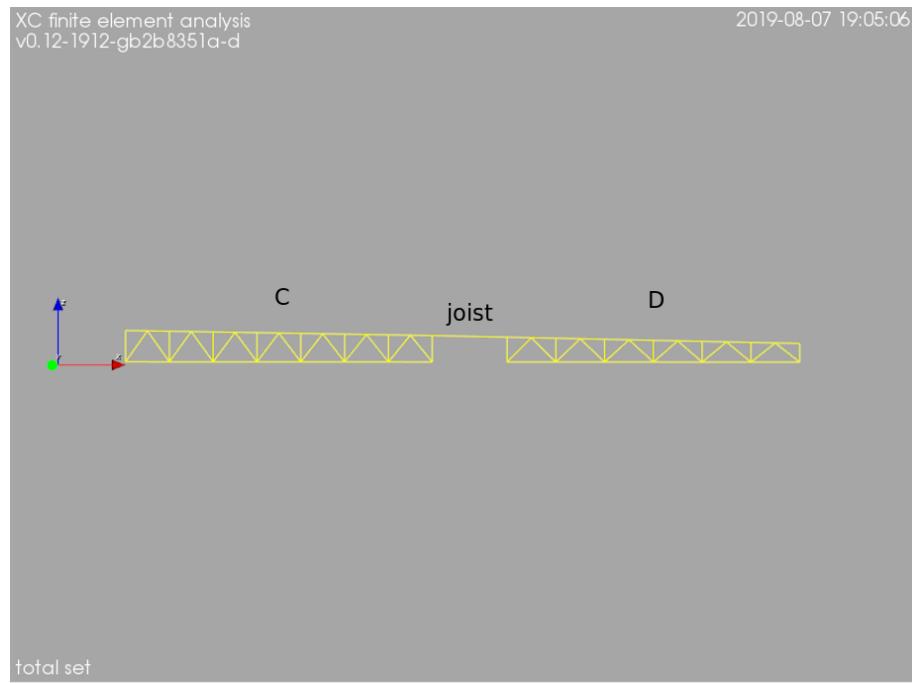


Figure 5: Roof trusses at zones C and D (see key plan in figure 1).

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

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

$$\Delta_{LL,C} = 4.50 \text{ mm} = \frac{L}{2224} < \frac{L}{540} \implies OK \quad (13)$$

$$\Delta_{LL,D} = 6.64 \text{ mm} = \frac{L}{772} < \frac{L}{540} \implies OK \quad (14)$$

$$\Delta_{TL,C} = 13.36 \text{ mm} = \frac{L}{748} < \frac{L}{360} \implies OK \quad (15)$$

$$\Delta_{TL,D} = 20.64 \text{ mm} = \frac{L}{462} < \frac{L}{360} \implies OK \quad (16)$$

### 8.2.5 Trusses C and D. Third floor

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

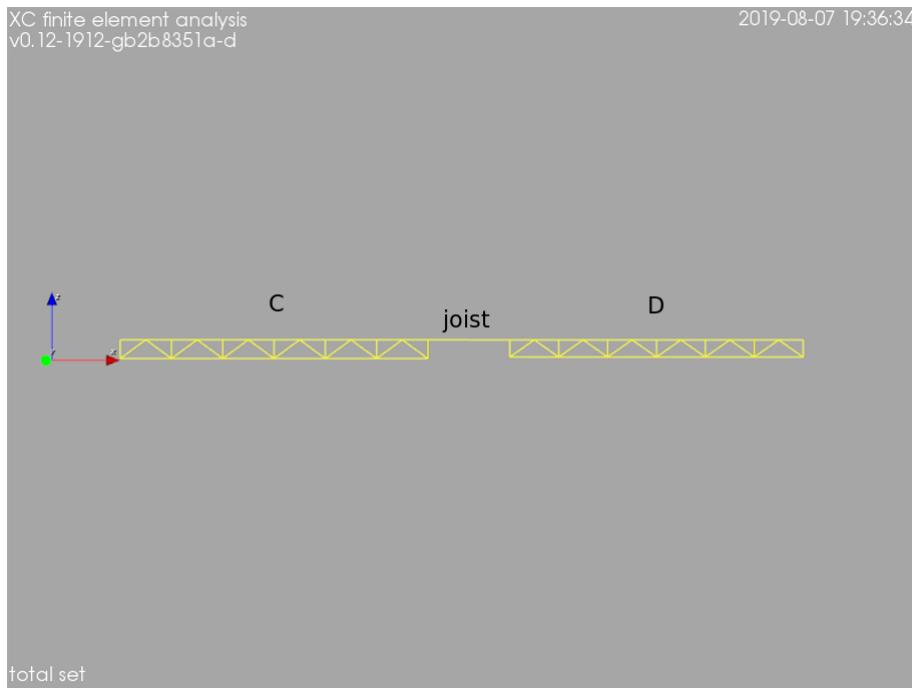


Figure 6: Third floor trusses at zones C and D (see key plan in figure 1).

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

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

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

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

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

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

### 8.2.6 Trusses C and D. Second floor

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

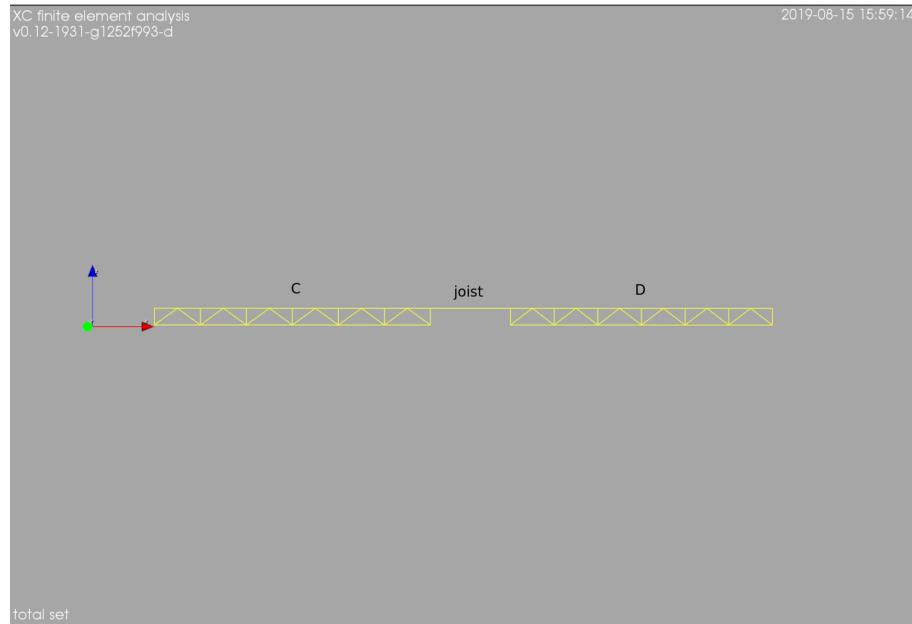


Figure 7: Second floor trusses at zones C and D (see key plan in figure 1).

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

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

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

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

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

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

### 8.2.7 Truss E. Roof

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

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

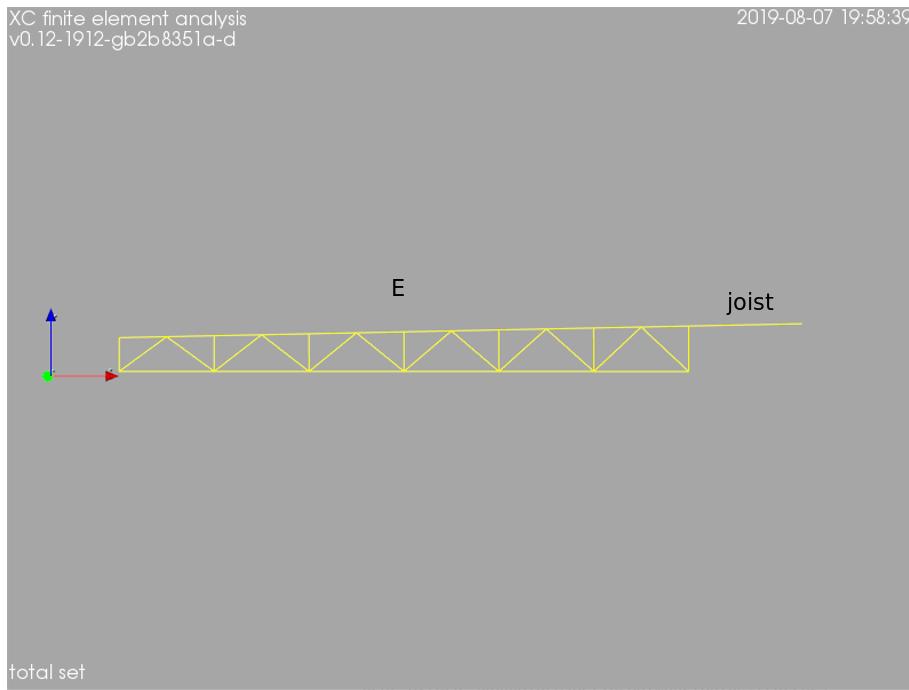


Figure 8: Roof truss at zone E (see key plan in figure 1).

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

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

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

(27)

### 8.2.8 Truss E. Third floor

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

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

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

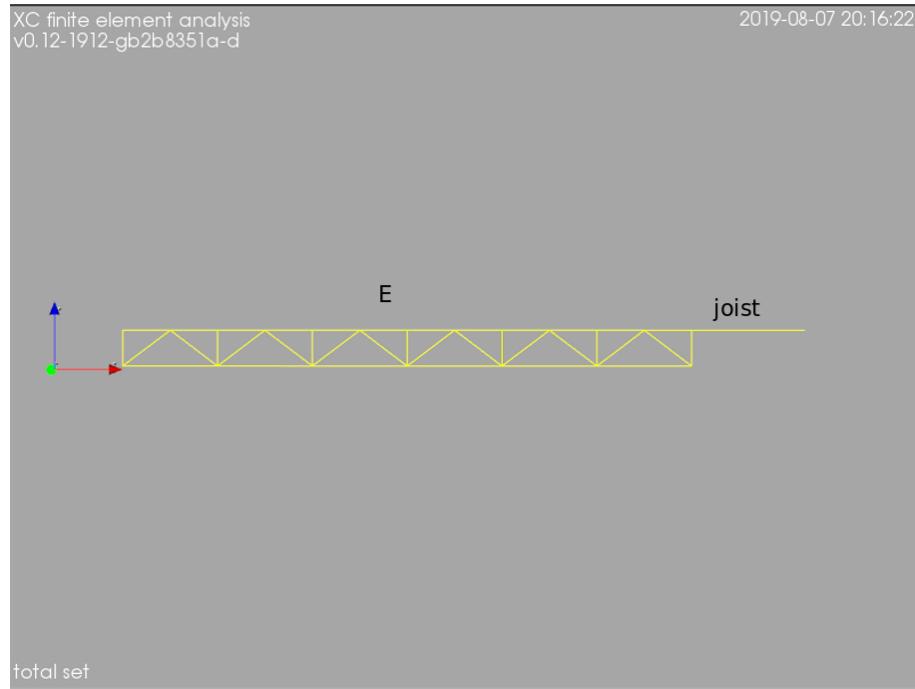


Figure 9: Third floor truss at zone E (see key plan in figure 1).

$$\Delta_{LL,E} = 14.65 \text{ mm} = \frac{L}{646} < \frac{L}{540} \implies OK \quad (28)$$

$$\Delta_{TL,E} = 23.30 \text{ mm} = \frac{L}{406} < \frac{L}{360} \implies OK \quad (29)$$

(30)

### 8.2.9 Truss E. Second floor

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

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

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

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

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

(33)

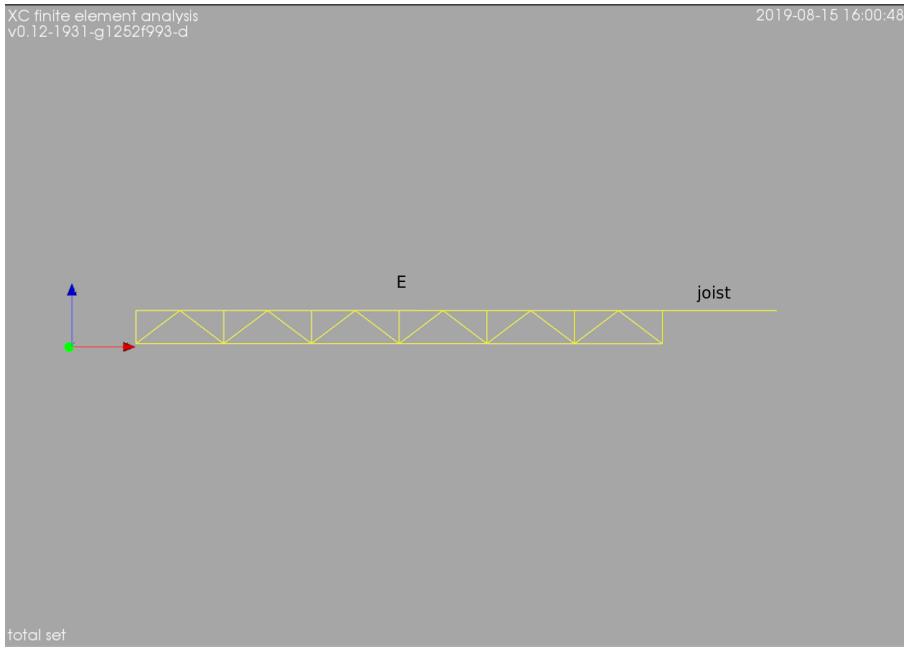


Figure 10: Second floor truss at zone E (see key plan in figure 1).

### 8.3 Joists

#### 8.3.1 Corridor floor sheathing

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

$$q_{live} = 1.92 \text{ kN/m}^2 (40 \text{ psf}) \quad (34)$$

$$q_{dead} = 0.72 \text{ kN/m}^2 (15 \text{ psf}) \quad (35)$$

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

#### Structural design of the panels.

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

**Bending stiffness.** The deflection obtained under live load is:

$$\Delta_{LL} = 1.34 \text{ mm} = \frac{\text{span}}{607} < \frac{\text{span}}{540} \implies OK \quad (36)$$

and the deflection under total load is:

$$\Delta_{TL} = 1.84 \text{ mm} = \frac{\text{span}}{441} < \frac{\text{span}}{360} \implies OK \quad (37)$$

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

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

$$\sigma_{max} = 1.69 \text{ MPa} < 6.94 \text{ MPa} = F'_b \implies OK \quad (38)$$

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

The maximum shear stress obtained under total load is:

$$\tau_{max} = 0.04 \text{ MPa} < 0.33 \text{ MPa} = F'_v \implies OK \quad (39)$$

**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:

$$\sigma_{max} = 4.59 \text{ MPa} < 6.94 \text{ MPa} = F'_b \implies OK \quad (40)$$

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

$$\tau_{max} = 0.12 \text{ MPa} < 0.33 \text{ MPa} = F'_v \implies OK \quad (41)$$

### 8.3.2 Corridor Joists

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

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

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

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

**Structural design of the joist.**

#### Loads.

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

**722.6.2 Walls, Floors and Roofs**

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

**TABLE 722.6.2(1) TIME ASSIGNED TO WALLBOARD MEMBRANES<sup>a, b, c, d</sup>**

DESCRIPTION OF FINISH	TIME <sup>e</sup> (minutes)
3/8-Inch wood structural panel bonded with exterior glue	5
15/32-Inch wood structural panel bonded with exterior glue	10
19/32-Inch wood structural panel bonded with exterior glue	15
3/8-Inch gypsum wallboard	10
1/2-Inch gypsum wallboard	15
5/8-Inch gypsum wallboard	30
1/2-Inch Type X gypsum wallboard	25
5/8-Inch Type X gypsum wallboard	40
Double 3/8-inch gypsum wallboard	25
1/2-Inch + 3/8-Inch gypsum wallboard	35
Double 1/2-Inch gypsum wallboard	40

For SI: 1 inch = 25.4 mm.

Table 9: Time assigned to wallboard membranes

**Internal forces.** Maximum induced moment:

$$M_{max} = w_{load} \frac{L^2}{8} = 1.54 \text{ kNm} \quad (45)$$

Maximum induced shear:

$$V_{max} = w_{load} \frac{L}{2} = 2.57 \text{ kN} \quad (46)$$

**Joist mechanical properties.** Joist section modulus:

$$S_s = 344.13 \times 10^{-6} \text{ m}^3 \quad (47)$$

Tabulated bending stress:

$$F_b = 21.59 \text{ MPa} \quad (48)$$

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

$$F'_b = 13.59 \text{ MPa} \quad (49)$$

Tabulated shear stress:

$$F_v = 1.97 \text{ MPa} \quad (50)$$

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

$$F'_v = 1.97 \text{ MPa} \quad (51)$$

**Structural bending check.** Design resisting moment:

$$M'_s = 4.67 \text{ kNm} \quad (52)$$

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

**Structural shear check.** Design resisting shear:

$$V'_s = 17.74 \text{ kN} \quad (53)$$

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

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

**Mechanical properties of the burned section.** Effective char depth:

$$a_{eff} = 0.7 \times 10^{-3} \times 30 + 7 \times 10^{-3} = 28 \text{ mm} \quad (54)$$

section modulus for a joist exposed on three sides:

$$S_s = 84.86 \times 10^{-6} \text{ m}^3 \quad (55)$$

shear area for a beam exposed on three sides:

$$A_f = 40.93 \text{ cm}^2 \quad (56)$$

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

$$F'_{b,f} = 38.74 \text{ MPa} \quad (57)$$

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

$$F'_{v,f} = 5.40 \text{ MPa} \quad (58)$$

**Structural bending check.** Design resisting moment:

$$M'_f = 3.28 \text{ kNm} \quad (59)$$

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

**Structural shear check.** Design resisting shear:

$$V'_f = 14.74 \text{ kN} \quad (60)$$

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

### 8.3.3 Joists under storage/HVAC floor

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

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

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

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.**

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

**Internal forces.** Maximum induced moment:

$$M_{max} = w_{load} \frac{L^2}{8} = 5.79 \text{ kNm} \quad (64)$$

Maximum induced shear:

$$V_{max} = w_{load} \frac{L}{2} = 7.94 \text{ kN} \quad (65)$$

**Joist mechanical properties.** Joist section modulus:

$$S_s = 817.90 \times 10^{-6} \text{ m}^3 \quad (66)$$

Tabulated bending stress:

$$F_b = 20.58 \text{ MPa} \quad (67)$$

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

$$F'_b = 20.58 \text{ MPa} \quad (68)$$

Tabulated shear stress:

$$F_v = 1.97 \text{ MPa} \quad (69)$$

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

$$F'_v = 1.97 \text{ MPa} \quad (70)$$

**Structural bending check.** Design resisting moment:

$$M'_s = 16.83 \text{ kNm} \quad (71)$$

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

**Structural shear check.** Design resisting shear:

$$V'_s = 27.36 \text{ kN} \quad (72)$$

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

**Bending stiffness.** The deflection obtained under live load is:

$$\Delta_{LL} = 3.45 \text{ mm} = \frac{\text{span}}{845} < \frac{\text{span}}{540} \implies OK \quad (73)$$

and the deflection under total load is:

$$\Delta_{TL} = 3.86 \text{ mm} = \frac{\text{span}}{754} < \frac{\text{span}}{360} \implies OK \quad (74)$$

## 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.**

$$w_{load} = 1.75 \text{ kN/m} \quad (75)$$

**Internal forces.** Maximum induced moment:

$$M_{max} = w_{load} \frac{L^2}{8} = 1.65 \text{ kNm} \quad (76)$$

Maximum induced shear:

$$V_{max} = w_{load} \frac{L}{2} = 2.40 \text{ kN} \quad (77)$$

**Structural bending check.** Design resisting moment:

$$M'_s = 14.96 \text{ kNm} \quad (78)$$

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

**Structural shear check.** Design resisting shear:

$$V'_s = 29.79 \text{ kN} \quad (79)$$

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

## 8. WOOD FRAMING

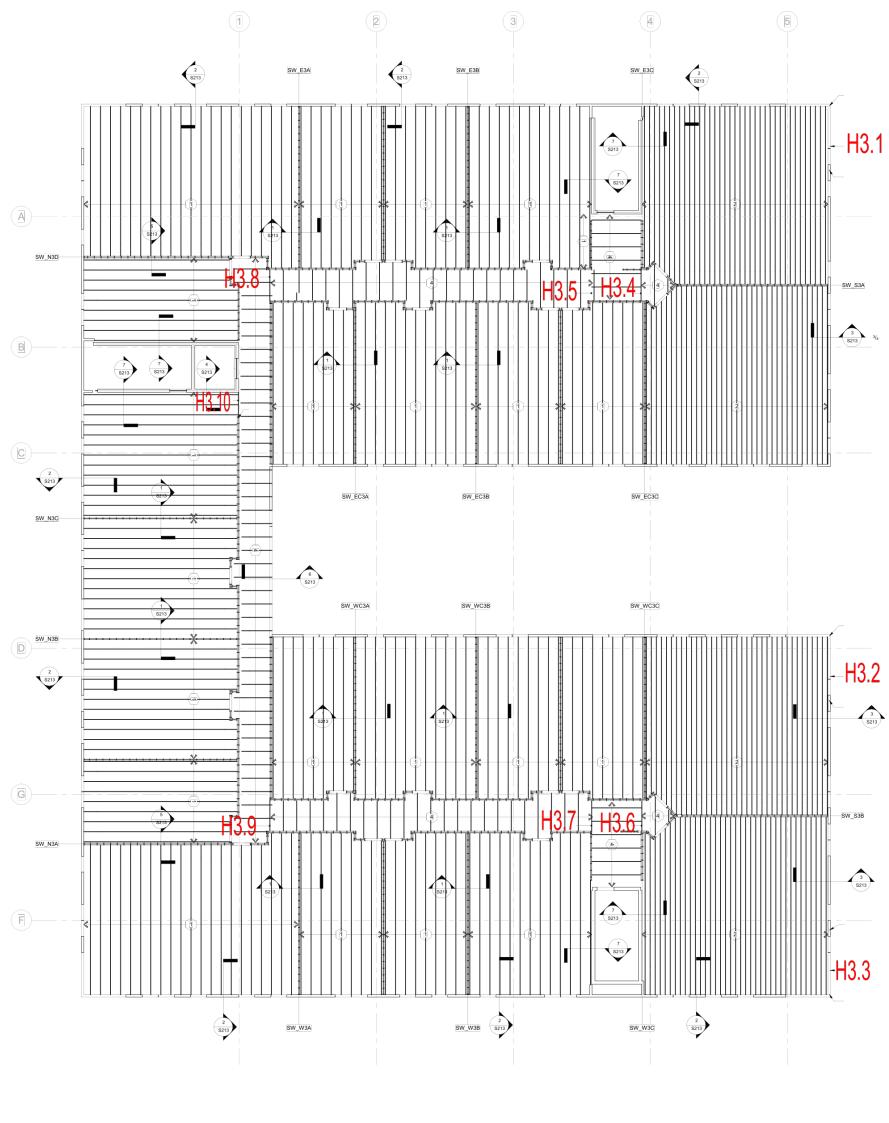


Figure 11: Headers key plan. Roof

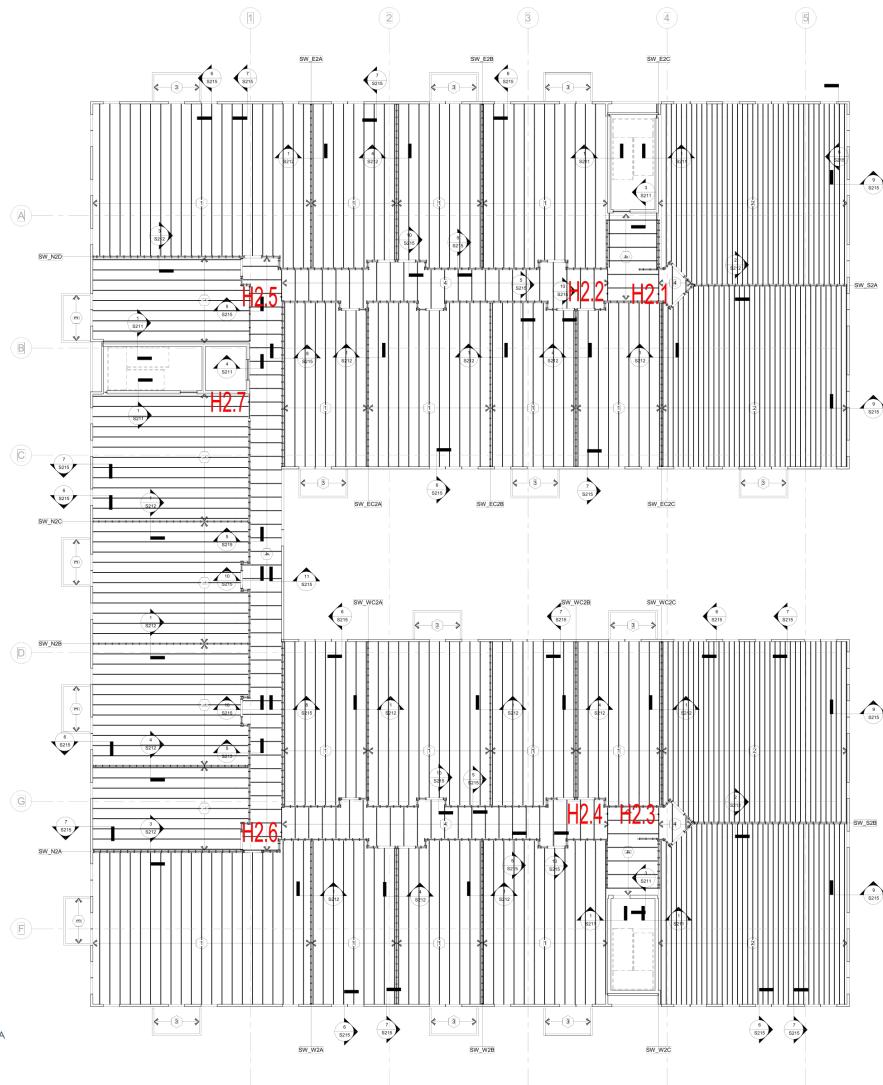


Figure 12: Headers key plan. Third floor

## 8. WOOD FRAMING

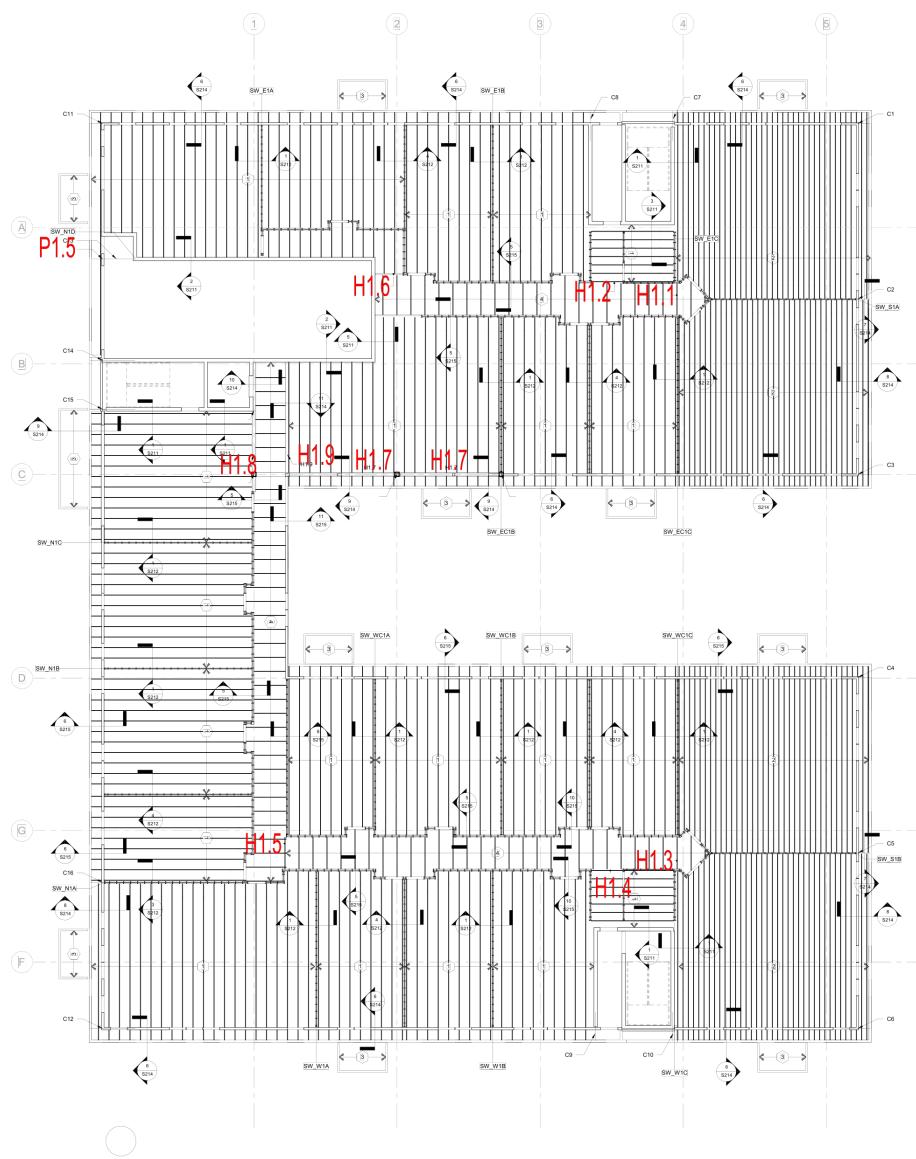


Figure 13: Headers key plan. Second floor

**Bending stiffness.** The deflection obtained is:

$$\Delta_{TL} = 0.72 \text{ mm} = \frac{L}{3781} < \frac{L}{360} \implies OK \quad (80)$$

#### 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.**

$$w_{load} = 5.26 \text{ kN/m} \quad (81)$$

**Internal forces.** Maximum induced moment:

$$M_{max} = w_{load} \frac{L^2}{8} = 2.29 \text{ kNm} \quad (82)$$

Maximum induced shear:

$$V_{max} = w_{load} \frac{L}{2} = 5.21 \text{ kN} \quad (83)$$

**Structural bending check.** Design resisting moment:

$$M'_s = 10.63 \text{ kNm} \quad (84)$$

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

**Structural shear check.** Design resisting shear:

$$V'_s = 21.44 \text{ kN} \quad (85)$$

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

**Bending stiffness.** The deflection obtained is:

$$\Delta_{TL} = 0.07 \text{ mm} = \frac{L}{27939} < \frac{L}{360} \implies OK \quad (86)$$

**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:

$$a_{eff} = 0.7 \times 10^{-3} \times 30 + 7 \times 10^{-3} = 28 \text{ mm} \quad (87)$$

section modulus for a joist exposed on three sides:

$$S_s = 133.70 \times 10^{-6} \text{ m}^3 \quad (88)$$

shear area for a beam exposed on three sides:

$$A_f = 51.37 \text{ cm}^2 \quad (89)$$

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

$$F'_{b,f} = 60.26 \text{ MPa} \quad (90)$$

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

$$F'_{v,f} = 5.40 \text{ MPa} \quad (91)$$

**Structural bending check.** Design resisting moment:

$$M'_f = 8.06 \text{ kNm} \quad (92)$$

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

**Structural shear check.** Design resisting shear:

$$V'_f = 18.51 \text{ kN} \quad (93)$$

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.**

$$w_{load} = 18.11 \text{ kN/m} \quad (94)$$

**Internal forces.** Maximum induced moment:

$$M_{max} = w_{load} \frac{L^2}{8} = 10.94 \text{ kNm} \quad (95)$$

Maximum induced shear:

$$V_{max} = w_{load} \frac{L}{2} = 27.61 \text{ kN} \quad (96)$$

**Structural bending check.** Design resisting moment:

$$M'_s = 36.65 \text{ kNm} \quad (97)$$

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

**Structural shear check.** Design resisting shear:

$$V'_s = 41.41 \text{ kN} \quad (98)$$

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

**Bending stiffness.** The deflection obtained is:

$$\Delta_{TL} = 0.33 \text{ mm} = \frac{L}{6027} < \frac{L}{360} \implies OK \quad (99)$$

**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:

$$a_{eff} = 0.7 \times 10^{-3} \times 30 + 7 \times 10^{-3} = 28 \text{ mm} \quad (100)$$

section modulus for a joist exposed on three sides:

$$S_s = 588.48 \times 10^{-6} \text{ m}^3 \quad (101)$$

shear area for a beam exposed on three sides:

$$A_f = 107.78 \text{ cm}^2 \quad (102)$$

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

$$F'_{b,f} = 55.74 \text{ MPa} \quad (103)$$

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

$$F'_{v,f} = 5.40 \text{ MPa} \quad (104)$$

**Structural bending check.** Design resisting moment:

$$M'_f = 32.8 \text{ kNm} \quad (105)$$

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

**Structural shear check.** Design resisting shear:

$$V'_f = 38.82 \text{ kN} \quad (106)$$

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.**

$$w_{load} = 4.77 \text{ kN/m} \quad (107)$$

**Internal forces.** Maximum induced moment:

$$M_{max} = w_{load} \frac{L^2}{8} = 25.63 \text{ kNm} \quad (108)$$

Maximum induced shear:

$$V_{max} = w_{load} \frac{L}{2} = 15.73 \text{ kN} \quad (109)$$

**Structural bending check.** Design resisting moment:

$$M'_s = 87.66 \text{ kNm} \quad (110)$$

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

**Structural shear check.** Design resisting shear:

$$V'_s = 79.87 \text{ kN} \quad (111)$$

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

**Bending stiffness.** The deflection obtained is:

$$\Delta_{TL} = 8.97 \text{ mm} = \frac{L}{735} < \frac{L}{600} \implies OK \quad (112)$$

#### 8.4.5 Facade headers

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

**Structural design of the header.**

**Design load.**

$$w_{load} = 75.91 \text{ kN/m} \quad (113)$$

**Internal forces.** Maximum induced moment:

$$M_{max} = w_{load} \frac{L^2}{8} = 8.26 \text{ kNm} \quad (114)$$

Maximum induced shear:

$$V_{max} = w_{load} \frac{L}{2} = 43.38 \text{ kN} \quad (115)$$

**Structural bending check.** Design resisting moment:

$$M'_s = 21.96 \text{ kNm} \quad (116)$$

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

**Structural shear check.** Design resisting shear:

$$V'_s = 50.53 \text{ kN} \quad (117)$$

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

**Bending stiffness.** The deflection obtained is:

$$\Delta_{TL} = 0.41 \text{ mm} = \frac{L}{2758} < \frac{L}{600} \implies OK \quad (118)$$

#### 8.4.6 Corridor headers

Simply supported 3.5x16 LSL 1.55E header.

**Structural design of the header.**

**Design load.**

$$w_{load} = 118.11 \text{ kN/m} \quad (119)$$

**Internal forces.** Maximum induced moment:

$$M_{max} = w_{load} \frac{L^2}{8} = 14.63 \text{ kNm} \quad (120)$$

Maximum induced shear:

$$V_{max} = w_{load} \frac{L}{2} = 72.0 \text{ kN} \quad (121)$$

**Structural bending check.** Design resisting moment:

$$M'_s = 48.00 \text{ kNm} \quad (122)$$

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

**Structural shear check.** Design resisting shear:

$$V'_s = 76.60 \text{ kN} \quad (123)$$

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

**Bending stiffness.** The deflection obtained is:

$$\Delta_{TL} = 0.26 \text{ mm} = \frac{L}{4581} < \frac{L}{600} \implies OK \quad (124)$$

## 8.5 Steel beams

### 8.5.1 Steel beam at courtyard facade

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

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

**Structural design of the beam.**

**Loads.**

$$w_{load} = 19.785 \text{ kN/m} \quad (125)$$

**Internal forces on each structural channel.** Maximum induced moment:

$$M_{max} = 170.61 \text{ kNm} \quad (126)$$

Maximum induced shear:

$$V_{max} = 102.71 \text{ kN} \quad (127)$$

CD_reactions									
comb	floor	truss	Rz	Rz	truss	Rz	Rz		
		name	(kN)	(kN)	name	(kN)	(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		
			<b>9.04</b>	<b>11.81</b>		<b>11.05</b>	<b>8.30</b>		
		<b>kN/m</b>	<b>14.84</b>	<b>19.37</b>		<b>18.13</b>	<b>13.62</b>		
EQ1608	3 <sup>rd</sup> floor	2C	3.78	4.86 2D		4.69	3.62		
EQ1609	3 <sup>rd</sup> floor	2C	9.64	12.14 2D		11.71	9.20		
EQ1610	3 <sup>rd</sup> floor	2C	3.78	4.86 2D		4.69	3.62		
EQ1611	3 <sup>rd</sup> floor	2C	8.17	10.32 2D		9.95	7.81		
EQ1612	3 <sup>rd</sup> floor	2C	3.78	4.86 2D		4.69	3.62		
EQ1613	3 <sup>rd</sup> floor	2C	8.17	10.32 2D		9.95	7.81		
EQ1615	3 <sup>rd</sup> floor	2C	2.27	2.92 2D		2.82	2.17		
			<b>9.64</b>	<b>12.14</b>		<b>11.71</b>	<b>9.20</b>		
		<b>kN/m</b>	<b>15.81</b>	<b>19.92</b>		<b>19.20</b>	<b>15.10</b>		
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		
			<b>17.22</b>	<b>22.13</b>		<b>21.00</b>	<b>16.11</b>		
		<b>kN/m</b>	<b>28.24</b>	<b>36.30</b>		<b>34.46</b>	<b>26.43</b>		
EQ1608	2 <sup>nd</sup> floor	1C	3.04	5.17 1D		5.29	3.16		
EQ1609	2 <sup>nd</sup> floor	1C	8.19	13.24 1D		13.56	8.52		
EQ1610	2 <sup>nd</sup> floor	1C	3.04	5.17 1D		5.29	3.16		
EQ1611	2 <sup>nd</sup> floor	1C	6.90	11.22 1D		11.49	7.18		
EQ1612	2 <sup>nd</sup> floor	1C	3.04	5.17 1D		5.29	3.16		
EQ1613	2 <sup>nd</sup> floor	1C	6.90	11.22 1D		11.49	7.18		
EQ1615	2 <sup>nd</sup> floor	1C	1.82	3.10 1D		3.17	1.90		
LIVE	2 <sup>nd</sup> floor	1C	5.15	8.08 1D		8.27	5.36		
			<b>8.19</b>	<b>13.24</b>		<b>13.56</b>	<b>8.52</b>		
		<b>kN/m</b>	<b>13.44</b>	<b>21.72</b>		<b>22.24</b>	<b>13.98</b>		
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		
			<b>24.12</b>	<b>33.35</b>		<b>32.49</b>	<b>23.29</b>		
		<b>kN/m</b>	<b>39.57</b>	<b>54.71</b>		<b>53.30</b>	<b>38.21</b>		

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

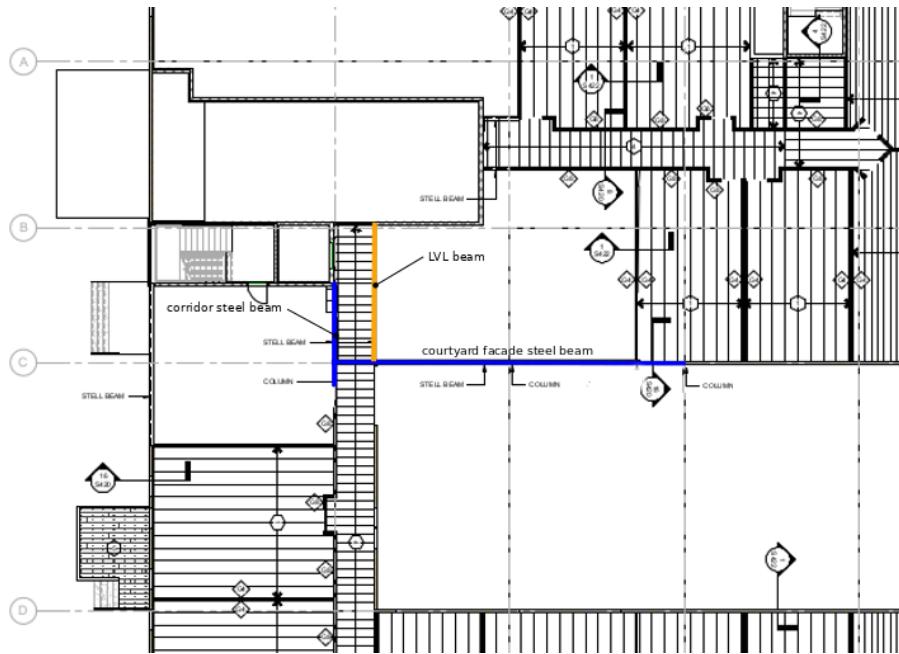


Figure 14: Second floor beams key plan.

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

$$V_u = 463.52 \text{ kN} \quad (128)$$

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

Resisting moment:

$$M_u = 171.88 \text{ kN} \cdot \text{m} \quad (129)$$

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

**Bending stiffness.** The deflection obtained is:

$$\Delta_{TL} = 18.72 \text{ mm} = \frac{\text{span}}{444} < \frac{L}{360} \implies OK \quad (130)$$

### 8.5.2 Steel beam at corridor

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

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

#### Structural design of the beam.

##### Loads.

$$w_{load} = 51.94 \text{ kN/m} \quad (131)$$

roof_truss_E_reactions									
comb	floor	truss	Rx	Ry	Rz	Rx	Ry	Rz	
		name	(kN)	(kN)	(kN)	(kN)	(kN)	(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	
					<b>8.62</b>			<b>11.04</b>	
						<b>kN/m</b>	<b>14.14</b>		<b>18.11</b>
EQ1608	3 <sup>rd</sup> floor	2E	0.00	0.00	3.59	0.00	0.00	4.68	
EQ1609	3 <sup>rd</sup> floor	2E	0.00	0.00	9.13	0.00	0.00	11.69	
EQ1610	3 <sup>rd</sup> floor	2E	0.00	0.00	3.59	0.00	0.00	4.68	
EQ1611	3 <sup>rd</sup> floor	2E	0.00	0.00	7.74	0.00	0.00	9.94	
EQ1612	3 <sup>rd</sup> floor	2E	0.00	0.00	3.59	0.00	0.00	4.68	
EQ1613	3 <sup>rd</sup> floor	2E	0.00	0.00	7.74	0.00	0.00	9.94	
EQ1615	3 <sup>rd</sup> floor	2E	0.00	0.00	2.15	0.00	0.00	2.81	
					<b>9.13</b>			<b>11.69</b>	
						<b>kN/m</b>	<b>14.97</b>		<b>19.18</b>
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	
					<b>16.36</b>			<b>20.98</b>	
						<b>kN/m</b>	<b>26.84</b>		<b>34.41</b>
EQ1608	2 <sup>nd</sup> floor	1E	0.00	0.00	2.91	0.00	0.00	4.88	
EQ1609	2 <sup>nd</sup> floor	1E	0.00	0.00	7.85	0.00	0.00	12.61	
EQ1610	2 <sup>nd</sup> floor	1E	0.00	0.00	2.91	0.00	0.00	4.88	
EQ1611	2 <sup>nd</sup> floor	1E	0.00	0.00	6.62	0.00	0.00	10.68	
EQ1612	2 <sup>nd</sup> floor	1E	0.00	0.00	2.91	0.00	0.00	4.88	
EQ1613	2 <sup>nd</sup> floor	1E	0.00	0.00	6.62	0.00	0.00	10.68	
EQ1615	2 <sup>nd</sup> floor	1E	0.00	0.00	1.75	0.00	0.00	2.93	
LIVE	2 <sup>nd</sup> floor	1E	0.00	0.00	4.94	0.00	0.00	7.73	
					<b>7.85</b>			<b>12.61</b>	
						<b>kN/m</b>	<b>12.88</b>		<b>20.69</b>
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	
					<b>22.98</b>			<b>31.66</b>	
						<b>kN/m</b>	<b>37.70</b>		<b>51.93</b>

Table 11: Steel beam at corridor. Trusses reactions.

**Internal forces on each structural channel.** Maximum induced moment:

$$M_{max} = 147.94 \text{ kNm} \quad (132)$$

Maximum induced shear:

$$V_{max} = 140.13 \text{ kN} \quad (133)$$

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

$$V_u = 254.63 \text{ kN} \quad (134)$$

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

Resisting moment:

$$M_u = 160.10 \text{ kN} \cdot m \quad (135)$$

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

**Bending stiffness.** The deflection obtained is:

$$\Delta_{TL} = 9.04 \text{ mm} = \frac{L}{439} < \frac{\text{span}}{360} \implies OK \quad (136)$$

## 8.6 Cantilevers

### 8.6.1 Cantilevers C1, C3, C4, C6, C11 and C12

Loads.

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

**Internal forces.** Maximum induced moment:

$$M_{max} = 37.98 \text{ kNm} \quad (137)$$

Maximum induced shear:

$$V_{max} = 65.41 \text{ kN} \quad (138)$$

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

$$M'_s = 44.89 \text{ kNm} \quad (139)$$

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

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

$$V'_s = 89.36 \text{ kN} \quad (140)$$

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

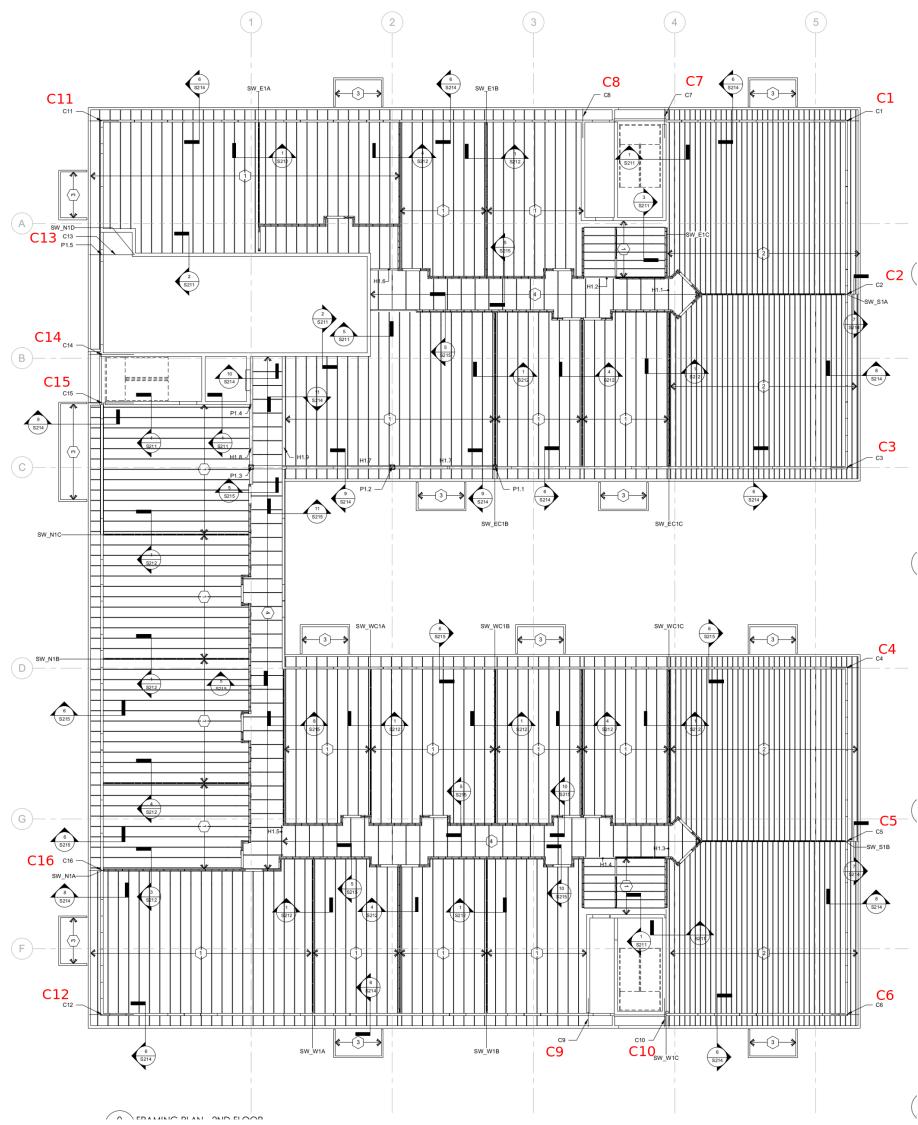


Figure 15: Second floor cantilevers key plan.

**Bending stiffness.** The deflection obtained under total load is:

$$\Delta_{TL} = 1.88 \text{ mm} = \frac{L}{506} < \frac{L}{360} \implies OK \quad (141)$$

### 8.6.2 Cantilevers C2 and C5

**Loads.**

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

**Internal forces.** Maximum induced moment:

$$M_{max} = 63.79 \text{ kNm} \quad (142)$$

Maximum induced shear:

$$V_{max} = 129.25 \text{ kN} \quad (143)$$

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

$$M'_s = 65.88 \text{ kNm} \quad (144)$$

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

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

$$V'_s = 151.60 \text{ kN} \quad (145)$$

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

**Bending stiffness.** The deflection obtained under total load is:

$$\Delta_{TL} = 1.95 \text{ mm} = \frac{L}{442} < \frac{L}{360} \implies OK \quad (146)$$

### 8.6.3 Cantilevers C7, C8, C9 and C10

**Loads.**

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

**Internal forces.** Maximum induced moment:

$$M_{max} = 17.77 \text{ kNm} \quad (147)$$

Maximum induced shear:

$$V_{max} = 18.71 \text{ kN} \quad (148)$$

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

$$M'_s = 29.93 \text{ kNm} \quad (149)$$

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

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

$$V'_s = 59.58 \text{ kN} \quad (150)$$

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

**Bending stiffness.** The deflection obtained under total load is:

$$\Delta_{TL} = 1.5 \text{ mm} = \frac{L}{632} < \frac{L}{360} \implies OK \quad (151)$$

#### 8.6.4 Cantilever C13

**Loads.**

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

**Internal forces.** Maximum induced moment:

$$M_{max} = 49.99 \text{ kNm} \quad (152)$$

Maximum induced shear:

$$V_{max} = 52.62 \text{ kN} \quad (153)$$

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

$$M'_s = 72.00 \text{ kNm} \quad (154)$$

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

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

$$V'_s = 114.90 \text{ kN} \quad (155)$$

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

**Bending stiffness.** The deflection obtained under total load is:

$$\Delta_{TL} = 1.5 \text{ mm} = \frac{L}{717} < \frac{L}{360} \implies OK \quad (156)$$

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

Table 12: Cantilever schedule.

### 8.6.5 Cantilever C14

Loads.

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

**Internal forces.** Maximum induced moment:

$$M_{max} = 33.33 \text{ kNm} \quad (157)$$

Maximum induced shear:

$$V_{max} = 35.08 \text{ kN} \quad (158)$$

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

$$M'_s = 48.00 \text{ kNm} \quad (159)$$

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

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

$$V'_s = 76.60 \text{ kN} \quad (160)$$

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

**Bending stiffness.** The deflection obtained under total load is:

$$\Delta_{TL} = 1.33 \text{ mm} = \frac{L}{717} < \frac{L}{360} \Rightarrow OK \quad (161)$$

## 8.7 Columns

### 8.7.1 Columns P3.1 to P3.6

Loads.

- Load from header: 4.8 kN

**Internal forces.** Maximum induced axial force:

$$N_{max} = 4.8 \text{ kN} \quad (162)$$

## 8. WOOD FRAMING

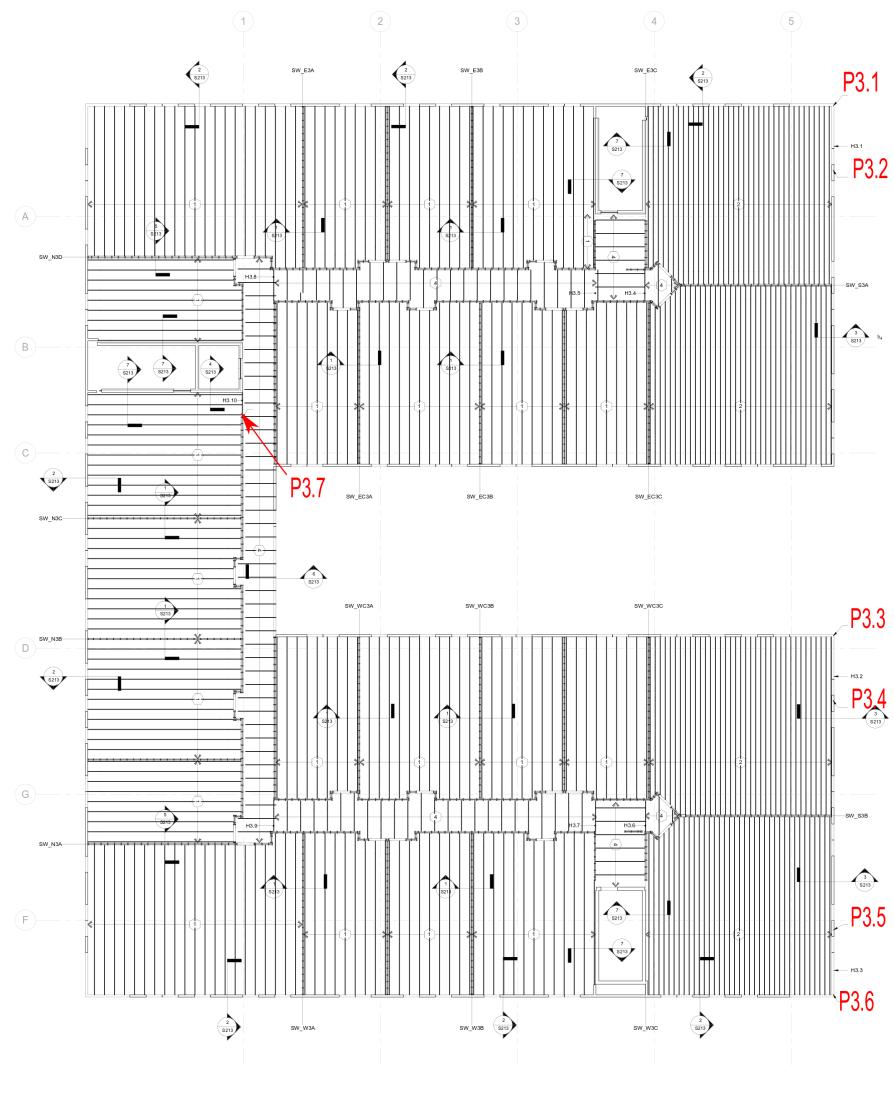


Figure 16: Columns key plan. Roof

## CALCULATION REPORT

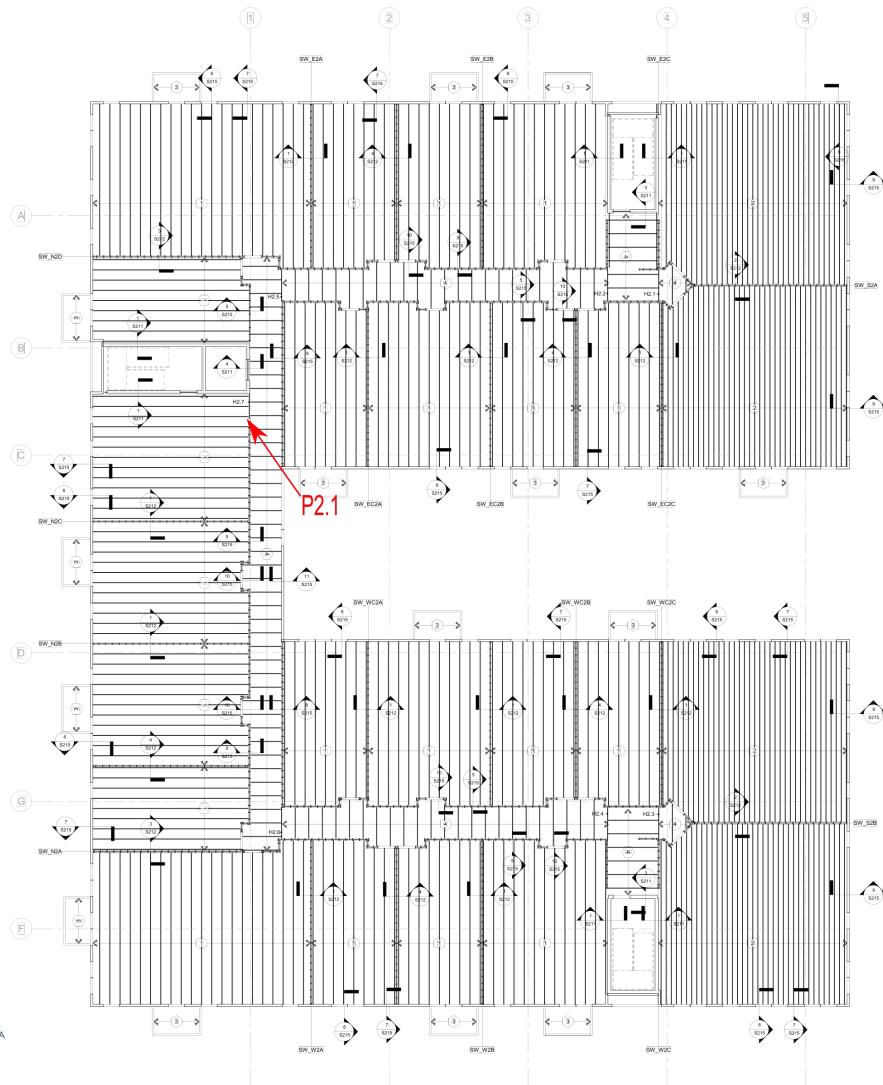


Figure 17: Columns key plan. Third floor

## 8. WOOD FRAMING

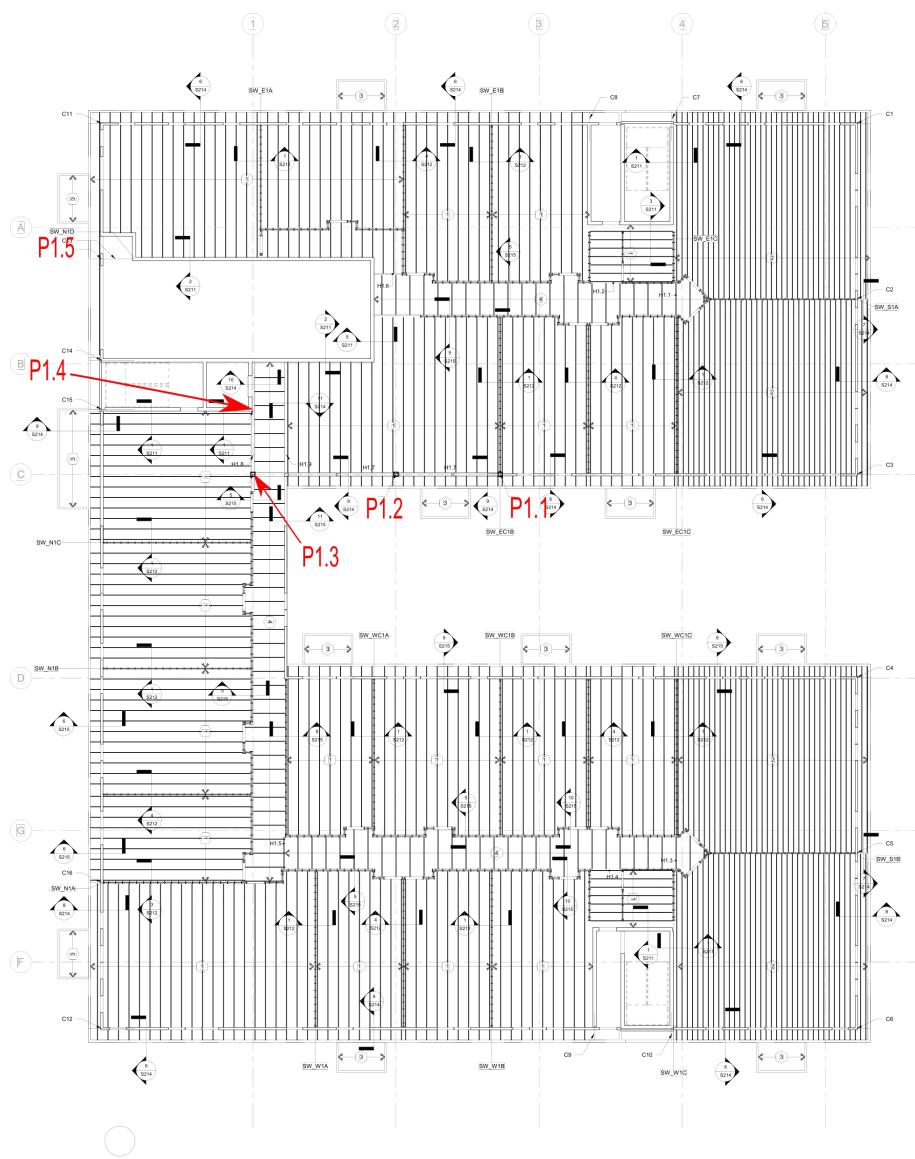


Figure 18: Columns key plan. Second floor

**Compression check.**

**Mechanical properties**

- Species: Spruce-pine-fir No.2
- $E_{min} = 2.55 \text{ GPa}$
- $F_c = 2.93 \text{ MPa.}$
- 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:  $C_P = 0.74$

$$N'_s = 42.46 \text{ kN} \quad (163)$$

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

**8.7.2 Column P3.7**

**Loads.**

- Load from header 3.10: 27.61 kN

**Internal forces.** Maximum induced axial force:

$$N_{max} = 27.61 \text{ kN} \quad (164)$$

**Compression check.**

**Mechanical properties**

- Species: Douglas fir-Larch dense structural
- $E_{min} = 4.27 \text{ GPa}$
- $F_c = 9.31 \text{ MPa.}$
- 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:  $C_P = 0.64$

$$N'_s = 74.34 \text{ kN} \quad (165)$$

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

**8.7.3 Column P2.1**

**Loads.**

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

**Internal forces.** Maximum induced axial force:

$$N_{max} = 55.22 \text{ kN} \quad (166)$$

**Compression check.**

#### Mechanical properties

- Species: Douglas fir-Larch dense structural
- $E_{min} = 4.27 \text{ GPa}$
- $F_c = 9.31 \text{ MPa}$ .
- 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:  $C_P = 0.64$

$$N'_s = 74.34 \text{ kN} \quad (167)$$

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

#### 8.7.4 2x6 wood stud capacity

**Compression capacity.**

#### Mechanical properties.

- Species: Hem-fir stud select structural.
- $E_{min} = 3.24 \text{ GPa}$
- $F_c = 8.27 \text{ MPa}$ .
- 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:  $C_P = 0.45$

$$N'_s = 19.90 \text{ kN} \quad (168)$$

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

**Loads.**

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

**Internal forces.** Maximum induced axial force:

$$N_{max} = 258.96 \text{ kNm} M_{y,max} = 5.18 \text{ kNm} M_{z,max} = 12.95 \text{ kNm} \quad (169)$$

**Mechanical properties.**

- Structural shape: HSS8x8x3/16
- Height: 3.51 m
- Steel: ASTM A500 Grade B ( $F_y = 315.0 \text{ MPa}$ )
- 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:  $P_n = \frac{844.37}{1.67} = 505.61 \text{ kN}$
- Nominal flexural strength:  $M_n = \frac{58.55}{1.67} = 35.06 \text{ kNm}$

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

$$\frac{P_d}{P_n} + \frac{8}{9} \left( \frac{M_{yd}}{M_{yn}} + \frac{M_{zd}}{M_{zn}} \right) = 0.97 < 1.0 \implies OK \quad (170)$$

### 8.7.6 Column P1.4

**Loads.**

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

**Internal forces.** Maximum induced axial force:

$$N_{max} = 125.49 \text{ kNm} M_{y,max} = 7.33 \text{ kNm} M_{z,max} = 6.27 \text{ kNm} \quad (171)$$

**Mechanical properties.**

- Structural shape: HSS7X7X3/16
- Height: 3.51 m
- Steel: ASTM A500 Grade B ( $F_y = 315.0 \text{ MPa}$ )
- 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:  $P_n = \frac{774.19}{1.67} = 463.59 \text{ kN}$
- Nominal flexural strength:  $M_n = \frac{51.02}{1.67} = 30.55 \text{ kNm}$

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

$$\frac{P_d}{P_n} + \frac{8}{9} \left( \frac{M_{yd}}{M_{yn}} + \frac{M_{zd}}{M_{zn}} \right) = 0.67 < 1.0 \implies OK \quad (172)$$

## 8.8 Bearing walls

### 8.8.1 Facade bearing walls at first floor

**Loads.**

- Vertical load from trusses: 43.34 kN/m
- Vertical load on each stud: 10.57 kN
- Wind load on stud height: 0.21 kN/m

**Internal forces.** Internal forces:

$$N_{max} = 10.56 \text{ kN} \quad (173)$$

$$M_{max} = 0.18 \text{ kN} \cdot m \quad (174)$$

**Bending and axial compression check.**

**Mechanical properties**

- Species: Hem-fir stud
- Spacing: 0.24 m
- Stud height: 2.64 m
- Repetitive member factor:  $C_r = 1.15$
- Size factor:  $C_F = 1.3$
- $E_{min} = 3.03 \text{ GPa}$
- $F'_c = 4.5 \text{ MPa}$ .
- $F'_b = 6.96 \text{ MPa}$ .
- Sections dimensions: (2x6"), effective (1.5x5.5")= 38.1 x 139.7 mm.
- Unbraced lenght x axis: 2.64 m
- Unbraced lenght y axis: 0.3 m
- Stud stability factor:  $C_P = 0.68$

$$N'_s = 23.94 \text{ kN} \quad (175)$$

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

$$CF = 0.46 < 1 \implies OK \quad (176)$$

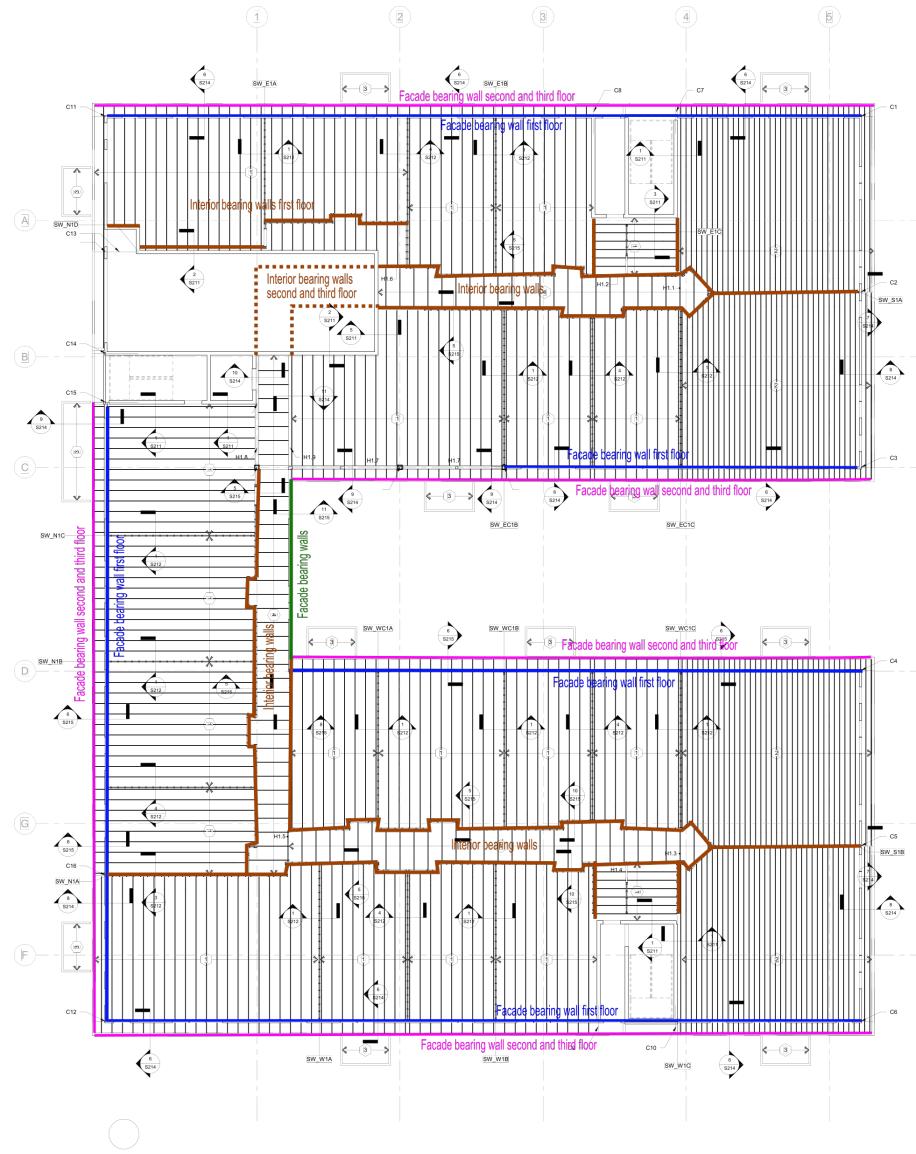


Figure 19: Bearing walls key plan. Roof

### 8.8.2 Facade bearing walls at second floor

**Loads.**

- Vertical load from trusses: 30.52 kN/m
- Vertical load on each stud: 7.44 kN
- Wind load on stud height: 0.21 kN/m

**Internal forces.** Internal forces:

$$N_{max} = 7.44 \text{ kN} \quad (177)$$

$$M_{max} = 0.18 \text{ kN} \cdot \text{m} \quad (178)$$

**Bending and axial compression check.**

**Mechanical properties**

- Species: Hem-fir stud
- Spacing: 0.24 m
- Stud height: 2.64 m
- Repetitive member factor:  $C_r = 1.15$
- Size factor:  $C_F = 1.3$
- $E_{min} = 3.03 \text{ GPa}$
- $F'_c = 4.5 \text{ MPa.}$
- $F'_b = 6.96 \text{ MPa.}$
- Sections dimensions: (2x6"), effective (1.5x5.5")= 38.1 x 139.7 mm.
- Unbraced lenght x axis: 2.64 m
- Unbraced lenght y axis: 0.3 m
- Stud stability factor:  $C_P = 0.68$

$$N'_s = 26.00 \text{ kN} \quad (179)$$

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

$$CF = 0.34 < 1 \implies OK \quad (180)$$

### 8.8.3 Facade bearing walls at third floor

**Loads.**

- Vertical load from trusses: 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:

$$N_{max} = 7.81 \text{ kN} \quad (181)$$

$$M_{max} = 0.36 \text{ kN} \cdot \text{m} \quad (182)$$

**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:  $C_F = 1.3$
- $E_{min} = 3.03 \text{ GPa}$
- $F'_c = 4.89 \text{ MPa.}$
- $F'_b = 6.96 \text{ MPa.}$
- Sections dimensions: (2x6"), effective (1.5x5.5")= 38.1 x 139.7 mm.
- Unbraced lenght x axis: 2.64 m
- Unbraced lenght y axis: 0.3 m
- Stud stability factor:  $C_P = 0.68$

$$N'_s = 26.00 \text{ kN} \quad (183)$$

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

$$CF = 0.62 < 1 \implies OK \quad (184)$$

#### 8.8.4 Interior bearing walls at first floor

##### Loads.

- Vertical load from trusses: 54.71 kN/m
- Vertical load on each stud: 13.34 kN
- Wind load on stud height: 0.0 kN/m

**Internal forces.** Internal forces:

$$N_{max} = 13.34 \text{ kN} \quad (185)$$

$$M_{max} = 0.26 \text{ kN} \cdot \text{m} \quad (186)$$

**Bending and axial compression check.**

### Mechanical properties

- Species: Hem-fir stud
- Spacing: 0.24 m
- Stud height: 2.64 m
- Repetitive member factor:  $C_r = 1.15$
- Size factor:  $C_F = 1.3$
- $E_{min} = 3.03 \text{ GPa}$
- $F'_c = 4.89 \text{ MPa}$
- $F'_b = 6.96 \text{ MPa}$ .
- Sections dimensions: (2x6"), effective (1.5x5.5")= 38.1 x 139.7 mm.
- Unbraced lenght x axis: 2.64 m
- Unbraced lenght y axis: 0.3 m
- Stud stability factor:  $C_P = 0.68$

$$N'_s = 26.00 \text{ kN} \quad (187)$$

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

$$CF = 0.74 < 1 \implies OK \quad (188)$$

#### 8.8.5 Interior bearing walls at second floor

##### Loads.

- Vertical load from trusses: 36.3 kN/m
- Vertical load on each stud: 17.70 kN
- Wind load on stud height: 0.0 kN/m

**Internal forces.** Internal forces:

$$N_{max} = 17.70 \text{ kN} \quad (189)$$

$$M_{max} = 0.35 \text{ kN} \cdot \text{m} \quad (190)$$

##### Bending and axial compression check.

**Mechanical properties**

- Species: Hem-fir stud
- Spacing: 0.24 m
- Stud height: 2.64 m
- Repetitive member factor:  $C_r = 1.15$
- Size factor:  $C_F = 1.3$
- $E_{min} = 3.03 \text{ GPa}$
- $F'_c = 4.89 \text{ MPa.}$
- $F'_b = 6.96 \text{ MPa.}$
- Sections dimensions: (2x6"), effective (1.5x5.5")= 38.1 x 139.7 mm.
- Unbraced lenght x axis: 2.64 m
- Unbraced lenght y axis: 0.3 m
- Stud stability factor:  $C_P = 0.68$

$$N'_s = 26.00 \text{ kN} \quad (191)$$

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

$$CF = 0.38 < 1 \implies OK \quad (192)$$

**8.8.6 Interior bearing walls at third floor**

**Loads.**

- Vertical load from trusses: 19.37 kN/m
- Vertical load on each stud: 9.45 kN
- Wind load on stud height: 0.0 kN/m

**Internal forces.** Internal forces:

$$N_{max} = 19.37 \text{ kN} \quad (193)$$

$$M_{max} = 0.19 \text{ kN} \cdot \text{m} \quad (194)$$

**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:  $C_F = 1.3$
- $E_{min} = 3.03 \text{ GPa}$
- $F'_c = 4.89 \text{ MPa}$ .
- $F'_b = 6.96 \text{ MPa}$ .
- Sections dimensions: (2x6"), effective (1.5x5.5")= 38.1 x 139.7 mm.
- Unbraced lenght x axis: 2.64 m
- Unbraced lenght y axis: 0.3 m
- Stud stability factor:  $C_P = 0.68$

$$N'_s = 26.00 \text{ kN} \quad (195)$$

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

$$CF = 0.43 < 1 \implies OK \quad (196)$$

## 8.9 Lateral. Diaphragms/Shear walls

### 8.9.1 East and West facades shear walls

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

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

The shear values obtained for each wall are as follows:

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

And the cumulated values are:

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

leading to the following dimensions:

CALCULATION REPORT

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	Bottom plate attachment (foundation)		Bottom plate attachment (floor to floor)	
									(in)	(in)	Number of bolts	Bolt spacing
E3A	WSP – sheathing	19/32	Y	1-1/2	10d	4	1430	3	(1)	U4-SDS2.5	-	-
E3B	WSP – sheathing	3/8	N	1-3/8	8d	6	560	-	-		-	-
E3C	WSP – sheathing	19/32	Y	1-1/2	10d	4	1430	6	(2)	U11-SDS2.5	-	-
E2A	WSP – sheathing	19/32	Y	1-1/2	10d	3	1860	7	(3)	U11-SDS2.5	-	-
E2B	WSP – sheathing	3/8	N	1-3/8	8d	6	560	1	(1)	U4-SDS2.5	-	-
E2C	WSP – sheathing	19/32	Y	1-1/2	10d	2	2435	11	(4)	19	-	-
E1A	WSP – sheathing	19/32	Y	1-1/2	10d	2	2435	13	(4)	19	7	36
E1B	WSP – sheathing	3/8	N	1-3/8	8d	6	560	-	-		11	36
E1C	WSP – sheathing	19/32	Y	1-1/2	10d	2	2435	9	(3)	19	11	36
W3A	WSP – sheathing	19/32	Y	1-1/2	10d	4	1430	3	(1)	U4-SDS2.5	-	-
W3B	WSP – sheathing	3/8	N	1-3/8	8d	6	560	-	-		-	-
W3C	WSP – sheathing	19/32	Y	1-1/2	10d	4	1430	6	(2)	U11-SDS2.5	-	-
W2A	WSP – sheathing	19/32	Y	1-1/2	10d	3	1860	7	(3)	U11-SDS2.5	-	-
W2B	WSP – sheathing	3/8	N	1-3/8	8d	6	560	1	(1)	U4-SDS2.5	-	-
W2C	WSP – sheathing	19/32	Y	1-1/2	10d	2	2435	11	(4)	19	-	-
W1A	WSP – sheathing	19/32	Y	1-1/2	10d	2	2435	13	(4)	19	9	30
W1B	WSP – sheathing	3/8	N	1-3/8	8d	6	560	-	-		11	36
W1C	WSP – sheathing	19/32	Y	1-1/2	10d	2	2435	9	(3)	19	11	36

### 8.9.2 Courtyard facades shear walls

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

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

The shear values obtained for each wall are as follows:

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

And the cumulated values are:

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

leading to the following dimensions:

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

## CALCULATION REPORT

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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 fasten- ers 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 fasten- ers 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 fasten- ers 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 fasten- ers 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 fasten- ers in 2 rows.

### 8.9.3 South facades shear walls

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

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

The shear values obtained for each wall are as follows:

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

And the cumulated values are:

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

leading to the following dimensions:

ID	(in)	(in)	(in)	(in)	(plf)	(kip)	HD		(in)			
S3A	WSP – sheathing	19/32	Y	1-1/2	10d	6	950	2	(1)	U4-SDS2.5	-	-
S3B	WSP – sheathing	19/32	Y	1-1/2	10d	6	950	2	(1)	U4-SDS2.5	-	-
S2A	WSP – sheathing	19/32	Y	1-1/2	10d	3	1860	6	(2)	U11-SDS2.5	-	-
S2B	WSP – sheathing	19/32	Y	1-1/2	10d	3	1860	6	(2)	U11-SDS2.5	-	-
S1A	WSP – sheathing	19/32	Y	1-1/2	10d	2	2435	11	(4)	19	10	36
S1B	WSP – sheathing	19/32	Y	1-1/2	10d	2	2435	11	(4)	19	10	36

#### 8.9.4 North facade shear walls

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

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

The shear values obtained for each wall are as follows:

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

And the cumulated values are:

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

CALCULATION REPORT

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leading to the following dimensions:

	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	Bottom plate attachment (foundation)	Bottom plate attachment (floor to floor)
ID N3A	WSP – sheathing	(in) 3/8	Y	(in) 1-3/8	8d	(in) 4	(plf) 840	(kip) 2	(1)	HD U4-SDS2.5	-	(in) -	wood screws 20 (d= 0.32 in) at 25 in. o/c; 30 fasteners in 2 rows.			
N3B	WSP – sheathing	3/8	N	1-3/8	8d	6	560	-	-		-	-	16d (d= 0.268 in) nails at 24 in. o/c; 16 fasteners in 1 row.			
N3C	WSP – sheathing	3/8	N	1-3/8	8d	6	560	-	-		-	-	16d (d= 0.268 in) nails at 21 in. o/c; 35 fasteners in 2 rows.			
N3D	WSP – sheathing	3/8	Y	1-3/8	8d	4	840	2	(1)	U4-SDS2.5	-	-	wood screws 20 (d= 0.32 in) at 25 in. o/c; 30 fasteners in 2 rows.			
N2A	WSP – sheathing	19/32	Y	1-1/2	10d	4	1430	4	(2)	U4-SDS2.5	-	-	wood screws 20 (d= 0.32 in) at 14 in. o/c; 52 fasteners in 2 rows.			
N2B	WSP – sheathing	19/32	Y	1-1/2	10d	6	950	-	-		-	-	16d (d= 0.268 in) nails at 13 in. o/c; 28 fasteners in 1 row.			
N2C	WSP – sheathing	19/32	Y	1-1/2	10d	6	950	1	(1)	U4-SDS2.5	-	-	16d (d= 0.268 in) nails at 12 in. o/c; 59 fasteners in 2 rows.			
N2D	WSP – sheathing	19/32	Y	1-1/2	10d	4	1430	4	(2)	U4-SDS2.5	-	-	wood screws 20 (d= 0.32 in) at 14 in. o/c; 52 fasteners in 2 rows.			
N1A	WSP – sheathing	19/32	Y	1-1/2	10d	3	1860	7	(3)	U11-SDS2.5	10	36	SDWS log screw (d= 0.197 in) at 12 in. o/c; 58 fasteners in 2 rows.			
N1B	WSP – sheathing	19/32	Y	1-1/2	10d	6	950	-	-		11	36	16d (d= 0.268 in) nails at 19 in. o/c; 39 fasteners in 2 rows.			
N1C	WSP – sheathing	19/32	Y	1-1/2	10d	6	950	3	(1)	U4-SDS2.5	11	36	wood screws 20 (d= 0.32 in) at 19 in. o/c; 40 fasteners in 2 rows.			
N1D	WSP – sheathing	19/32	Y	1-1/2	10d	3	1860	7	(3)	U11-SDS2.5	10	36	SDWS log screw (d= 0.197 in) at 12 in. o/c; 60 fasteners in 2 rows.			

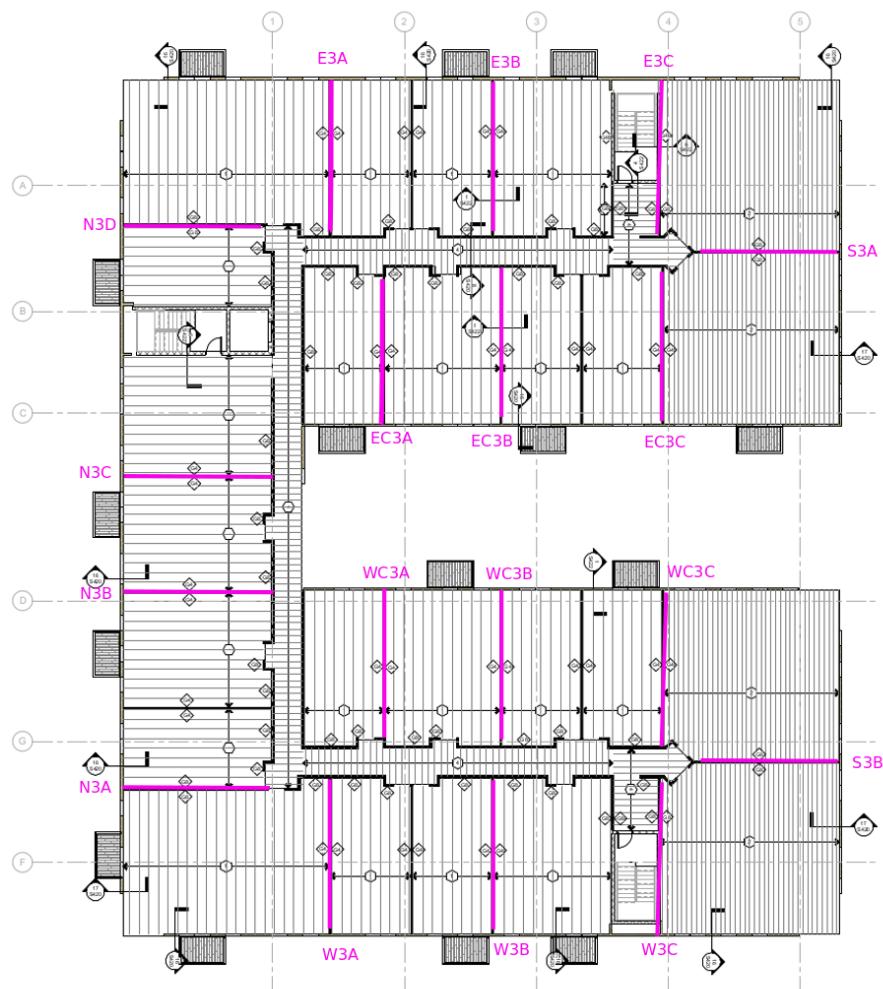


Figure 20: Shear walls on the third floor.

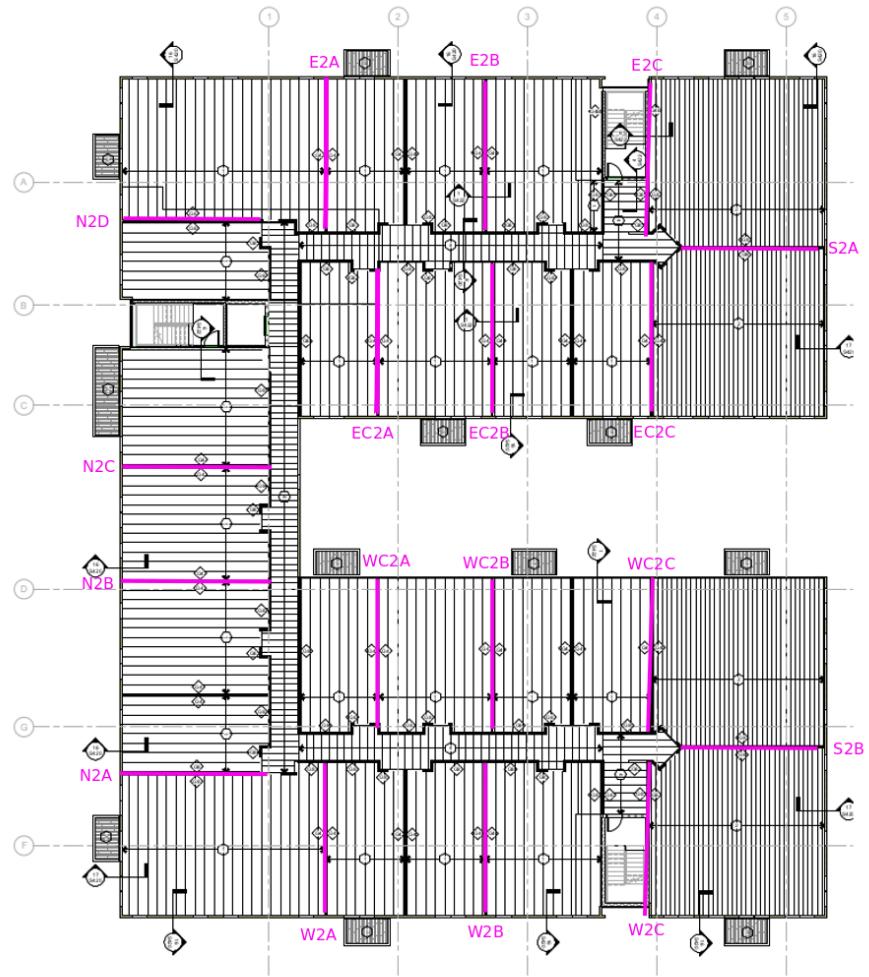


Figure 21: Shear walls on the second floor.

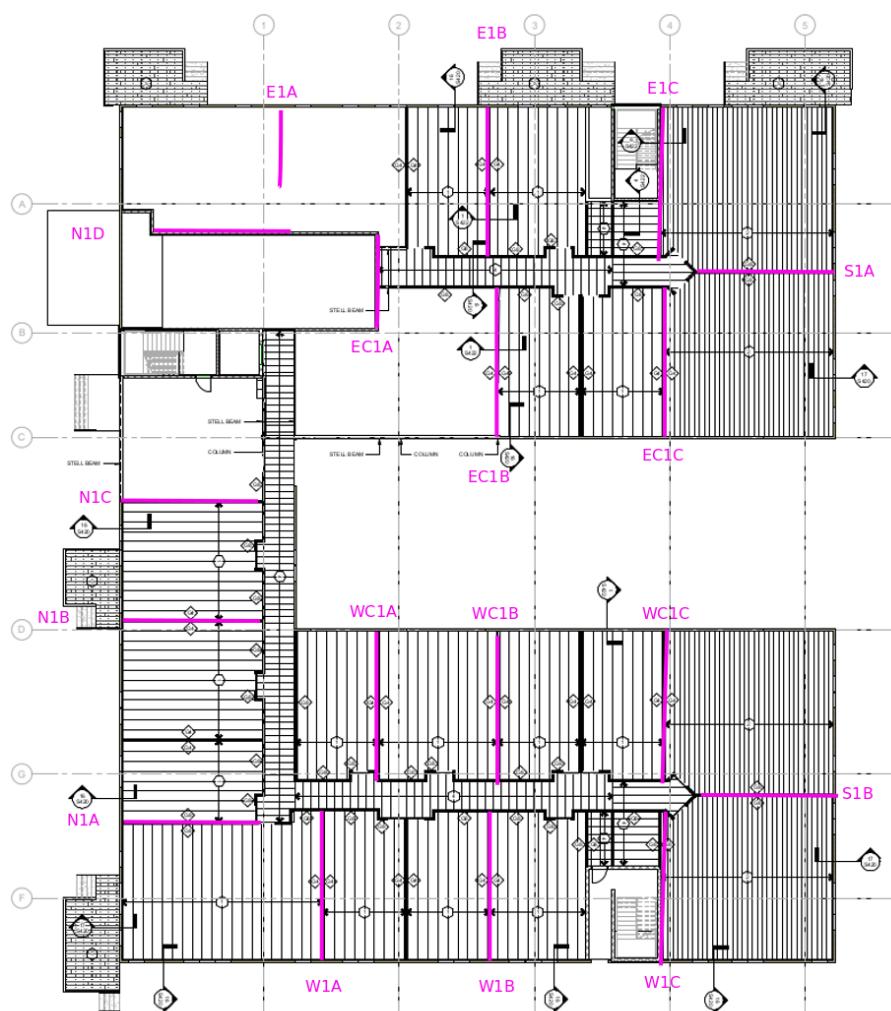


Figure 22: Shear walls on the first floor.

## 9 Basement

### 9.1 Structural model

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

Linear loads are expressed in kN/m and surface loads in kN/m<sup>2</sup>, where:

$$\begin{aligned}1 \text{ kN/m} &= 68.52178 \text{ lb/ft} \\1 \text{ kN/m}^2 &= 20.885434 \text{ psf}\end{aligned}$$

## 9. BASEMENT

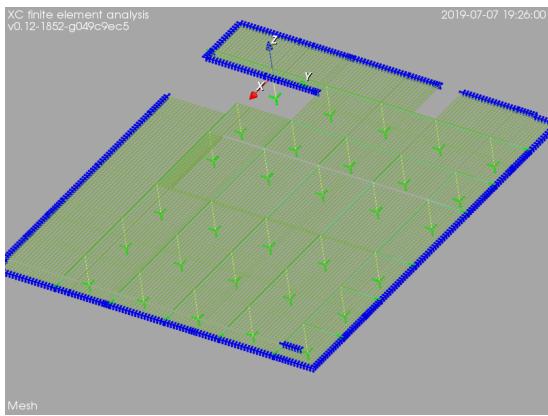


Figure 23: Elastic model, mesh.

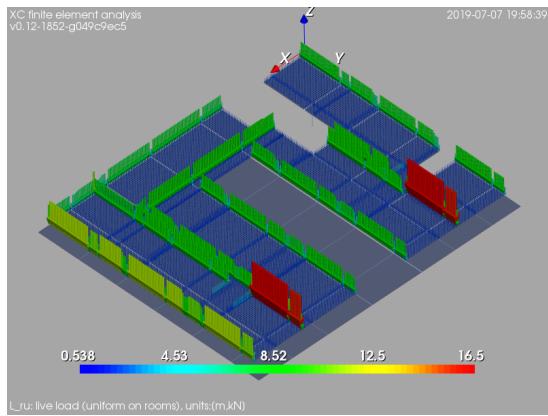


Figure 26: Load case Lrs: live load (staggered pattern on rooms) [units: kN,m].

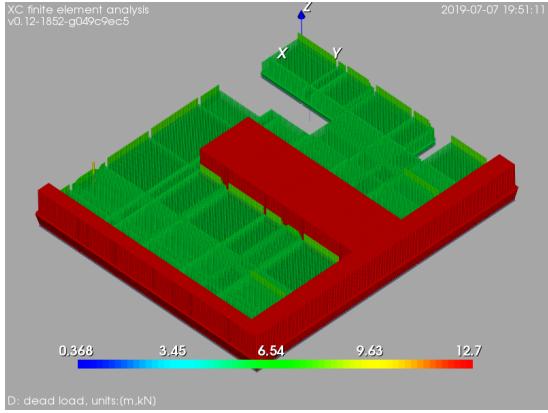


Figure 24: Load case D: dead load (include slab self-weight) [units: kN,m].

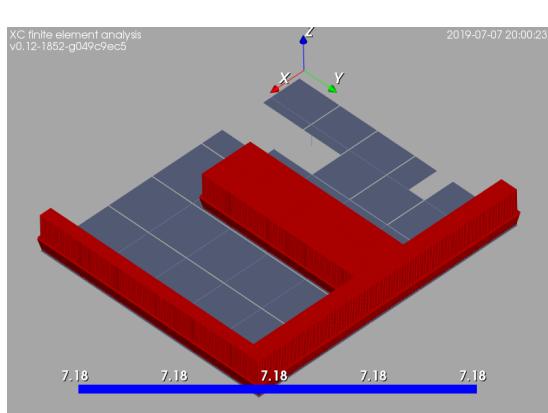


Figure 27: Load case Lpu: live load (uniform on patios) [units: kN,m].

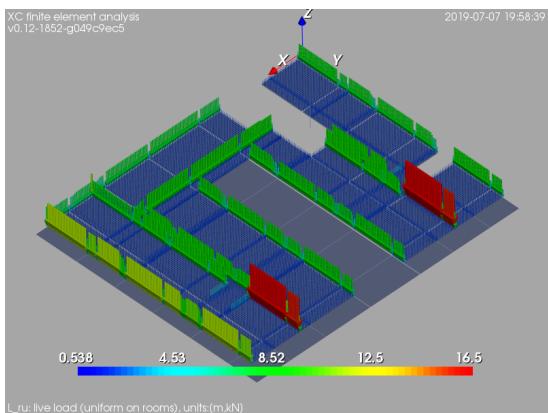


Figure 25: Load case Lru: live load (uniform on rooms) [units: kN,m].

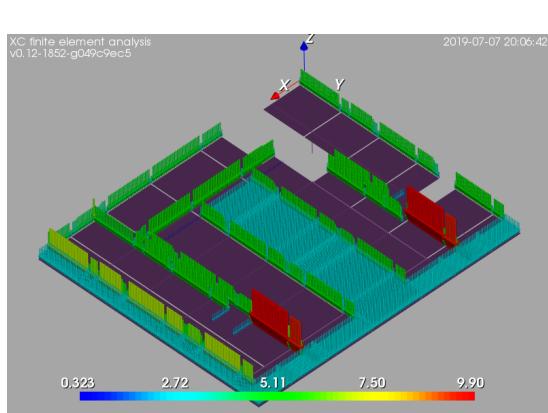


Figure 28: Load case S: snow [units: kN,m].

## CALCULATION REPORT

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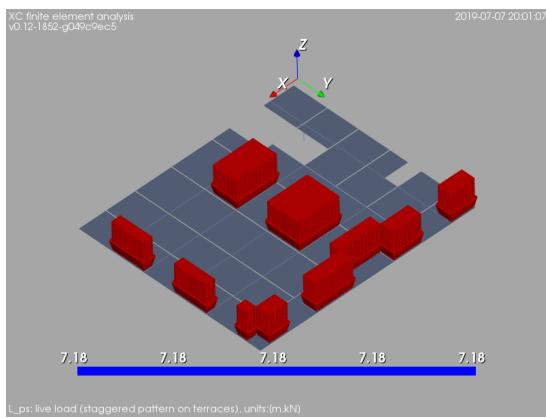


Figure 29: Load case Lps: live load (staggered pattern on patios) [units: kN,m].

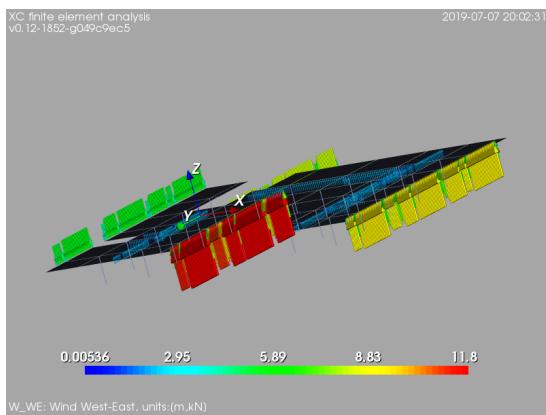


Figure 30: Load case W\_WE: wind West-East [units: kN,m].

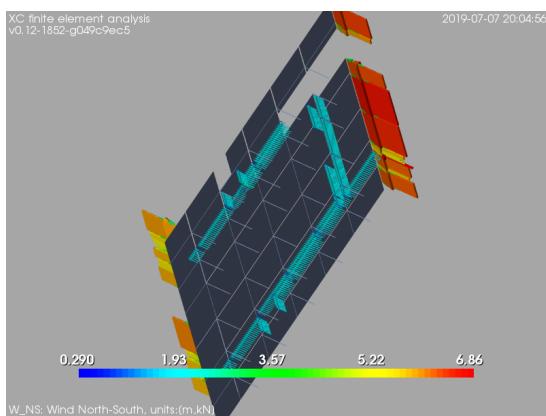


Figure 31: Load case W\_NS: wind North-South [units: kN,m].

## 9. BASEMENT

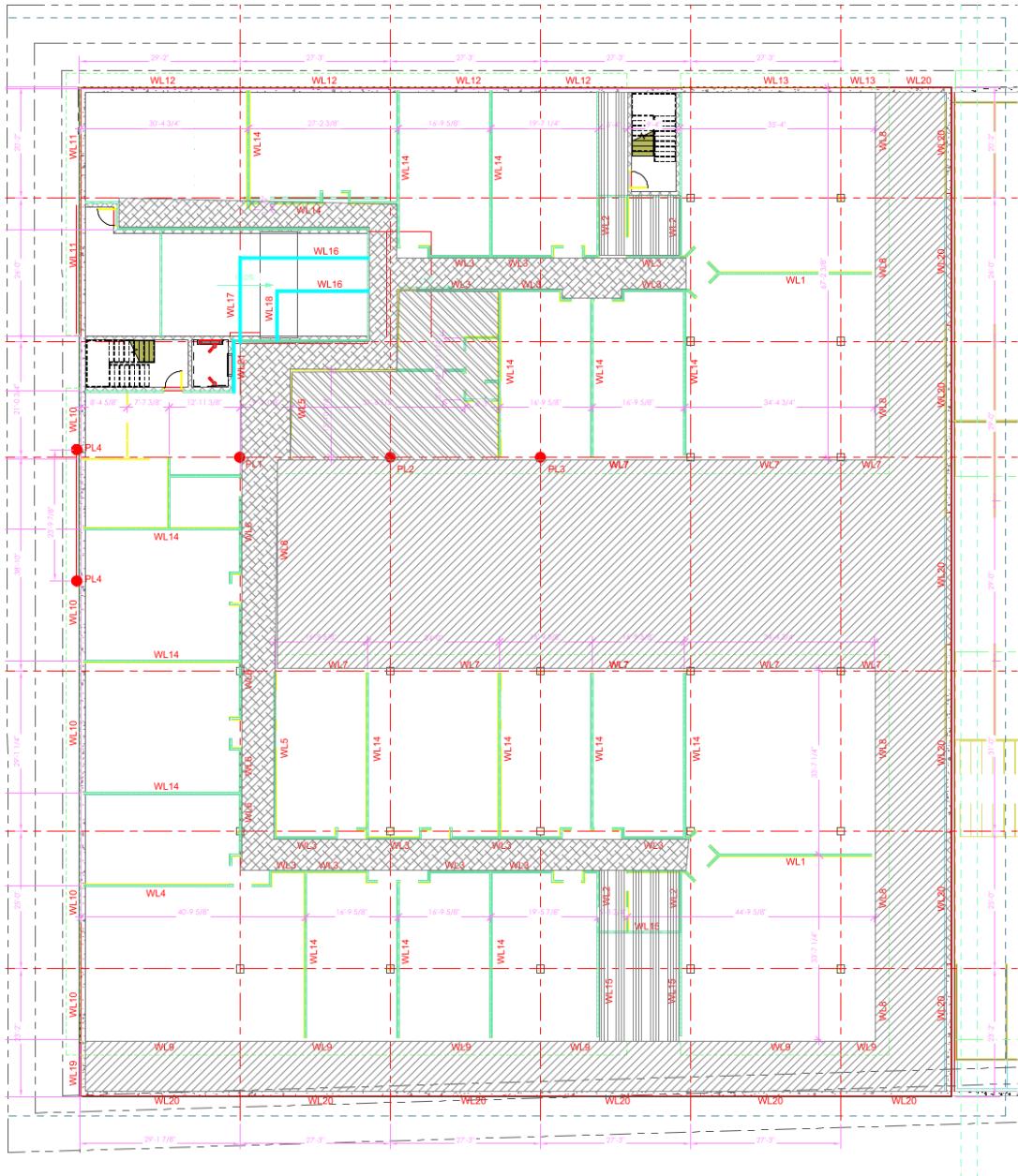


Figure 32: Load layout on first floor.

POINT LOAD SCHEDULE			
MARK	DEAD LOAD Kib	LIVE LOAD Kib	SNOW LOAD Kib
PL1	15.24	25.01	15.01
PL2	30.47	50.09	30.09
PL3	15.24	25.01	15.01
-	-	-	6.01

SUPERIMPOSED UNIFORM LOAD SCHEDULE				
MARK.	DEAD LOAD PSF	LIVE LOAD PSF	SHOW 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 ENTRYS
	20.00	100.00	N/A	STORE FIRST FLOOR
	20.00	40.00	N/A	PRIVATE ROOMS MULTIFAMILY DWELLING

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 FOOTING EXCAVATIONS SHALL BE INSPECTED AND APPROVED BY THE GEOTECHNICAL ENGINEER PRIOR TO PLACING CONCRETE

Table 17: Column footing schedule.

## 9.2 Footings

### 9.2.1 Loads

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

### 9.2.2 Load combinations

The load combinations are shown in tables 7 and 8.

### 9.2.3 Footing dimensions and reinforcement

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

### 9.2.4 Limit state checking

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

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

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

## 9.3 Basement walls

### 9.3.1 Introduction

The design is based in the following assumptions:

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

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

Table 18: Soil bearing pressures. Capacity factors

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

Table 19: Shear design. Capacity factors

Footing	Worst combination	Vertical load (kN)	thickness (m)	L (m)	d (m)	c (m)	Vd/l kN/m	Vu kN/m	CF
A1	SLS04.a	-356.20	0.51	2.74	0.46	0.41	92.90	517.97	0.18
A2	SLS02.a	-644.78	0.51	2.74	0.46	0.41	168.16	517.97	0.32
A3	SLS02.a	-950.92	0.51	2.74	0.46	0.41	248.00	517.97	0.48
A4	SLS02.a	-881.82	0.51	2.74	0.46	0.41	229.98	517.97	0.44
A5	SLS04.b	-933.03	0.51	2.74	0.46	0.41	243.33	517.97	0.47
B1	SLS02.a	-429.53	0.51	2.90	0.46	0.41	113.28	517.97	0.22
B2	SLS02.a	-670.79	0.51	2.90	0.46	0.41	176.91	517.97	0.34
B3	SLS02.a	-1,030.65	0.51	2.90	0.46	0.41	271.82	517.97	0.52
B4	SLS02.a	-968.24	0.51	2.90	0.46	0.41	255.36	517.97	0.49
B5	SLS04.b	-972.32	0.51	2.90	0.46	0.41	256.43	517.97	0.50
C1	SLS02.a	-1,460.67	0.69	3.20	0.62	0.41	320.25	699.26	0.46
C2	SLS02.a	-1,742.45	0.64	3.51	0.57	0.41	410.79	647.47	0.63
C3	SLS02.a	-1,750.31	0.64	3.51	0.57	0.41	412.64	647.47	0.64
C4	SLS02.a	-1,751.62	0.64	3.51	0.57	0.41	412.95	647.47	0.64
C5	SLS02.a	-1,660.02	0.64	3.51	0.57	0.41	391.35	647.47	0.60
D1	SLS02.a	-1,648.12	0.69	3.35	0.62	0.41	365.00	699.26	0.52
D2	SLS04.a	-1,950.01	0.64	3.66	0.57	0.41	462.88	647.47	0.71
D3	SLS04.a	-1,959.12	0.64	3.66	0.57	0.41	465.05	647.47	0.72
D4	SLS04.a	-1,960.14	0.64	3.66	0.57	0.41	465.29	647.47	0.72
D5	SLS04.a	-1,838.69	0.64	3.66	0.57	0.41	436.46	647.47	0.67
F1	SLS02.a	-1,090.88	0.64	3.05	0.57	0.41	250.18	647.47	0.39
F2	SLS02.a	-997.32	0.64	3.05	0.57	0.41	228.72	647.47	0.35
F3	SLS02.a	-1,011.87	0.64	3.05	0.57	0.41	232.06	647.47	0.36
F4	SLS02.a	-1,005.30	0.64	3.05	0.57	0.41	230.55	647.47	0.36
F5	SLS04.b	-741.90	0.64	3.05	0.57	0.41	170.14	647.47	0.26
G1	SLS02.a	-1,496.86	0.69	3.35	0.62	0.41	331.50	699.26	0.47
G2	SLS02.a	-1,256.06	0.64	3.35	0.57	0.41	293.79	647.47	0.45
G3	SLS02.a	-1,227.94	0.64	3.35	0.57	0.41	287.22	647.47	0.44
G4	SLS02.a	-1,137.75	0.64	3.35	0.57	0.41	266.12	647.47	0.41
G5	SLS04.b	-1,167.26	0.64	3.35	0.57	0.41	273.02	647.47	0.42

Table 20: Two-way shear design. Capacity factors

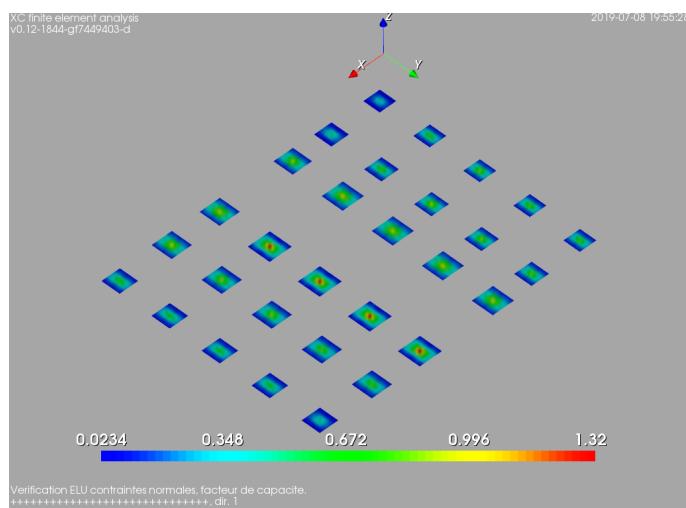


Figure 33: Flexure in the longitudinal direction. Capacity factor.

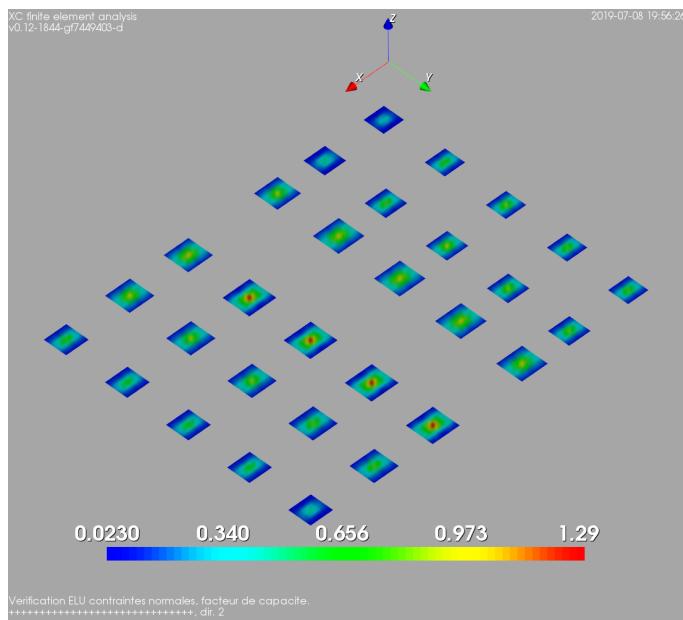


Figure 34: Flexure in the transverse direction. Capacity factor.

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

### 9.3.2 Load determination

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

**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
	LiveL	21.67	21.67
	Wind_NS	-15.12	-15.12
	Wind_WE	-1.33	-1.33
	DeadL	31.54	31.54
South	SnowL	8.04	16.08
	LiveL	14.22	28.44
	Wind_NS	4.97	9.95
	Wind_WE	-0.23	-0.46
	DeadL	20.58	41.15
East	SnowL	11.96	11.96
	LiveL	23.75	23.75
	Wind_NS	-0.07	-0.07
	Wind_WE	12.97	12.97
	DeadL	30.87	30.87
West	SnowL	15.02	15.02
	LiveL	27.15	27.15
	Wind_NS	-0.20	-0.20
	Wind_WE	-13.20	-13.20
	DeadL	29.81	29.81

### 9.3.3 Load combinations

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

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

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

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

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

Table 21: Concrete walls reinforcing schedule

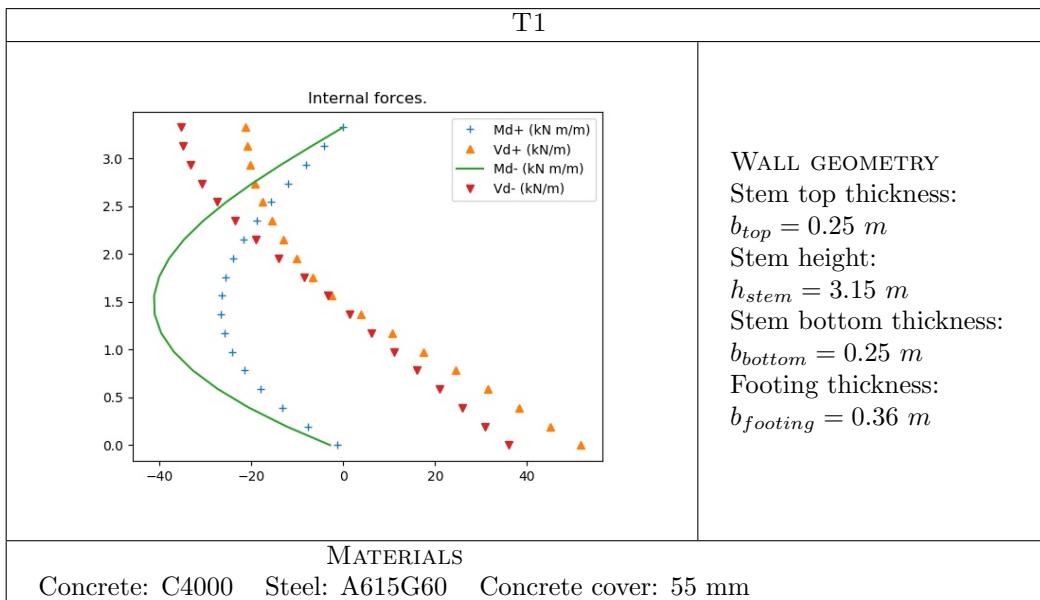


Table 22: Wall materials and dimensions T1

### 9.3.4 Stem dimensions and reinforcement

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

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

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

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

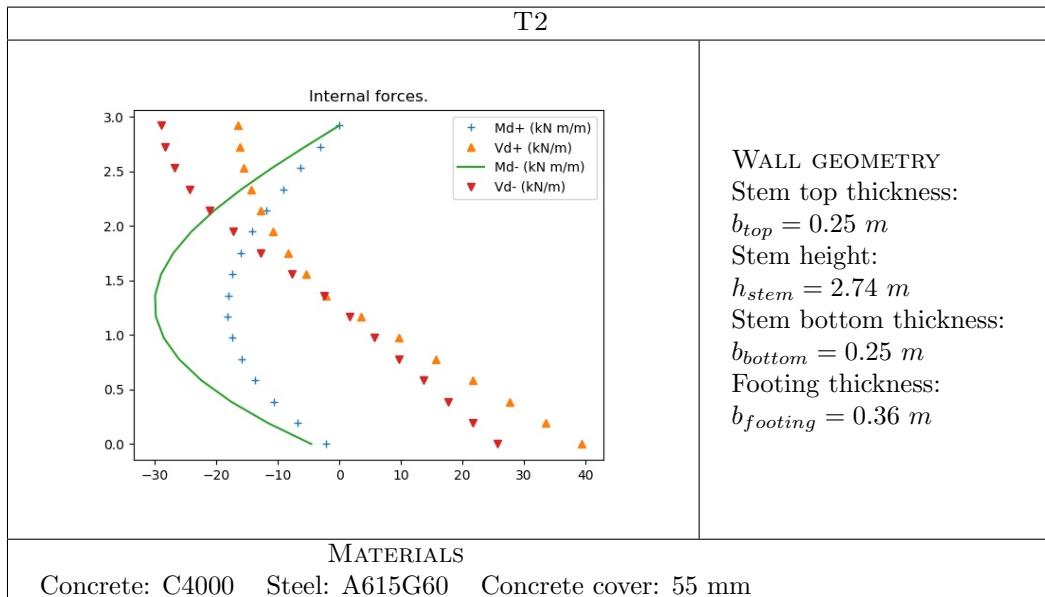


Table 23: Wall materials and dimensions T2

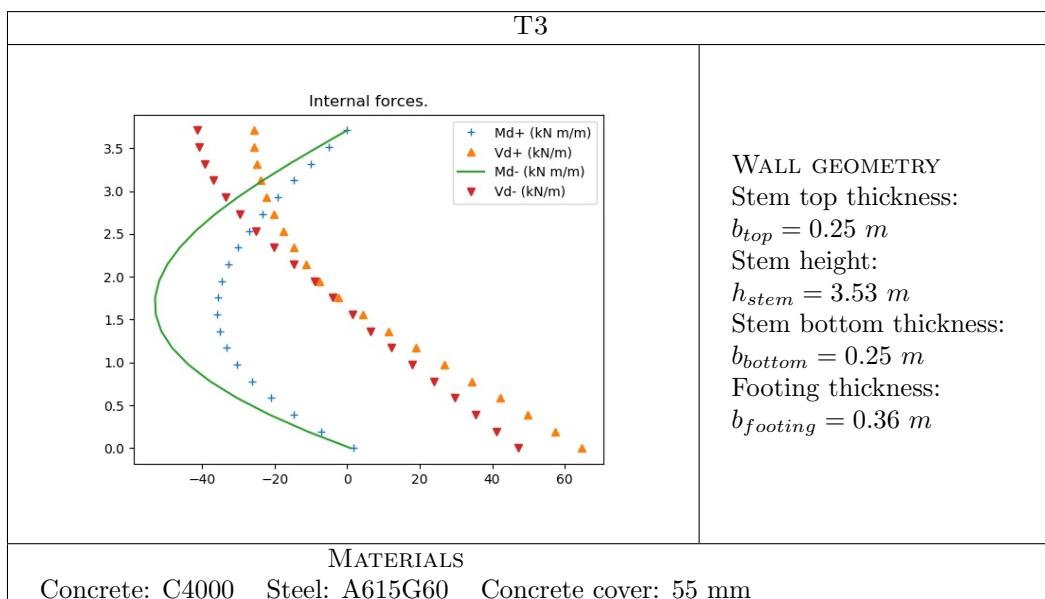


Table 24: Wall materials and dimensions T3

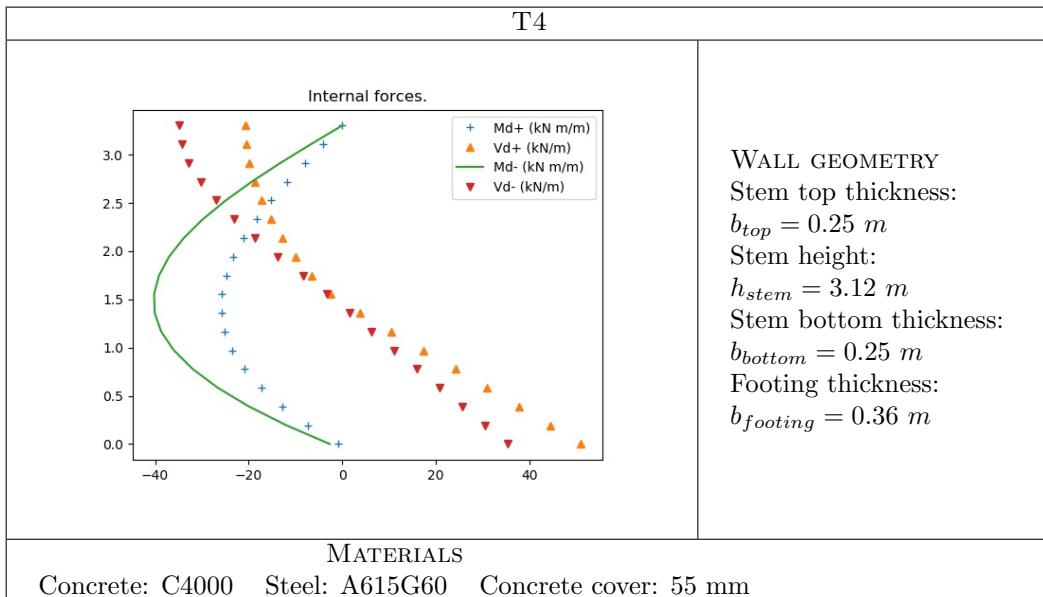


Table 25: Wall materials and dimensions T4

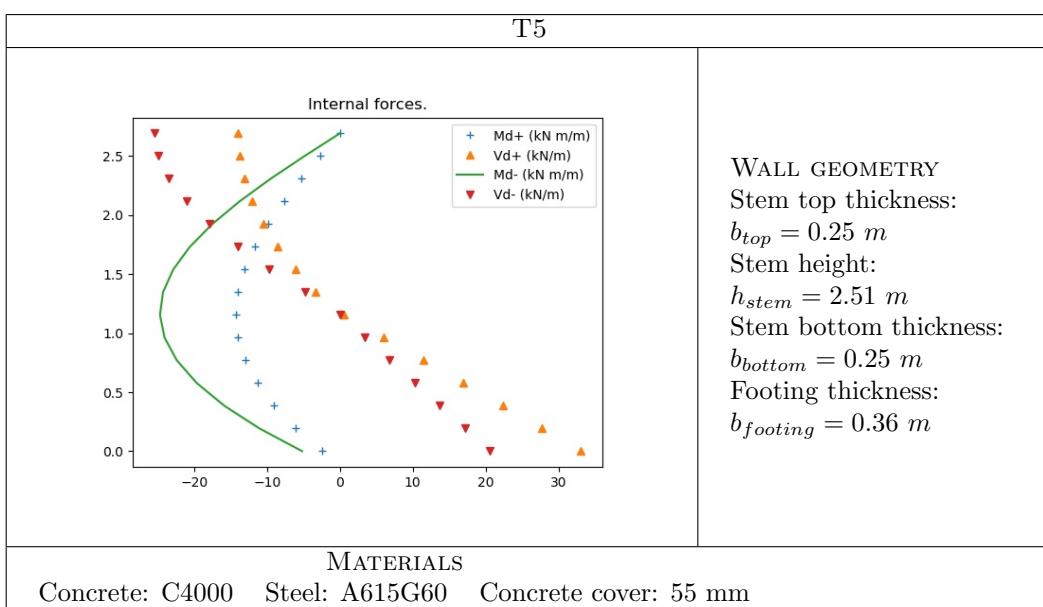


Table 26: Wall materials and dimensions T5

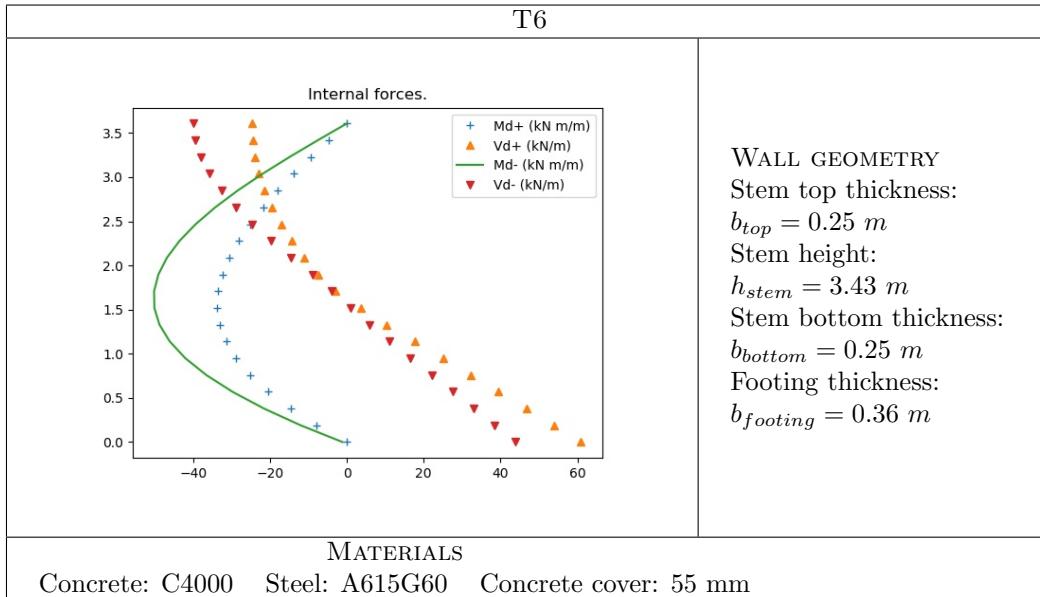


Table 27: Wall materials and dimensions T6

**Reinforcement checks.**

WALL VERTICAL REINFORCEMENTS	
<b>T1 wall. Inside stem reinforcement:</b>	RC section dimensions; $b = 1.00 \text{ m}$ , $h = 0.25 \text{ m}$ diam: 16 mm, spacing: 300 mm reinf. development $L=0.34 \text{ m}$ (22 diameters). area: $As = 6.67 \text{ cm}^2/\text{m}$ areaMin: $4.56 \text{ cm}^2/\text{m}$ $F(As) = 1.46 \text{ OK!}$ Bending check: $Md = 40.09 \text{ kN m}$ , $MR = 41.36 \text{kN m}$ $F(M) = 1.03 \text{ OK!}$ Shear check: $Vd = 7.61 \text{ kN}$ , $VR = 199.37 \text{ kN}$ $F(V) = 26.21 \text{ OK!}$
<b>T2 wall. Inside stem reinforcement:</b>	RC section dimensions; $b = 1.00 \text{ m}$ , $h = 0.25 \text{ m}$ diam: 16 mm, spacing: 400 mm reinf. development $L=0.34 \text{ m}$ (22 diameters). area: $As = 5.00 \text{ cm}^2/\text{m}$ areaMin: $4.56 \text{ cm}^2/\text{m}$ $F(As) = 1.10 \text{ OK!}$ Bending check: $Md = 29.02 \text{ kN m}$ , $MR = 31.02 \text{kN m}$ $F(M) = 1.07 \text{ OK!}$ Shear check: $Vd = 6.84 \text{ kN}$ , $VR = 199.37 \text{ kN}$ $F(V) = 29.15 \text{ OK!}$
<b>T3 wall. Inside stem reinforcement:</b>	RC section dimensions; $b = 1.00 \text{ m}$ , $h = 0.25 \text{ m}$ diam: 19 mm, spacing: 300 mm reinf. development $L=0.61 \text{ m}$ (32 diameters). area: $As = 9.47 \text{ cm}^2/\text{m}$ areaMin: $4.56 \text{ cm}^2/\text{m}$ $F(As) = 2.08 \text{ OK!}$ Bending check: $Md = 51.88 \text{ kN m}$ , $MR = 58.26 \text{kN m}$ $F(M) = 1.12 \text{ OK!}$ Shear check: $Vd = 7.98 \text{ kN}$ , $VR = 199.37 \text{ kN}$ $F(V) = 24.98 \text{ OK!}$
<b>T4 wall. Inside stem reinforcement:</b>	RC section dimensions; $b = 1.00 \text{ m}$ , $h = 0.25 \text{ m}$ diam: 16 mm, spacing: 300 mm reinf. development $L=0.34 \text{ m}$ (22 diameters). area: $As = 6.67 \text{ cm}^2/\text{m}$ areaMin: $4.56 \text{ cm}^2/\text{m}$ $F(As) = 1.46 \text{ OK!}$ Bending check: $Md = 39.19 \text{ kN m}$ , $MR = 41.36 \text{kN m}$ $F(M) = 1.06 \text{ OK!}$
..../..	

WALL VERTICAL REINFORCEMENTS (CONT.)
Shear check: $V_d = 7.46 \text{ kN}$ , $VR = 199.37 \text{ kN}$ $F(V) = 26.73 \text{ OK!}$
<b>T5 wall. Inside stem reinforcement:</b>
RC section dimensions; $b = 1.00 \text{ m}$ , $h = 0.25 \text{ m}$ diam: 16 mm, spacing: 400 mm reinf. development $L = 0.34 \text{ m}$ (22 diameters). area: $A_s = 5.00 \text{ cm}^2/\text{m}$ areaMin: 4.56 $\text{cm}^2/\text{m}$ $F(A_s) = 1.10 \text{ OK!}$ Bending check: $M_d = 23.62 \text{ kN m}$ , $MR = 31.02 \text{kN m}$ $F(M) = 1.31 \text{ OK!}$ Shear check: $V_d = 6.42 \text{ kN}$ , $VR = 199.37 \text{ kN}$ $F(V) = 31.05 \text{ OK!}$
<b>T6 wall. Inside stem reinforcement:</b>
RC section dimensions; $b = 1.00 \text{ m}$ , $h = 0.25 \text{ m}$ diam: 19 mm, spacing: 300 mm reinf. development $L = 0.61 \text{ m}$ (32 diameters). area: $A_s = 9.47 \text{ cm}^2/\text{m}$ areaMin: 4.56 $\text{cm}^2/\text{m}$ $F(A_s) = 2.08 \text{ OK!}$ Bending check: $M_d = 49.02 \text{ kN m}$ , $MR = 58.26 \text{kN m}$ $F(M) = 1.19 \text{ OK!}$ Shear check: $V_d = 8.13 \text{ kN}$ , $VR = 199.37 \text{ kN}$ $F(V) = 24.54 \text{ OK!}$

SHEAR CHECK
<b>T1 wall. Shear check:</b> Shear check: $V_d = 42.99 \text{ kN}$ , $VR = 199.37 \text{ kN}$ $F(V) = 4.64 \text{ OK!}$
<b>T2 wall. Shear check:</b> Shear check: $V_d = 31.78 \text{ kN}$ , $VR = 199.37 \text{ kN}$ $F(V) = 6.27 \text{ OK!}$
<b>T3 wall. Shear check:</b> Shear check: $V_d = 55.00 \text{ kN}$ , $VR = 199.37 \text{ kN}$ $F(V) = 3.63 \text{ OK!}$
<b>T4 wall. Shear check:</b> Shear check: $V_d = 42.35 \text{ kN}$ , $VR = 199.37 \text{ kN}$ $F(V) = 4.71 \text{ OK!}$
<b>T5 wall. Shear check:</b> Shear check: $V_d = 26.03 \text{ kN}$ , $VR = 199.37 \text{ kN}$ $F(V) = 7.66 \text{ OK!}$
<b>T6 wall. Shear check:</b> Shear check: $V_d = 51.43 \text{ kN}$ , $VR = 199.37 \text{ kN}$ $F(V) = 3.88 \text{ OK!}$

**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 ( $143.64 \text{ kN/m}^2$ ).

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

## 9.4 Ramp

### 9.4.1 Design criteria

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

### 9.4.2 Acceptance criteria

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

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

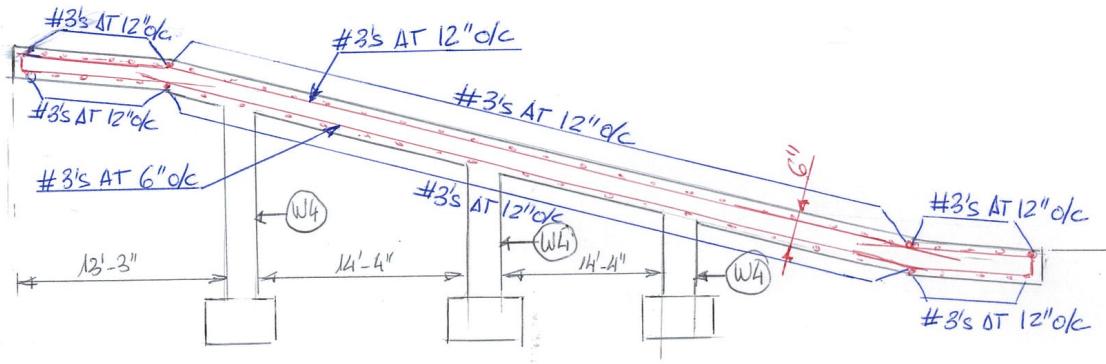


Figure 35: Ramp. Reinforcing layout.

## CALCULATION REPORT

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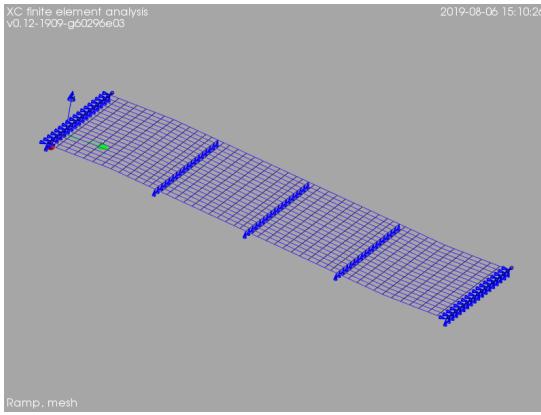


Figure 36: Ramp elastic model, mesh.

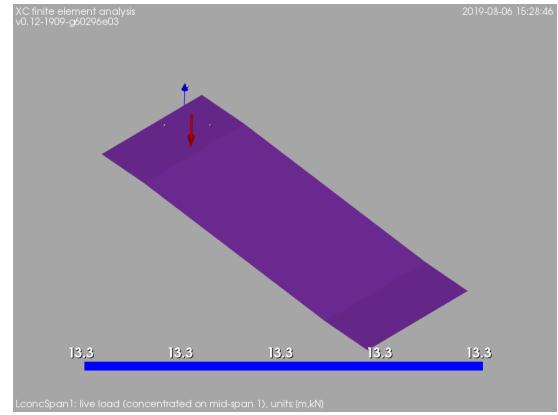


Figure 39: Load case  $Lv_{conc,s1}$ : concentrated live load (vehicles) on mid-span 1 [units: kN].

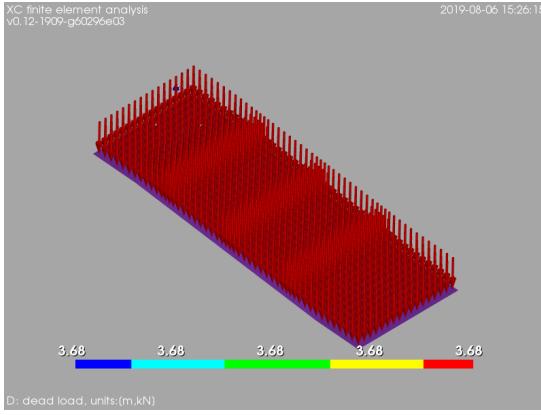


Figure 37: Load case D: dead load (include slab self-weight) [units:  $kN/m^2$ ].

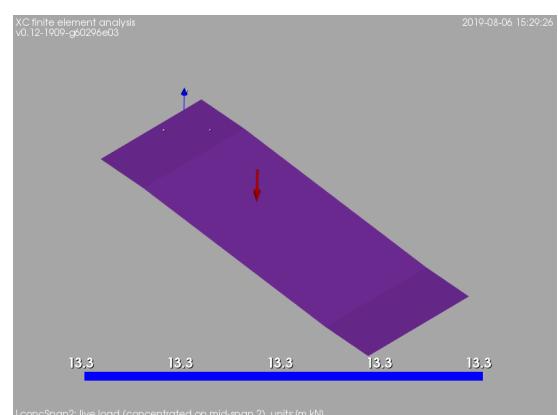


Figure 40: Load case  $Lv_{conc,s2}$ : concentrated live load (vehicles) on mid-span 2 [units: kN].

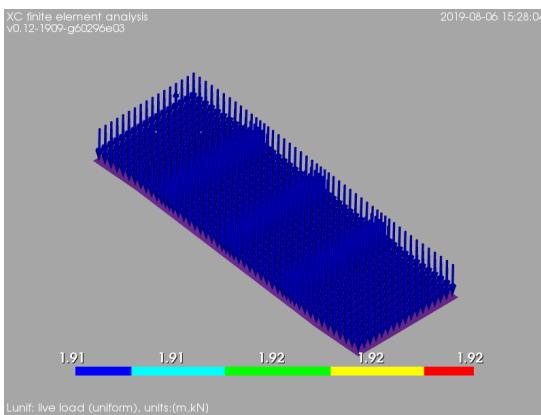


Figure 38: Load case  $Lv_{unif}$ : uniform live load (vehicles) [units:  $kN/m^2$ ].

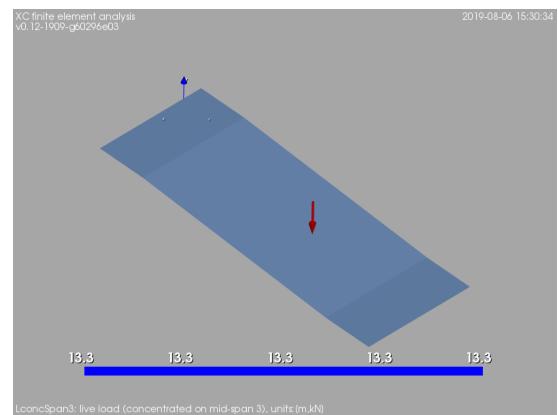


Figure 41: Load case  $Lv_{conc,s3}$ : concentrated live load (vehicles) on mid-span 3 [units: kN].

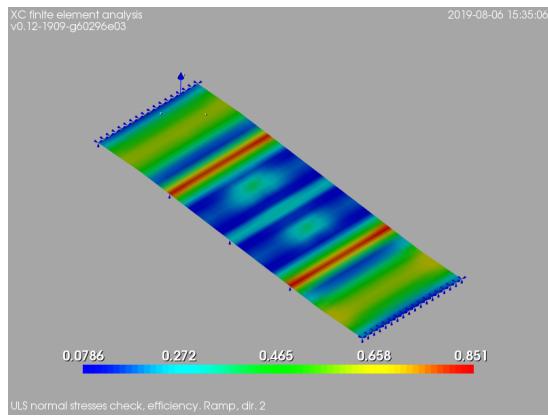


Figure 42: ULS normal stresses check. Efficiency in longitudinal direction

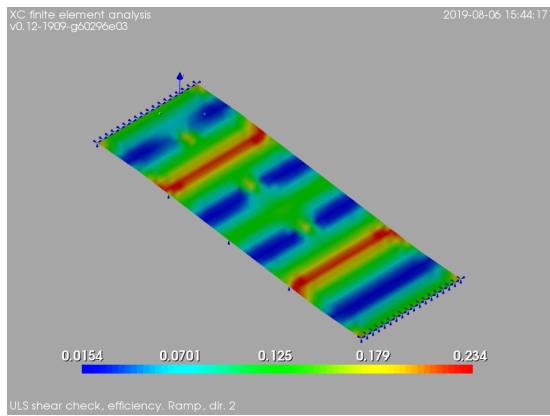


Figure 45: ULS shear check. Efficiency in longitudinal direction

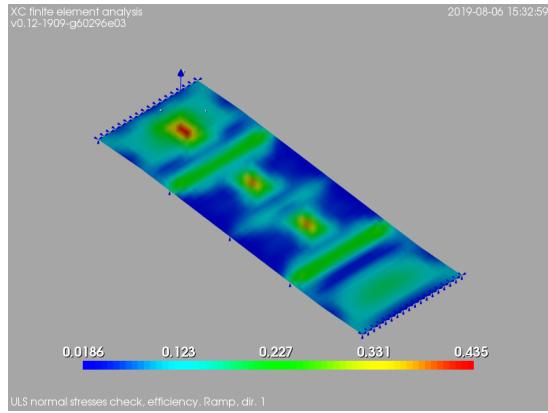


Figure 43: ULS normal stresses check. Efficiency in transversal direction

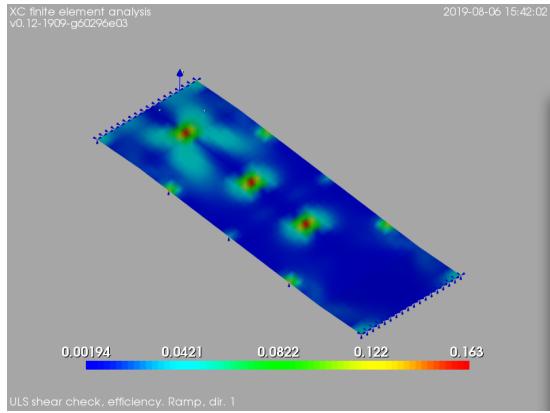


Figure 46: ULS shear check. Efficiency in transversal direction

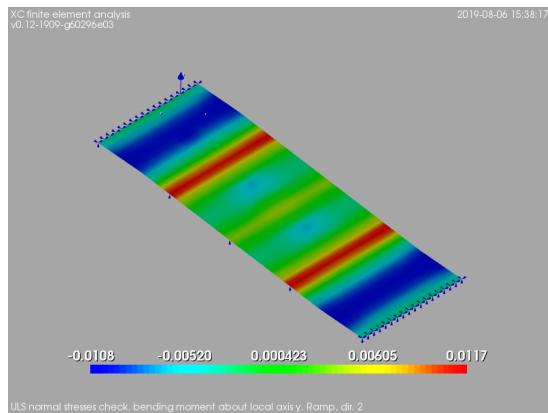


Figure 44: ULS normal stresses check. Bending moment in longitudinal direction [units: kN.m].

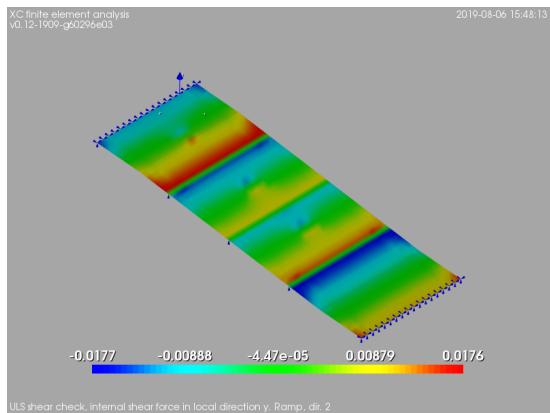


Figure 47: ULS shear check. Internal shear force in longitudinal direction [units: kN].

## CALCULATION REPORT

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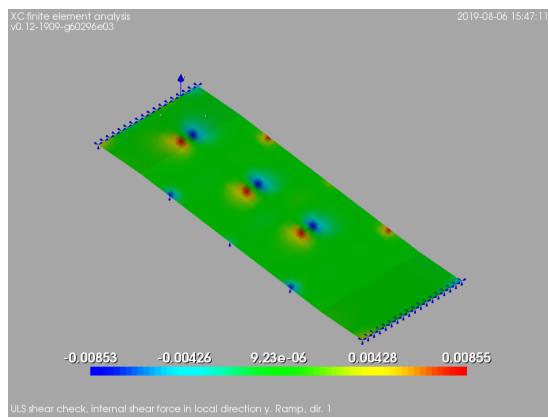


Figure 48: ULS shear check. Internal shear force in transversal direction [units: kN].

# Appendices

## A Loading criteria

### A.1 Dead loads

#### Materials

Wood structural panel	$36.0 \text{ pcf} = 5655 \frac{\text{newton}}{\text{meter}^3}$
Concrete reinforced stone (including gravel)	$150.0 \text{ pcf} = 23563 \frac{\text{newton}}{\text{meter}^3}$
Steel	$489.0 \text{ pcf} = 76816 \frac{\text{newton}}{\text{meter}^3}$
Gypsum crete	$115.0 \text{ pcf} = 18065 \frac{\text{newton}}{\text{meter}^3}$
Gypsum, loose	$70.0 \text{ pcf} = 10996 \frac{\text{newton}}{\text{meter}^3}$
Earth (not submerged) sand and gravel (wet)	$120.0 \text{ pcf} = 18850 \frac{\text{newton}}{\text{meter}^3}$
Water	$62.4 \text{ pcf} = 9802 \frac{\text{newton}}{\text{meter}^3}$
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 $38.4\text{in} \times 102\text{in}$	3.2 psf = 153 pascal
Fiber cement panels, small scale $9.6\text{in} \times 102\text{in}$	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
Sheathing	
Roof sheathing	3.5 psf = 167 pascal
Floor sheathing	2.5 psf = 120 pascal
Ceilings	2.5 psf = 120 pascal
Deck composite sleepers (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) + maintenance	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	$p_g = 60.0 \text{ psf} = 2873 \text{ pascal}$	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
<b>Snow load flat roof</b>	$p_f = 0.7 \times C_e \times C_t \times I_s \times p_g = 0.7 \times 1.0 \times 1.0 \times 1.0 \times 60.0 = 42.0 \text{ psf} = 2873 \text{ pascal}$	ASCE 7. Sect. 7.3

#### A.4 Wind loads

Alternate all-heights method.

$$\text{Ultimate design wind speed } V_{ult} = 115 \frac{\text{miles}}{\text{hour}} = 51 \frac{\text{meters}}{\text{second}}$$

$$\text{Velocity pressure exposure coefficient } K_z = 0.72$$

$$\text{Topographic factor } K_{zt} = 1.0$$

*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*

*IBC-2018, figure 1609.3(1). Risk category II building*

*ASCE 7, table 27.3.1. Exposure B, height above ground level  $z \approx 33$  feet*

*ASCE 7, sect. 26.8*

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

Description	$C_{net} + \text{Internal pressure}$	$C_{net} - \text{Internal pressure}$
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

*IBC-2018, Table 1609.6.2, enclosed*

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

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

Description	$P_{net} + \text{Internal pressure}$	$P_{net} - \text{Internal pressure}$
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

*IBC-2018, sect. 1609.6.3*

### 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		$S_1 \leq 0.04 \text{ and } S_s \leq 0.15 \rightarrow \text{SDS A}$	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
Design spectral response acceleration parameters		$S_{M1} = F_a \cdot S_1 = 0.038$ $S_{DS} = \frac{2}{3}S_{MS} = 0.03$ $S_{D1} = \frac{2}{3}S_{M1} = 0.025$	IBC-2018, sect. 163.3.4 IBC-2018, sect. 163.3.4

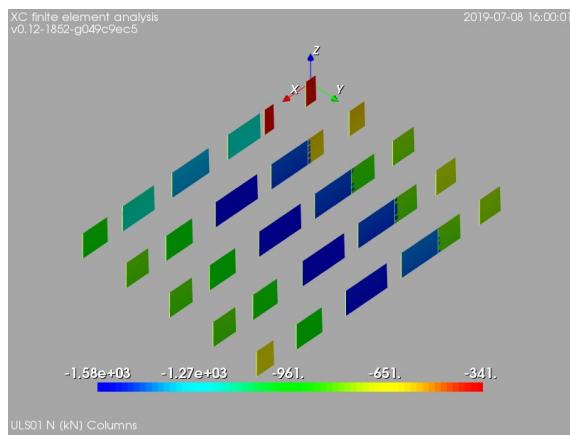


Figure 49: ULS01: 1.4\*D. Columns, internal axial force [kN]

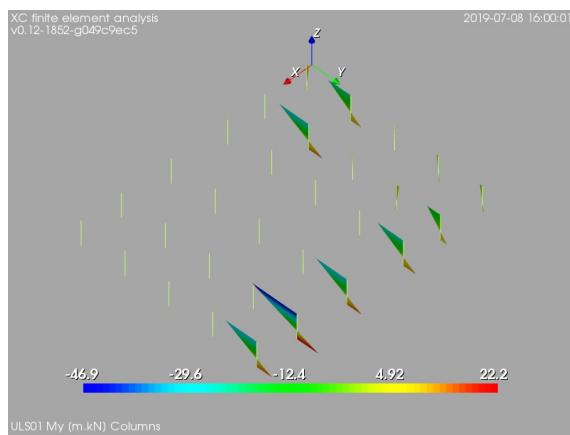


Figure 50: ULS01: 1.4\*D. Columns, bending moment about local axis y [m.kN]

## B Calculation results. Internal forces on columns

### B.1 Ultimate limit states

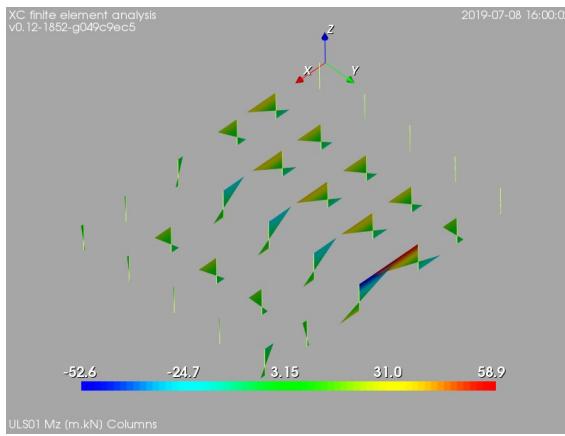


Figure 51: ULS01: 1.4\*D. Columns, bending moment about local axis  $z$  [m.kN]

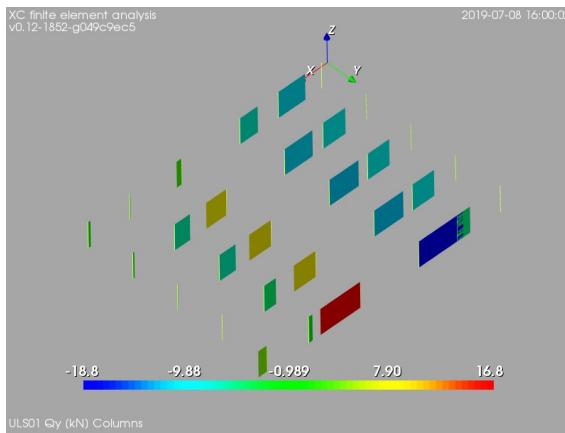


Figure 52: ULS01: 1.4\*D. Columns, internal shear force in local direction  $y$  [kN]

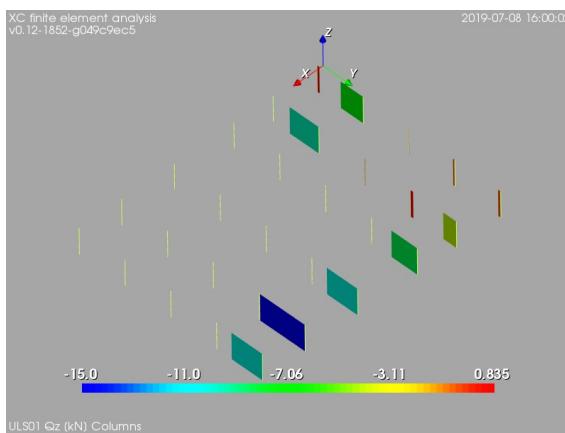


Figure 53: ULS01: 1.4\*D. Columns, internal shear force in local direction  $z$  [kN]

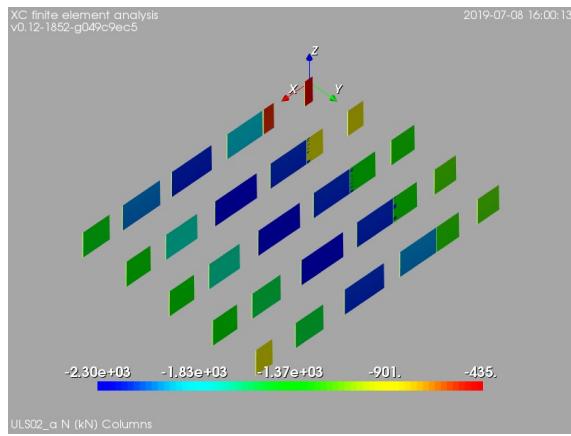


Figure 54: ULS02\_a:  $1.2*D + 1.6*Lru + Lpu + 0.5*S$ . Columns, internal axial force [kN]

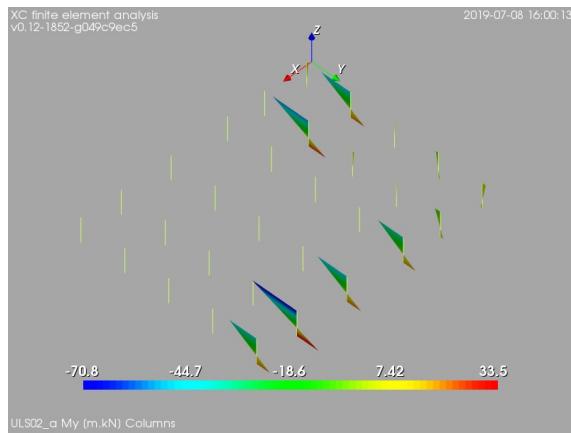


Figure 55: ULS02\_a:  $1.2*D + 1.6*Lru + Lpu + 0.5*S$ . Columns, bending moment about local axis y [m.kN]

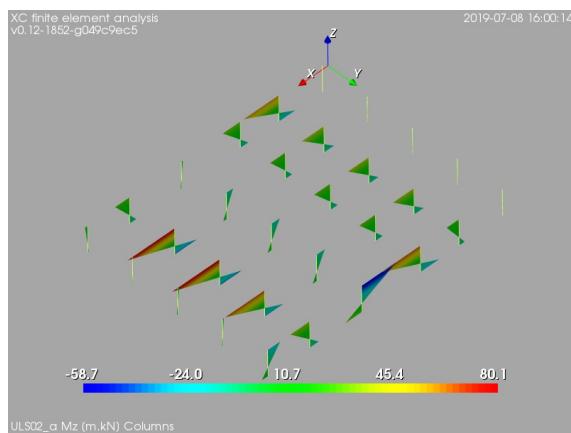


Figure 56: ULS02\_a:  $1.2*D + 1.6*Lru + Lpu + 0.5*S$ . Columns, bending moment about local axis z [m.kN]

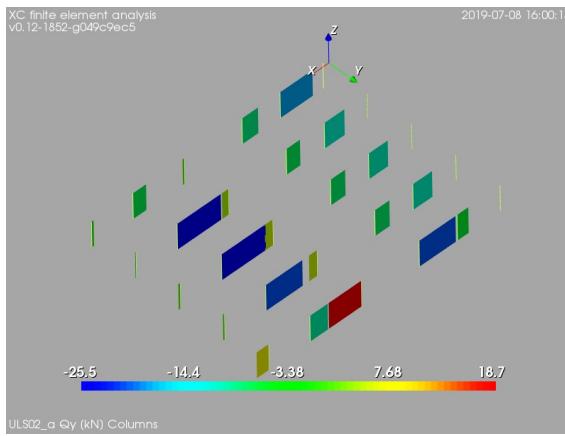


Figure 57: ULS02\_a:  $1.2*D + 1.6*Lru + Lpu + 0.5*S$ . Columns, internal shear force in local direction y [kN]

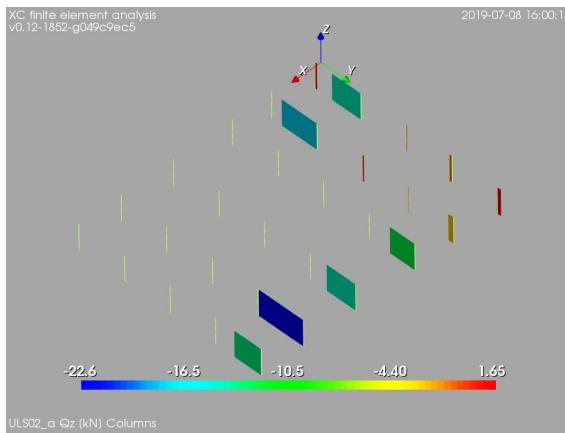


Figure 58: ULS02\_a:  $1.2*D + 1.6*Lru + Lpu + 0.5*S$ . Columns, internal shear force in local direction z [kN]

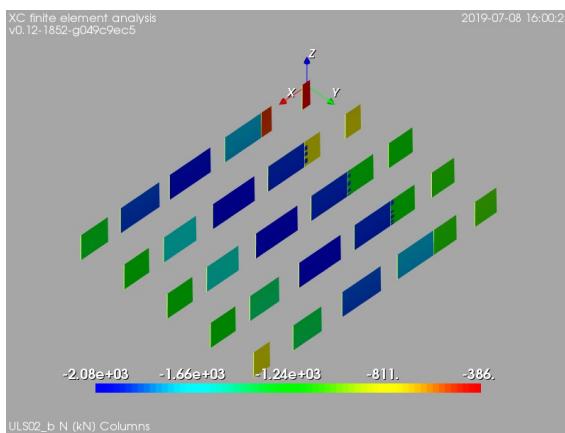


Figure 59: ULS02\_b:  $1.2*D + 1.6*Lrs + Lps + 0.5*S$ . Columns, internal axial force [kN]

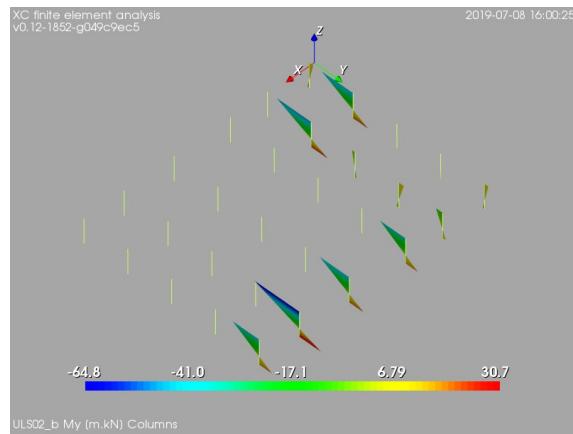


Figure 60: ULS02\_b:  $1.2*D + 1.6*Lrs + Lps + 0.5*S$ . Columns, bending moment about local axis y [m.kN]

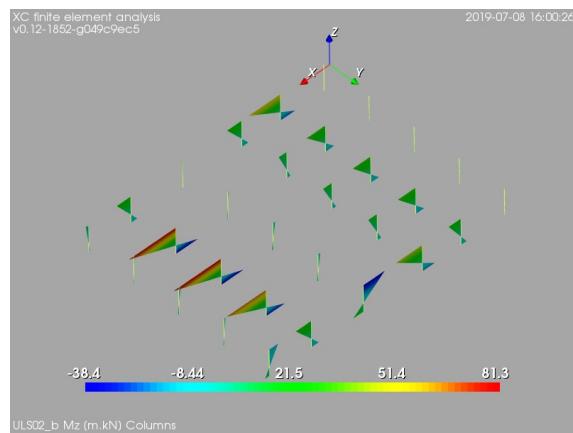


Figure 61: ULS02\_b:  $1.2*D + 1.6*Lrs + Lps + 0.5*S$ . Columns, bending moment about local axis z [m.kN]

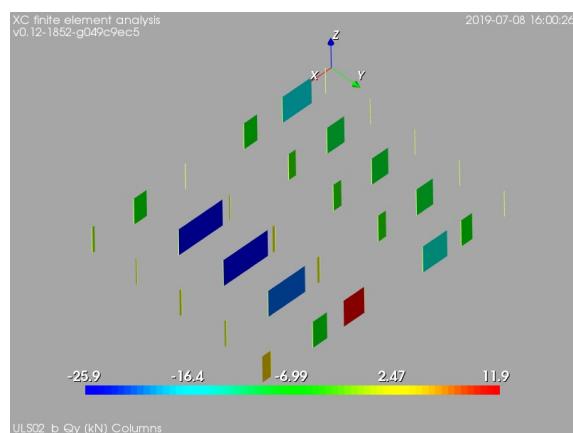


Figure 62: ULS02\_b:  $1.2*D + 1.6*Lrs + Lps + 0.5*S$ . Columns, internal shear force in local direction y [kN]

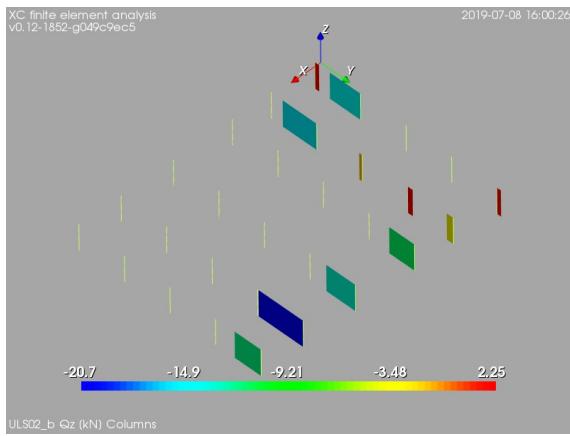


Figure 63: ULS02\_b:  $1.2*D + 1.6*Lrs + Lps + 0.5*S$ . Columns, internal shear force in local direction z [kN]

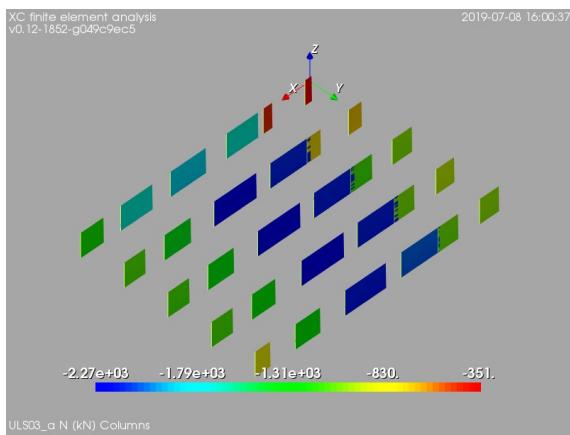


Figure 64: ULS03\_a:  $1.2*D + 1.6*S + 0.5*Lru + Lpu$ . Columns, internal axial force [kN]

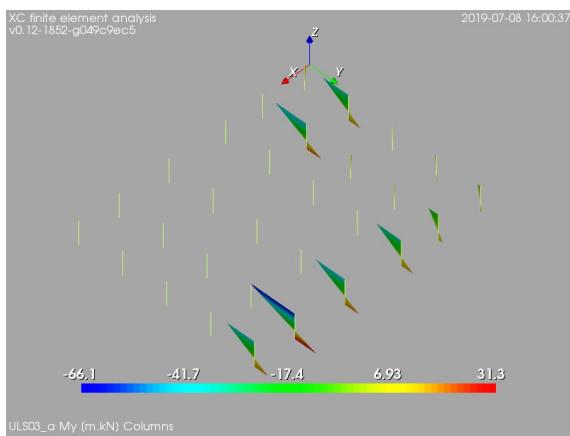


Figure 65: ULS03\_a:  $1.2*D + 1.6*S + 0.5*Lru + Lpu$ . Columns, bending moment about local axis y [m.kN]

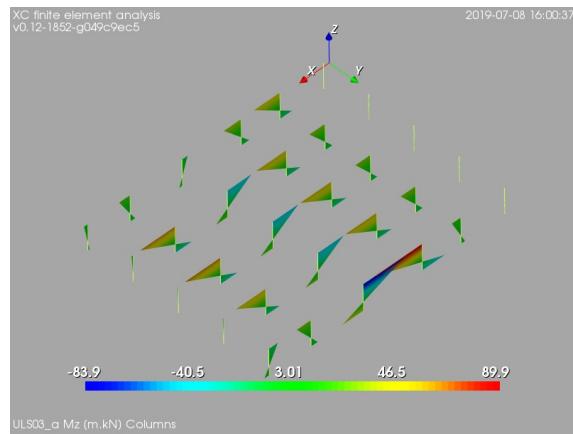


Figure 66: ULS03\_a:  $1.2*D + 1.6*S + 0.5*Lru + Lpu$ . Columns, bending moment about local axis z [m.kN]

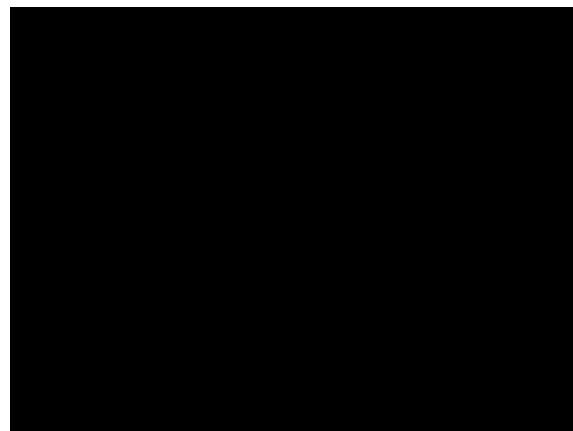


Figure 67: ULS03\_a:  $1.2*D + 1.6*S + 0.5*Lru + Lpu$ . Columns, internal shear force in local direction y [kN]

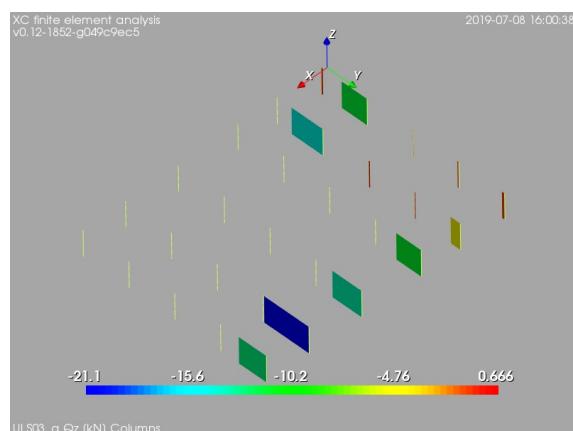


Figure 68: ULS03\_a:  $1.2*D + 1.6*S + 0.5*Lru + Lpu$ . Columns, internal shear force in local direction z [kN]

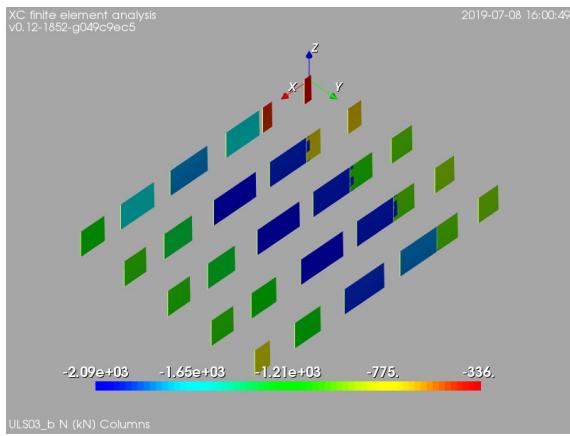


Figure 69: ULS03\_b:  $1.2*D + 1.6*S + 0.5*Lrs + Lps$ . Columns, internal axial force [kN]

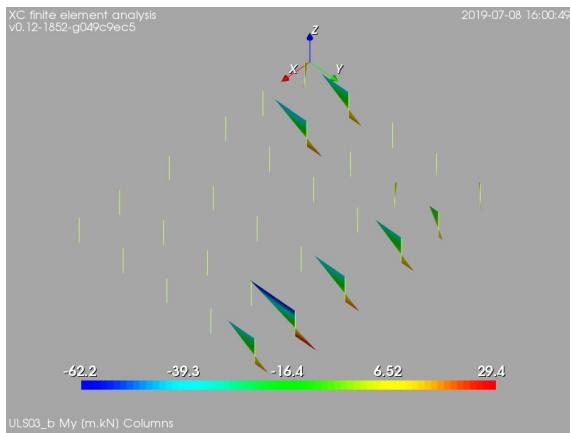


Figure 70: ULS03\_b:  $1.2*D + 1.6*S + 0.5*Lrs + Lps$ . Columns, bending moment about local axis y [m.kN]

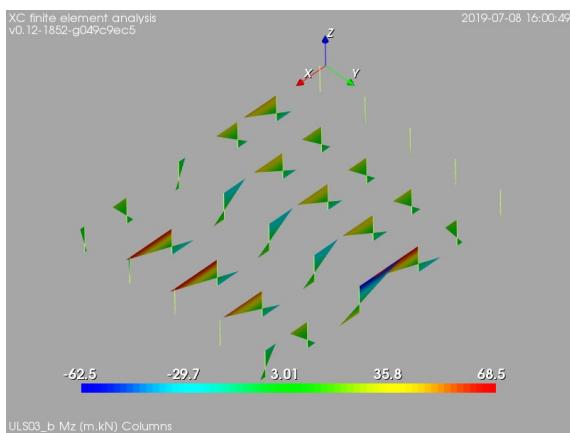


Figure 71: ULS03\_b:  $1.2*D + 1.6*S + 0.5*Lrs + Lps$ . Columns, bending moment about local axis z [m.kN]

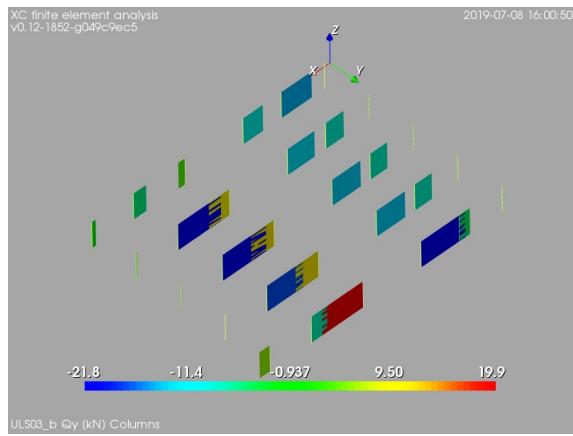


Figure 72: ULS03\_b:  $1.2*D + 1.6*S + 0.5*Lrs + Lps$ . Columns, internal shear force in local direction y [kN]

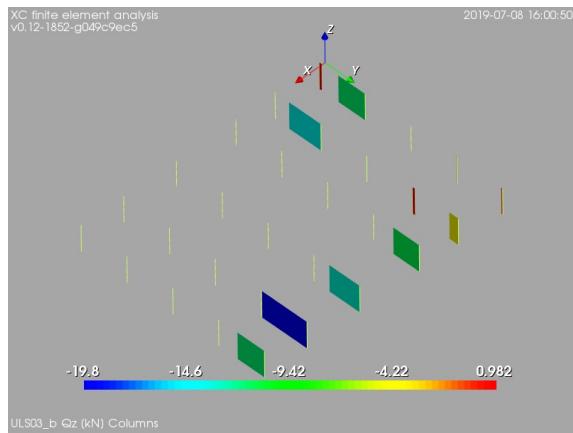


Figure 73: ULS03\_b:  $1.2*D + 1.6*S + 0.5*Lrs + Lps$ . Columns, internal shear force in local direction z [kN]

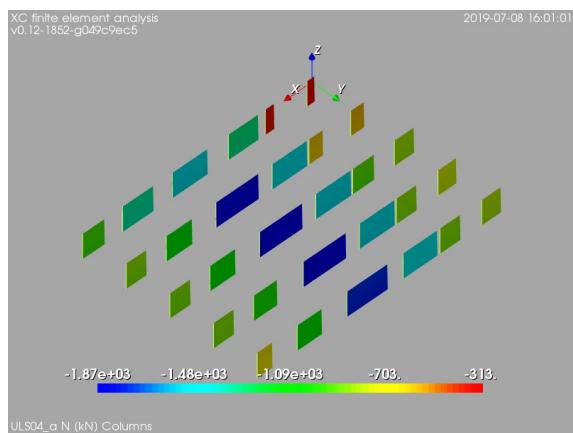


Figure 74: ULS04\_a:  $1.2*D + 1.6*S + 0.5*W\_WE$ . Columns, internal axial force [kN]

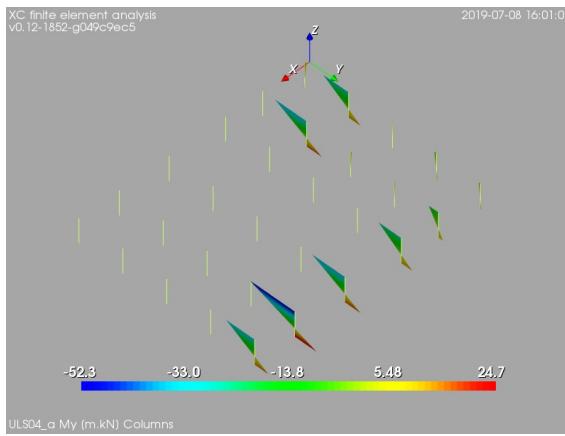


Figure 75: ULS04\_a:  $1.2*D + 1.6*S + 0.5*W\_WE$ . Columns, bending moment about local axis  $y$  [ $\text{m.kN}$ ]

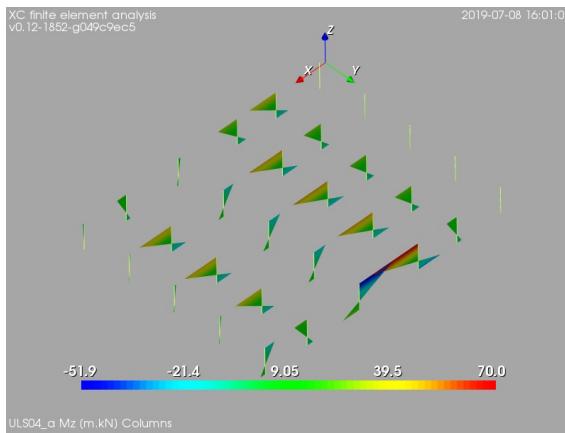


Figure 76: ULS04\_a:  $1.2*D + 1.6*S + 0.5*W\_WE$ . Columns, bending moment about local axis  $z$  [ $\text{m.kN}$ ]

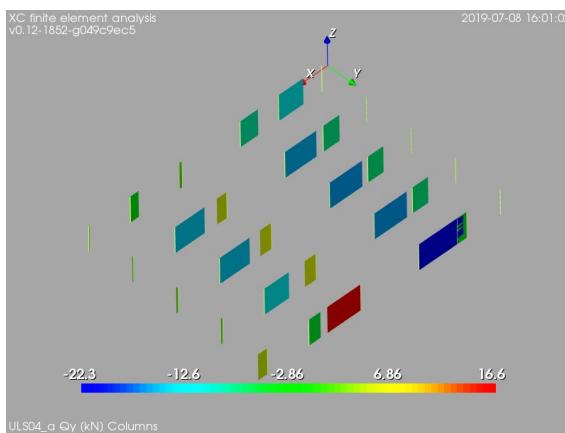


Figure 77: ULS04\_a:  $1.2*D + 1.6*S + 0.5*W\_WE$ . Columns, internal shear force in local direction  $y$  [ $\text{kN}$ ]

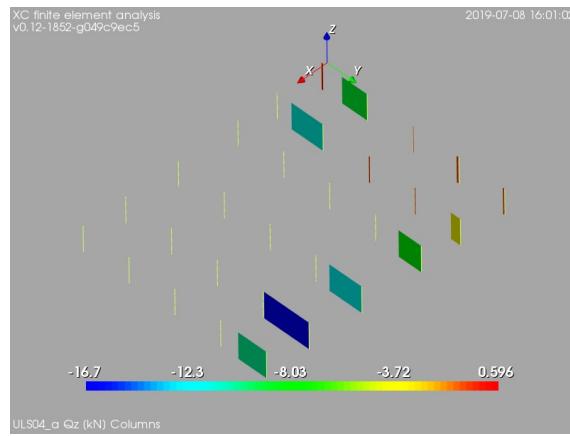


Figure 78: ULS04\_a:  $1.2*D + 1.6*S + 0.5*W\_WE$ . Columns, internal shear force in local direction z [kN]

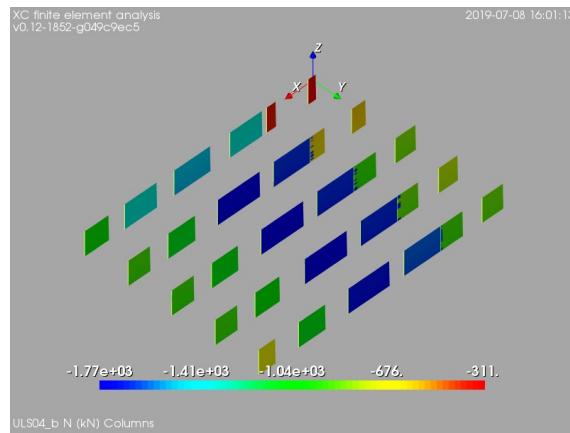


Figure 79: ULS04\_b:  $1.2*D + 1.6*S + 0.5*W\_NS$ . Columns, internal axial force [kN]

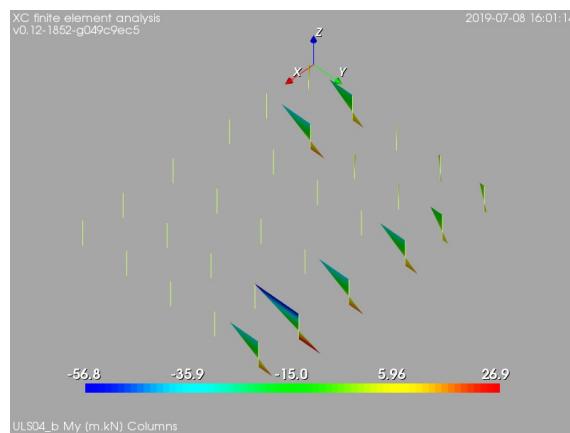


Figure 80: ULS04\_b:  $1.2*D + 1.6*S + 0.5*W\_NS$ . Columns, bending moment about local axis y [m.kN]

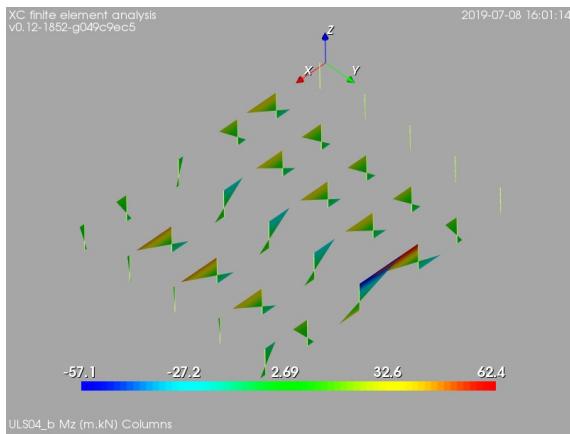


Figure 81: ULS04\_b:  $1.2*D + 1.6*S + 0.5*W_{NS}$ . Columns, bending moment about local axis z [m.kN]

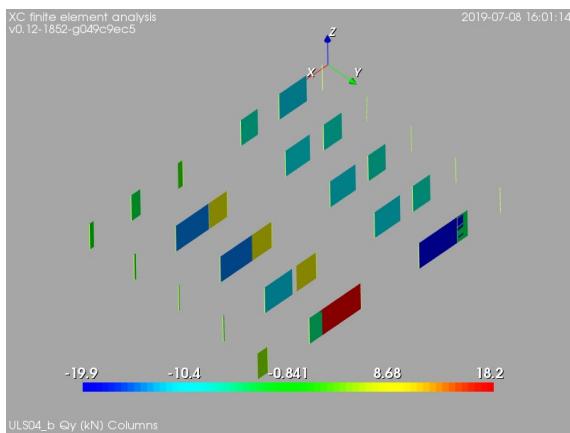


Figure 82: ULS04\_b:  $1.2*D + 1.6*S + 0.5*W_{NS}$ . Columns, internal shear force in local direction y [kN]

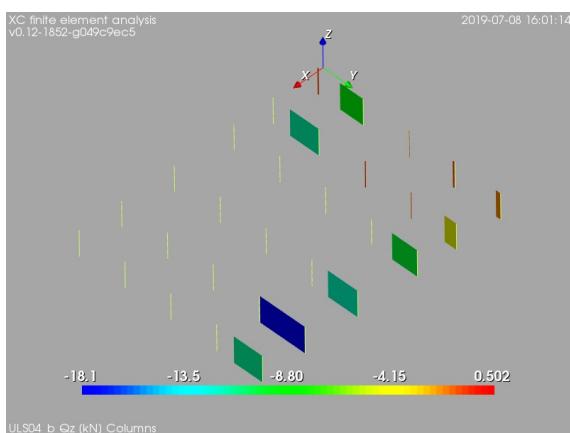


Figure 83: ULS04\_b:  $1.2*D + 1.6*S + 0.5*W_{NS}$ . Columns, internal shear force in local direction z [kN]

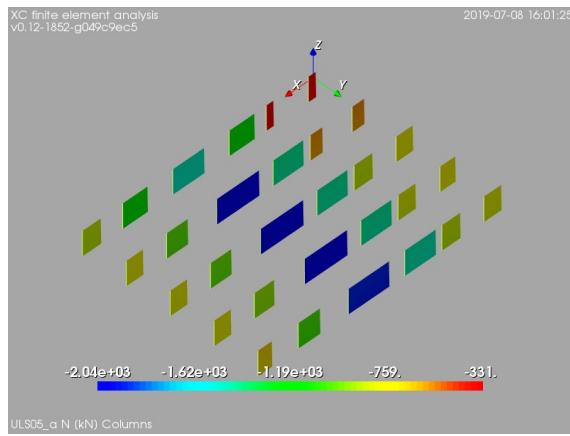


Figure 84: ULS05\_a: 1.2\*D + W\_WE + 0.5\*Lru + Lpu. Columns, internal axial force [kN]

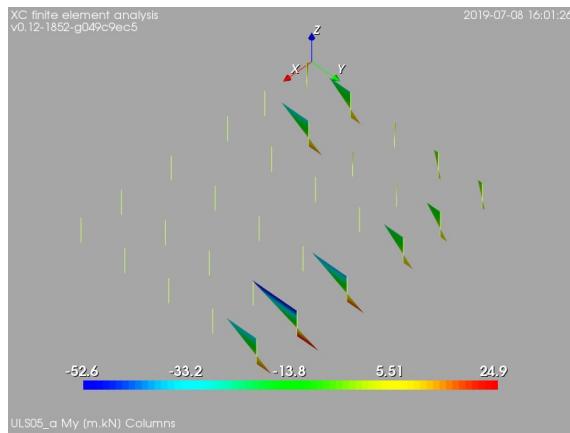


Figure 85: ULS05\_a: 1.2\*D + W\_WE + 0.5\*Lru + Lpu. Columns, bending moment about local axis y [m.kN]

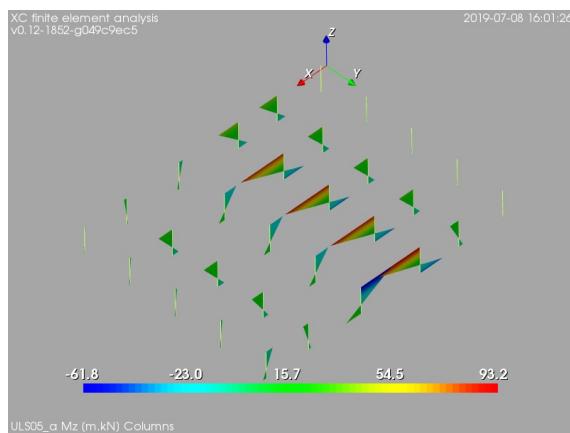


Figure 86: ULS05\_a: 1.2\*D + W\_WE + 0.5\*Lru + Lpu. Columns, bending moment about local axis z [m.kN]

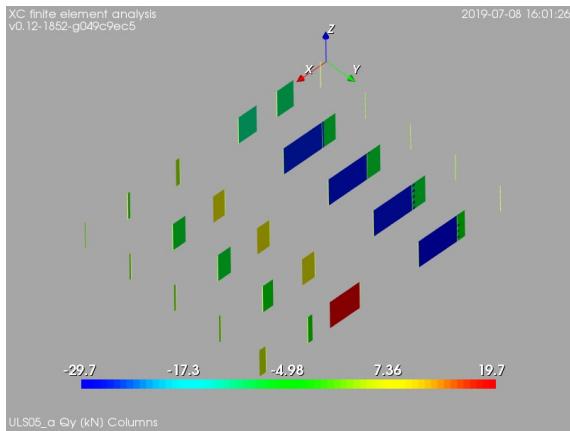


Figure 87: ULS05\_a: 1.2\*D + W\_WE + 0.5\*Lru + Lpu. Columns, internal shear force in local direction y [kN]

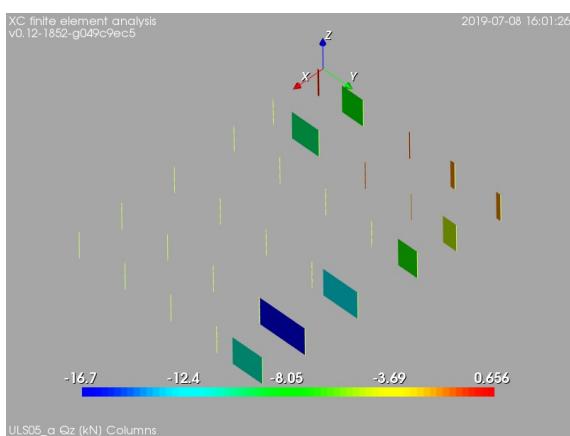


Figure 88: ULS05\_a: 1.2\*D + W\_WE + 0.5\*Lru + Lpu. Columns, internal shear force in local direction z [kN]

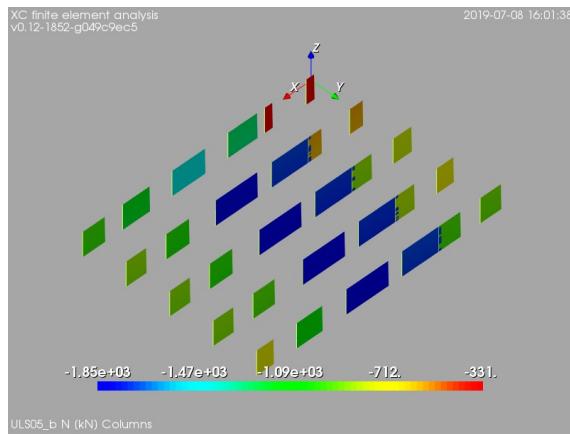


Figure 89: ULS05\_b:  $1.2*D + W_{NS} + 0.5*L_{ru} + L_{pu}$ . Columns, internal axial force [kN]

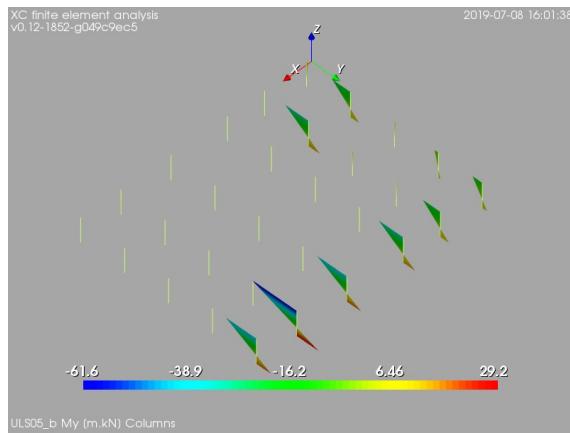


Figure 90: ULS05\_b:  $1.2*D + W_{NS} + 0.5*L_{ru} + L_{pu}$ . Columns, bending moment about local axis y [m.kN]

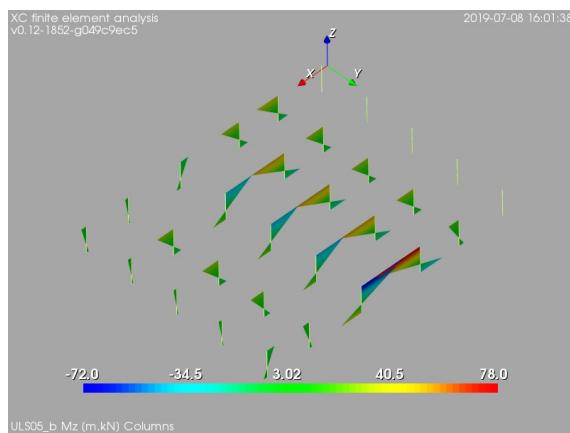


Figure 91: ULS05\_b:  $1.2*D + W_{NS} + 0.5*L_{ru} + L_{pu}$ . Columns, bending moment about local axis z [m.kN]

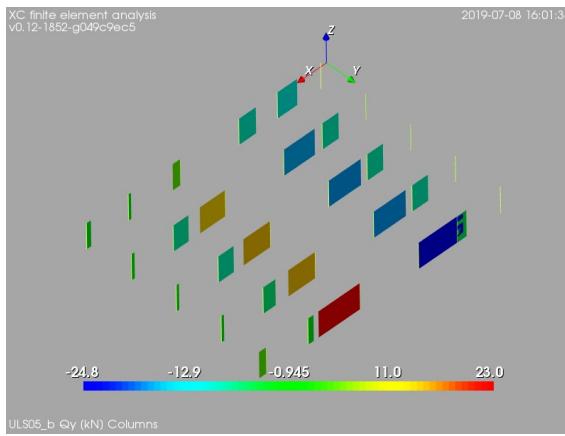


Figure 92: ULS05\_b: 1.2\*D + W\_NS + 0.5\*Lru + Lpu. Columns, internal shear force in local direction y [kN]

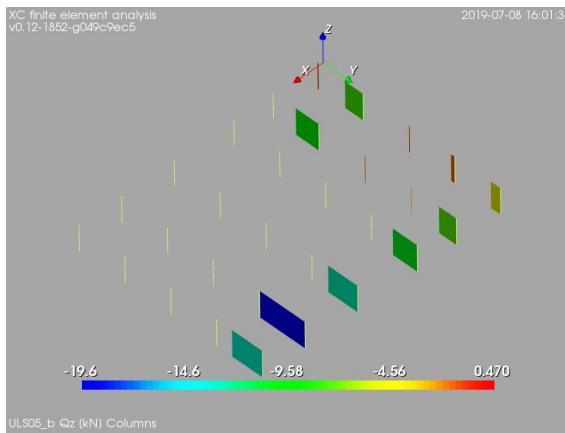


Figure 93: ULS05\_b: 1.2\*D + W\_NS + 0.5\*Lru + Lpu. Columns, internal shear force in local direction z [kN]

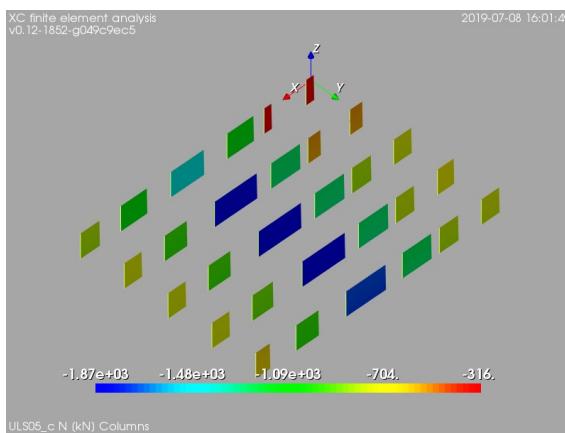


Figure 94: ULS05\_c: 1.2\*D + W\_WE + 0.5\*Lrs + Lps. Columns, internal axial force [kN]

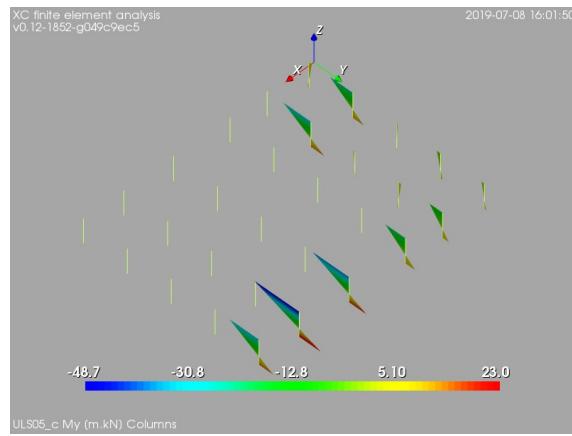


Figure 95: ULS05\_c: 1.2\*D + W\_WE + 0.5\*Lrs + Lps. Columns, bending moment about local axis y [m.kN]

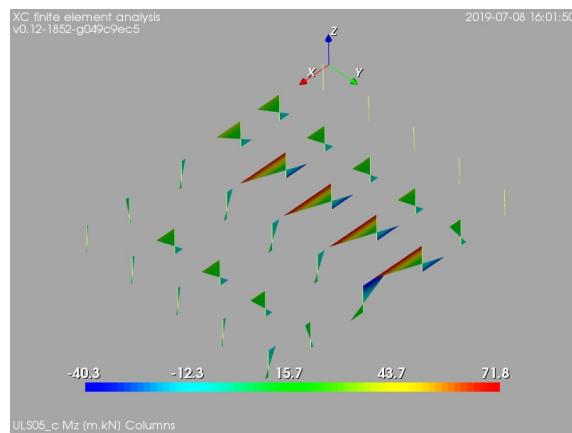


Figure 96: ULS05\_c: 1.2\*D + W\_WE + 0.5\*Lrs + Lps. Columns, bending moment about local axis z [m.kN]

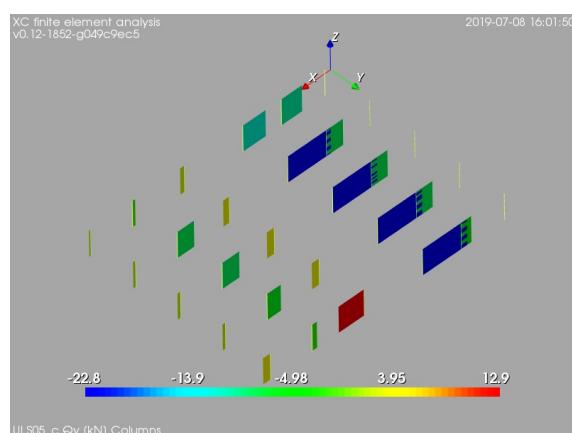


Figure 97: ULS05\_c: 1.2\*D + W\_WE + 0.5\*Lrs + Lps. Columns, internal shear force in local direction y [kN]

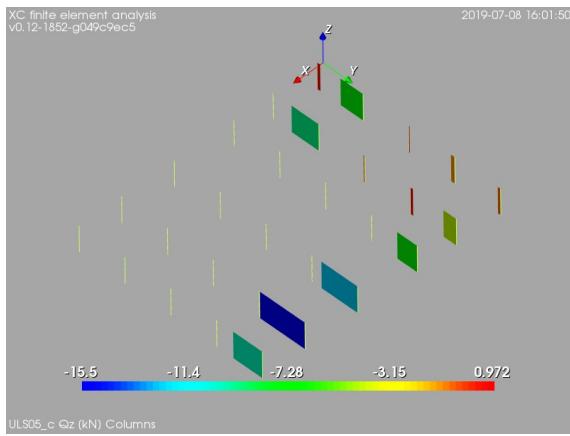


Figure 98: ULS05\_c:  $1.2*D + W\_WE + 0.5*Lrs + Lps$ . Columns, internal shear force in local direction z [kN]

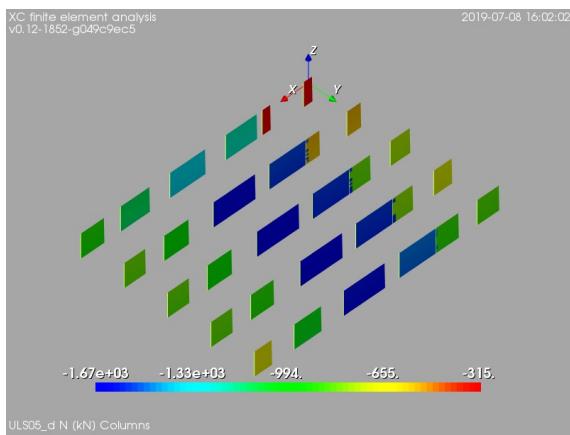


Figure 99: ULS05\_d:  $1.2*D + W\_NS + 0.5*Lrs + Lps$ . Columns, internal axial force [kN]

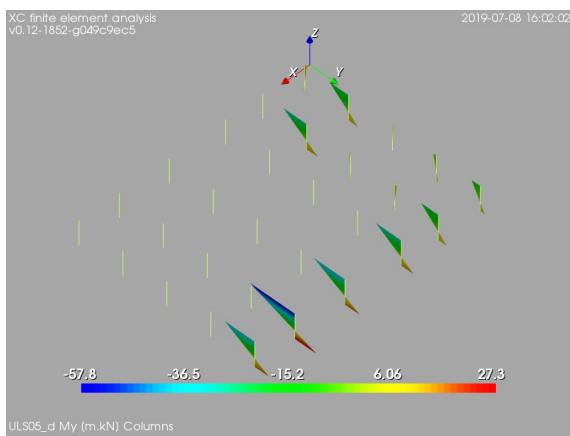


Figure 100: ULS05\_d:  $1.2*D + W\_NS + 0.5*Lrs + Lps$ . Columns, bending moment about local axis y [m.kN]

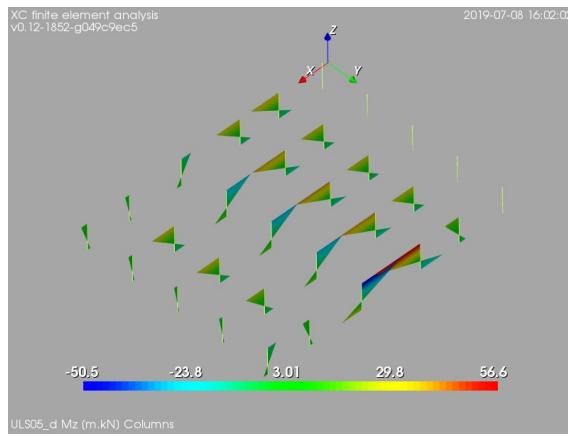


Figure 101: ULS05\_d: 1.2\*D + W\_NS + 0.5\*Lrs + Lps. Columns, bending moment about local axis z [m.kN]

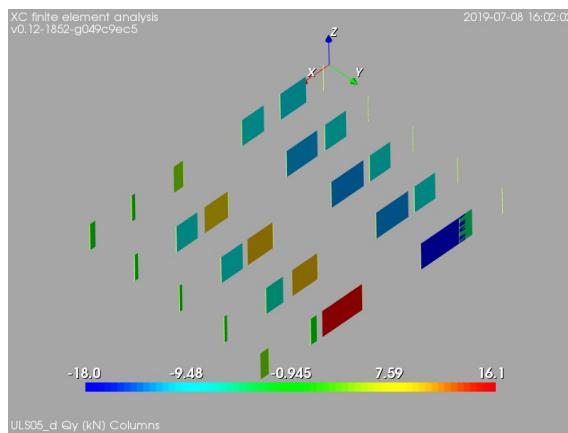


Figure 102: ULS05\_d: 1.2\*D + W\_NS + 0.5\*Lrs + Lps. Columns, internal shear force in local direction y [kN]

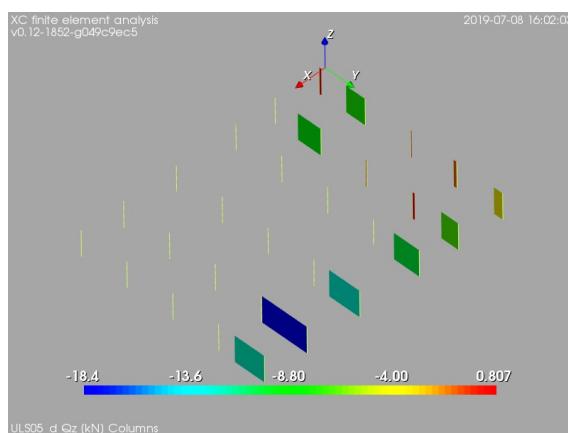


Figure 103: ULS05\_d: 1.2\*D + W\_NS + 0.5\*Lrs + Lps. Columns, internal shear force in local direction z [kN]

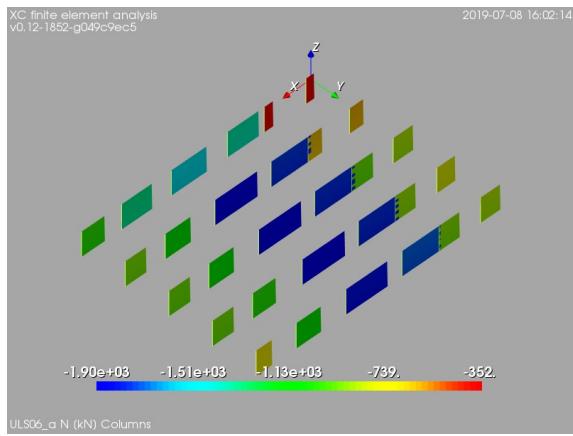


Figure 104: ULS06\_a:  $1.2*D + 0.5*Lru + Lpu + 0.2*S$ . Columns, internal axial force [kN]

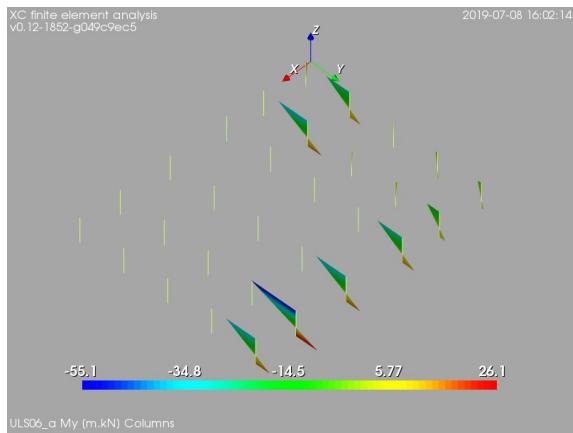


Figure 105: ULS06\_a:  $1.2*D + 0.5*Lru + Lpu + 0.2*S$ . Columns, bending moment about local axis y [m.kN]

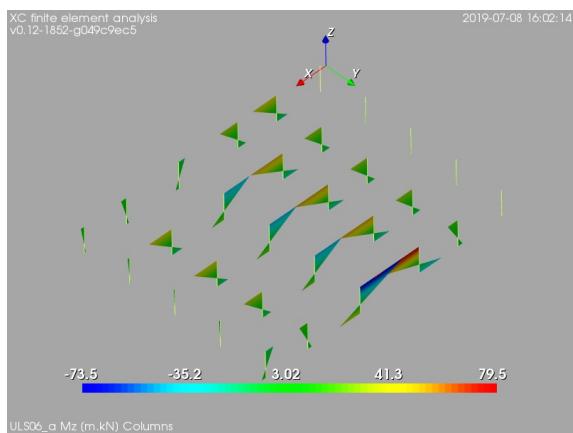


Figure 106: ULS06\_a:  $1.2*D + 0.5*Lru + Lpu + 0.2*S$ . Columns, bending moment about local axis z [m.kN]

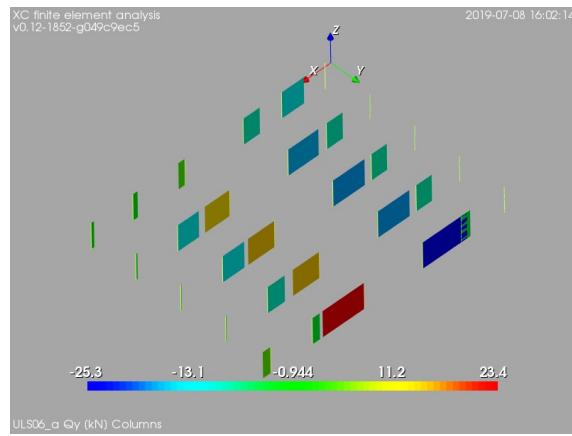


Figure 107: ULS06\_a: 1.2\*D + 0.5\*Lru + Lpu + 0.2\*S. Columns, internal shear force in local direction y [kN]

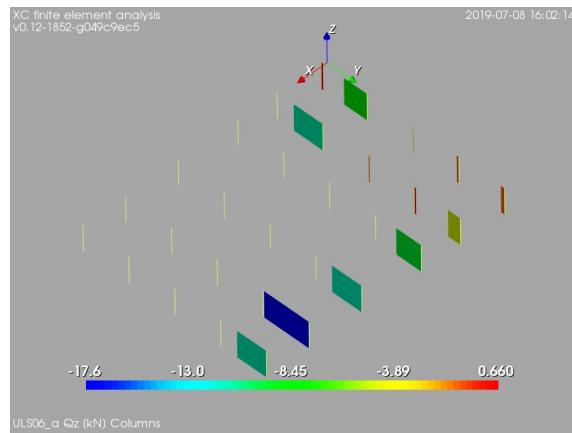


Figure 108: ULS06\_a: 1.2\*D + 0.5\*Lru + Lpu + 0.2\*S. Columns, internal shear force in local direction z [kN]

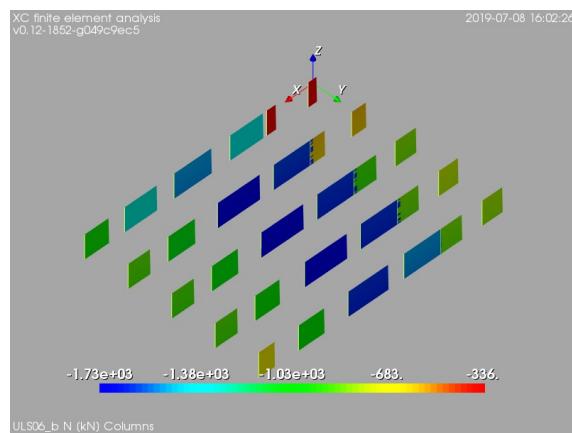


Figure 109: ULS06\_b: 1.2\*D + 0.5\*Lrs + Lps + 0.2\*S. Columns, internal axial force [kN]

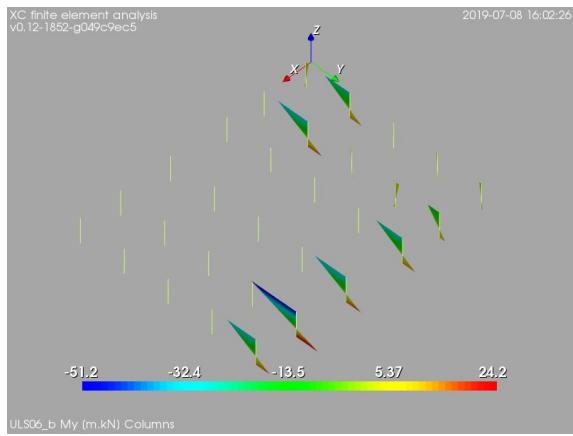


Figure 110: ULS06\_b:  $1.2*D + 0.5*Lrs + Lps + 0.2*S$ . Columns, bending moment about local axis y [m.kN]

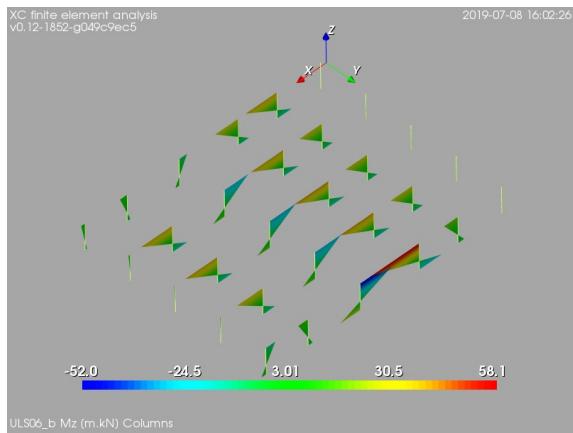


Figure 111: ULS06\_b:  $1.2*D + 0.5*Lrs + Lps + 0.2*S$ . Columns, bending moment about local axis z [m.kN]

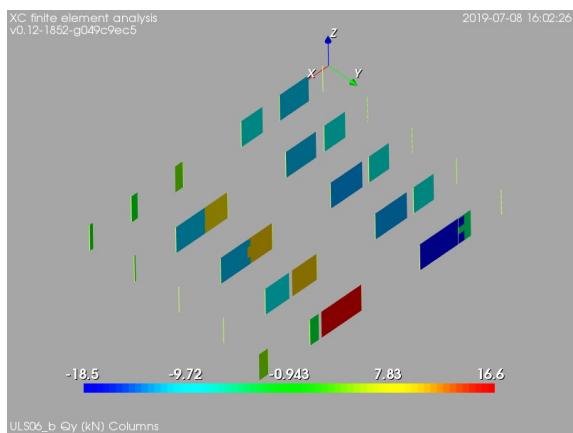


Figure 112: ULS06\_b:  $1.2*D + 0.5*Lrs + Lps + 0.2*S$ . Columns, internal shear force in local direction y [kN]

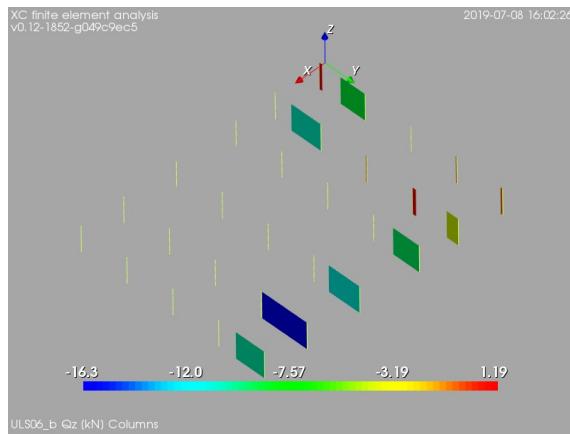


Figure 113: ULS06\_b:  $1.2*D + 0.5*Lrs + Lps + 0.2*S$ . Columns, internal shear force in local direction z [kN]

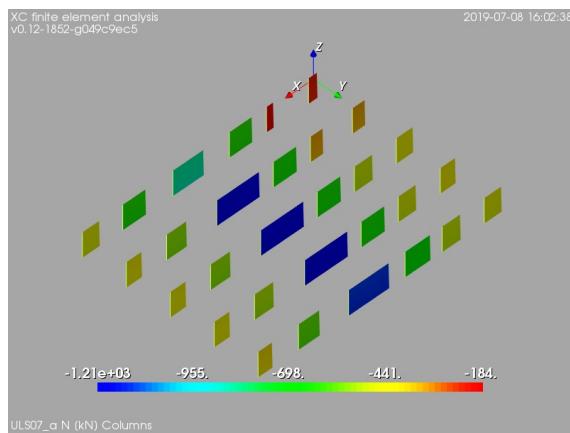


Figure 114: ULS07\_a:  $0.9*D + W\_WE$ . Columns, internal axial force [kN]

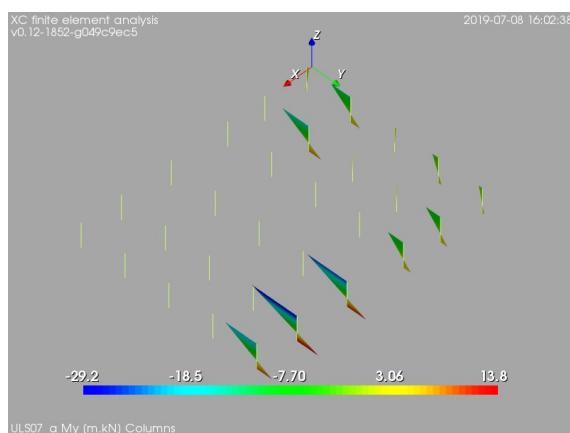


Figure 115: ULS07\_a:  $0.9*D + W\_WE$ . Columns, bending moment about local axis y [m.kN]

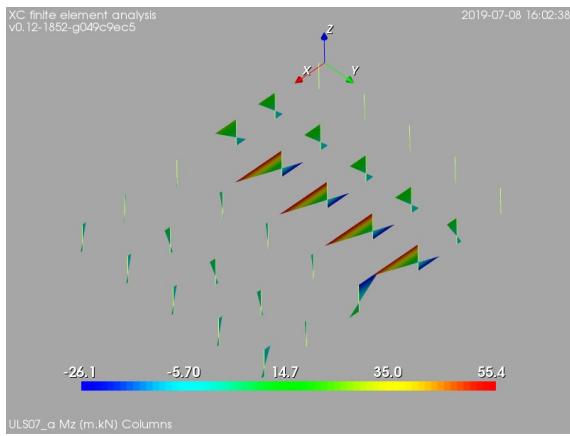


Figure 116: ULS07\_a: 0.9\*D + W\_WE. Columns, bending moment about local axis z [m.kN]

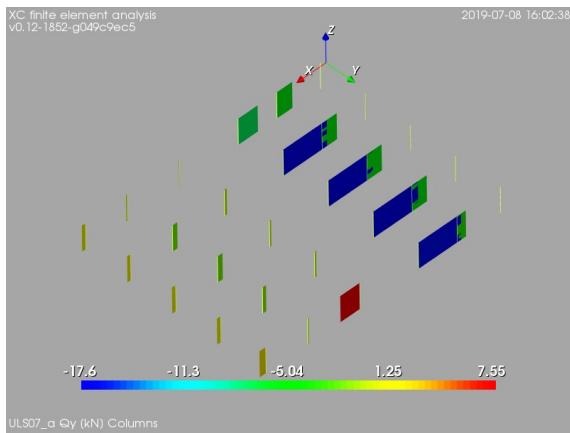


Figure 117: ULS07\_a: 0.9\*D + W\_WE. Columns, internal shear force in local direction y [kN]

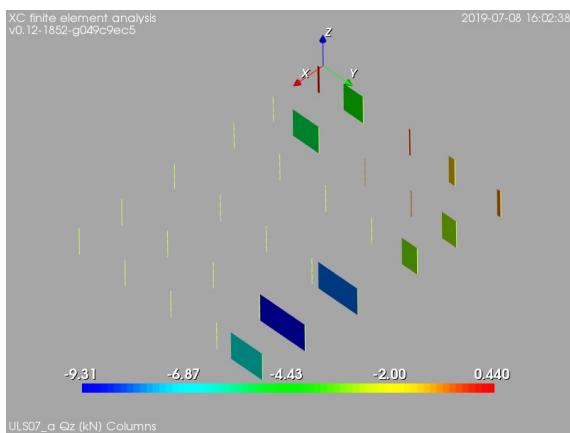


Figure 118: ULS07\_a: 0.9\*D + W\_WE. Columns, internal shear force in local direction z [kN]

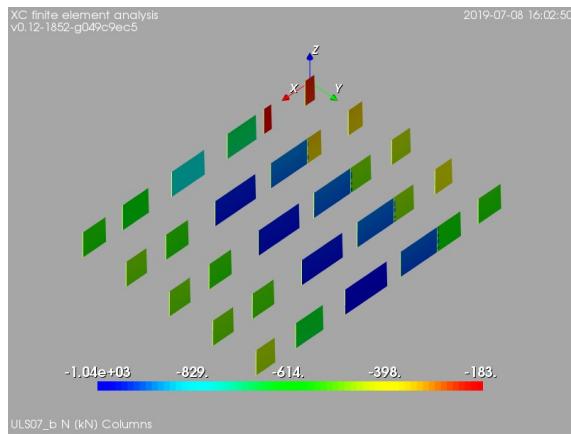


Figure 119: ULS07\_b: 0.9\*D + W\_NS. Columns, internal axial force [kN]

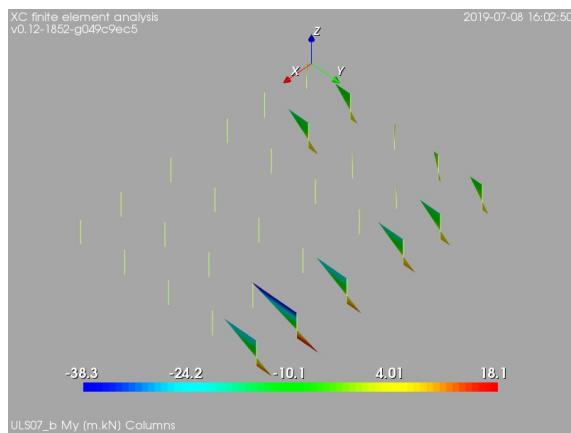


Figure 120: ULS07\_b: 0.9\*D + W\_NS. Columns, bending moment about local axis y [m.kN]

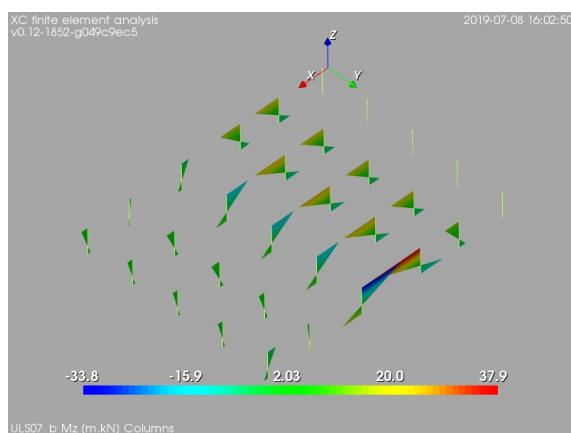


Figure 121: ULS07\_b: 0.9\*D + W\_NS. Columns, bending moment about local axis z [m.kN]

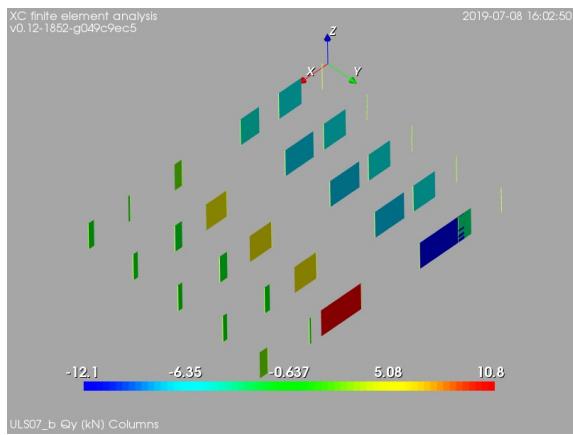


Figure 122: ULS07\_b: 0.9\*D + W\_NS. Columns, internal shear force in local direction y [kN]

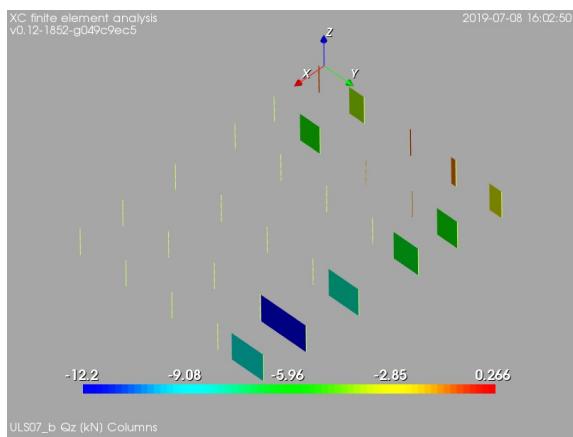


Figure 123: ULS07\_b: 0.9\*D + W\_NS. Columns, internal shear force in local direction z [kN]

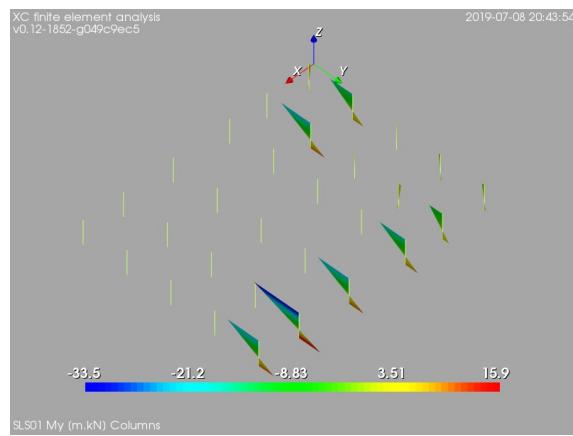


Figure 124: SLS01: 1.0\*D. Columns, bending moment about local axis y [m.kN]

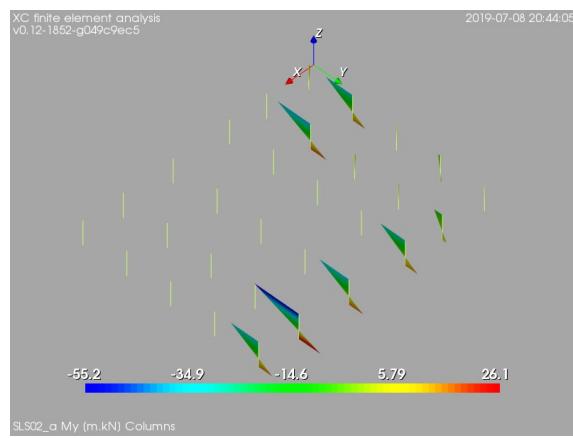


Figure 125: SLS02\_a: 1.0\*D + 1.0\*Lru + Lpu + 0.3\*S. Columns, bending moment about local axis y [m.kN]

## B.2 Serviceability limit states

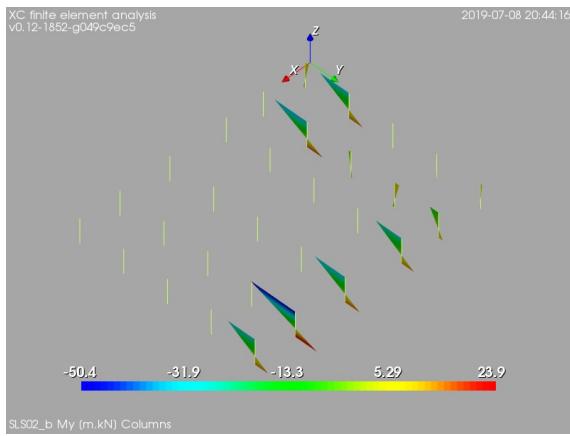


Figure 126: SLS02\_b:  $1.0*D + 1.0*Lrs + Lps + 0.3*S$ . Columns, bending moment about local axis y [m.kN]

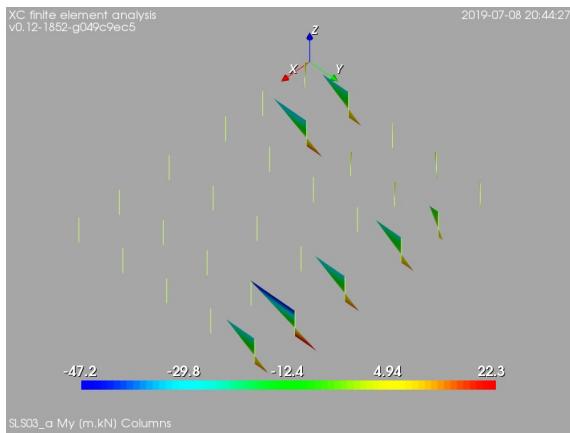


Figure 127: SLS03\_a:  $1.0*D + 1.0*S + 0.3*Lru + 0.3*Lpu$ . Columns, bending moment about local axis y [m.kN]

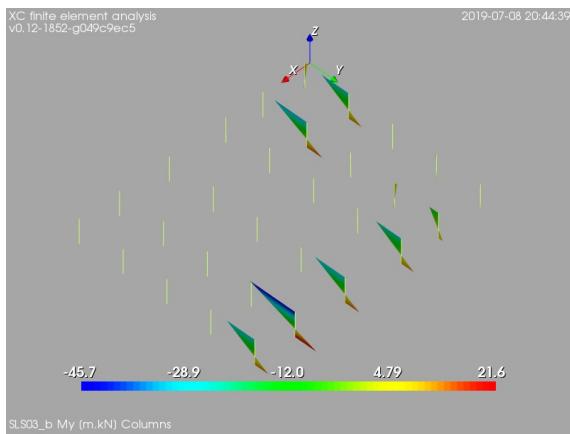


Figure 128: SLS03\_b:  $1.0*D + 1.0*S + 0.3*Lrs + 0.3*Lps$ . Columns, bending moment about local axis y [m.kN]

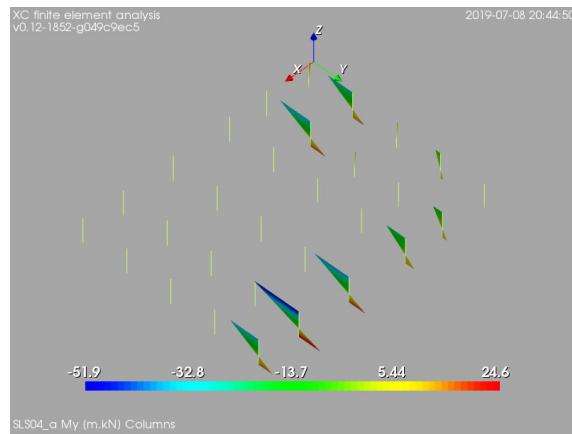


Figure 129: SLS04\_a: 1.0\*D + W\_WE + 1.0\*Lru + Lpu. Columns, bending moment about local axis y [m.kN]

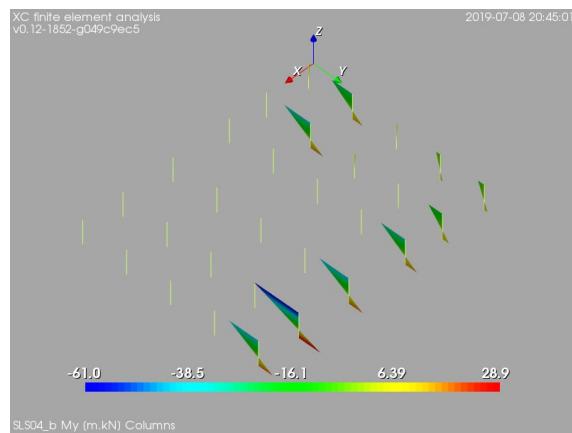


Figure 130: SLS04\_b: 1.0\*D + W\_NS + 1.0\*Lru + Lpu. Columns, bending moment about local axis y [m.kN]

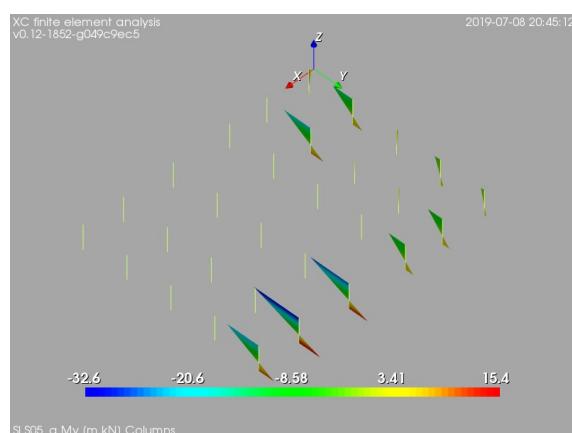


Figure 131: SLS05\_a: 1.0\*D + W\_WE. Columns, bending moment about local axis y [m.kN]

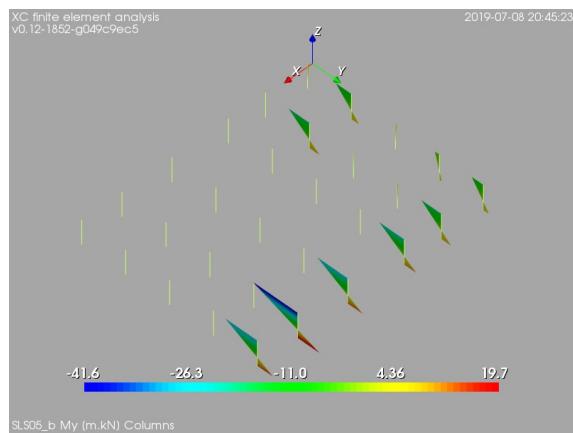


Figure 132: SLS05\_b: 1.0\*D + W\_NS. Columns, bending moment about local axis y [m.kN]