Structural Technical Report 1
Structural Concept / Structural Existing Conditions Report

PricewaterhouseCoopers
Oslo, Norway

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Structural Option
AE 481W Senior Thesis
The Pennsylvania State University
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Table of Contents

Executive Summary

The purpose of this report is to assess the existing conditions of the PricewaterhouseCoopers building and to gain an understanding of the procedures used in its structural design. It encompasses a structural discussion, code overview, material summary, determination of design loads and spot checks.

The design codes used by the structural engineer to determine loads on the structure are *Norwegian Standards*. As the *Eurocodes* will be implemented across Europe within the next couple of years, this technical report makes an attempt to follow the *Eurocodes* when possible. However, with limited guidance on procedures alternative codes have additionally been used. Lateral loads have been determined in accordance with ASCE 07 and spot checks follow LRFD provisions.

An effort has been made to summarize the properties of the materials used in the building as well as their designations in accordance with various standards.

The gravity loads on the structure were found to be directly in accordance with the *Eurocodes*. Lateral loads were calculated to be greater than those determined by design engineer and needs further review. Reasons for the discrepancies could be differences in design codes, reference location and assumptions as well as errors in manual calculation.

A spot check of one column and one beam indicates the members of the structure are adequate to carry design loads. The reason for the calculated member capacities being considerably larger than the loads is most likely due to simplifying assumptions made in calculations.

1 - Structural Discussion

1.1 Introduction

The superstructure of the PricewaterhouseCoopers (*PwC)* building consists of concrete plank decking on steel frame with cast in place concrete cores. Along the exterior of the building the concrete planks typically rest on HSQ profile beams, while along the interior they rest on steel angles connected to the concrete core. The exterior steel beams are supported by circular steel columns filled with reinforced concrete. A grand opening at the center of the facade is allowed through the use of three steel trusses. To provide lateral resistance there are concrete cores located centrally in each leg of the building. The cores are integrated into substructure which is comprised of cast in place concrete. The foundation uses steel and concrete piles driven between 30 and 40m to bedrock.

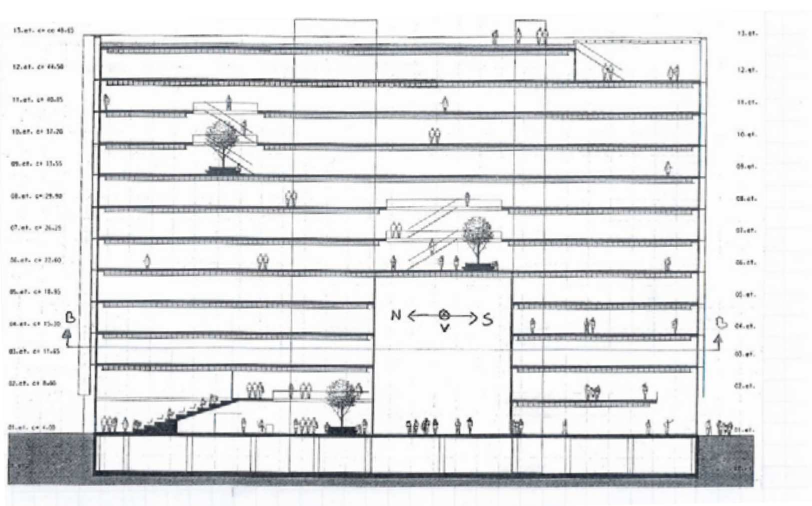


Figure 1. Building Section

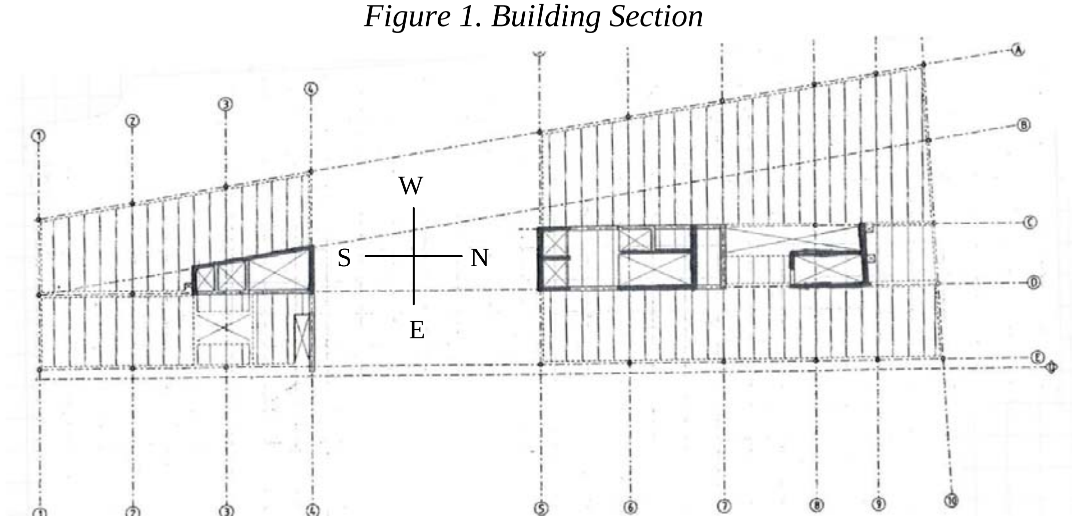


Figure 2. Typical framing plan for floors 1 – 4

1.2 Floor and Roof System of the Superstructure

The floor and roof system of the superstructure is precast hollow core concrete plank on steel framing. The concrete planks are HD265's and have approximate section dimensions of 1.2m x 0.3m (figure 3). Spans range from 5m to 10m and are run in the East-West direction. Due to the buildings shape, edges of some of the planks are cut at an angle.

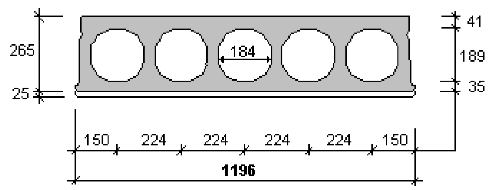


Figure 3: HD 265 element

Along the exterior of the building the concrete planks typically rest on HSQ profile beams. The beams are welded steel shapes fabricated by prefabrication engineer, *Contiga*. Connections between beams and deck elements are made with cast in place concrete containing stirrups that loop around shear tabs on the beams (figure 5). Beams with concrete elements on either side, have steel reinforcing bars that span across the top of the beam and between element joints (figure 5).

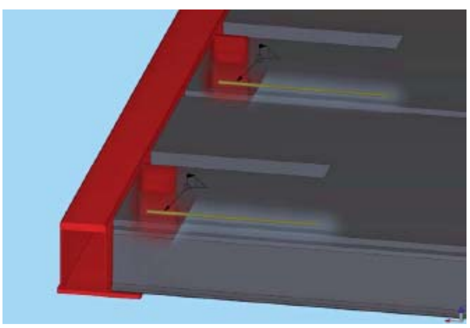


Figure 4: Principle connection of deck elements to one sided HSQ beam.

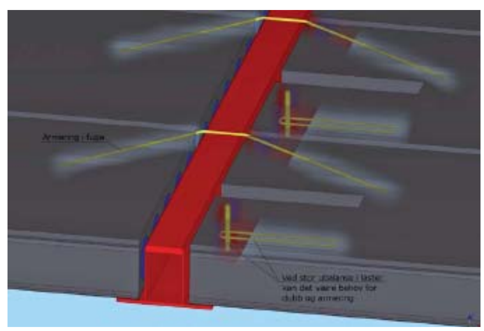


Figure 5: Principle connection of deck elements to two sided HSQ beam.

Interior planks typically rest on steel L150x15 steel angles (figure 6). The angles are welded to steel plates at the top and the bottom with 5mm fillet welds. The plates contain steel tabs that are cast into the concrete core. Beams are similarly connected to the concrete wall using steel angles and plates with steel tabs (figure7)

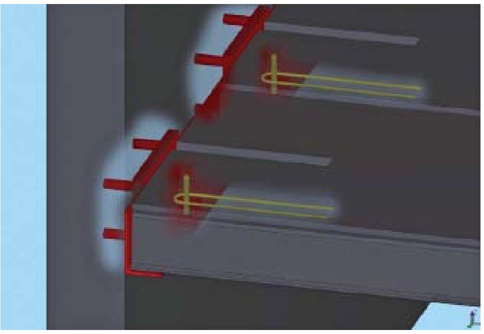


Figure 6: Principle connection of deck element to concrete wall

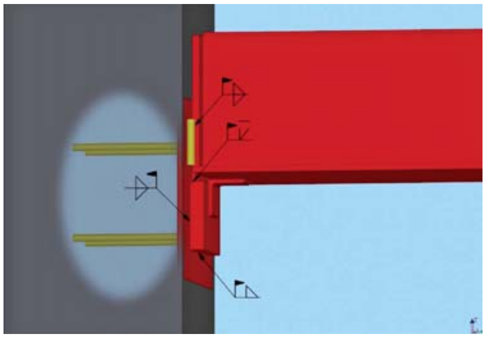


Figure 7: principle connection of steel beam to to concrete wall

Figure 4,5,6, and 7 are from report Hulldeker på Stål bæresystem provided by Norsk Stålforbund and Betongelement Foreningen.

1.3 Superstructure Columns

Hollow circular steel columns filled with reinforced concrete support the beams along the exterior of the building. They have typical spacing of 7.2 m and sizes ranging from Ø406.4mm x 8mm at level 1 to Ø323.9 x 6.3 at level 12. At the base of the building columns are connected at to steel plates with 6mm fillet welds (figure 9)

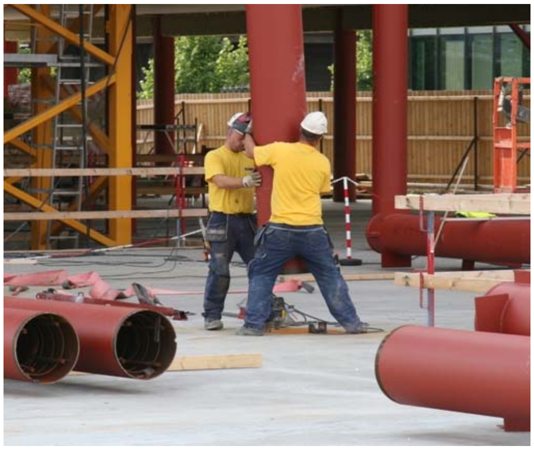


Figure 8: Placing hollow steel column on steel base plate

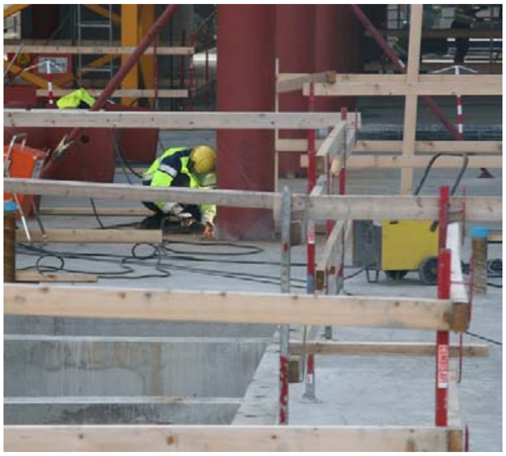


Figure 9: Welding column to steel plate

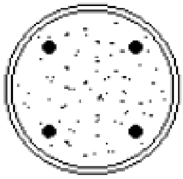


Figure10: Typical column cross section

According to *Design guide for concrete filled columns* by *Corus UK limited*, advantages to concrete filled structural hollow sections are:

- They provide architects and engineers with a robust and inherently fire resistant column.
- During construction the steel sections dispenses with the need for formwork and erection schedule is not depended on concrete curing time.
- During finishing concrete, filling is protected against mechanical damage.
- When completed, columns provide greater usable floor area, higher visibility, reduced maintenance, and are aesthetically pleasing

1.4 Lateral System

Lateral resistance is provided by cast in place concrete cores, located at the center of each leg of the building. Concrete plank decking acts as a rigid diaphragm that transfers loads to the shear walls. The building is tall and narrow in the short direction and therefore requires thick shear walls. Walls are typically 400mm thick in the short direction and 300mm in the long direction. The narrow building shape also causes large overturning moments. Cores are integrated into the cast in place concrete substructure and acts as a base to distribute the overturning moments to the foundation.

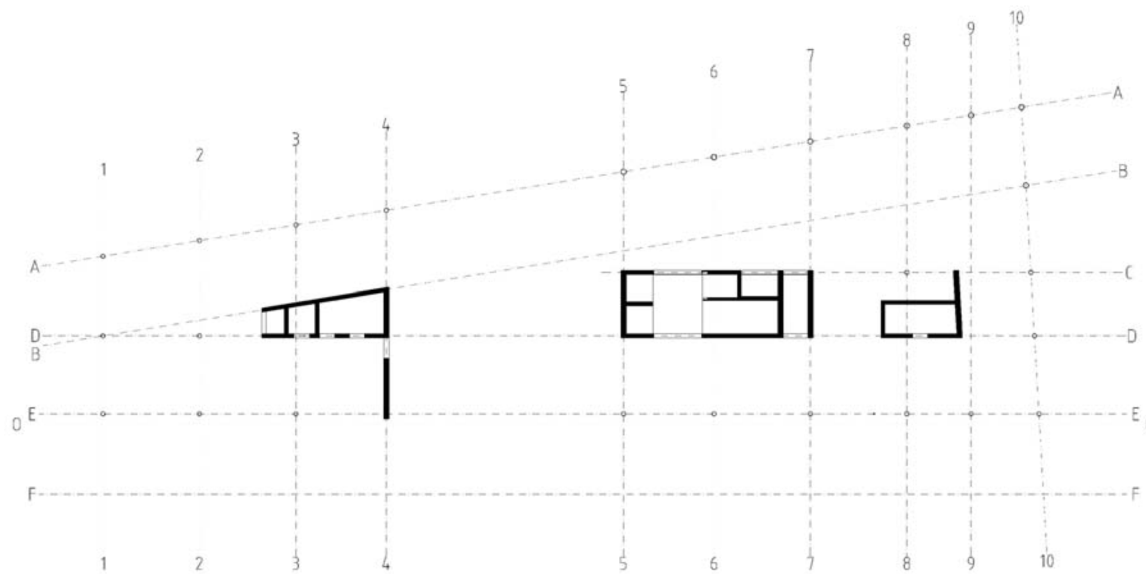


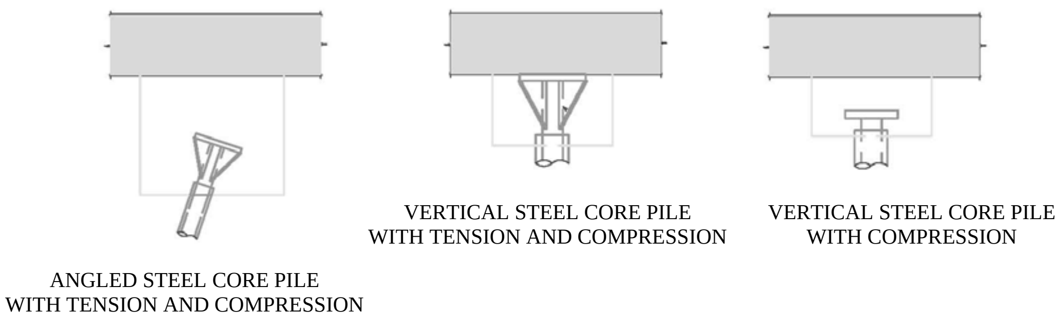
Figure11: Plan showing location of shear walls

1.5 Substructure

There are two stories below grade comprised of cast in place concrete. The slab at the lowest level is 500mm thick with recessed areas for elevator shafts. Floor slabs are 300mm thick except in the areas that are below outdoor areas where slab thickness has been increased to 400mm.

1.6 Foundation:

The foundation uses steel and concrete piles to transfer axial tension, axial compression and lateral loads to the ground. There are five different types of piles used which are driven between 30 and 40m into bedrock. Pile capacities are dependent on pile type, connection type, and whether bending is about strong or weak axis.



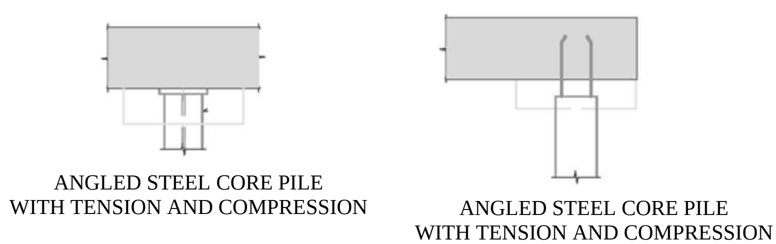


Figure12: Typical pile types

The barcode strip is being built in sections. This meant that the PwC building stood complete before the next building to the west had begun. Therefore uneven loads from ground pressure to the west were accounted for in the design.

Figure13: Image showing building to the west being erected after the PWC (to the left is complete)

Figure14: Excavated Barcode site

1.6 The Grand Entrance

The grand opening at the center of the façade is created by using three trusses comprised of hollow circular steel tubing for diagonal/vertical members and HSQ profiles for horizontal members (Figure 15,16,17). During construction the structure was supported by three temporary columns that were removed after the integrity of the truss was intact.

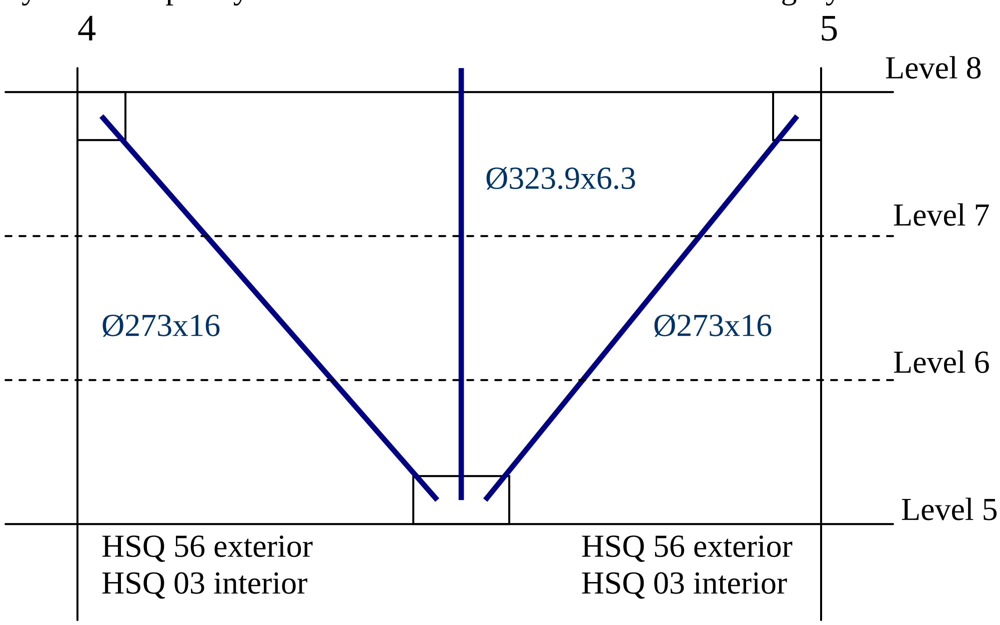


Figure 15: Truss Elevation

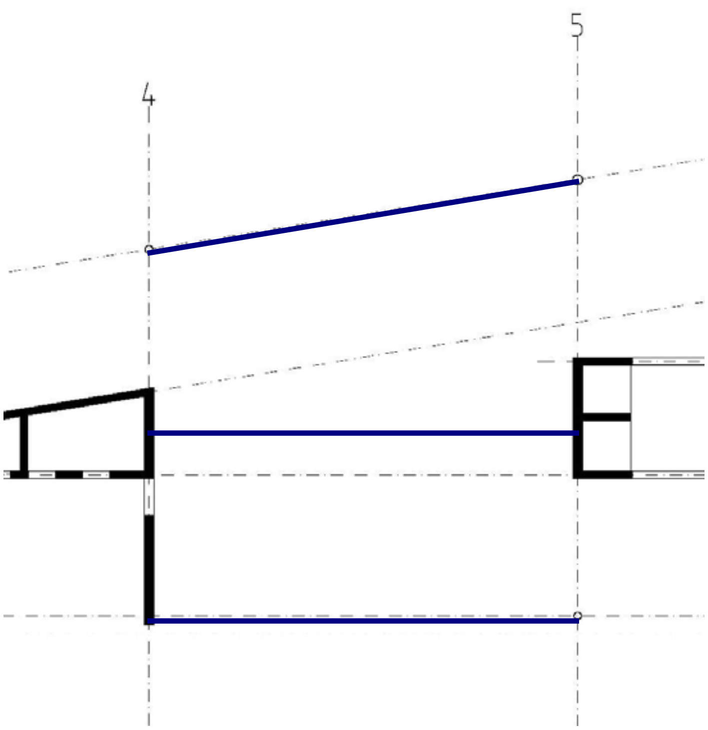


Figure 16: Truss Plan

Figure 17: Truss Images

2 - Design codes

2.1 Norwegian Standards Used in Original Design:

- **NS 3472,** 3. edition September 2001: Prosjektering av stålkonstruksjoner. Beregning og dimensjonering. - *Design of steel structures*
- **NS 3473,** 6. edition September 2003: Prosjektering av betongkonstruksjoner. Beregnings- og konstruksjonsregler. - *Design of conctete structures*
- **NS 3490,** 2. edition December 2004: Prosjektering av konstruksjoner. Krav til pålitelighet. - *Basis of structural design*
- **NS 3491-1,** 1. edition December 1998: Prosjektering av konstruksjoner. Dimensjonerende laster. Del 1: Egenlaster og nyttelaster. - *general actions - Densities, self weight, imposed loads for buildings*
- **NS 3491-3,** 1. edition March 2001: Prosjektering av konstruksjoner. Dimensjonerende laster. Del 3: Snølaster. – *action on structures, snow loads*
- **NS 3491-4,** 1. edition May 2002: Prosjektering av konstruksjoner. Dimensjonerende laster. Del 4: Vindlaster. – *action on structures, wind loads*
- **NS 3491-5,** 1. edition June 2003: Prosjektering av konstruksjoner. Dimensjonerende laster. Del 5: Termiske påvirkninger.
- **NS 3491-7,** 1. edition November 2000: Prosjektering av konstruksjoner. Dimensjonerende laster. Del 7: Ulykkeslaster. – *action on structures, accidental actions*
- **NS 3491-12,** 1. edition December 2004: Prosjektering av konstruksjoner. Dimensjonerende laster. Del 12: Laster fra seismiske påkjenninger. - action on structures, seismic loads

2.2 Codes and Reference Standards used in Senior Thesis:

In the past Norway has operated using national design standards. As part of an effort to decrease trade barriers between EU countries the Eurocodes have been developed, which are unified design codes for buildings and civil engineering works for all of Europe. Norway is currently in the transition period where National and Eurocodes coexist. The Norwegian versions of the Eurocodes and the national annexes are still under production and aim to be completed by 2009. According to the time schedule the transition will period last from year 2008 – 2010, after which national standards will be withdrawn.

This Senior Thesis will make an attempt to use the Eurocodes for design purposes when possible. However, with limited guidance on procedures alternative design codes and standards may be used. Below are the reference standards that will be used for Senior Thesis.

2.2.1 Eurocodes:

- **NS-EN 1990:** 2002 + NA:2008 Eurocode 1:Basis of structural design
- **NS-EN 1991-1-1:** 2002 Eurocode 1: Actions on structures – Part 1-1: General actions – Densities, self weight, imposed loads for buildings
- **NS-EN 1991-1-3:** 2003 Eurocode 1: Action on structures – Part 1-3: General actions – Snow loads
- **NS 3491-4**, 1. edition May 2002: Prosjektering av konstruksjoner. Dimensjonerende laster. Del 4: Vindlaster. – action on structures, wind loads
- **NS-EN 1992-1-1:** 2004 Eurocode 2: Design of concrete structures – Part 1-1: General rules and rules for buildings
- **NS-EN 1993-1-1**:2005 Eurocode 5 + NA:2008: Design of steel structures – Part 1-1: gereral rules for buildings.
- **NS-EN 1993-1-8: 2005 Eurokode 3:** Design of steel structures – Part 1-8: Design of joints
- **NS-EN 1998:2004+NA:2008:Eurocode 8 –** Design of structures for earthquake resistance – Part1: General rules, seismic actions and rules for buildings.

2.2.2 Alternative Codes:

- **ASCE 7 2005:** Minimum Design Loads for Buildings and Other Structures
- **IBC 2006:** International Building Code

3 - Materials

3.1 Steel

Metric

Item	Euronorm	ASTM	Fu (N/mm^2)	Fy (N/mm^2)	Ea (N/mm^2)	Va	Density (kg/m^3)
Columns	S355	A572Gr50	355	510	210 000	.3	7 850
Beams	S355	A572Gr50	355	510	210 000	.3	7 850
Reinforcing	B500C	-	-	500	210 000	-	-
Piles	HISAR460	*still need to determine*					

Imperial

Item	Euronorm	ASTM	Fu (ksi)	Fy (ksi)	Ea (ksi)	Va	Density (lb/ft^3)
Columns	S355	A572Gr50	51	74	30 500	.3	50
Beams	S355	A572Gr50	51	74	30 500	.3	50
Reinforcing	B500C	-	-	72	30 500	-	-

Notes

1. Metric densities are converted to imperial form using 1 lb/ ft = 157 kg/m3
2. Metric material strengths are converted to imperial form using 1 psi = .006894 N/mm^2. Values are rounded down to nearest whole number.

3.2 Concrete

Metric

Item	Norwegian Standard	Eurocode CEN	f_{ck} (N/mm^2)	f_{ctm} (N/mm^2)	E_{cm} (N/mm^2)
Cast in place	B35	C35/45	35	3.2	33 500
Prefabricated	B45	C45/55	45	3.8	36 000
Columns	B45	C45/55	45	3.8	36 000

Imperial

Item	Norwegian Standard	Eurocode CEN	f_{ck} (ksi)	f_{ctm} (ksi)	E_{cm} (ksi)
Cast in place	B35	C35/45	5	0.46	4 850
Prefabricated	B45	C45/55	6.5	0.55	5 222
Columns	B45	C45/55	6.5	0.55	5 222

f_{ck} *- compressive cylinder strength at 28days*
f_{ctm} *- value of mean axial tensile strength of concrete*
E_{cm} *– Secant modulous of elasticity*

Notes

1. Metric material strengths are converted to imperial form using 1psi = .006894 N/mm^2. Values are been rounded down to nearest whole number.

4 Gravity and Lateral Loads

4.1 Dead Loads

Dead load calculations have been determined in accordance with EN 1991-1, 2002. In each case the load determined by the codes matched that used by the design engineer. For purpose of providing a reference for future calculation, as well as review by advisor, the unit weights are also shown in imperial form.

Concrete Decking

Level	Thickness (mm)	Reference	Density (kN/m^3)	Unit Weight (kN/m^2)	Unit Weight (psf)
Slab on grade	500	EN 1991-1 2002, Table A.1	25	12.5	262
Deck over K1 axis E-F/ 5-10	450	EN 1991-1 2002, Table A.1	25	11.3	237
Deck over K1 axis A-E/ 4-5	400	EN 1991-1 2002, Table A.1	25	10	210
Typical basement slab	300	EN 1991-1 2002, Table A.1	25	7.5	157
Concrete plank	-	Contiga website - SD 265	-	3.7	74

Floor, Ceiling, Partitions, M.E.P.

Level	Reference	Unit Weight (kN/m^2)	Unit Weight (psf)
K2	Design Value	1	21
K1	Design Value	1.5	32
1st floor	Design Value	1	21
2-12 floor	Design Value	1.5	31
Roof	Design Value	0.5	11

Other

Item	Reference	Unit Weight (kN/m^2)	Unit Weight (psf)
Façade	Design Value	0.7	15
Outdoor terrace	Design Value	2	42
concrete steps	Design Value[1]	6.5	137
steel steps	Design Value	2.5	52

Notes

1. Where "Design Value" has been listed under reference, the unit weight shown is that obtained by structural design engineer. Architectural details of floor/roof/façade finishes were not provided prior to this technical report. Without that information accurate unit loads were difficult calculate and therefore found it more suitable to list design values.
2. Metric unit weights have been converted to imperial form using 1psf = .04784kN/m2. Values are rounded up to nearest whole number

4.2 Live Loads

Live load calculations have been determined according to the EN 1991-1, 2002 and NS-EN 1991-1-3, 2003 In each case the load determined by the codes matched that used by the design engineer.

Imposed Floor Loads

Area	Reference	Category	Unit Weight (kN/m^2)	Concentrated Load (kN)	Unit Weight (psf)
Office spaces	EN 1991-1 2002, Table NA.6.2	B	2	2	42
Cafeteria	EN 1991-1 2002, Table NA.6.2	C1	3	4	63
Outdoor terrace	EN 1991-1 2002, Table NA.6.2	C1	3	4	63
Auditorium	EN 1991-1 2002, Table NA.6.2	C2	4	4	84
Corridors	EN 1991-1 2002, Table NA.6.2	C3	5	4	105
Technical Rooms	Design Value[1]	-	5	4	105
Archives (stationary)	Design Value[1]	-	7	4	147
Archives (On rollers)	Design Value[1]	-	12	4	251
Outdoor "under opening"	Design Value[1]	-	20	105	418
Outdoor	Design Value[1]	-	20	190	418

Snow Loads

Snow Load	NS-EN 1991-1-3: 2003	-	2.8	-	60

Notes

1. Where "Design Value" has been listed under reference, the value shown is that obtained by structural design engineer. Architectural details of floor/roof/façade finishes were not obtained prior this technical report. Without that information accurate unit loads could not be calculated and therefore it was more suitable to list design values.
2. Metric unit weights have been converted to imperial form using 1psf = .04784kN/m2. Values are been rounded up to nearest whole number

4.3 Wind Loads

For the purpose of determining approximate wind load on the structure, loads have been calculated in accordance with ASCE 7, 05 for the region Boston, MA. The reason for not using NS 3491-4, which is the applicable code for Oslo, Norway, is that I as of current do not have enough knowledge on the procedures. This is something that I might look into, as the values obtained did not match that of the design engineer. The site Boston, MA was chosen due to its similar climate, urban density, and coastal exposure.

The design engineer determined an average positive pressure per story of 83.3 kN in the North/West direction. For comparison purposes the average force on one story using Method 2 of ASCE 07 was 361 kN. Since the value I obtained was profoundly larger I assume there is a mistake in my calculations. Other reasons for discrepancies are noted below.

- Basic wind speed of Oslo may not match that of Boston.
- Determination of site class was an estimation
- The natural frequency determined through ASCE 07 may be incorrect. It was determined to be .5s in both directions. This appears be to low, which means the building is modeled to be more flexible than it actually is. I believe this error may have been caused by incorrect choice of equation in ASCE 07.

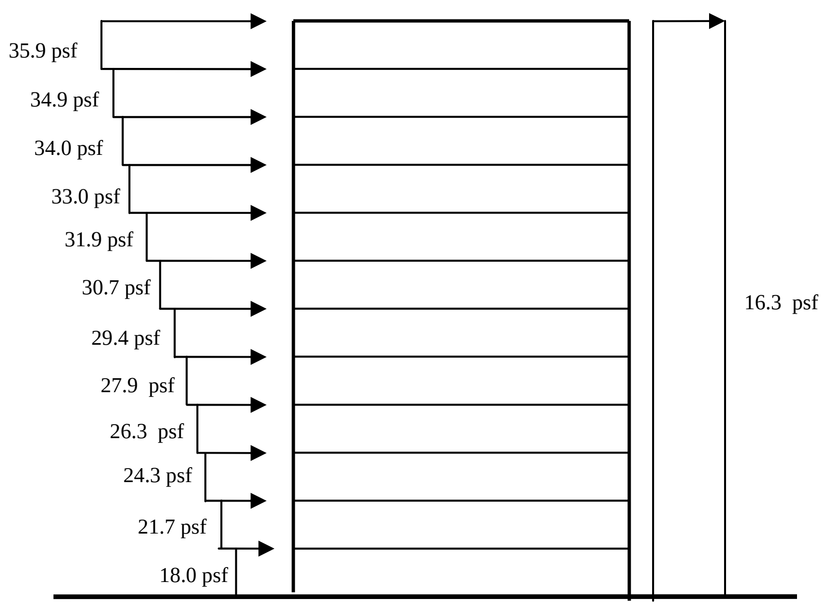


Figure 18: Wind pressure in the North / South direction.
For information on calculations see Appendix A

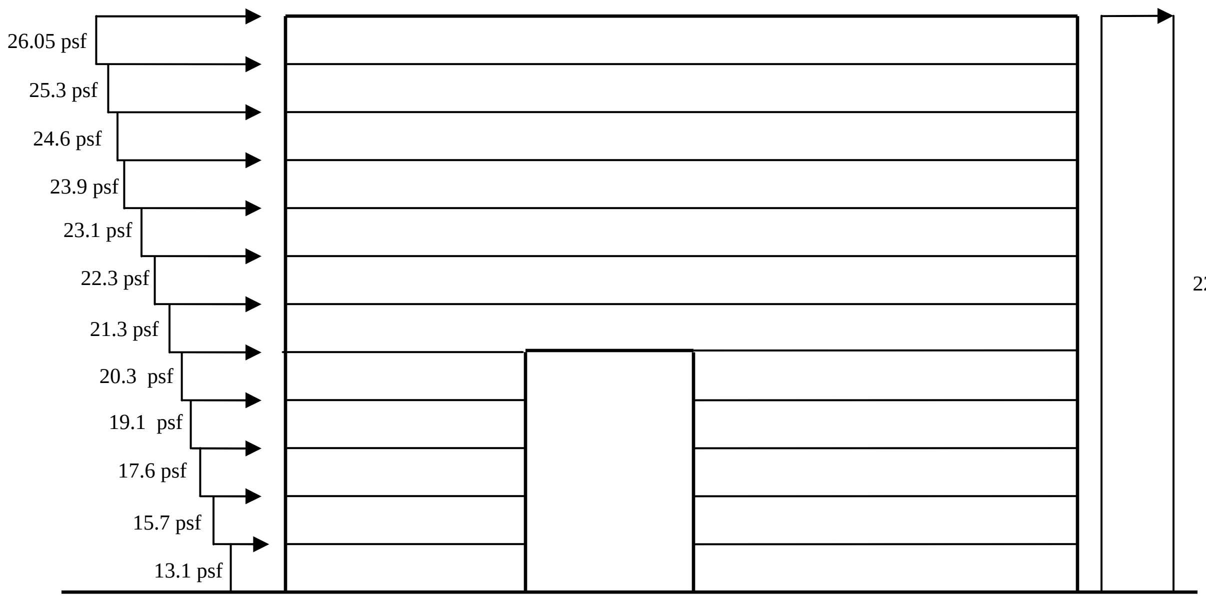


Figure 19: Wind pressure in the East / West direction
For information on calculations see Appendix A

4.4 Seismic Loads

For the purpose of determining approximate seismic loads on the structure, loads have been calculated in accordance with ASCE 7, 05 for the region Boston, MA. The site was chosen for similar reasons to that stated under wind loading (4.3).

I did not manage to obtain seismic loads determined by the structural engineer and am therefore unable to make a comparison. As with the wind there are a number assumptions made during calculation that might have caused discrepancies and needs further review. My main concern is that the natural frequency of the structure calculated was to low.

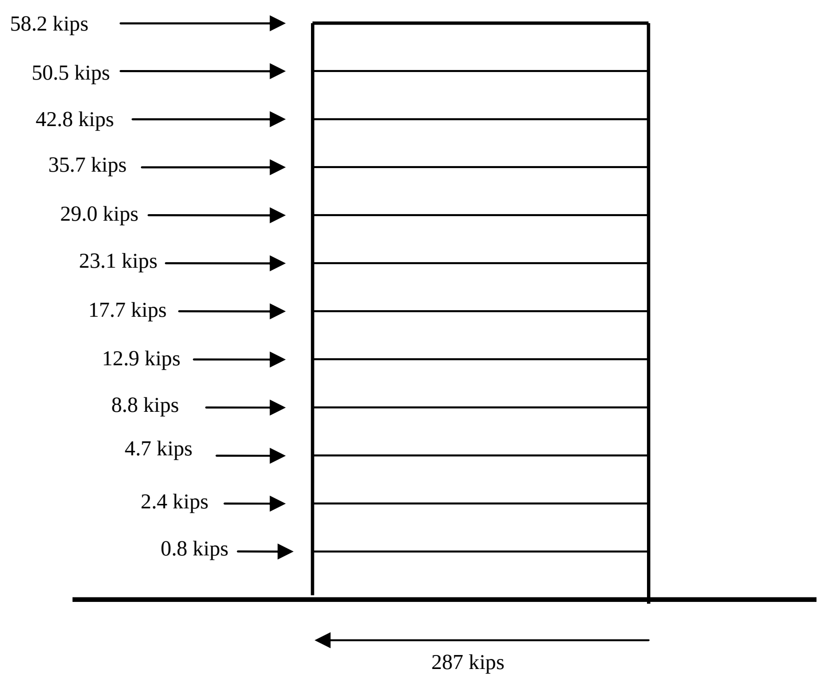


Figure 20: Distribution of lateral seismic forces on structure
For information on calculations see Appendix B

5 Spot Checks

5.1 Fourth Story Beam

An exterior fourth story beam was spot checked for this analysis. The beam was checked for bending and shear in accordance with LRFD provisions. The calculations determined the beam capacity to be larger than the applied load, however there were numerous simplifying assumptions made. Mainly the beam is experiencing torsion which was neglected in the calculations and would decrease the capacity of the beam. Another factor is that only two load combinations were checked, 1.2D +1.6L and 1.4D. Lastly the codes used to check the member were not the same as those used by the design engineer, which could create discrepancies. If these considerations were taken into account I believe the beam would be more adequately sized. A summary of findings is listed below.

Beam: SB - 155
Profile: HSQ53
Span: 26.3ft
Mu: 140 kips
ΦMn: 461 kips
Vu: 23.6kips
ΦVn: 732 kips

5.2 First story composite column

A typical first story composite column was checked for this analysis. The column was checked for axial gravity load in accordance with LRFD provisions. The column was assumed to have pinned connections at either side and contain no lateral loads. From the calculations it was determined that the design load was approximately half of the columns capacity. Reasons for the beam appearing to be oversized could be the design engineer chose a column that would keep uniformity throughout the building. Other reasons are differences in design codes and manuals used. A summary of findings is listed below.

Column: SS - 115
Size: Ø323.6*6.3mm
Pu: 500 kips
ΦPn: 1073 kips

Appendix

A.1 Wind Load Calculations

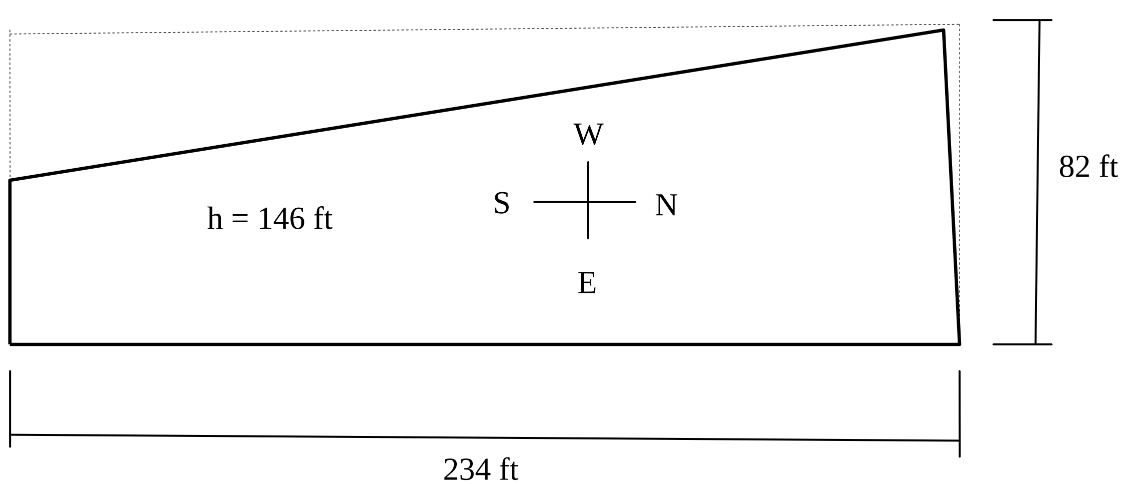
A1.1 Summary of values determined from ASCE 07 method 2

Velocity Pressure	
V - Basic Wind speed	120
Occupancy Category	III
K_d	0.85
Importance Factor	1
Exposure Category	B
K_{zt}	1

Internal pressure coefficients
Enclosed Structure
GC_{pi} = +/- 0.18

Pressure Coefficients	
C_p Windward wall	0.8
Cp Leeward wall	-0.5
C_p Side wall	-0.7

Gust Factor		
	N/S	E/W
L	82	234
B	234	82
h	147	147
n_1	0.5	0.5
g_Q, g_v	3.4	3.4
g_r	3.29	3.29
I_z	0.25	0.25
Q	0.77	0.8
V	120	120
V_z	101.3	101.3
N_1	2.22	2.22
R_n	0.8	0.08
R_h	0.25	0.25
R_B	0.17	0.4
R_L	0.147	0.055
R	1.35	1.06
G	1.3	0.943



A1.2 Wind Pressure

North / South

Floor	height (ft)	Kz	qz	Pressure (psf)						
				N/S Windward			N/S Leeward			Total
Roof	146	1.102	34.53	35.91	+/-	6.22	-22.45	+/-	6.22	58.36
12	133	1.072	33.58	34.93	+/-	6.22	-22.45	+/-	6.22	57.37
11	121	1.043	32.69	34.00	+/-	6.22	-22.45	+/-	6.22	56.44
10	109	1.013	31.73	33.00	+/-	6.22	-22.45	+/-	6.22	55.44
9	97	0.979	30.68	31.90	+/-	6.22	-22.45	+/-	6.22	54.35
8	85	0.943	29.56	30.74	+/-	6.22	-22.45	+/-	6.22	53.18
7	73	0.903	28.30	29.43	+/-	6.22	-22.45	+/-	6.22	51.88
6	61	0.858	26.89	27.96	+/-	6.22	-22.45	+/-	6.22	50.41
5	49	0.806	25.26	26.27	+/-	6.22	-22.45	+/-	6.22	48.72
4	37	0.744	23.32	24.25	+/-	6.22	-22.45	+/-	6.22	46.70
3	25	0.666	20.86	21.69	+/-	6.22	-22.45	+/-	6.22	44.14
2	13	0.553	17.33	18.03	+/-	6.22	-22.45	+/-	6.22	40.47
1	0									0.00

East / West

Floor	height (ft)	Kz	qz	Pressure (psf)						
				N/S Windward			N/S Leeward			Total
Roof	146	1.102	34.53	26.05	+/-	6.22	-16.28	+/-	6.22	42.33
12	133	1.072	33.58	25.33	+/-	6.22	-16.28	+/-	6.22	41.62
11	121	1.043	32.69	24.66	+/-	6.22	-16.28	+/-	6.22	40.94
10	109	1.013	31.73	23.94	+/-	6.22	-16.28	+/-	6.22	40.22
9	97	0.979	30.68	23.14	+/-	6.22	-16.28	+/-	6.22	39.42
8	85	0.943	29.56	22.30	+/-	6.22	-16.28	+/-	6.22	38.58
7	73	0.903	28.30	21.35	+/-	6.22	-16.28	+/-	6.22	37.63
6	61	0.858	26.89	20.28	+/-	6.22	-16.28	+/-	6.22	36.57
5	49	0.806	25.26	19.06	+/-	6.22	-16.28	+/-	6.22	35.34
4	37	0.744	23.32	17.59	+/-	6.22	-16.28	+/-	6.22	33.87
3	25	0.666	20.86	15.74	+/-	6.22	-16.28	+/-	6.22	32.02
2	13	0.553	17.33	13.08	+/-	6.22	-16.28	+/-	6.22	29.36
1	0									0.00

A1.3 Base Shear

North / South

Floor	Height (ft)	Tributary Area Above (ft)	Tributary Area Below (ft)	Force (kips)	Shear (kips)
Roof	146	0	558	33	33
12	133	558	491	61	93
11	121	491	491	56	149
10	109	491	498	55	204
9	97	498	484	54	258
8	85	484	491	52	311
7	73	491	491	52	362
6	61	491	491	50	413
5	49	491	491	49	461
4	37	491	491	47	508
3	25	491	491	45	553
2	13	491	538	43	596
1	0	538	0	22	618

Total Base Shear =	618 kips

East / West

Floor	Height (ft)	Tributary Area Above (ft)	Tributary Area Below (ft)	Force (kips)	Shear (kips)
Roof	146	0	1593	67	67
12	133	1593	1401	126	193
11	121	1401	1401	116	309
10	109	1401	1420	114	423
9	97	1420	1382	112	535
8	85	1382	1401	109	643
7	73	1401	1401	107	750
6	61	1401	1401	104	854
5	49	1401	1401	101	955
4	37	1401	1401	97	1052
3	25	1401	1401	92	1144
2	13	1401	1535	90	1234
1	0	1535	0	45	1279

Total Base Shear =	1279 kips

A.2 Seismic Calculation

A.2.1 ASCE 7 Calculation Summary

Location	Boston, Mass	
Latitude	42.35	
Longitude	-71.06	
Site Class	D	Table 20.3 – 1
Ss	0.28	USGA Java Motion Parameter:
S1	0.068	USGA Java Motion Parameter:
Fa	1.577	Table 11.4-1
Fv	2.4	Table 11.4-2
S_{MS}	0.44156	Eq 11.4-1
S_{M1}	0.1632	Eq 11.4-2
SD_s	0.293	USGA Java Motion Parameter:
SD_1	0.108	USGA Java Motion Parameter:
Occupancy Category	II	IBC Table 1604.5
T	2	Sec 12.8.2
SDC	B	Table 11.6-1

N/W Direction		
R	5.5	Table 12.2-1
I	1	Table 11.5-1
T_L	6	Figure 22-15
H	147	
Cs	0.01	

E/W Direction		
R	5.5	Table 12.2-1
I	1	Table 11.5-1
T_L	6	Figure 22-15
H	147	
Cs	0.01	

Story	w_x (kips)	h_x	k	$w_x h_x$^k	C_{vx}	F_x (kips)	V_x(kips)
Roof	2402	146	1.75	14811132	0.20	58.2	
12	2473	133	1.75	12856810	0.18	50.5	58
11	2473	121	1.75	10898144	0.15	42.8	109
10	2473	109	1.75	9079848	0.12	35.7	152
9	2473	97	1.75	7383724	0.10	29.0	187
8	2473	85	1.75	5879786	0.08	23.1	216
7	2473	73	1.75	4507279	0.06	17.7	239
6	2473	61	1.75	3294083	0.05	12.9	257
5	2471	49	1.75	2244895	0.03	8.8	270
4	2165	37	1.75	1205323	0.02	4.7	279
3	2165	25	1.75	609009	0.01	2.4	284
2	2165	13	1.75	195804	0.00	0.8	286
1	0	0	1.75	0	0.00	0	287
Totals	28682	NA	NA	72965837		286.8	287

V = Cs*W	**286.81844**

A2.2 Total Building Dead Load

Floor

Level	Tributary Area (ft2)	Deck + Super Imposed (psf)	Beam + Connections (psf)	Dead Load (kips)
Roof	14391	127	11	1986
12	14391	105	11	1669
11	14391	105	11	1669
10	14391	105	11	1669
9	14391	105	11	1669
8	14391	105	11	1669
7	14391	105	11	1669
6	14391	105	11	1669
5	14391	105	11	1669
4	11756	105	11	1364
3	11756	105	11	1364
2	11756	105	11	1364
1	0	0	0	0

Façade

Story	Perimiter (ft)	Trib. Height (ft)	Wall Load (psf)	Wall load (kips)
Roof	581	6	15	52.29
12	581	12	15	104.58
11	581	12	15	104.58
10	581	12	15	104.58
9	581	12	15	104.58
8	581	12	15	104.58
7	581	12	15	104.58
6	581	12	15	104.58
5	587	12	15	105.66
4	587	12	15	105.66
3	587	12	15	105.66
2	587	12	15	105.66
1	587	6	15	52.83

Concrete Core

Level	Story Height (ft)	North/ South		East / West		Density (lb/ft3)	Load (kips)
		Thickness (ft)	Dist (ft)	Thickness (ft)	Dist (ft)		
1 to 12	**12**	**1**	**196**	**1.3**	**136**	**150**	**671.04**

Columns

Story	Coulmns	Approx weight per col - per story (lb)	Dead Load (kips)
12	28	0.5	28.5
11	28	0.5	28.5
10	28	0.5	28.5
9	28	0.5	28.5
8	28	0.5	28.5
7	28	0.5	28.5
6	28	0.5	28.5
5	28	0.5	28.5
4	24	0.5	24.5
3	24	0.5	24.5
2	24	0.5	24.5
1	24	0.5	24.5

A3 Snow Loads

Snow loads are calculated in accordance with NS-EN 1991-1-3: 2003 Eurocode 1

Item	Reference	Value
u_1 - snow load shape coefficient	Table 5.2	0.8
C_e - Exposure coefficient	Table 5.1	1
C_t - Thermal coefficient	Note section 5.2 (8)	1
S_k - Characteristic snow level on ground	Table NA4.1(901)	3.5 kN/m2

$s = u_i C_e C_t s_k$

$s = (.8)(1)(1)(3.5)$

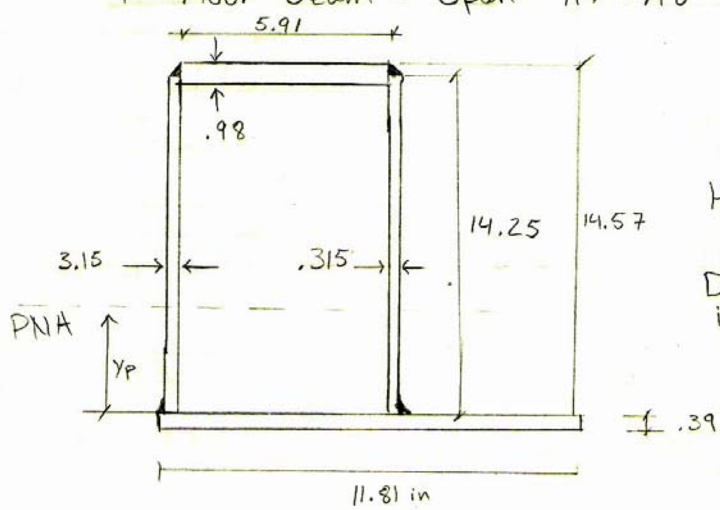
$$\boxed{\mathbf{s = 2.8\ kN/m^2}}$$

Imperial s = 60 psf

A4 Beam Spot Check

Beam Spot Check SB-155

4th Floor Beam - Span A7 - A8



HSQ 53

Dimentions in inches

NTS

- Model Shape as four rectangular plates
- Span = 23.6 ft
- A572 Gr 50 Steel

  $F_y = 74$ ksi
- Neglect Torsion
- Only Considder Load Combination 1.2D + 1.6L

⇒ Check Beam for bending and shear

Determine Plastic Moment Strength

1) Determine Plastic Neutral Axis

$A_{top} = (5.91)(.98) = 5.79 \text{ in}^2$

$2 \times A_{flange} = 2 \times (.315)(14.25) = 2 \times 4.48$

$A_{bottom} = (11.81)(.39) = 4.6 \text{ in}^2$

$A_{Total} = 19.36 \text{ in}^2$

$\frac{A_T}{2} = 9.68 \text{ in}^2 \Rightarrow$ PNA in Flange

$y_p = \frac{9.68 - 4.6}{(2)(.315)}$

$y_p = 8.06$ in from bottom

2) Determine the plastic Section moduloas

$$Z = (5.79)\left(\frac{14.57 - \frac{.98}{2}}{2}\right) + 2\left[\left(\frac{8.06 - \frac{14.25}{2}}{2}\right)(4.48)\right.$$
$$+ (4.6)\left(8.06 + \frac{.39}{2}\right)$$

$Z = 40.76 + 4.18 + 37.27$

$Z = 83.21 \text{ in}^3$

3) Determine Plastic Moment Strength

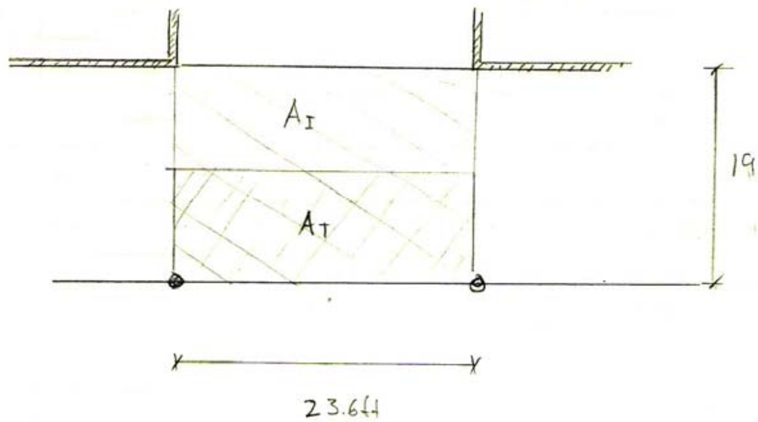
$M_p = F_y Z$

$M_p = (74)(83.21)$

$M_p = 6157$ ft-kips

$\phi M_p = \frac{(.9)(6157)}{\frac{12 \text{ in}}{\text{ft}}} = \boxed{461.7 \text{ ft-kips}}$

Determine Loads on Beam



$A_T = (23.6)\left(\frac{19.1}{2}\right) = 25.38\ ft^2$

$A_I = (23.6)(19.1) = 450.76\ ft^2$

Live Load Reduction - ASCE 7-05

$L_r = \ell_o\left(.25 + \frac{15}{\sqrt{A_I}}\right)$

$L_r = \ell_o\left(.25 + \frac{15}{\sqrt{450.76}}\right)$

$L_r = \ell_o(.95) > .4 \therefore OK$

Live Load

Floor = (42 psf)(9.55 ft)(.95) = 381 lb/ft

Dead Load

Decking = (74 psf)(9.55 ft) = 707 lb/ft

Facade = (15 psf)(12 ft) = 180 lb/ft

Flr/Ciel/MEP = (31 psf)(9.55 ft) = 296 lb/ft

- lb / ft on Column

$$w = 1.2D + 1.6L$$
$$w = (1.2)(707 + 180 + 296) + 1.6(381)$$
$$w = 2 \text{ kip/ft}$$

- Determine Bending Moment

$$M_u = \frac{w\ell^2}{8}$$
$$M_u = \frac{(2)(23.6)^2}{8}$$
$$\boxed{M_u = 139.24 \text{ ft-kips}}$$

$$\boxed{\phi M_p > M_u \text{ Ok in bending}}$$

Check Beam For Shear

1) Determine Shear on the Beam

$$V_u = \frac{w\ell}{2} = \frac{(\cancel{2})(23.6)}{\cancel{2}} = 23.6^k$$

2) Determine Allowable Shear Strength

$$V_n = (0.6)(F_y)(A_w)(C_v)$$
$$V_n = (0.6)(74)(19.36)(1)$$
$$V_n = 859.5$$

3) Determine Design Strength

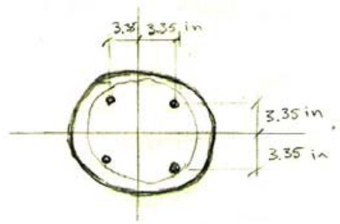
$$\phi V_n = (.9)(859.5)$$
$$\phi V_n = 773.6$$

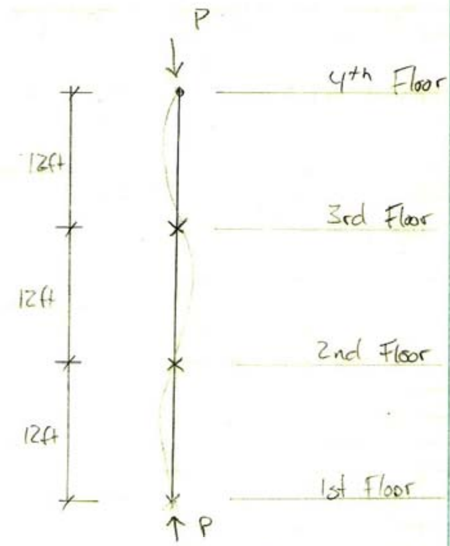
$$\boxed{\phi V_n > V_u \text{ Beam OK in Shear}}$$

A5 Column Spot Check

Column Spot Check: SS-115

Exterior Gravity Column
No lateral Loads
Composite Column





Steel Shape - Ø323.6 * 6.3 (mm)

$F_y = 74$ ksi
$F_u = 51$ ksi
$I_s = 190.5\ in^4$
$A_s = 9.67\ in^2$
E = 30 500 ksi

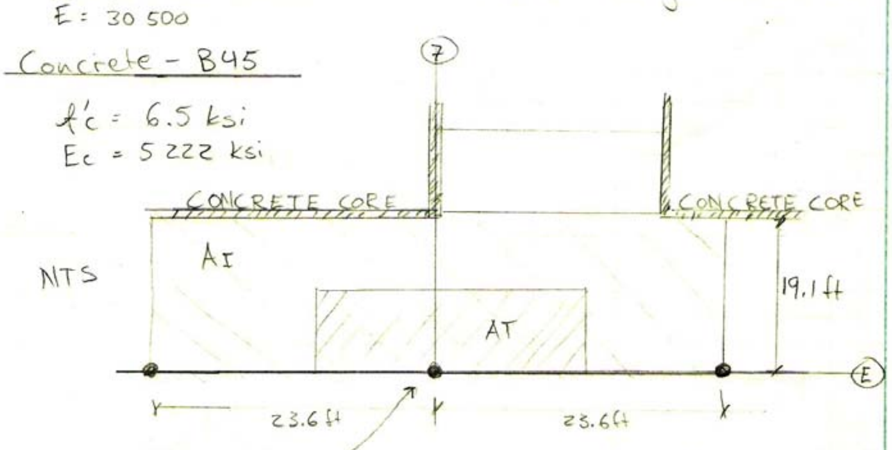
Assume Pin-Pin Connection
Effective Length Factor = 1
Effective Length = (1)(12) = 12'

Reinforcing - D16 (mm)

$F_y = 72$ ksi
$A_s = .31\ in^2$
E = 30 500

Concrete - B45

$f'_c = 6.5$ ksi
$E_c = 5\,222$ ksi



Typical Framing Plan Fl. 1-12

Load on 1st Story Column

- $A_T = 23.6 + \frac{19.1}{2} = 225\,ft^2$
- $A_I = (23.6)\left(\frac{19.1}{2}\right)(11\ \text{floors}) = 2480$
- Live Load Reduction Factor

$L_r = l_o\left(.25 + \frac{15}{\sqrt{A_I}}\right)$

$L_r = l_o\left(.25 + \frac{15}{\sqrt{2480}}\right)$

$L_r = l_o(.55) > .4\ \therefore \text{OK}$

- Live Load

Roof $= (63\,psf)(225\,ft^2) = 14.2^k$

Floor $= (42\,psf)(225\,ft^2)(10\ \text{floors})(.6) = 56.7^k$

Snow $= (60\,psf)(225\,ft^2) = 13.5^k$

Total $= 84.4^k$

- Dead Load

Roof $= (11\,psf)(225\,ft^2) = 2.5^k$

Flr/Ciel/MEP/Part $= (31\,psf)(225\,ft^2)(10\ \text{floors}) = 69.7^k$

Deck $= (74\,psf)(225\,ft^2)(11\ \text{floors}) = 183^k$

Façade $= (15\,psf)(12\,ft)(23.6\,ft) = 48.8^k$

Total Dead $= 304^k$

- Axial Load On Column

$P_u = (1.2)(304) + (1.6)(84.4)$

$\boxed{P_u = 500^k}$

CAMPAD

Determine Nominal Strength of Composite Column

1) Determine Area of Components

$A_s = 9.67 in^2$
$A_{sr} = 4(.31 in^2) = 1.24 in^2$
$A_c = \pi r^2 - A_s - A_{sr}$
$= (\pi)(6.37^2) - 9.67 - 1.24$
$= 127.5 in^4$

2) Check minimum requirements:

$\frac{D}{t} < 0.15 \frac{E}{F_y}$ for round HSS

$\frac{12.75}{.24} < 0.15 \frac{210\ 000}{510}$

$53.125 < 61.76 \therefore$ ok

3) Determine P_o and P_e:

$C_2 = .95$ for round HSS

$C_4 = 1$ for filled HSS

$C_5 = .6 + 2\left(\frac{A_s}{A_c + A_s}\right) \leq 0.9$ HSS filled

$= .6 + 2\left(\frac{9.67}{127.5 + 9.67}\right) = .74 \leq 0.9$

$I_s = I_y = 190.5 in^4$

$I_{sr} = (4)(1)(3.35^2) = 44.9 in^4$

$I_c = \left(\frac{1}{4}\right)(\pi)(12.7^4) - 190.9 - 44.9 = 20\ 196 in^4$

d

$P_o = (9.67)(74) + (1.24)(72) + (.95)(127.5)(6.5)$

$P_o = 1592.17^k$

$EI_{eff} = E_s I_s + C_4 E_s I_s + C_5 E_c I_c$

$EI_{eff} = (30500)(190.5) + (1)(30\,500)(44.9) + (.74)(5\,222)(20196)$

$EI_{eff} = 85.22 \times 10^6$

$P_e = \dfrac{(\pi^2)(EI_{eff})}{(KL)^2} = \dfrac{(\pi^2)(85.22 \times 10^6)}{((12)(12))^2} = 40\,562$ ksi

4) Determine Controlling Column Strength

$\dfrac{P_e}{P_o} = \dfrac{40\,562}{1\,592.17} = 25.47 > .44$

$\Rightarrow P_e > .44 P_o$

5) Determine nominal compressive strength

$P_n = P_o (0.658)^{\frac{P_o}{P_e}}$

$= (1592.17)(0.658)^{\frac{1592.17}{40\,562}}$

$= 1430$ kips

6) LRFD Design Strength

$\phi P_n = (.75)(1430) = \boxed{1073 \text{ kips} = \phi P_n}$

$\boxed{\phi P_n > P_u \therefore \text{Column S-115 OK}}$