Design of a 16-story hotel building in Southeast Louisiana, USA

Department of Civil and Environmental Engineering

NAZARBAYEV UNIVERSITY

ENG 400: Capstone II

Final Report

Group 4

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Date of submission: 2024

Declaration

We hereby declare that this report entitled "Preliminary Design of a 16-story hotel building in Southeast Louisiana, USA" is the result of our own project work except for quotations and citations, which have been duly acknowledged. We also declare that it has not been previously or concurrently submitted for any other degree at Nazarbayev University.

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Introduction

The main goal involved performing a literature review and carrying out analytical analysis and calculations to create a comprehensive design for the hospitality facility. As part of this project, we completed the architectural, structural, geotechnical, and environmental design for a high-rise hotel. The project's ultimate aim was the creation of a 16-story hotel building in a strong wind environment.

The project's specific objectives include:

1. Structural and Architectural:
    a. Incorporate LEED principles for sustainable building design, construction, operation, and performance. Conduct a life cycle cost analysis using Life-365 software and account for corrosion.
    b. Evaluate internal forces, drift, and deflections using design software validated by manual calculations.
    c. Plan member design, joint design, serviceability design, and structural detailing using design software confirmed by manual calculations.
    d. Create detailed construction drawings for structural components.
    e. Select appropriate materials based on project requirements and sustainability standards.
2. Environmental:
    a. Select waste management option and estimate its potential
    b. Design Waste collection plan
    c. Build an integrated solid waste management plan
3. Geotechnical
    a. Determine the axial bearing capacity and settlement analysis of the foundation using design software, validated by manual calculations.
    b. Design the foundation in detail, specifying thickness, depth, strength, and reinforcement, utilizing design software and confirming with manual calculations.
    c. Develop a comprehensive construction procedure for foundation installation.
    d. Create detailed geotechnical drawings, including foundation specifications.
4. Construction Management

a. Ensure that project will execute successfully through proper planning of budget, time and resources
b. Divide the scope of the work into smaller and manageable structures by using WBS
c. Assess, monitor and mitigate all the risks of the project
d. Provide the safety environment on the project
e. Ensure the high quality of materials, services, activities during the project execution

1. Structural part

From the Capstone I we have selected the materials and assigned the loads on the frame. In table 1-1, list significant loads per story .

Table 1-1. Loads used for SAP2000

Slab	DL (kPa)	LL (kPa)	Exterior walls UDL (kN/m)
1-2 floors	7.031586	4.79	6.17
3-16 floors	6.588244	1.92	5.00
roof	3.85725	0.96	2.00
elevator roof	4.71425	0.96	0

1.1. Lateral drift analysis and check under wind and seismic loads

For calculations of drift, frame 3B was considered. The frame is shown in Figure 1-1.

Figure 1-1. Layout of Frames

The constant parameters are shown in Table 1-2.

Table 1-2. The constant parameters

m	$E\ (kPa)$	Beam Size	$I_b\ (m^4)$	$L_b\ (m)$
8	3.00E+07	600x300	0.0054	5

, where

m - number of spans
E - modulus of elasticity of the concrete members
I_b - moment of inertia of the beam
L_b - length of the beam

Also, column sizes and heights vary by the floors. For the first 4 floors, column sizes are 750 x 750 mm. For 5th-8th floors it is 650 x 650 mm. Then it changes to 500 x 500 mm. And the top 4 floors have columns with dimensions 350 x 350 mm. The heights for the 1st and 2nd floors are 4.2 m, and for other typical floors are 3.5 m.

1.1.1. SAP2000 Wind drift check

The ASCE 7-16 standard does not provide a specific drift limit for wind design like it does for seismic design. However, the non-mandatory Appendix CC (Serviceability Considerations) of ASCE 7-16 mentions that typical building design uses drift limits ranging from 1/600 to 1/400 of the building or story height without going into further detail. Common wind drift limits vary from H/100 to H/600 for total building drift and h/200 to h/600 for interstory drift, depending on the building type and the cladding or partition materials used. The most common drift limits are H (or h)/400 to H (or h)/500, according to the ASCE Task Committee on Drift Control of Steel Building Structures, 1988.

Therefore, in order to check the building for lateral drift due to wind, the absolute displacement of each floor was taken from the SAP2000. Then the interstory displacement was calculated and both interstory and absolute values were checked with 1/400H (h).

$$\Delta_{allow}(absolute) = \frac{H}{400} = \frac{57400}{400} = 143.5\ mm$$

$$\Delta_{allow}(interstory) = \frac{h}{400} = \frac{3500}{400} = 8.75\ mm$$

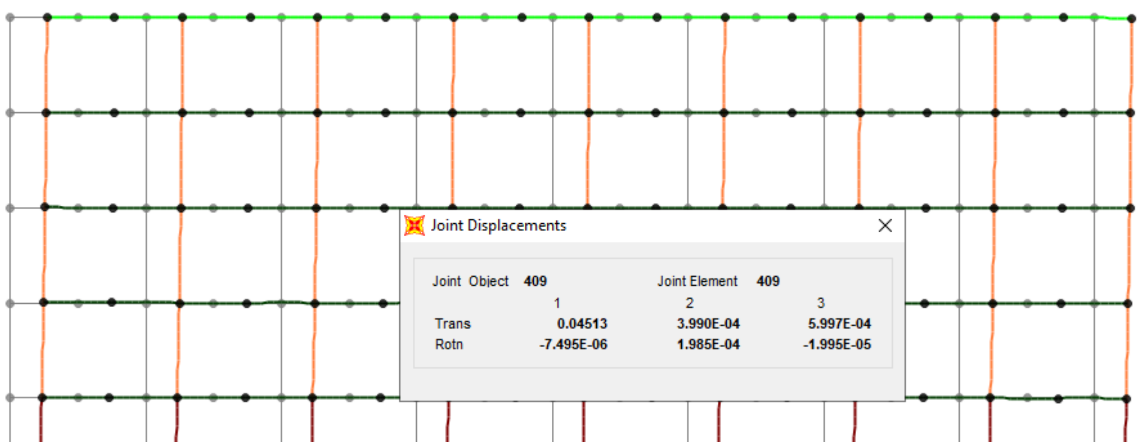


Figure 1-2. 3B frame displacement due to wind

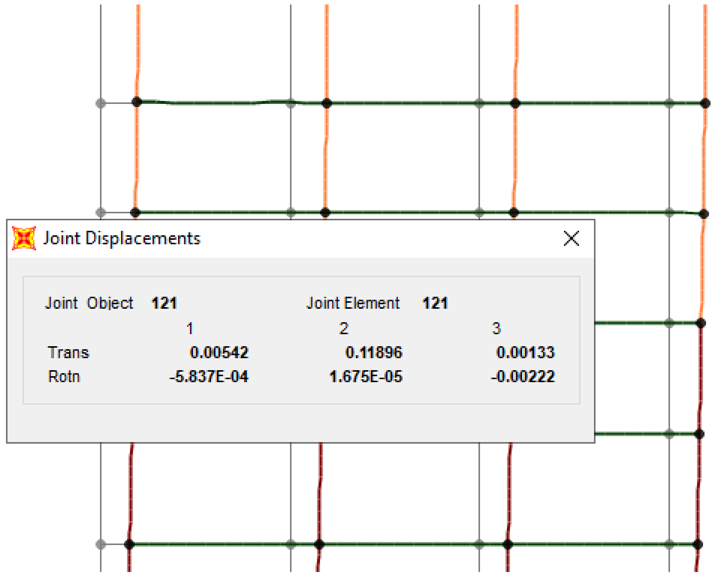


Figure 1-3. 9A frame displacement due to wind

Table 1-3. Frame 3B and 9A displacement

Frame 3B		Frame 9A	
Displacement (mm)	delta interstory (mm)	Displacement (mm)	delta interstory (mm)
45	1	118.9	7.5
44	1	111.4	6.3
43	2	105.1	8.5
41	3	96.6	7.4
38	2	89.2	7.6
36	5	81.6	8.1
31	3	73.5	6.5
28	3	67	8.2
25	4	58.8	6.2
21	3	52.6	7.3
18	4	45.3	7.7
14	4	37.6	8.1
10	4	29.5	6.4
6	2	23.1	7.8
4	2	15.3	7.2
2	2	8.1	8.1

1.1.2. SAP2000 Seismic drift check

Allowable drift for ACI code is 2%. Also, the obtained displacement must be multiplied by a factor $C_d = 5.5$.

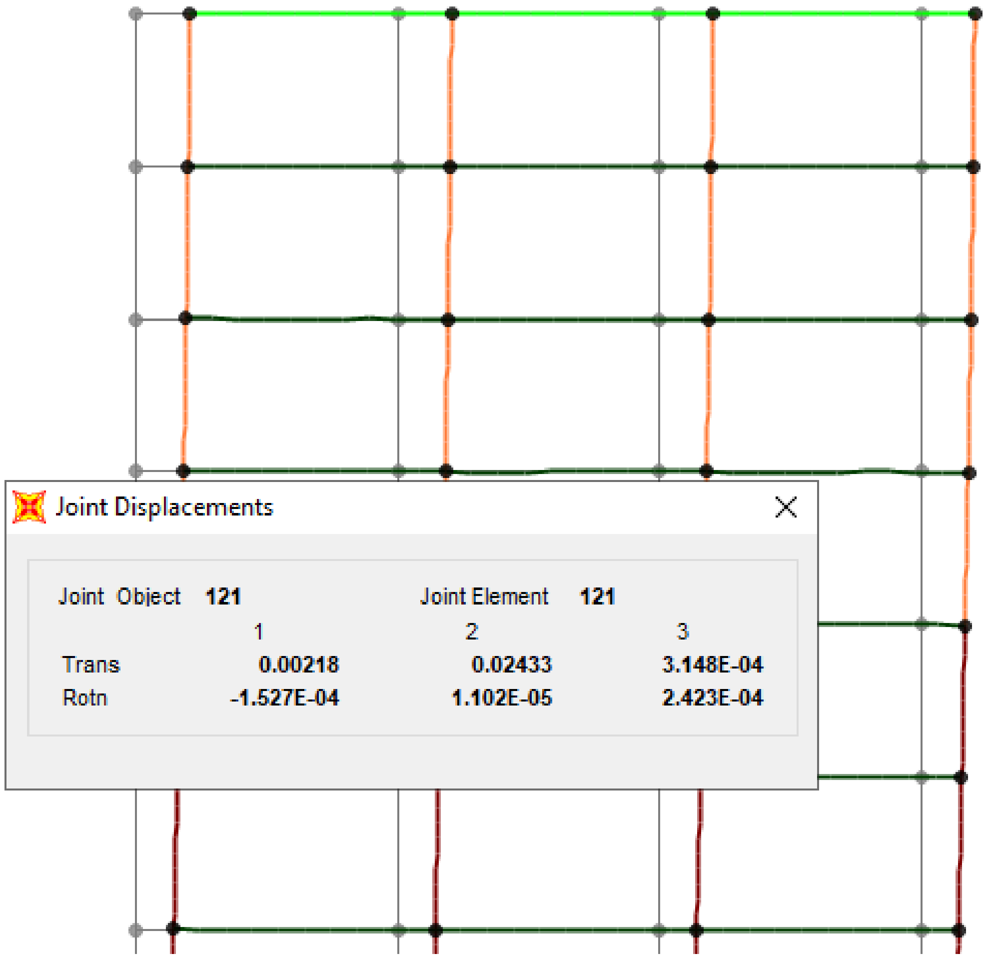


Figure 1-4. 9A frame displacement due to seismic loads

Table 1-4. Frame 9A drift due to seismic loads

Displacement (mm)	delta interstory (mm)	factored displacement (mm)	drift (%)
24	1	5.5	0.16%
23	1	5.5	0.16%
22	1	5.5	0.16%
21	1	5.5	0.16%
20	2	11	0.31%
18	1	5.5	0.16%
17	2	11	0.31%
15	2	11	0.31%
13	2	11	0.31%
11	1	5.5	0.16%
10	2	11	0.31%
8	2	11	0.31%
6	2	11	0.31%
4	2	11	0.31%
2	1	5.5	0.16%
1	1	5.5	0.16%

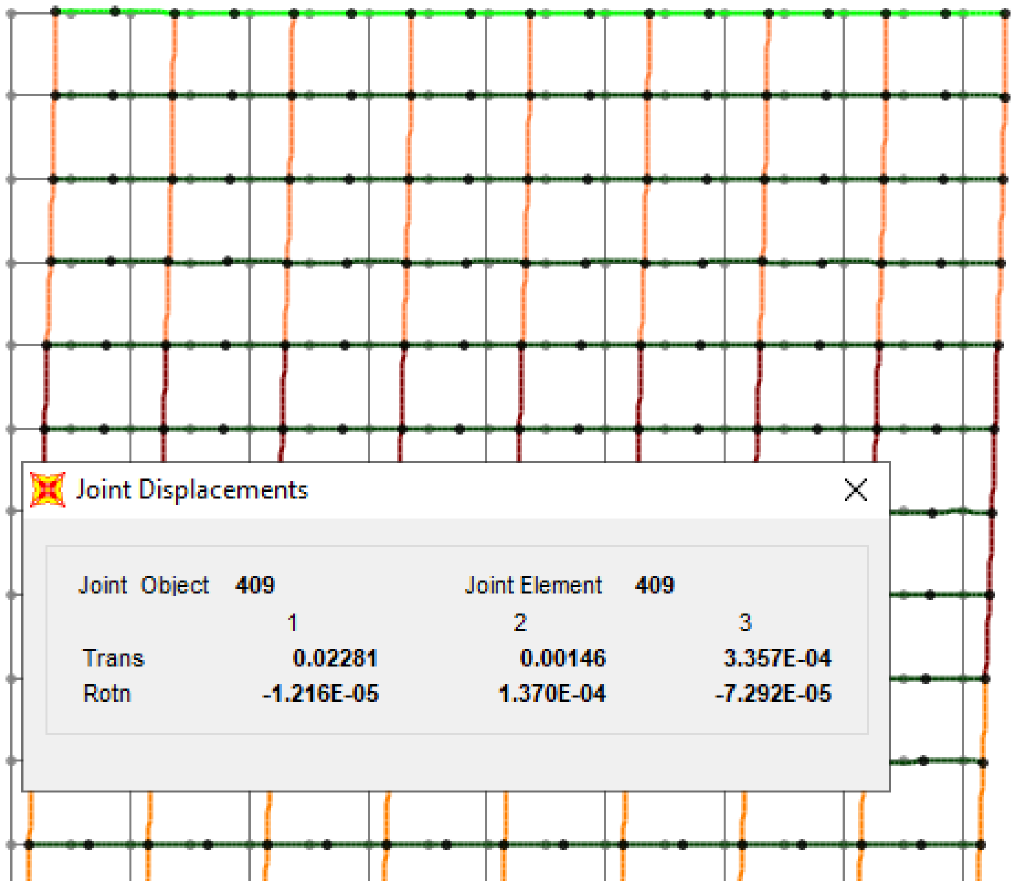


Figure 1-5. 3B frame displacement due to seismic loads

Table 1-5. Frame 3B drift due to seismic loads

Displacement (mm)	delta interstory (mm)	factored displacement (mm)	drift (%)
23	1	5.5	0.16%
22	1	5.5	0.16%
21	1	5.5	0.16%
20	2	11	0.31%
18	1	5.5	0.16%
17	1	5.5	0.16%
16	2	11	0.31%
14	2	11	0.31%
12	1	5.5	0.16%
11	2	11	0.31%
9	2	11	0.31%
7	1	5.5	0.16%
6	2	11	0.31%
4	2	11	0.31%
2	1	5.5	0.16%
1	1	5.5	0.16%

1.2. Internal force (M, N, V) analysis under all possible loads for all structural members

For the internal force analysis one row of external columns, one row of internal columns, one row of longitudinal beams, and one row of transverse beams were chosen, as shown in figure 1.6 below. The values were taken from SAP2000 under UDCON2, which is 1.2DL+1.6LL, and shown in tables 1.6-1.10.

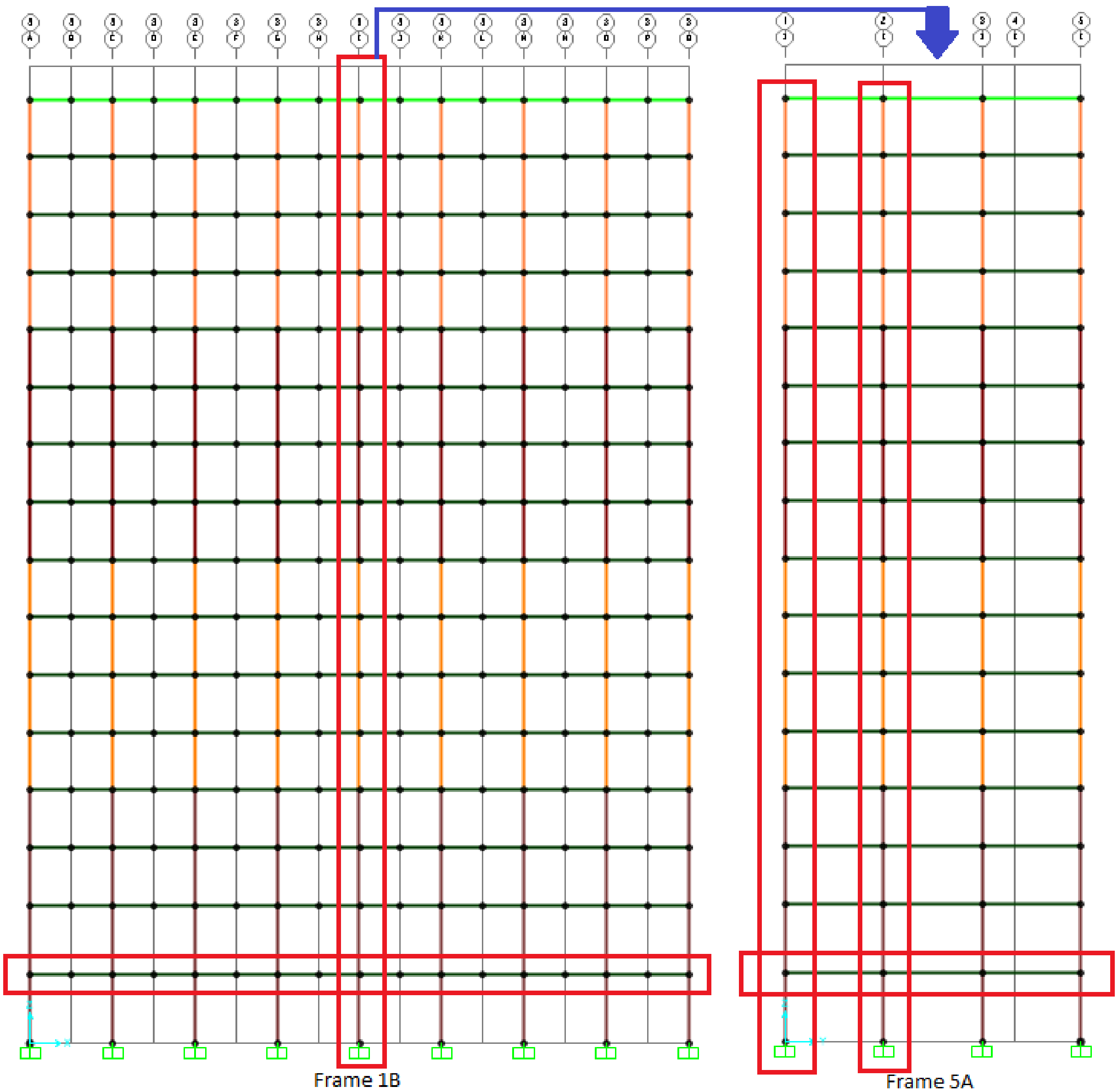


Figure 1-6. Chosen structural elements for internal force analysis

Table 1-6. Exterior column of Frame 5A under 1.2*DL+1.6*LL

Frame 5A	Station	P	V2	V3	T	M2	M3
Floor	m	KN	KN	KN	KN-m	KN-m	KN-m
1	0	-4975.15	-8.87E-13	-31.931	-3.50E-15	-41.8078	-4.23E-12
	4.2	-4878.955	-8.87E-13	-31.931	-3.50E-15	92.3024	-5.01E-13
2	0	-4542.106	-9.27E-13	-44.195	-6.00E-15	-109.4085	-2.76E-12
	4.2	-4445.912	-9.27E-13	-44.195	-6.00E-15	76.2085	1.13E-12
3	0	-4188.462	-9.72E-13	-37.133	-5.30E-15	-64.5918	-1.93E-12
	3.5	-4108.3	-9.72E-13	-37.133	-5.30E-15	65.3751	1.48E-12
4	0	-3848.97	-1.00E-12	-48.517	-4.28E-15	-80.9397	-1.84E-12
	3.5	-3768.808	-1.00E-12	-48.517	-4.28E-15	88.8684	1.67E-12
5	0	-3508.045	-9.97E-13	-37.87	-9.79E-16	-61.4879	-1.73E-12
	3.5	-3452.377	-9.97E-13	-37.87	-9.79E-16	71.0583	1.76E-12
6	0	-3190.026	-1.04E-12	-46.878	5.86E-16	-83.4535	-1.82E-12
	3.5	-3134.358	-1.04E-12	-46.878	5.86E-16	80.6198	1.84E-12
7	0	-2869.769	-1.04E-12	-45.073	1.54E-15	-80.7171	-1.76E-12
	3.5	-2814.101	-1.04E-12	-45.073	1.54E-15	77.0383	1.86E-12
8	0	-2547.391	-1.07E-12	-55.962	3.02E-15	-90.8002	-1.80E-12
	3.5	-2491.723	-1.07E-12	-55.962	3.02E-15	105.0661	1.95E-12
9	0	-2224.475	-9.61E-13	-40.15	1.76E-15	-63.6602	-1.61E-12
	3.5	-2188.848	-9.61E-13	-40.15	1.76E-15	76.8657	1.75E-12
10	0	-1921.79	-9.85E-13	-50.711	2.02E-15	-89.6456	-1.67E-12
	3.5	-1886.163	-9.85E-13	-50.711	2.02E-15	87.8423	1.78E-12
11	0	-1616.884	-9.00E-13	-48.063	2.31E-15	-85.6061	-1.54E-12
	3.5	-1581.257	-9.00E-13	-48.063	2.31E-15	82.6136	1.61E-12
12	0	-1309.81	-9.97E-13	-61.094	3.05E-15	-97.7505	-1.56E-12
	3.5	-1274.183	-9.97E-13	-61.094	3.05E-15	116.0799	1.93E-12
13	0	-1004.87	-7.18E-13	-35.724	2.29E-15	-55.7695	-1.21E-12
	3.5	-984.83	-7.18E-13	-35.724	2.29E-15	69.2648	1.30E-12
14	0	-721.097	-6.24E-13	-45.3	2.14E-15	-80.2658	-1.07E-12
	3.5	-701.057	-6.24E-13	-45.3	2.14E-15	78.2842	1.12E-12
15	0	-435.211	-5.10E-13	-45.847	2.26E-15	-78.8475	-8.37E-13
	3.5	-415.171	-5.10E-13	-45.847	2.26E-15	81.6163	9.47E-13
16	0	-149.925	-3.46E-13	-38.297	5.57E-15	-72.4771	-6.27E-13
	3.5	-129.885	-3.46E-13	-38.297	5.57E-15	61.562	5.82E-13

Table 1-7. Interior column of Frame 5A under 1.2*DL+1.6*LL

Frame 5A	Station	P	V2	V3	T	M2	M3
Floor	m	KN	KN	KN	KN-m	KN-m	KN-m
1	0	-6696.52	-8.77E-13	0.761	-3.50E-15	1.8423	-4.20E-12
	4.2	-6631.78	-8.77E-13	0.761	-3.50E-15	-1.3532	-5.19E-13
2	0	-6352.21	-9.27E-13	-0.808	-6.00E-15	-1.473	-2.75E-12
	4.2	-6301.36	-9.27E-13	-0.808	-6.00E-15	1.9194	1.15E-12
3	0	-6094.169	-9.45E-13	-4.367	-5.30E-15	-9.2818	-1.91E-12
	3.5	-6014.008	-9.45E-13	-4.367	-5.30E-15	6.004	1.40E-12
4	0	-5603.922	-9.42E-13	-6.501	-4.28E-15	-11.4969	-1.72E-12
	3.5	-5523.76	-9.42E-13	-6.501	-4.28E-15	11.2572	1.57E-12
5	0	-5114.959	-9.38E-13	-6.364	-9.79E-16	-11.0673	-1.65E-12
	3.5	-5059.291	-9.38E-13	-6.364	-9.79E-16	11.2072	1.63E-12
6	0	-4651.832	-9.29E-13	-9.354	5.86E-16	-16.9061	-1.63E-12
	3.5	-4596.164	-9.29E-13	-9.354	5.86E-16	15.8316	1.62E-12
7	0	-4190.71	-9.55E-13	-10.71	1.54E-15	-19.4467	-1.59E-12
	3.5	-4135.042	-9.55E-13	-10.71	1.54E-15	18.0386	1.76E-12
8	0	-3731.5	-1.02E-12	-14.601	3.02E-15	-24.014	-1.72E-12
	3.5	-3675.832	-1.02E-12	-14.601	3.02E-15	27.0891	1.83E-12
9	0	-3272.74	-9.40E-13	-10.51	1.76E-15	-17.0153	-1.57E-12
	3.5	-3237.113	-9.40E-13	-10.51	1.76E-15	19.7705	1.72E-12
10	0	-2833.743	-9.78E-13	-14.073	2.02E-15	-24.6286	-1.66E-12
	3.5	-2798.115	-9.78E-13	-14.073	2.02E-15	24.6285	1.77E-12
11	0	-2396.766	-9.86E-13	-15.05	2.31E-15	-26.4957	-1.63E-12
	3.5	-2361.139	-9.86E-13	-15.05	2.31E-15	26.1793	1.82E-12
12	0	-1961.788	-1.10E-12	-19.343	3.05E-15	-31.4232	-1.79E-12
	3.5	-1926.161	-1.10E-12	-19.343	3.05E-15	36.2774	2.06E-12
13	0	-1524.829	-7.70E-13	-9.282	2.29E-15	-15.1395	-1.28E-12
	3.5	-1504.789	-7.70E-13	-9.282	2.29E-15	17.346	1.42E-12
14	0	-1098.225	-7.64E-13	-11.608	2.14E-15	-20.0303	-1.28E-12
	3.5	-1078.184	-7.64E-13	-11.608	2.14E-15	20.5979	1.39E-12
15	0	-673.684	-6.59E-13	-12.29	2.26E-15	-21.316	-1.12E-12
	3.5	-653.643	-6.59E-13	-12.29	2.26E-15	21.6982	1.18E-12
16	0	-248.662	-4.62E-13	-10.314	5.57E-15	-18.954	-8.36E-13
	3.5	-228.621	-4.62E-13	-10.314	5.57E-15	17.1457	7.80E-13

Table 1-8. Longitudinal beams internal force analysis under 1.2*DL+1.6*LL

Longitudinal beams	Station	V2	T	M3
ID	m	KN	KN-m	KN-m
589	0	-95.39	-44.84	-104.69
589	1	-81.061	-44.8375	-16.4598
589	2	-66.73	-44.8375	57.4357
589	3	64.599	43.7858	58.5468
589	4	78.931	43.7858	-13.2179
589	5	93.262	43.7858	-99.3142
590	0	-94.511	-44.3023	-103.4255
590	1	-80.179	-44.3023	-16.0808
590	2	-65.847	-44.3023	56.9324
590	3	65.474	44.2895	57.1279
590	4	79.805	44.2895	-15.5117
590	5	94.137	44.2895	-102.4828
591	0	-94.386	-44.3621	-103.0604
591	1	-80.055	-44.3621	-15.8401
591	2	-65.723	-44.3621	57.0487
591	3	65.708	44.3595	57.0585
591	4	80.039	44.3595	-15.8152
591	5	94.371	44.3595	-103.0204
592	0	-94.379	-44.3638	-103.045
592	1	-80.048	-44.3638	-15.8315
592	2	-65.716	-44.3638	57.0505
592	3	65.717	44.3636	57.0505
592	4	80.048	44.3636	-15.832
592	5	94.38	44.3636	-103.0461
593	0	-94.38	-44.3636	-103.0461
593	1	-80.048	-44.3636	-15.832
593	2	-65.717	-44.3636	57.0505
593	3	65.716	44.3638	57.0505
593	4	80.048	44.3638	-15.8315
593	5	94.379	44.3638	-103.045
594	0	-94.371	-44.3595	-103.0204
594	1	-80.039	-44.3595	-15.8152

594	2	-65.708	-44.3595	57.0585
594	3	65.723	44.3621	57.0487
594	4	80.055	44.3621	-15.8401
594	5	94.386	44.3621	-103.0604
595	0	-94.137	-44.2895	-102.4828
595	1	-79.805	-44.2895	-15.5117
595	2	-65.474	-44.2895	57.1279
595	3	65.847	44.3023	56.9324
595	4	80.179	44.3023	-16.0808
595	5	94.511	44.3023	-103.4255
596	0	-93.262	-43.7858	-99.3142
596	1	-78.931	-43.7858	-13.2179
596	2	-64.599	-43.7858	58.5468
596	3	66.73	44.8375	57.4357
596	4	81.061	44.8375	-16.4598
596	5	95.393	44.8375	-104.6869

Table 1-9. Transverse beams internal force analysis under 1.2*DL+1.6*LL

Transverse beams	Station	V2	T	M3
ID	m	KN	KN-m	KN-m
633	0	-113.521	1.40E-16	-113.262
633	0.5	-94.713	1.40E-16	-61.2037
633	1	-75.904	1.40E-16	-18.5494
633	1.5	-57.096	1.40E-16	14.7006
633	2	-38.288	1.40E-16	38.5466
633	2.5	-19.479	1.40E-16	52.9884
633	3	-0.671	1.40E-16	58.026
633	3.5	18.137	1.40E-16	53.6596
633	4	36.945	1.40E-16	39.8889
633	4.5	55.754	1.40E-16	16.7142
633	5	74.562	1.40E-16	-15.8647
633	5.5	93.37	1.40E-16	-57.8477
633	6	112.178	1.40E-16	-109.2349
634	0	-112.792	-2.04E-17	-112.6995
634	0.5	-93.984	-2.04E-17	-61.0056

634	1	-75.175	-2.04E-17	-18.7159
634	1.5	-56.367	-2.04E-17	14.1697
634	2	-37.559	-2.04E-17	37.6512
634	2.5	-18.751	-2.04E-17	51.7285
634	3	0.058	-2.04E-17	56.4017
634	3.5	18.866	-2.04E-17	51.6707
634	4	37.674	-2.04E-17	37.5356
634	4.5	56.483	-2.04E-17	13.9964
634	5	75.291	-2.04E-17	-18.947
634	5.5	94.099	-2.04E-17	-61.2945
634	6	112.907	-2.04E-17	-113.0461
635	0	-112.035	6.78E-16	-108.8075
635	0.5	-93.226	6.78E-16	-57.4922
635	1	-74.418	6.78E-16	-15.5811
635	1.5	-55.61	6.78E-16	16.9259
635	2	-36.802	6.78E-16	40.0287
635	2.5	-17.993	6.78E-16	53.7274
635	3	0.815	6.78E-16	58.022
635	3.5	19.623	6.78E-16	52.9124
635	4	38.432	6.78E-16	38.3987
635	4.5	57.24	6.78E-16	14.4809
635	5	76.048	6.78E-16	-18.8411
635	5.5	94.856	6.78E-16	-61.5673
635	6	113.665	6.78E-16	-113.6975

Table 1-10. Internal Force

Frame	Station	Output Case	CaseType	P	V2	V3	T	M2	M3
Text	m	Text	Text	KN	KN	KN	KN-m	KN-m	KN-m
1	0	DCON 1	1.4*DL	-3553.666	-13.034	-13.979	-2.79E-15	-18.6041	-17.4025
1	2.1	DCON 1	1.4*DL	-3497.553	-13.034	-13.979	-2.79E-15	10.7512	9.9681
1	4.2	DCON 1	1.4*DL	-3441.44	-13.034	-13.979	-2.79E-15	40.1065	37.3388
1	0	DCON 2	1.2*DL+1.6 LL	-3493.654	-17.119	-20.822	-3.33E-15	-26.9756	-22.8571
1	2.1	DCON 2	1.2*DL+1.6 LL	-3445.557	-17.119	-20.822	-3.33E-15	16.7514	13.0925
1	4.2	DCON 2	1.2*DL+1.6 LL	-3397.46	-17.119	-20.822	-3.33E-15	60.4783	49.0421
1	0	DCON 3	1.2*DL+1*L L+1*W	-2922.863	45.236	-17.278	-0.1061	-21.5284	293.117
1	2.1	DCON 3	1.2*DL+1*L L+1*W	-2874.766	45.236	-17.278	-0.1061	14.7548	198.1217
1	4.2	DCON 3	1.2*DL+1*L L+1*W	-2826.669	45.236	-17.278	-0.1061	51.0379	103.1263
1	0	DCON 4	1.2*DL+1*L L-1*W	-3728.704	-75.013	-17.737	0.1061	-24.1509	-332.8757
1	2.1	DCON 4	1.2*DL+1*L L-1*W	-3680.607	-75.013	-17.737	0.1061	13.0959	-175.348
1	4.2	DCON 4	1.2*DL+1*L L-1*W	-3632.51	-75.013	-17.737	0.1061	50.3427	-17.8202
1	0	DCON 33	0.9*DL+1* W-44	-1955.609	31.393	2.712	-5.0534	51.5023	195.2861
1	2.1	DCON 33	0.9*DL+1* W-45	-1919.536	31.393	2.712	-5.0534	45.8081	129.3609
1	4.2	DCON 33	0.9*DL+1* W-46	-1883.463	31.393	2.712	-5.0534	40.1139	63.4357
1	0	DCON 34	0.9*DL-1* W-44	-2613.391	-48.15	-20.684	5.0534	-75.4219	-217.6607
1	2.1	DCON 34	0.9*DL-1* W-45	-2577.318	-48.15	-20.684	5.0534	-31.9851	-116.5447
1	4.2	DCON 34	0.9*DL-1* W-46	-2541.245	-48.15	-20.684	5.0534	11.4516	-15.4287

1	0	DCON 51	1.24*DL+1*LL+1*E	-3227.238	5.598	-16.768	-0.5285	-16.8472	94.0119
1	2.1	DCON 51	1.24*DL+1*LL+1*E	-3177.538	5.598	-16.768	-0.5285	18.3647	82.2554
1	4.2	DCON 51	1.24*DL+1*LL+1*E	-3127.838	5.598	-16.768	-0.5285	53.5767	70.499
1	0	DCON 52	1.24*DL+1*LL-1*E	-3627.396	-36.12	-19.045	0.5285	-29.8951	-134.765
1	2.1	DCON 52	1.24*DL+1*LL-1*E	-3577.696	-36.12	-19.045	0.5285	10.1003	-58.9121
1	4.2	DCON 52	1.24*DL+1*LL-1*E	-3527.996	-36.12	-19.045	0.5285	50.0957	16.9407
1	0	DCON 55	0.86*DL+1E	-1982.887	12.853	-7.448	-0.5285	-4.9043	103.6983
1	2.1	DCON 55	0.86*DL+1E	-1948.418	12.853	-7.448	-0.5285	10.7365	76.7071
1	4.2	DCON 55	0.86*DL+1E	-1913.948	12.853	-7.448	-0.5285	26.3773	49.7158
1	0	DCON 56	0.86*DL-1E	-2383.046	-28.866	-9.726	0.5285	-17.9522	-125.0786
1	2.1	DCON 56	0.86*DL-1E	-2348.576	-28.866	-9.726	0.5285	2.4721	-64.4605
1	4.2	DCON 56	0.86*DL-1E	-2314.106	-28.866	-9.726	0.5285	22.8964	-3.8424

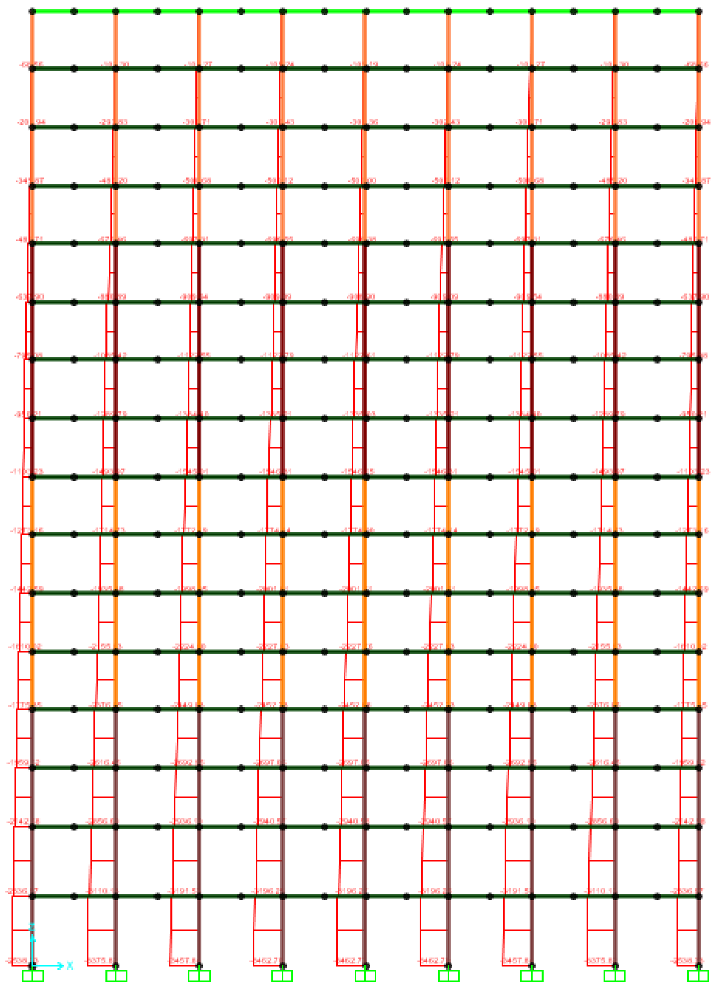


Figure 1-7. Frame axial force diagram under the Dead load

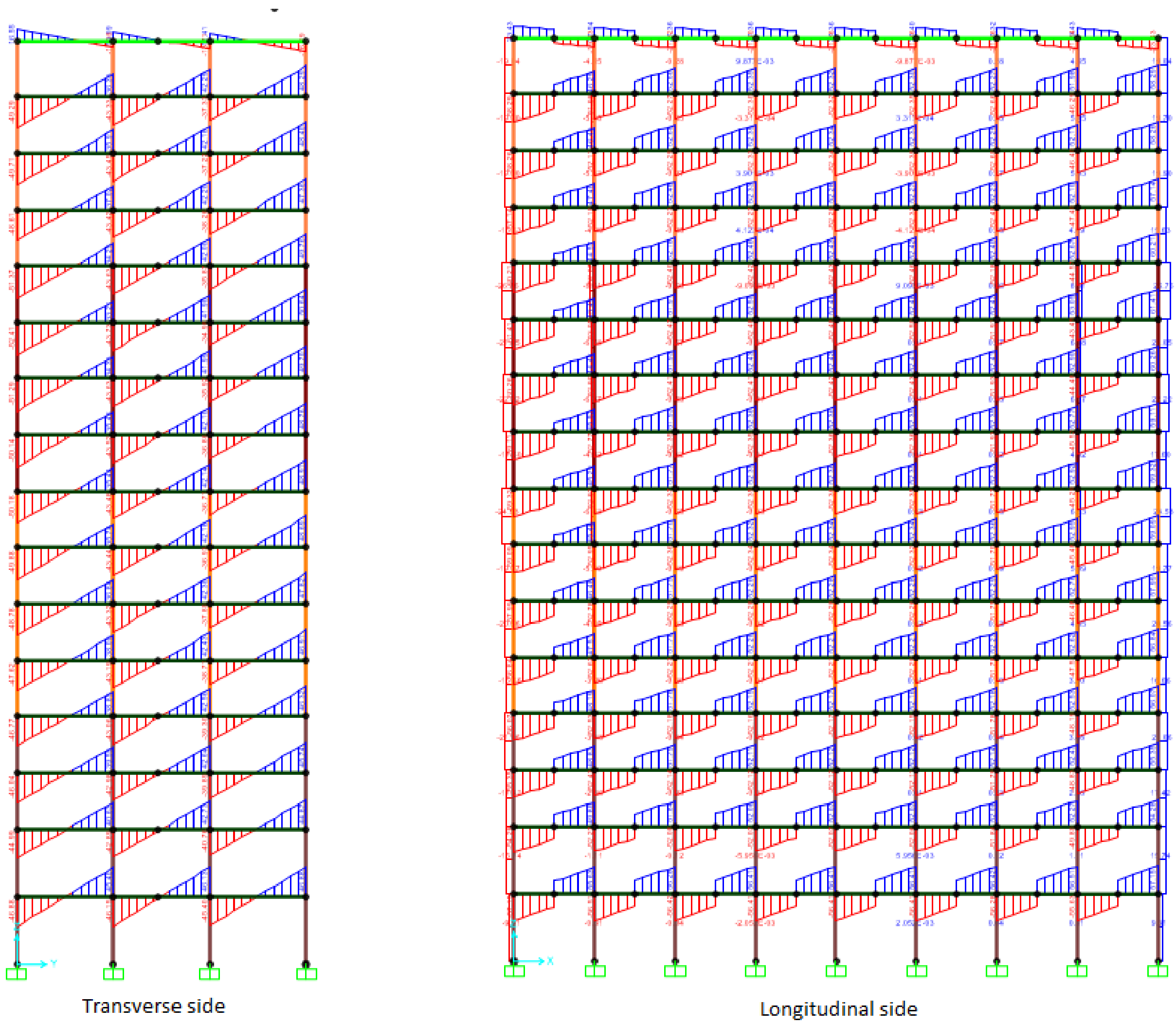


Figure 1-8. Frame shear 2-2 diagram under the Dead load

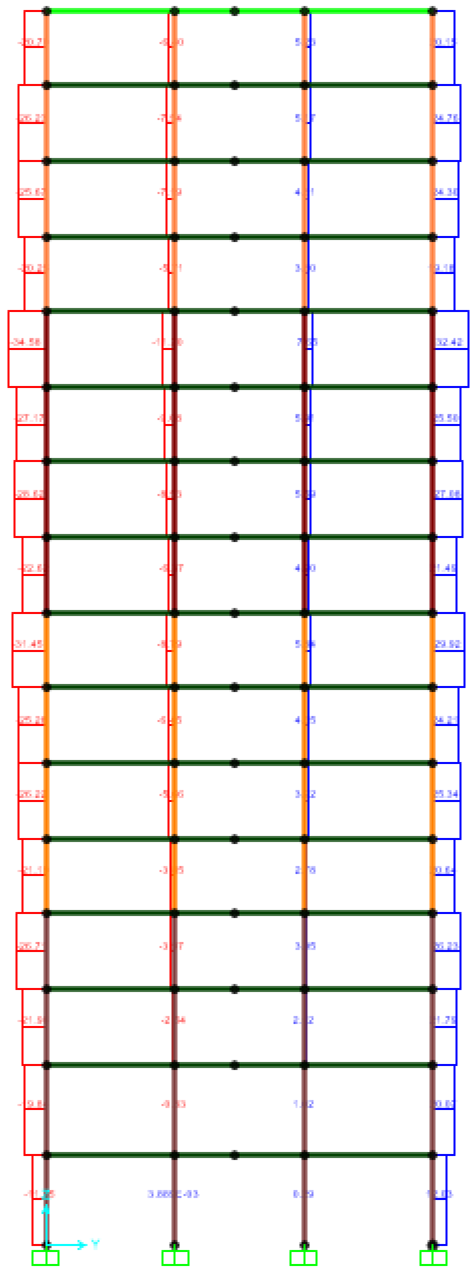


Figure 1-9. Frame shear 3-3 diagram under the Dead load

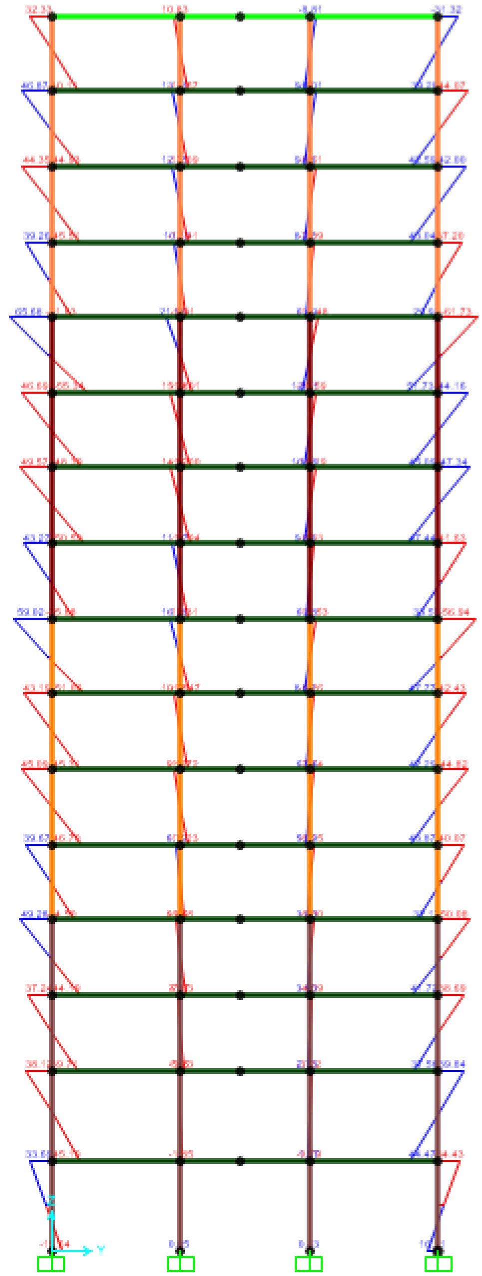


Figure 1-10. Frame Moment 2-2 diagram under the Dead load

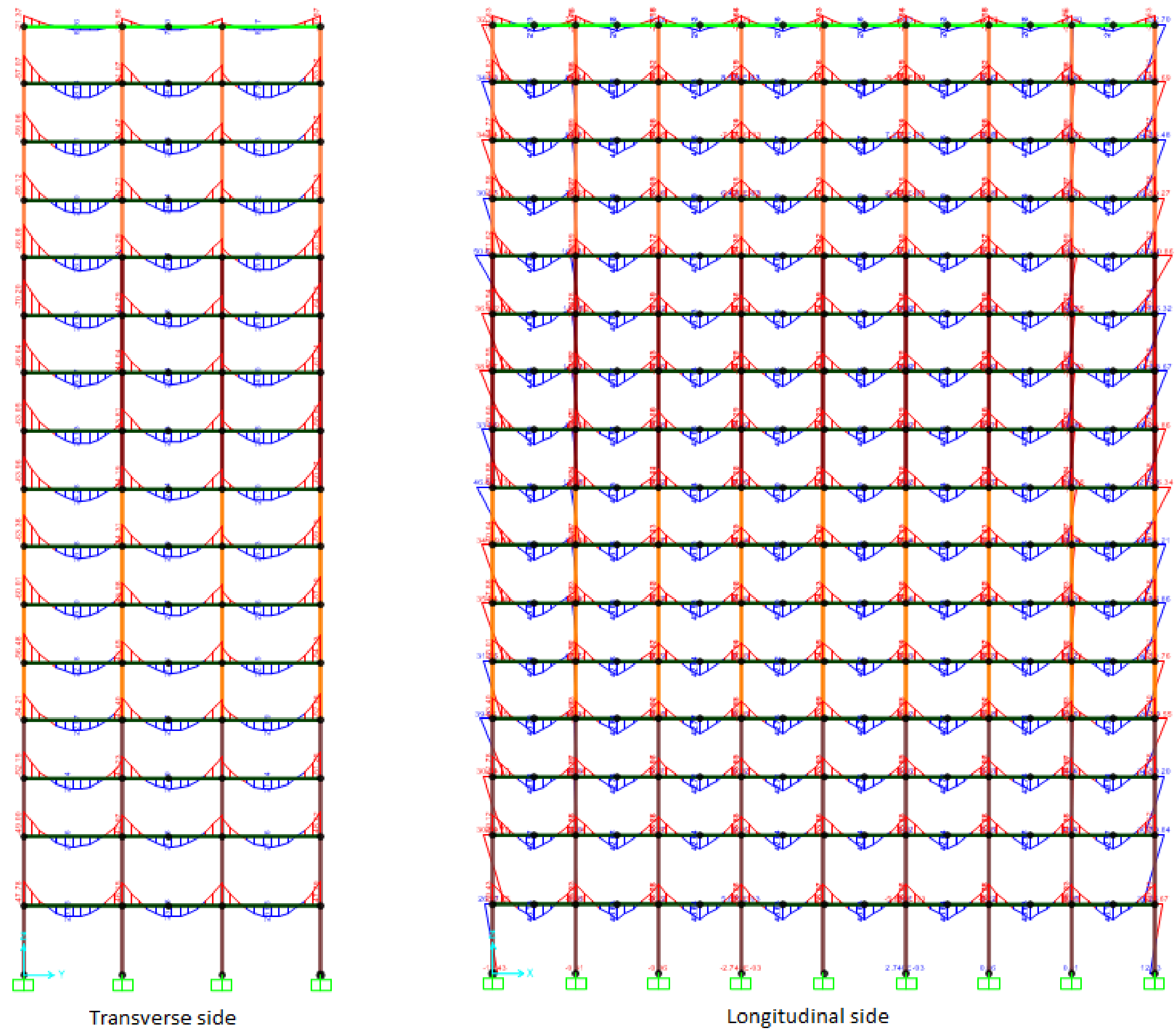


Figure 1-11. Frame moment 3-3 diagram under the Dead load

1.3. Hand calculation verification for internal forces

1.3.1. One vertical load case (Live load)

Space of the minor beam is 2.5 m and the span is 6 m. Loads in kPa are different for Roof, restaurant, residential, SPA and 1st floors. Also the height of the first two floors are 4.2 m and others 3.5 m.

The minor beam is simply supported, so the point of zero moment is located at the both ends of the beam, the basic method is applied. While the major beams have both fixed ends, the approximate method is used.

$$Live\ load\ reduction\ =\ 1\ or\ higher$$

$$W_u^{minor} = Live\ Load\ in\ kPa\ *\ Space\ of\ minor\ beam$$

$$R_{end}^{minor} = W_u^{minor} *\ Span\ of\ minor\ beam$$

$$P_{minor} = R_{end}^{minor} *\ 2$$

According to the approximate method the middle span is 0.8L, 4 m and the end span is 0.1L. 0.5 m.

$$R_{middle}^{major} = P_{minor} / 2$$

$$R_{end}^{major} = R_{middle}^{major} =\ N$$

$$M^{middle} = P_{minor} *\ Middle\ span / 4$$

$$M^{end} = R_{end}^{major} *\ End\ span$$

$$M^{end} = M^{top} + M^{bottom}$$

$$M^{top} = M^{end} * \frac{E_1 I_1}{L_1} / (\frac{E_1 I_1}{L_1} + \frac{E_2 I_2}{L_2})$$

$$M^{bottom} = M^{end} * \frac{E_2 I_2}{L_2} / (\frac{E_1 I_1}{L_1} + \frac{E_2 I_2}{L_2})$$

$$V\ *\ column\ height\ = M^{top} + M^{bottom}$$

Calculation of live load internal forces:

$$W_u^{minor} = 1.92\ kPa\ *\ 2.5\ m = 4.8\ kN/m$$

$$R_{end}^{minor} = 4.8\ kN/m\ *\ 6m/2 = 14.4kN$$

$$P_{minor} = 14.4\ *\ 2 = 28.8\ kN$$

$$R_{middle}^{major} = 28.8\ kN/2 = 14.4\ kN$$

$$M^{middle} = 28.8\ kN\ *\ 4m\ /\ 4 = 14.85\ kNm$$

$$M^{end} = 14.4kN\ *\ 0.5\ m = 7.2\ kNm$$

$$I = bh^3/12 = h^4 12 = 0.35^4/12 = 0.00521\ m^4$$

$$M^{top} = 7.2kNm\ * \frac{E_1 I_1}{L_1}\ /\ (\frac{E_1 I_1}{L_1} + \frac{E_2 I_2}{L_2}) = 7.2/2 = 3.6\ kNm$$

$$M^{bottom} = 7.2kNm\ * \frac{E_2 I_2}{L_2}\ /\ (\frac{E_1 I_1}{L_1} + \frac{E_2 I_2}{L_2}) = 7.2/2 = 3.6kNm$$

$$V = (M^{top} + M^{bottom})/column\ height = 7.2kNm/3.5m = 2.057\ kN$$

Table 1-11. Live Load Internal Forces' hand calculation results

	I , m^4	Mmiddle . kNm	Mend, kNm	Mtop, kNm	Mbottom , kNm	V col , kN	V beam , kN	N ext, kN	N int, kN
Roof	0.00342	14.85	3.7125		3.713	1.06	7.425	7.425	14.85
Slab 16	0.00342	71.85	17.9625	8.981	8.981	5.13	35.925	43.35	86.7
Slab 15	0.00342	28.8	7.2	3.600	3.600	2.06	14.4	57.75	115.5
Slab 14	0.00342	28.8	7.2	3.600	3.600	2.06	14.4	72.15	144.3
Slab 13	0.00342	28.8	7.2	1.731	5.469	2.06	14.4	86.55	173.1
Slab 12	0.01080	28.8	7.2	3.600	3.600	2.06	14.4	100.95	201.9
Slab 11	0.01080	28.8	7.2	3.600	3.600	2.06	14.4	115.35	230.7
Slab 10	0.01080	28.8	7.2	3.600	3.600	2.06	14.4	129.75	259.5
Slab 9	0.01080	28.8	7.2	2.092	5.108	2.06	14.4	144.15	288.3
Slab 8	0.02637	28.8	7.2	3.600	3.600	2.06	14.4	158.55	317.1
Slab 7	0.02637	28.8	7.2	3.600	3.600	2.06	14.4	172.95	345.9
Slab 6	0.02637	28.8	7.2	3.600	3.600	2.06	14.4	187.35	374.7
Slab 5	0.02637	28.8	7.2	2.343	4.857	2.06	14.4	201.75	403.5
Slab 4	0.05468	28.8	7.2	3.600	3.600	2.06	14.4	216.15	432.3
Slab 3	0.05468	28.8	7.2	3.927	3.273	2.06	14.4	230.55	461.1
Slab 2	0.05468	71.85	17.9625	8.981	8.981	4.28	35.925	266.475	532.95
Slab 1	0.05468	71.85	17.9625	17.963		4.28	35.925	302.4	604.8

Interior columns can bear twice the weight of exterior columns. Therefore, to determine the axial forces in the interior columns, simply double the values of the exterior columns.

Table 1-12 were constructed to illustrate the comparison of axial, shear forces, and bending moment. The axial forces in the columns exhibit a close match for each floor. Nonetheless, there are slight variations between the values obtained from the SAP2000 model and those from hand calculations, likely stemming from the approximations inherent in the hand calculations, which rely on several assumptions. It's important to acknowledge that before was introduced certain degrees of approximation, potentially leading to slight discrepancies when compared with more detailed analyses or numerical simulations.

Table 1-12. The Internal Forces from hand calculation and SAP2000

	Hand Calc			SAP2000		
	V beam , kN	M, kNm	N , kN	V beam , kN	M, kNm	N , kN
Roof	7.43	14.85	14.85	5.41	11.42	14.959
Slab 16	17.08	44.55	86.70	6.62	11.74	45.597
Slab 15	14.40	28.80	115.50	6.76	11.75	75.925
Slab 14	14.40	28.80	144.30	6.80	11.99	106.24
Slab 13	14.40	28.80	173.10	5.37	8.43	136.643
Slab 12	14.40	28.80	201.90	9.39	15.04	167.12
Slab 11	14.40	28.80	230.70	7.40	13.11	197.489
Slab 10	14.40	28.80	259.50	7.75	13.63	227.746
Slab 9	14.40	28.80	288.30	6.14	9.68	257.978
Slab 8	14.40	28.80	317.10	8.60	13.86	288.164
Slab 7	14.40	28.80	345.90	6.91	12.21	318.227
Slab 6	14.40	28.80	374.70	7.14	12.53	348.161
Slab 5	14.40	28.80	403.50	5.72	9.07	377.975
Slab 4	14.40	28.80	432.30	7.47	12.41	407.686
Slab 3	14.40	28.80	461.10	4.91	7.55	437.252
Slab 2	17.08	44.55	532.95	8.86	23.55	510.29
Slab 1	17.08	44.55	604.80	7.66	10.23	561.319

1.3.2. One horizontal load case (Wind)

For Hand Calculations of Internal Forces Portal Frame method was applied. The distribution of the lateral force to the columns were in the following way:

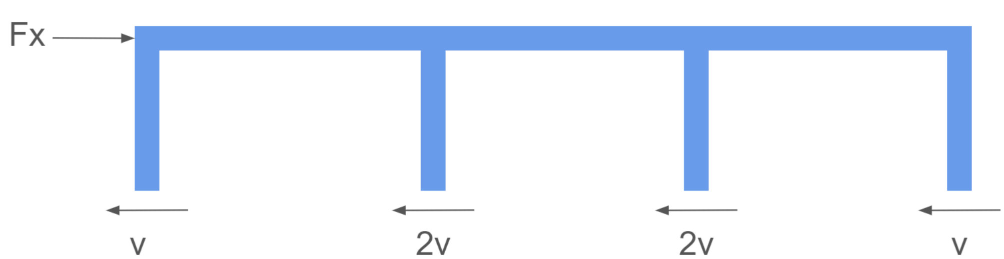


Figure 1- 12. Portal Frame method

Horizontal load case was chosen Wind lateral force towards 18 m and 5A frame was selected for analytical calculations. By computing internal forces: shear and axial in all members of 5A frame, the shear forces and moments at the ground level are identified. The cumulative wind force on the 1st floor is $660\ kN$. The numerical results from hand calculations and SAP 2000 are depicted in Table 1-13. Figure 1-13 shows joint reactions in the Joint Local Coordinate system in SAP 2000.

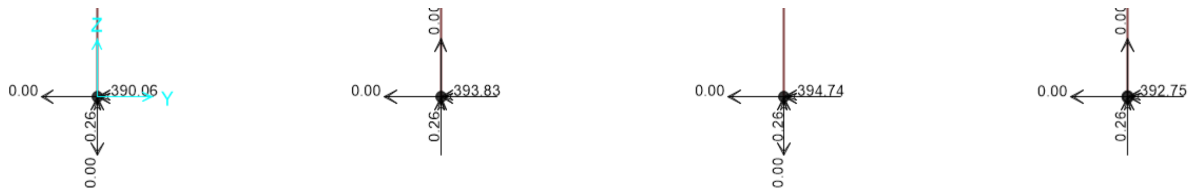


Figure 1-13. Joint Reactions due to wind load on the frame 5A

Table 1-13.WindLoad Internal Forces' hand calculation results

Floors	V ext col, kN	V beam, kN	Mbeam, kNm	Mmid col, kNm	N axial, kNm
16	-8.6	-	-15.1	15.1	-
15	-16.6	5.0	59.2	44.1	5.0
14	-24.4	14.7	131.0	71.8	19.7
13	-32.1	23.9	229.9	98.9	43.7
12	-39.6	33.0	355.3	125.5	76.6
11	-46.9	41.8	506.8	151.4	118.4
10	-54.1	50.5	683.6	176.8	168.9
9	-61.1	58.9	885.1	201.5	227.9
8	-67.8	67.2	1110.7	225.5	295.0
7	-74.3	75.2	1359.4	248.8	370.2
6	-80.6	82.9	1630.6	271.2	453.1
5	-86.6	90.4	1923.3	292.7	543.5
4	-92.3	97.6	2236.4	313.1	641.1
3	-97.6	104.4	2568.8	332.4	745.5
2	-103.5	110.8	2957.1	388.3	856.3
1	-110.3	129.4	3406.2	449.1	985.7
0	-110.3	149.7	231.7	-	1135.4

Table 1-14. Internal Forces from hand calculations and SAP 2000

Floors	SAP2000			Hand calculation		
	V, kN	M, kNm	N axial, kNm	V, kN	M, kNm	N axial, kNm
16	3.3	5.8	8.6	2.5	15.1	5.0
15	10.0	10.9	25.2	9.9	44.1	19.7
14	20.0	16.7	49.6	21.8	71.8	43.7
13	33.1	21.8	81.7	38.3	98.9	76.6
12	50.3	27.3	121.3	59.2	125.5	118.4
11	70.2	32.3	168.3	84.5	151.4	168.9
10	93.5	37.9	222.4	113.9	176.8	227.9
9	119.4	42.0	283.4	147.5	201.5	295.0
8	149.2	46.6	351.2	185.1	225.5	370.2
7	181.5	50.7	425.6	226.6	248.8	453.1
6	216.7	56.0	506.2	271.8	271.2	543.5
5	254.3	60.0	592.8	320.5	292.7	641.1
4	295.5	67.1	685.1	372.7	313.1	745.5
3	338.7	77.5	782.8	428.1	332.4	856.3
2	383.6	127.0	886.3	492.9	388.3	985.7
1	430.9	216.5	996.6	567.7	449.1	1135.4

The difference in the hand and the numerical modeling findings can be explained by the fact that the hand calculations are based on the approximate method: Portal Frame. There are assumptions in the analytical calculations, while SAP 2000 results are based on the Finite Element Method, which can be the reason for distinct results.

1.4. Structural member design (size or check reinforcement) using software

1.4.1. All slabs panels

Table 1-15. Slab reinforcement design from SAP 2000

# story	As/s (in2/in)
1-4	0.001755
5-8	0.001322
9-12	0.001309
13-16	0.001302

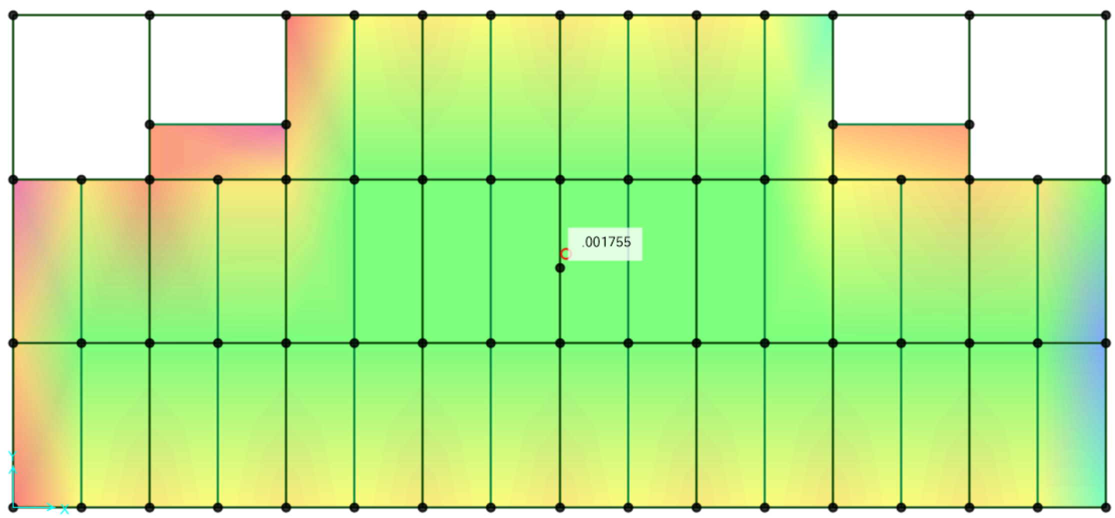


Figure 1-14. 1-4 floor slab reinforcement

Due to the fact that SAP2000 shows values less than minimum allowable, the slabs were designed using hand calculations, which is explained in the following chapter.

1.4.2. All major and minor beams

Major and minor beams were designed with flexural and shear reinforcement. The torsion reinforcement is not needed, because the torsion check om SAP2000 design has shown null.

1.4.2.1. Major beam

In order to deal with lateral drift the size of major beams was changed to the following dimensions:

Table 1-16. Sizes of structural members

Beam design	Major Beam	Minor Beam	Roof Major
h (m)	0.7	0.4	0.5
b (m)	0.35	0.2	0.25

Table B.6 Minimum Thickness of Beams and One-Way Slabs

Member	Yield Strength f_y (MPa)	Simply Supported	One End Continuous	Both Ends Continuous	Cantilever
Solid one-way slabs	280	L/25	L/30	L/35	L/12.5
	350	L/22	L/27	L/31	L/11
	420	L/20	L/24	L/28	L/10
Beams or ribbed one-way slabs	280	L/20	L/23	L/26	L/10
	350	L/18	L/20.5	L/23.5	L/9
	420	L/16	L/18.5	L/21	L/8

Figure 1-15. Minimum beam thickness according to ACI code

One end continuous is the most critical value for our case, which is:

$L/18.5 = 6000/18.5 = 324.324 < 350$ (ok)

1.4.2.1.1 Flexural reinforcement

Table 1-17. Area of steel in major beams taken from SAP2000 (in^2)

Roof	0.568 0.354	0.180 0.592	0.568 0.363
9-15 floor	1.151 0.673	0.372 1.104	1.151 0.749
5-8 floor	1.321 0.865	0.429 1.094	1.314 0.861
1-4 floor	1.477 0.966	0.478 1.278	1.476 0.964

Table above shows the critical values of area of steel for major beams, based on these results the following bar sizes were chosen.

Table 1-18. Negative reinforcement for longitudinal major beam

Beam Major -ve	1-4 floor	5-8 floor	9-15 floor	Roof
As (in2)	1.47	1.32	1.15	0.57
Bar#	4 #6 (1.76 in2)	4 #6 (1.76 in2)	4 #5 (1.24 in2)	4 #4 (0.8 in2)
Spacing (mm)	75.416	75.416	76.458	44.166

Table 1-19. Positive reinforcement for longitudinal major beam

Beam Major "+ve"	1-4 floor	5-8 floor	9-12 floor	13-16 floor
As (in2)	1.28	1.85	1.21	0.64
Bar#	5 #6 (2.2 in2)	5 #6 (2.2 in2)	4 #5 (1.24 in2)	4 #4 (0.8 in2)
Spacing (mm)	56.562	56.562	76.458	44.166

According to ACI code, minimum spacing between longitudinal bars is either 1 inch, nominal bar diameter D, or 4/3 nominal maximum aggregate size. Both major and minor beam spacing is greater than minimum allowable.

Table 1-20. Bar diameter in metric units

Bar # (US)	Metric	Diameter (mm)
3	10	9.5
4	13	12.7
5	16	15.9
6	19	19.1
7	22	22.2
8	25	25.4
9	29	28.7
10	32	32.2

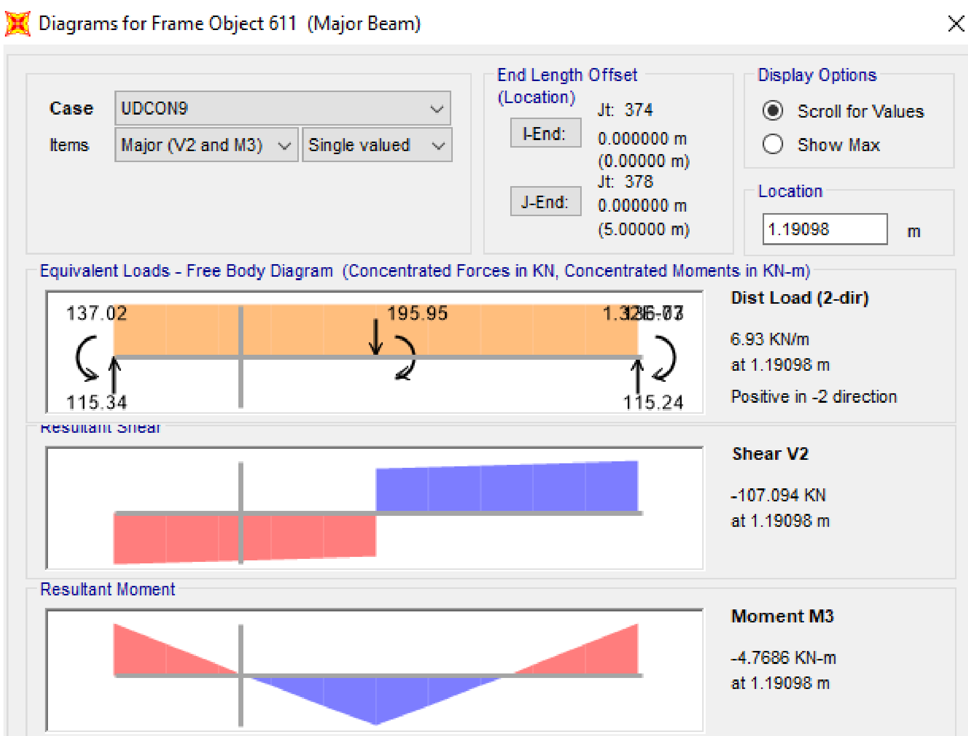


Figure 1-16. 1st floor major longitudinal beam (5m)

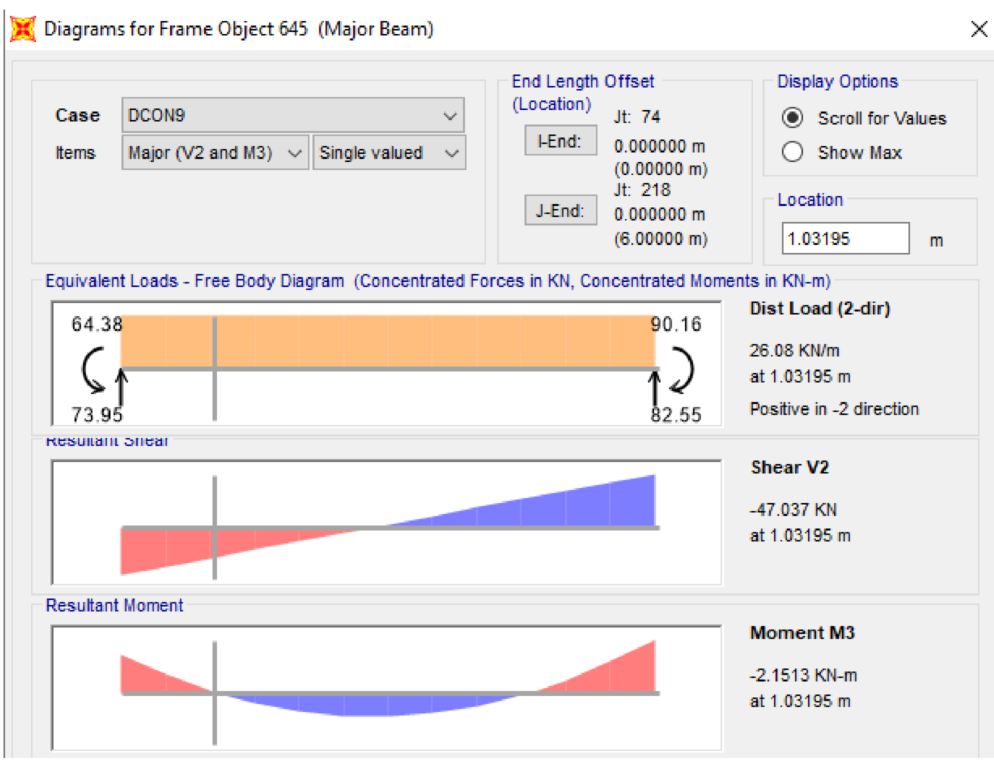


Figure 1-17. 1st floor major transverse beam (6m)

Figure above shows the profile of resultant moment of typical major beam, which shows that negative moment ends at approximately 1.2 m from end of the beam. This is where the top reinforcement cut will be done.

1.4.2.1.2 Shear reinforcement

b_w (in.)	10	11	12	13	14	15	16	18	20	22	24	b_w
S_{max} (in.) no. 3 stirrups	24	24	22	20.3	18.9	17.6	16.5	14.7	13.2	12	11	$264/b_w$
S_{max} (in.) no. 4 stirrups	24	24	24	24	24	24	24	24	24	21.8	20	$480/b_w$

Figure 1-18. Maximum spacing values for shear reinforcement according to ACI code

For the shear reinforcement #4 stirrups were chosen. Area of 2 legs is $0.4\ in^2$. Using this area and Av/s values the spacing was calculated. The results were rounded to the lower integer. The values that exceed maximum spacing were replaced by S_{max} taken from Figure 1-18.

Table 1-21. Major beam Design of shear reinforcement and spacing of using SAP2000

Beam Major	1-4 floor	5-8 floor	9-15 floor	Roof
Av/s (in^2/in)	0.041	0.035	0.031	0.011
Calculated s (in)	9.756	11.429	12.903	36.364
Chosen s (in)	9.000	11.000	12.000	24.000
Chosen s (mm)	225.000	275.000	300.000	600.000

1.4.2.2. Minor beam

1.4.2.2.1 Flexural reinforcement

Table 1-22. Area of steel in minor beams taken from SAP2000 (in^2)

Roof	0.348 0.092 0.348 0.185 0.347 0.185
12-15 floor	0.799 0.252 0.781 0.385 0.599 0.377
2-11 floor	1.005 0.313 0.965 0.479 0.594 0.461
1 floor	1.434 0.348 1.258 0.668 1.188 0.592

Table 1-23. Negative reinforcement for longitudinal minor beam

Beam Minor -ve	1 floor	2-11 floor	12-15 floor	Roof
As (in2)	1.434	1.005	0.799	0.35
Bar#	4 #6 (1.76 in2)	4 #5 (1.24 in2)	4 #4 (0.8 in2)	3 #4 (0.6 in2)
spacing	25.41666667	26.45833333	27.5	41.25

Table 1-24. Positive reinforcement for longitudinal minor beam

Beam Minor "+ve"	1 floor	2-11 floor	12-15 floor	Roof
As (in2)	1.188	0.594	0.599	0.35
Bar#	4 #5 (1.24 in2)	4 #4 (0.8 in2)	4 #4 (0.8 in2)	3 #4 (0.6 in2)
spacing	26.45833333	27.5	27.5	41.25

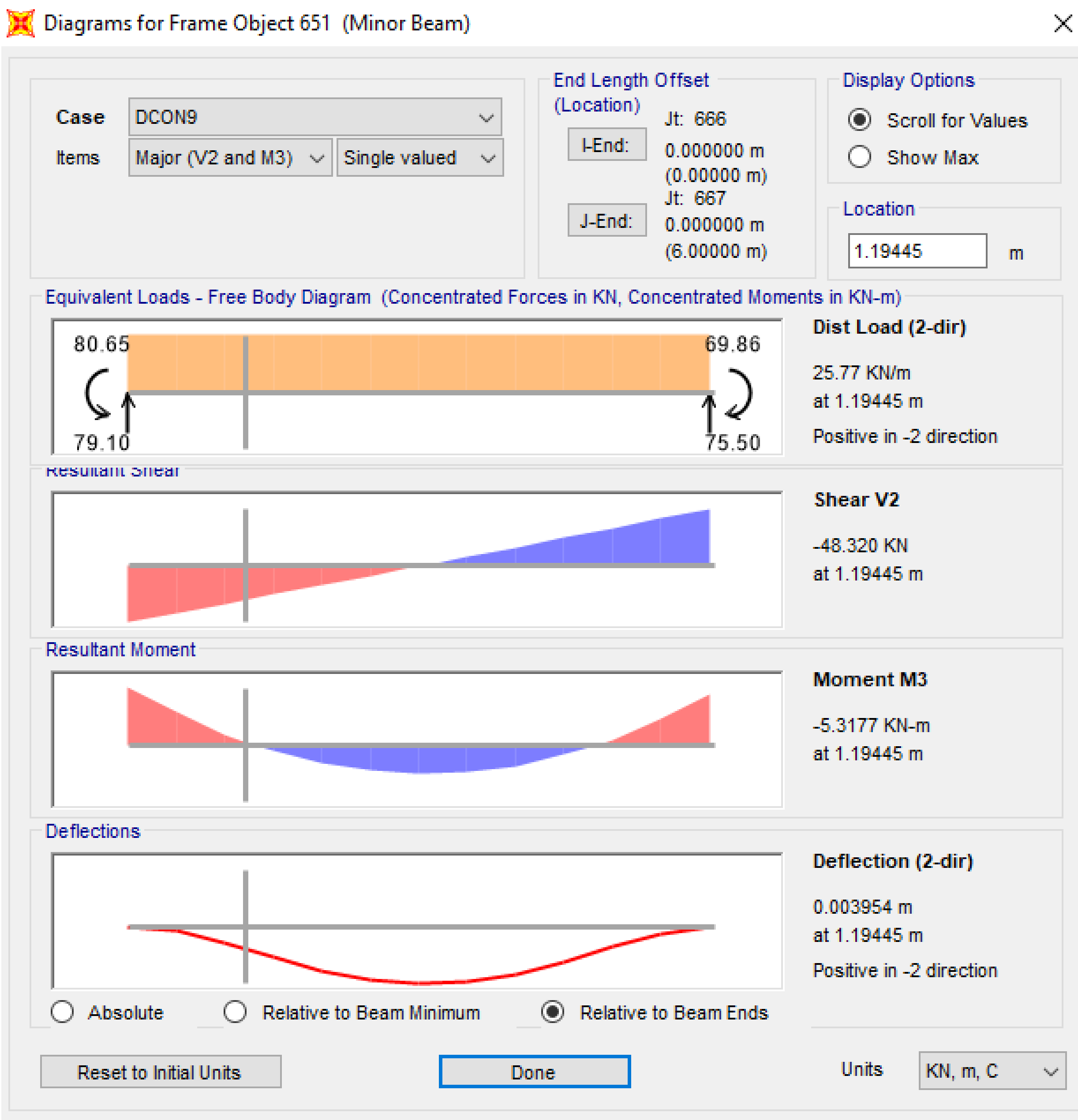


Figure 1-19. 1st floor minor beam

Negative moment ends at approximately 1 m from the end of the beam, just as in the major beams. So the cut of the top reinforcement will be the same.

1.4.2.2.2 Shear reinforcement

Maximum spacing for long bars of minor beams is the same as for the major beams explained above.

Table 1-25. Minor beam Design of shear reinforcement and spacing of using SAP2000

Beam Minor	1 floor	2-11 floor	12-15 floor	Roof
As/s (in2/in)	0.026	0.013	0.011	0.007
Calculated s (in)	15.385	30.769	36.364	57.143
Chosen s (in)	15.000	24.000	24.000	24.000
Chosen s (mm)	375.000	600.000	600.000	600.000

1.4.3. All columns

According to ACI Code 25.2.3, the minimum longitudinal bar spacing for columns must be at least the greatest of (a) through (c). For the column longitudinal bars #8, #6 and #10 were used. (a) 1.5 in; (b) 1.5 db ; (c) (4/3) db

Table 1-26. The column longitudinal bar spacing

Floors	1-4 floor	5-8 floor	9-12 floor	13-16 floor
Bar#	12 #10 (15.24 in2)	12 #8 (9.48 in2)	8 #8 (6.32 in2)	8 #6 (3.52 in2)
db, in	1.27	1	1	0.75
Spacing a)	1.5	1.5	1.5	1.5
Spacing b)	1.69	1.33	1.33	1.00
Spacing c)	1.91	1.50	1.50	1.13
Min longitudinal spacing, in	1.91 (49mm)	1.5 (39mm)	1.5 (38mm)	1.5 (38mm)
Min spacing, in	0.6 (16mm)	0.5 (13mm)	0.8 (12mm)	0.8 (12mm)
Actual Spacing, in	32.8 (820)	26.8 (670)	20.8 (520)	14.8 (370)

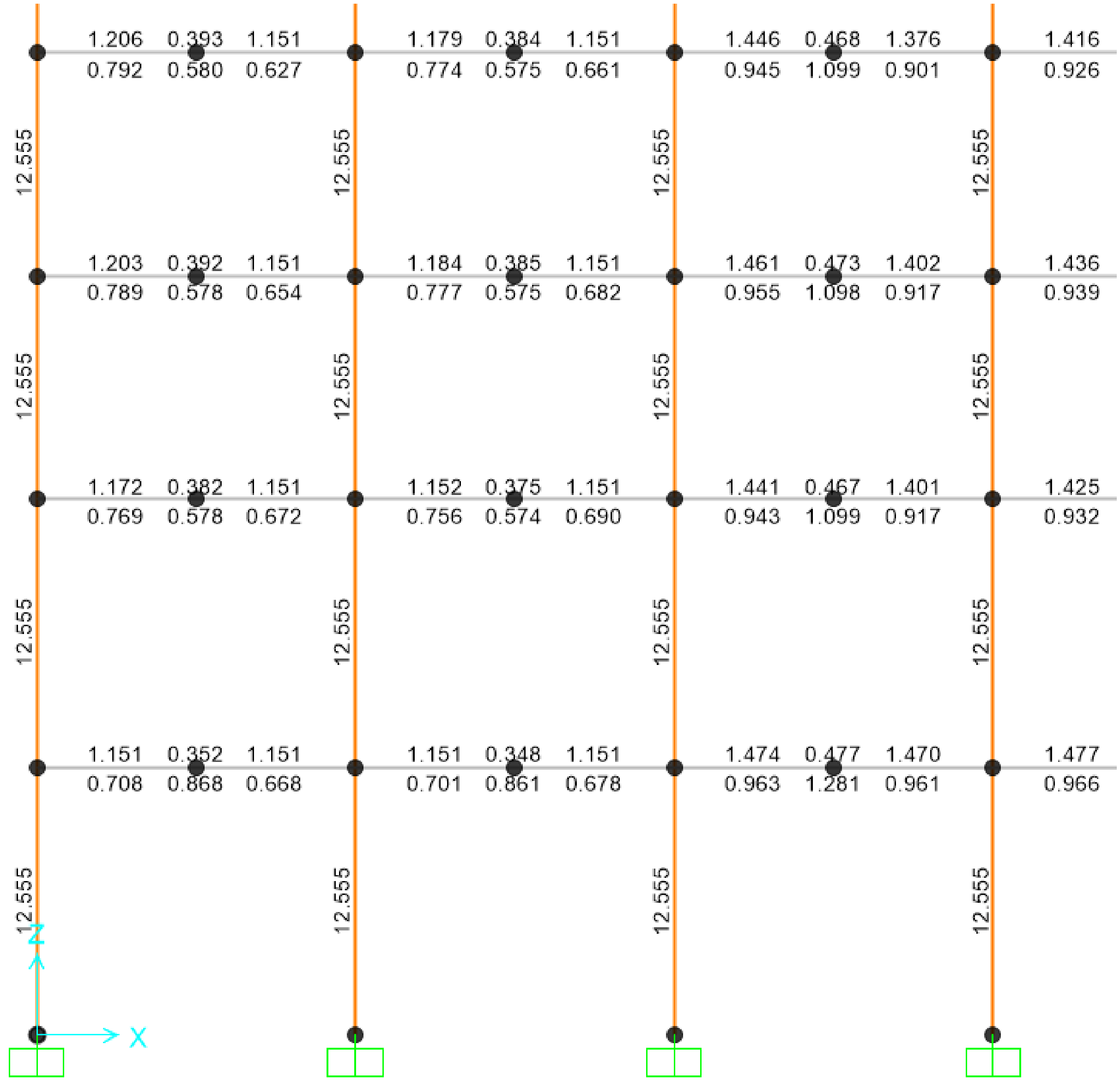


Figure 1-20. Area of Column reinforcement at 1-4th floor

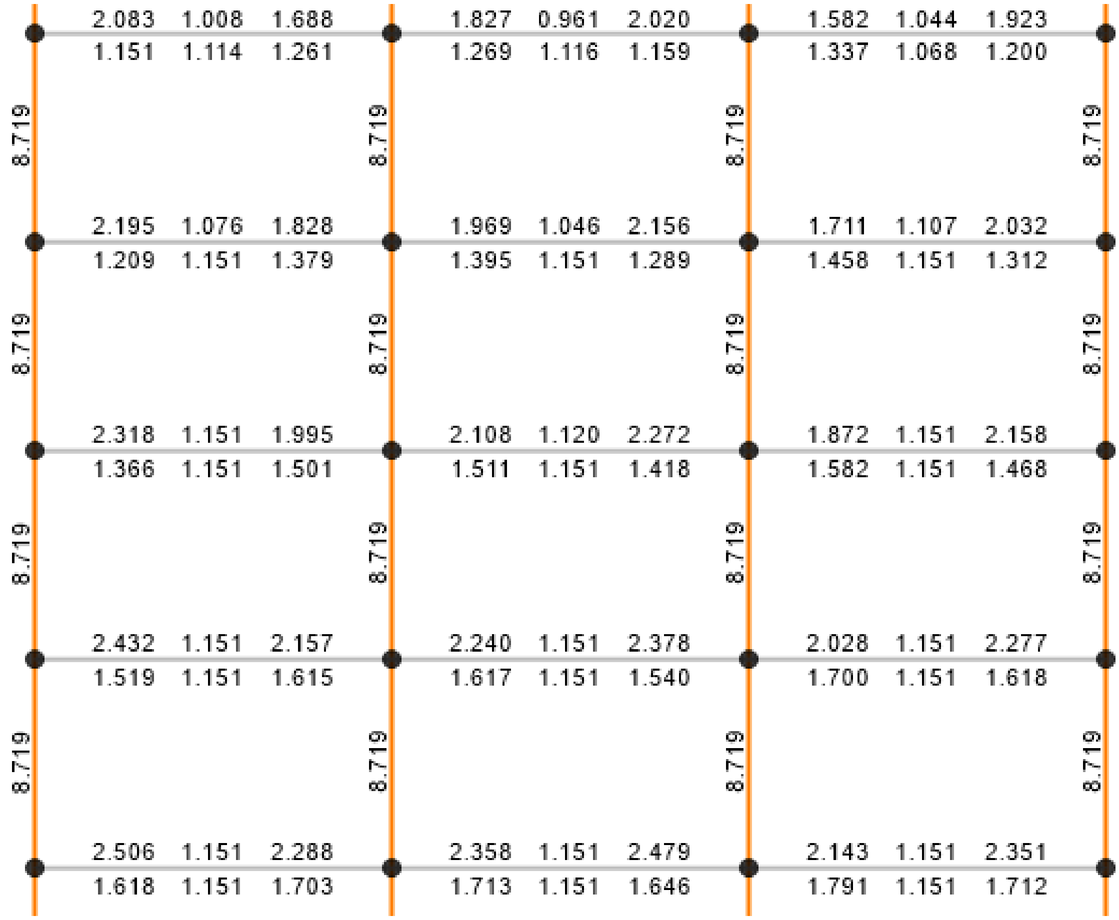


Figure 1-21. Area of Column reinforcement at 5-8th floor

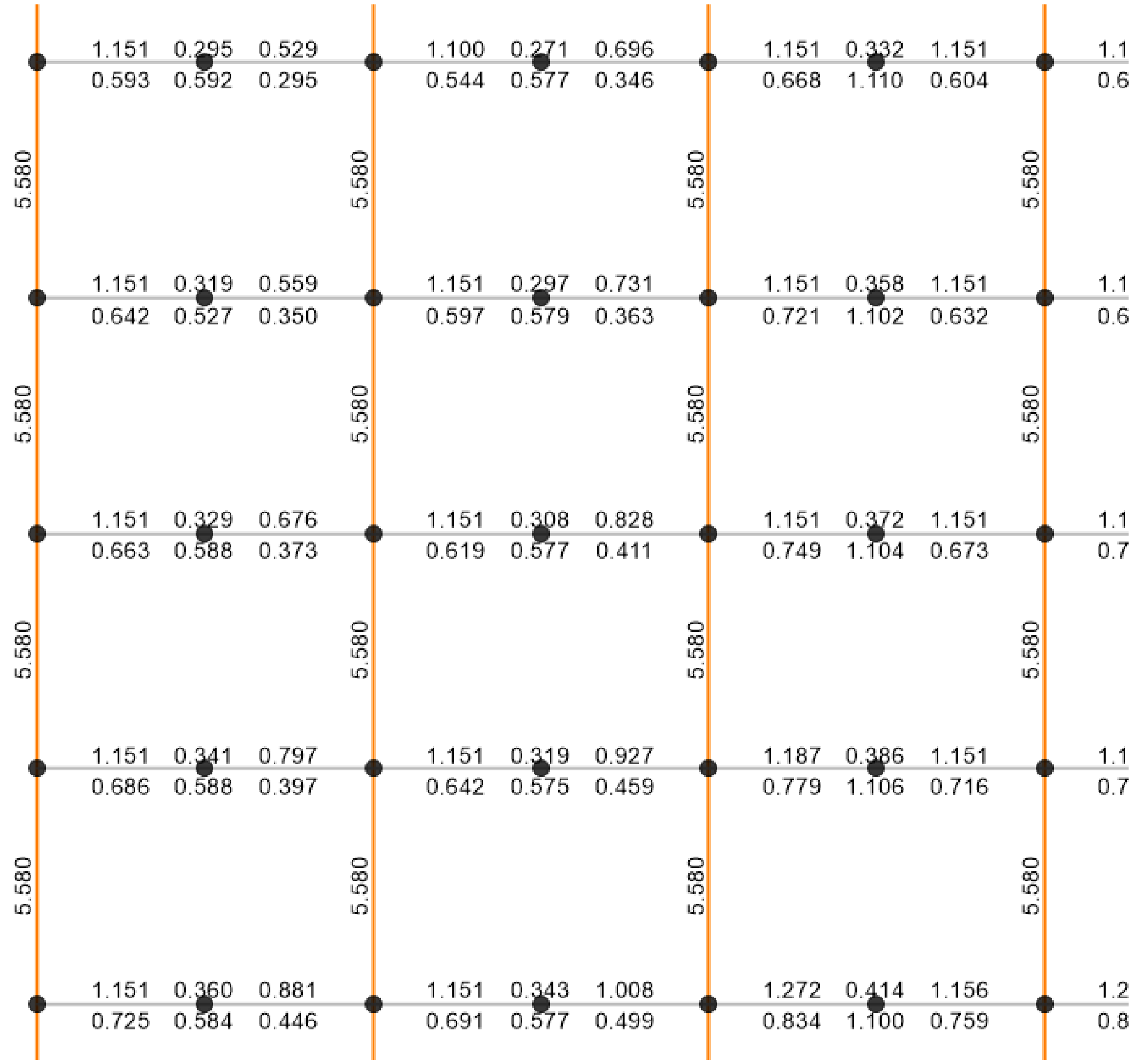


Figure 1-22. Area of Column reinforcement at 9-12th floor

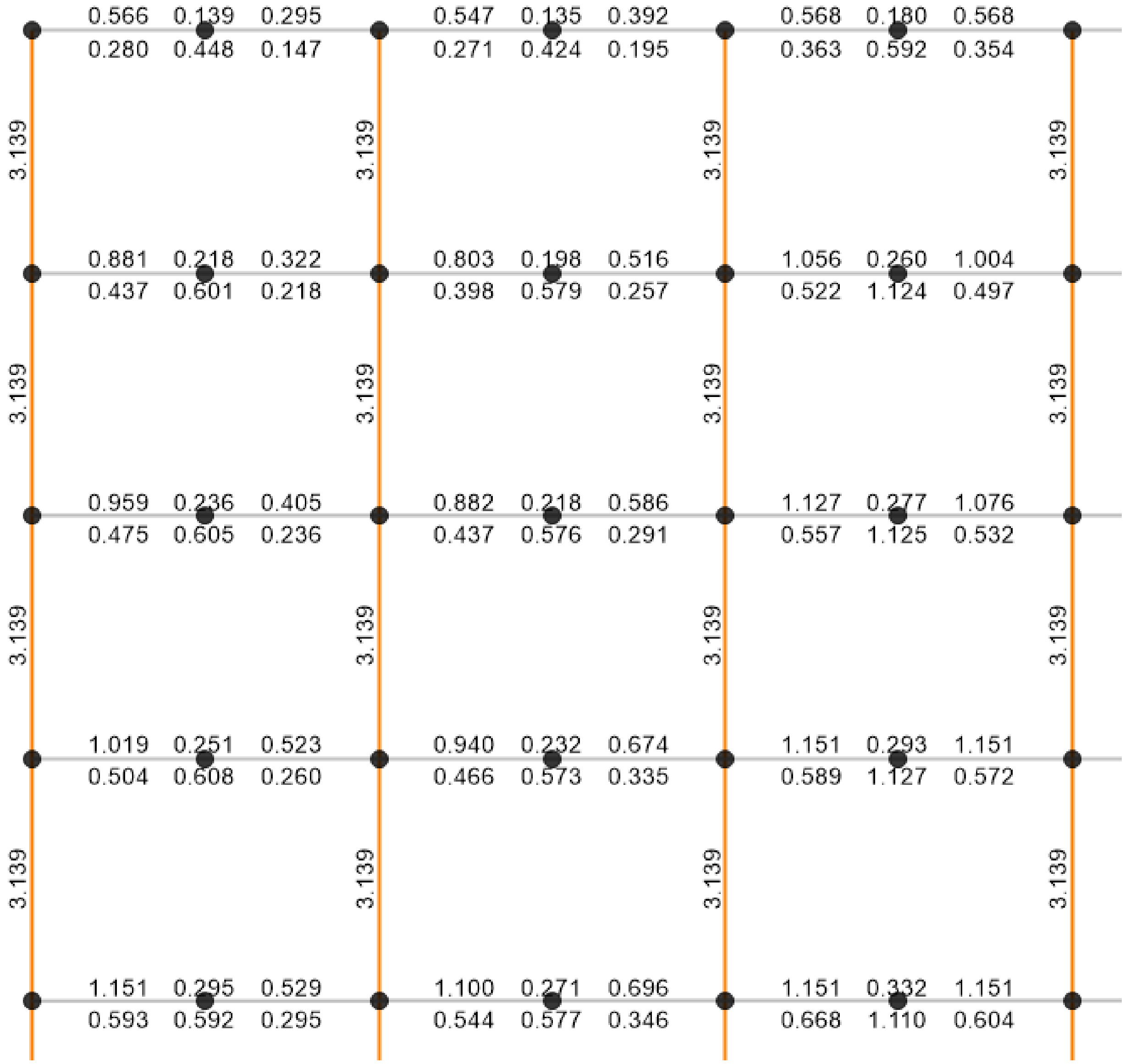


Figure 1-23. Area of Column reinforcement at 13-16th floor

Table 1-27. Reinforcement design of columns on all floors

Column	1-4 floor	5-8 floor	9-12 floor	13-16 floor
As (in2)	12.55	8.72	5.58	3.14
Bar#	12 #10 (15.24 in2)	12 #8 (9.48 in2)	8 #8 (6.32 in2)	8 #6 (3.52 in2)

As it can be observed from Figures 1-23, the reinforcement of External and Internal Columns are the same.

1.5. Hand calculation verification for structural member design (size or check reinforcement)

1.5.1. At least one slab panel

We need to find several parameters to design the slab reinforcement.

Table 1-28. 2nd-floor slab calculation inputs

1st Floor	SI units (mm)	Imperial units (in)
Column size	750	29.5276
L	2500	98.4252
Ln	1750	68.89764
L2	6000	236.2205
fc	ksi	3
fy	ksi	60
unit weight of concrete	pcf	150

The basic minimum for fill and simple surfaces like patios or sidewalks is typically between 2,000 and 3,000 psi. ACI establishes 2,500 psi as the minimum for structural concrete. Pavements, slabs, and footings can go up to 4,000 psi. Suspended slabs, beams, and girders, often found in bridges, may require 5,000 psi (Lyset T, 2022).

$$l_n = l - C_c - 0.5 * D_{rebar}$$

C_c - a clear cover of the reinforcement

D_{rebar} - diameter of the rebar

l_n - a clear span of the slab

Table 1-29. Loads of the 1st floor slab

Loads	kPa	psf
SW		62.5
DL	6.2572	130.685
Total DL		193.185
LL	4.79	100.041
Checking code requirements		
LL/DL	0.5178	≤3
w_u	391.8876	0.3918

Table 1-30. Primary reinforcement design

Primary / Flexural Reinforcement							
Points	Sign	Moment Coeff.	Mu (k-in)	Rn	Ro	As	Bar# & Spacing
A	-ve	0.04167	127.149	38.57561	0.000647	0.6025	#5 @ 6 c/c
B	"+ve"	0.07143	217.970	66.12962	0.001116	1.0387	#5 @ 3.5 c/c
C	-ve	0.10000	305.158	92.58147	0.001572	1.4622	#6 @ 3.5 c/c
D	-ve	0.09091	277.417	84.16497	0.001426	1.3269	#6 @ 3.5 c/c
E	"+ve"	0.06250	190.724	57.86342	0.000975	0.9074	#5 @ 3.5 c/c

$$M_u = coefficient \cdot w_u \cdot l_n^2$$

w_u - factored uniformly distributed load

Coefficients are taken from the ACI guidelines

$$R_n = \frac{M_u}{\phi b d^2}$$

ϕ - 0.9 factor

b - width of the slab

d - thickness of the slab from top to the center of the bottom reinforcement (excluding the bottom cover and half the rebar dia)

$$\rho = \frac{0.85 \cdot f_c'}{f_y} \cdot (1 - \sqrt{1 - \frac{2 \cdot R_n}{0.85 \cdot f_c'}})$$

ρ - reinforcement ratio

$$A_s = \rho b d$$

A_s - Area of the rebars

Using the calculated area of the rebars the bar size and spacing can be chosen from the ACI code (Table A. 14)

The spacing needs to be checked with the following criteria:

a) $s = 3 \cdot h = 15\, in$

b) $s = 18\, in$

c) $s = \frac{15 \cdot 40000}{2/3 \cdot f_y} - 2.5 \cdot C_C = 13.125\, in$

d) $s = \frac{12 \cdot 40000}{2/3 \cdot f_y} = 12\, in$

A similar procedure applies to the design of the secondary reinforcement.

Table 1-31. Secondary reinforcement design

Secondary / Temperature & Shrinkage Reinforcement			
As,min	2.1259845		
#5 bar	0.3067961576		
s	1.731693665	2	inches
#5 @ 2 c/c			

$$A_{s,min} = 0.0018 \cdot bh$$

Table 1-32. Spacing check according to ACI code

Spacing Checking		
1	25	inches
2	18	inches

Table 1-33. Shear force resistance check (ACI code)

Shear Force				
Points	Ultimate Shear	Ro	Shear Capacity	Checking
A	1.293756267	0.00064786	26.44913702	>
B	1.293756267	0.00111683	31.71365751	>
C	1.293756267	0.00157210	35.54213494	>
D	1.293756267	0.00142670	34.41072807	>
E	1.293756267	0.00097559	30.3160158	>

1.5.2. At least one major and minor beam

Table 1-33. Hand Calculated Reinforcement design of longitudinal major beam

Sign	Mu (k-ft)	Rn	Ro (%)	As (in^2)	Bar#
-ve	46.75	214.4432	0.37%	0.6249	2 #5 (0.62 in^2)
"+ve"	39.25	180.0405	0.31%	0.5217	2 #5 (0.62 in^2)
-ve	46.75	214.4432	0.37%	0.6249	2 #5 (0.62 in^2)

Table 1-34. SAP2000 Reinforcement design of longitudinal major beam

Sign	As (in^2)	Bar#
-ve	1.5440	4 #6 (1.76 in^2)
"+ve"	0.5870	2 #5 (0.62 in^2)
-ve	1.4210	4 #6 (1.76 in^2)

1.5.3. At least one column

The column design is done considering it as an unbraced sway column. For the hand calculation the interior column on the 16th floor (last floor). A column with a high slenderness ratio will have a considerable reduction in strength, whereas a low slenderness ratio means that the column is relatively short and the reduction in strength may not be significant.

The column parameters are 450*450*3500

Firstly, the slenderness ratio should be determined.

$$\frac{kLu}{r} > 22$$

Where k is the effective length factor, Lu is the unsupported length of the column, r is the radius of Gyration.

To determine the effective length factor alignment chart is used, where elastic modulus is the same for beam and column.

$$\Psi = \frac{\Sigma(EI/L)_{column}}{\Sigma(EI/L)_{beam}}$$

$$I_{beam} = \frac{1}{12}bh^3 = 0.35 * 450mm * (450mm)^3/12 = 2873.43\ in^4$$

$$I_{column} = \frac{1}{12}bh^3 = 0.70 * 350mm * (700mm)^3/12 = 16824.51\ in^4$$

$$\Psi_A = \frac{\Sigma(EI/L)_{column}}{\Sigma(EI/L)_{beam}} = \frac{16824.51\ in^4/137.08\ in}{2*2873.43\ in^4/196.85\ in} = 4.18$$

$$\Psi_B = \frac{\Sigma(EI/L)_{column}}{\Sigma(EI/L)_{beam}} = \frac{2*16824.51\ in^4/137.08\ in}{2*2873.43\ in^4/196.85\ in} = 8.36$$

From the alignment chart the k is equal to 2.4.

$$Lu = 3500\ mm - 450mm/2 - 700mm/2 = 2925mm$$

$$r = 0.3 * h\ ()$$

$$\frac{kLu}{r} = \frac{2.4*2925mm}{0.3*450mm} = 52 > 22$$

The slenderness ratio is more than 22 so the column is considered slender, there is need for moment magnification. The loads applied to columns are dead load 36k, wind load 11k, live load 0 and roof live load 5.63k. The moments are dead load M_2 is 4.5k-in, M_1 is 4k-in, wind load M_2 is 75k-in, M_1 is 70k-in, roof live load M_2 is 1.27k-in, M_1 is 1.1k-in.

To determine the Euler buckling load $(EI)_{eff}$ is calculated to account for creep, cracks.

$$\beta_{dns} = \frac{1.2D(sustained)}{P_u} = \frac{1.2*26.1}{45.66} = 0.68$$

$$I_g = \frac{1}{12}bh^3 = 350mm * (700mm)^3/12 = 24035\ in^4$$

$$E_c = 57000\sqrt{4000} = 3604996$$

$$(EI)_{eff} = \frac{0.4E_c I_g}{1+\beta_{dns}} = \frac{0.4*24035}{1+0.68} = 20584835261$$

$$P_c = \frac{\pi^2 (EI)_{eff}}{(klu)^2} = \frac{3.14^2*20584835261}{(2.4*115.16)^2} = 2657051$$

$$\delta = \frac{1}{1-(\Sigma P_u / 0.75\Sigma P_c)} \geq 1.0$$

Table 1-35. Load combination and moment magnification

Case	P_u	M_u	M_{2ns}	M_{2s}	ΣP_u	δ	M_2
1.4D	36.42	6.30	6.30	-	36.42	1.019	6.3
1.2D+1.6L+0.5RL	34.03	6.04	6.04	-	34.03	1.017	6.0
1.2D+1.6RL+0.5W	46.04	44.93	7.43	37.5	46.04	1.024	45.8
1.2D+1W+1L+0.5RL	45.66	81.035	6.035	75	45.66	1.023	82.8
1.2D-1W+1L+0.5RL	22.41	-68.965	6.035	-75	22.41	1.011	-69.8
0.9D+1W	35.04	79.05	4.05	75	35.04	1.018	80.4
0.9D-1W	11.79	-70.95	4.05	-75	11.79	1.006	-71.4

$M_c = 82.8$ k-in

$$\gamma = (17.72in - 5in)/17.72in = 0.717$$

$$P_n = P_u/0.65 = 45.66/0.65 = 70.25k$$

$$e = M_c/P_u = 82.8/70.25 = 1.81in$$

$$K_n = \frac{P_n}{f_c' A_g} = \frac{70.25k}{4k*(17.72\ in)^2} = 0.0556$$

$$R_n = \frac{P_n}{f_c' A_g}\frac{e}{h} = \frac{0.0314*1.81}{17.72} = 0.0057$$

With the calculated Kn and Rn, ρ is determined via alignment chart. The ρ is 0.01.

$$A_{st} = 0.01 * 17.72^2 = 3.18\, in^2$$

Therefore, use bar 8#6 for 16th floor internal column reinforcement which is the same as in the SAP2000 simulation.

1.5.3.1. P-M check

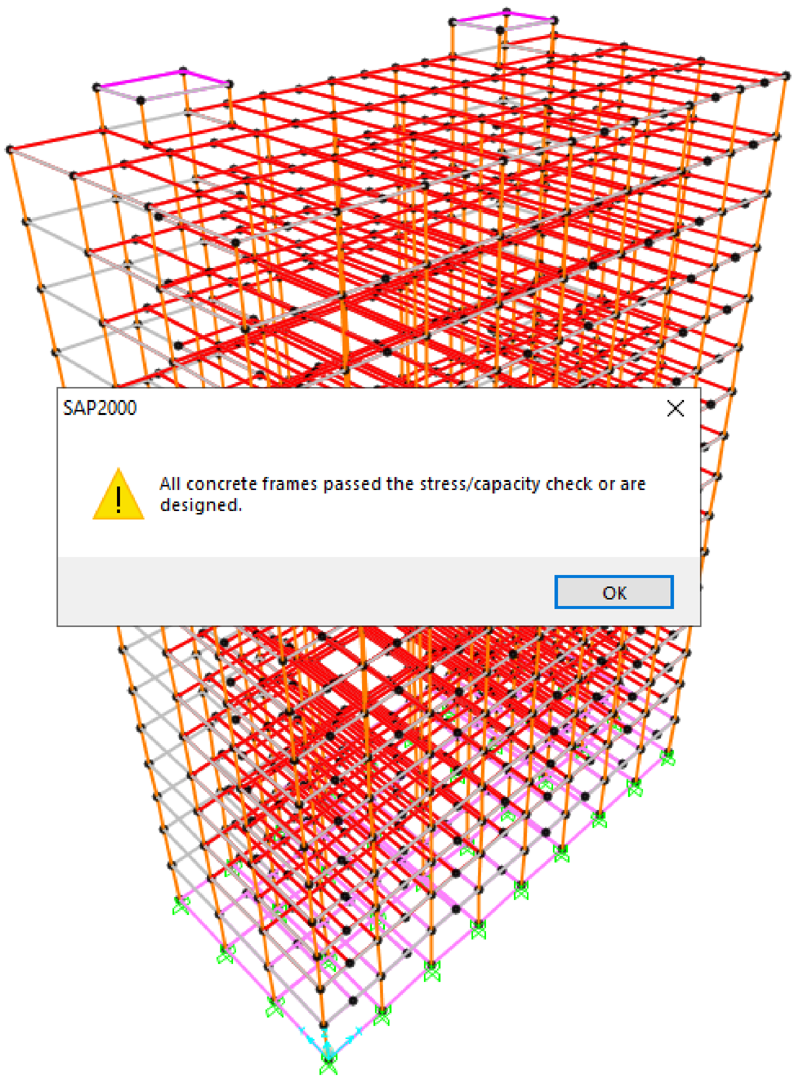


Figure 1-24. P-M interaction for columns check in SAP2000

According to ACI 318, assuming the balanced condition, the P and m is calculated, resulting in Table 1-24.

$\varphi P_n = \varphi P_o$

$P_0 = \varphi[0.85 f_c'(A_g - A_{st}) + (f_y A_{st})]$

$P_0 = 0.8(0.85 * 28 * (202500 - 2272) + (420 * 2272)) = 4575.8 kN$

$\beta = 0.85$ for 4ksi

$c = \frac{0.003}{0.003+0.00207} * 450 = 266.3 mm$

$a = 266.3 * 0.85 = 226.3 mm$

$\varepsilon = \frac{c-d}{d} * 0.003 = 266.3 - 40/266.3 * 0.003 = 0.00255$

By recalculating the values for different C values, the following results were obtained. To plot the P-M interaction diagram (Figure 1-25.), different values of load and moment were calculated for various e values.

Table 1-36. Load and Moment calculator results

P, kN	M. kNm
2841.15	0
2841.15	94.01
2641.78	147.17
2221.58	188.81
1773.01	215.50
1292.98	229.83
1122.24	248.91
802.95	254.55
326.485	191.52

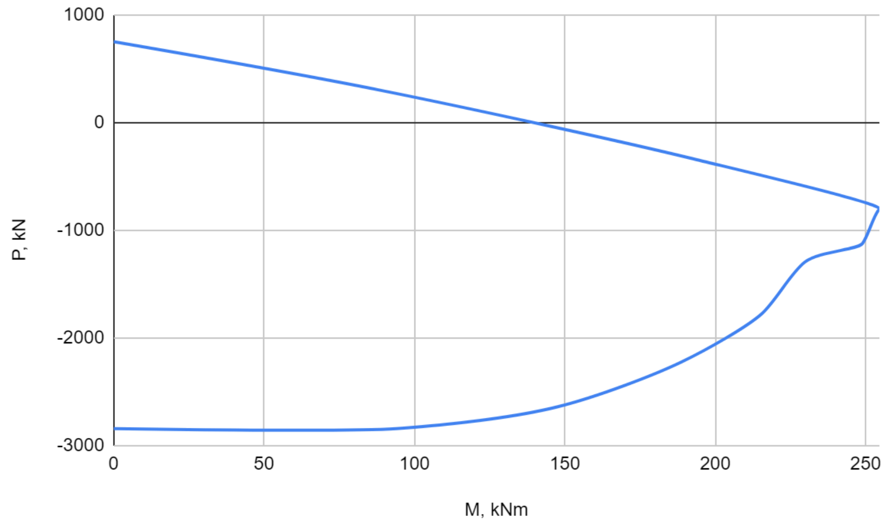


Figure 1-25. Interaction Diagram

1.5.3.2 Check for shear

Maximum axial capacity of column calculated for further shear check procedure.

AXIAL FORCE & BIAXIAL MOMENT DESIGN FOR PU, M2, M3

	Rebar Area	Design Pu	Design M2	Design M3	Minimum M2	Minimum M3
	0.002	131.608	16.513	3.782	3.782	3.782

AXIAL FORCE & BIAXIAL MOMENT FACTORS

	Cm Factor	Delta_ns Factor	Delta_s Factor	K Factor	L Length
Major Bending(M3)	0.400	1.000	1.000	1.000	3.500
Minor Bending(M2)	0.400	1.000	1.000	1.000	3.500

SHEAR DESIGN FOR V2,V3

	Rebar Av/s	Shear Vu	Shear phi*Vc	Shear phi*Vs	Shear Vp
Major Shear(V2)	6.396E-04	76.005	0.000	76.005	76.005
Minor Shear(V3)	3.971E-04	47.183	0.000	47.183	47.183

Figure 1-26. Axial Force and Biaxial Moment Design of the column

From SAP 2000 we have Major Shear $V_{u1} = 76.005\ kN$ and Major Shear $V_{u2} = 47.183\ kN$.

$$V_c = 0.17(1 + \frac{N_u}{14A_g})\lambda\sqrt{f_c'}b_w d = 0.17(1 + \frac{4575.8}{14*202500})\sqrt{28} * 450 * (450 - 67) = 166.2kN$$

For major shear:

$$\varphi V_c = 0.75 * 166.2 = 124.6kN\text{f}$$

$\varphi V_c/2 = 62.34kN < 76kN$, the shear reinforcement is needed

Referring to ACI Code, Table 25.7.3.6, the spacing of ties is provided in Table 1-37. The 16th floor internal column side is 18 in for the #6 bar, the spacing is 12in (305mm).

Table 1-37. Maximum spacing of ties

Column least side or diameter (in)	Spacing of toes (in) for Bar					
	#6	#7	#8	#9	#10	#11
12	12	12	12	12	12	12
14	12	14	14	14	14	14
16	12	14	16	16	16	16
18	12	14	16	18	18	18
20	12	14	16	18	18	20
22-40	12	14	16	18	18	22
Ties	#3	#3	#3	#3	#3	#4

Table 1-38. The design results of shear reinforcement of 16th floor column from SAP2000

shear reinforcement	SAP2000
Av/s	0.025
Av	0.4
s (in)	16
s (mm)	400

The shear reinforcement from SAP2000 is adequate. The spacing from SAP2000 is more so we use 400mm for the final design of the column.

1.5.3.3. Check for biaxial bending

To assess potential biaxial bending in corner columns of the building, Dr. Boris Bresler's reciprocal interaction equation was applied.The axial loads and biaxial moment were provided from SAP2000.

$P_u/0.85 = 131.68/0.85 = 154.92\,kN$

$\gamma = \frac{17.71-2*2.44}{17.71} = 0.724$; $\rho_g = \frac{8*0.750}{17.71*17.71} = 0.01913$

$P_u = 131.608kN$; $M_x = 16.513\,kNm$; $M_y = 3.782\,kNm$

$e_x = \frac{16.513}{131.608} = 0.125m = 4.92in$

$e_y = \frac{3.782}{131.608} = 0.028m = 1.10in$

Bending about x-axis:

$\frac{e_x}{h} = \frac{4.92}{17.71} = 0.27;\ R_n = 0.15$

$P_{nx} = \frac{R_n f_c' A_g h}{e_x} = \frac{0.15*4000*17.71^2*17.71}{4.92} = 677.4k$

Bending about y-axis:

$\frac{e_y}{h} = \frac{1.10}{17.71} = 0.062;\ R_n = 0.06$

$P_{ny} = \frac{R_n f_c' A_g h}{e_x} = \frac{0.06*4000*17.71^2*17.71}{1.10} = 1211.9k$

$P_0 = 0.85 f_c' A_g + f_y A_s = 0.85 * 4 * 17.71^2 + 8 * 0.75 * 60 = 1426.4k$

$\frac{1}{P_{ni}} = \frac{1}{P_{nx}} + \frac{1}{P_{ny}} - \frac{1}{P_0} = \frac{1}{677.4} + \frac{1}{1211.9} - \frac{1}{1426.4}$

$P_{ni} = 624.87\,k = 2779.5\,kN > 154.92$kN

The column is structurally safe when subjected to biaxial bending.

1.6. Structural joint design

All cast-in-place joints

The required development length in compression is:

$$l_{dc} = (\frac{f_y \psi_r}{50\lambda\sqrt{f_c'}})d_b \geq 0.0003 f_y \psi_r d_b \,()$$

Where, ψ_r is the confining reinforcement factor equal to 1 for this case and λ is a normal weight modification factor. The development length in compression for roof major beam rebar is calculated below:

$$l_{dc} = (\frac{60,000*1}{50*1*\sqrt{4000}})0.5in \geq 0.0003 * 60k * 0.5 \approx 10in \text{ (250mm)}$$

The minimum diameter of bend, measured on the main bar of a standard hook is $4d_b$ for #3-5 bars and $6d_b$ for #6-8 bars (ACI code 25.4.3).

Table 1-39. The Major beam development length in compression and tension and the number of ties calculation results.

floor	Rebar	Develop length (mm)		Hook (mm)	Dia of bar (mm)	Ln (mm)	Spacing ties (mm)	Number of ties
		Tension	Compression					
roof	#4	475	250.0	50.8	12.7	5550	300	18.5
9-15 floor	#5	609.6	304.8	63.6	15.9	5400	300	18.0
5-8 floor	#6	736.6	381.0	114.6	19.1	5250	300	17.5
1-4 floor	#6	736.6	381.0	114.6	19.1	5100	300	17.0

The development length in compression for column rebar in 13-16th levels is calculated below:

$$l_{dc} = (\frac{60,000*1}{50*1*\sqrt{4000}})0.75in \approx 15\ in \text{ (381 mm)}$$

The minimum diameter of bend, measured on the main bar of a standard hook is $4d_b$ for #3-5 bars and $6d_b$ for #6-8 bars (ACI code 25.4.3).

Table 1-40. The Column development length in compression and the number of ties calculation results.

Floor	Rebar	Develop length in compression (mm)	Dia of bar (mm)	Ln (mm)	Spacing ties (mm)	Number of ties
13-16	#6	381	19.1	3050	300	10.2
9-12	#8	483	25.4	2900	300	9.7
5-8	#8	483	25.4	2750	300	9.2
4-3	#10	635	32.2	2600	300	8.7
1-2	#10	635	32.2	3300	300	11.0

Lap Splice in compression of rebars in the joint of two different sized columns was calculated through the following formula:

$I_{sc} = 0.0005 f_y d_b$ for $f_y \leq 60\,000\, psi$

$I_{sc} = (0.0009 f_y - 24) d_b$ for $f_y > 60\,000\, psi$ and $\leq 80\,000\, psi$

As, $f_y = 60\,000\, psi$, lap splice in compression in 13-16th floors:

$I_{sc} = 0.0005(60000\, psi)(0.75\, in) = 22.5\, in\, (571.5\, mm)$

Table 1-41. The Column lap splice in compression.

Floor	Rebar	Dia of bar (mm)	Lap Splice (mm)
13-16	#6	19.1	571.5
9-12	#8	25.4	760
5-8	#8	25.4	760
4-3	#10	32.2	967.7
1-2	#10	32.2	967.7

1.7. Structural detailing (splice and anchorage) and drawings (professional)

i. Slab

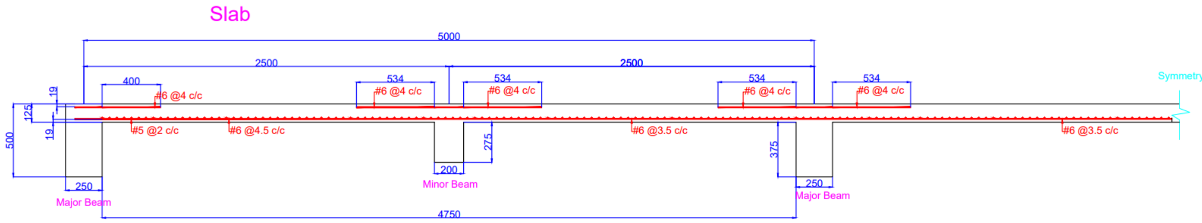


Figure 1-27. Slab design detailing

ii. Beams

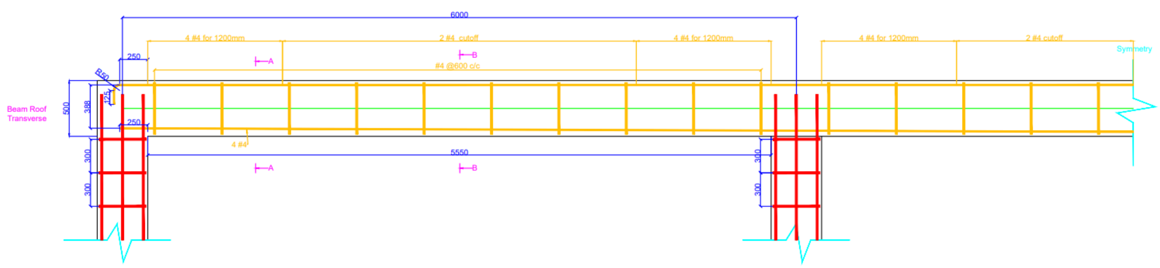


Figure 1-28. Transverse major beam on roof design detailing

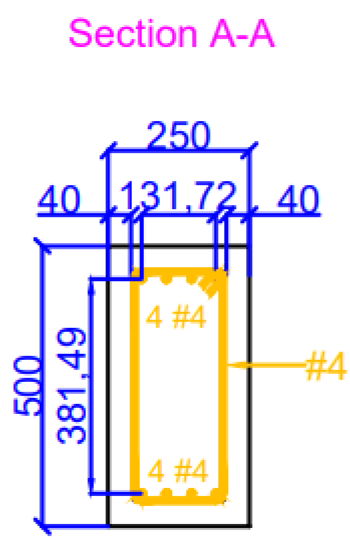


Figure 1-29. Transverse major beam on roof design cross section

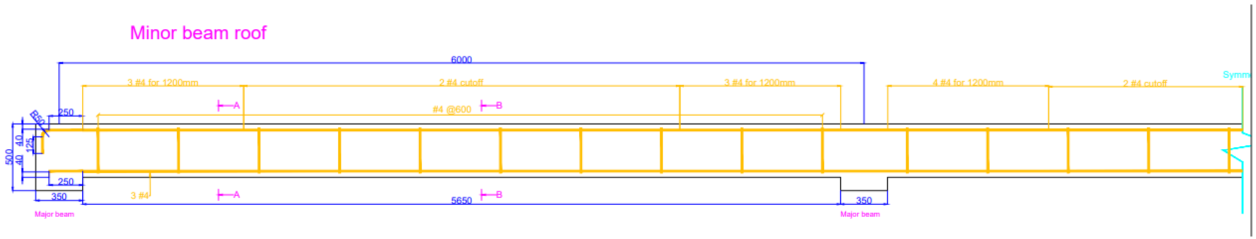


Figure 1-30. Minor beam on roof design detailing

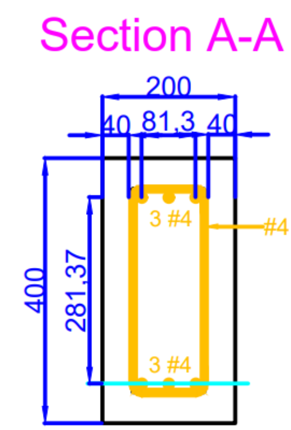


Figure 1-31. Minor beam on roof design cross section

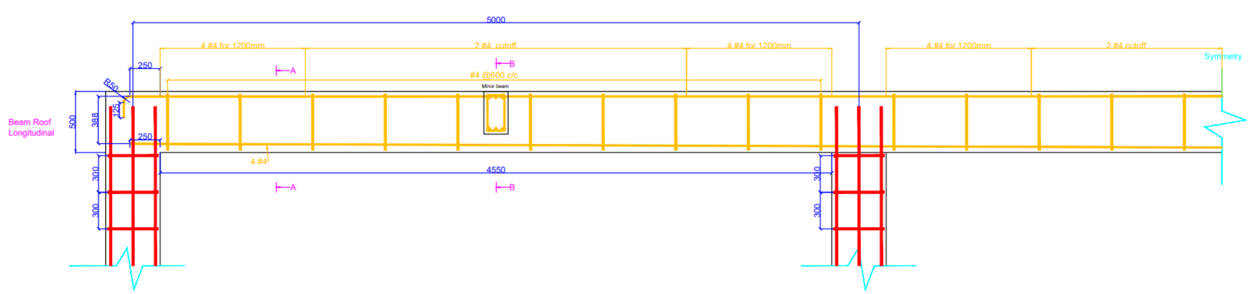


Figure 1 -32. Longitudinal major beam on design detailing

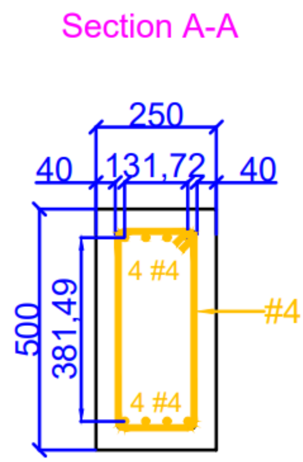


Figure 1-33. Longitudinal major beam on design detailing cross section

iii. Columns

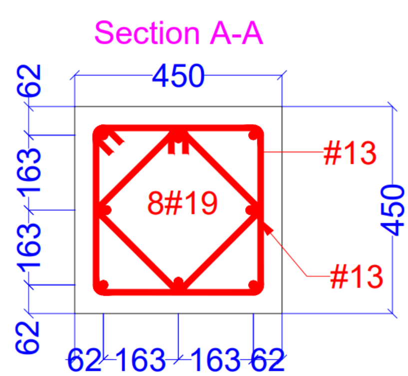


Figure 1-34. 13-16th floors column design cross section

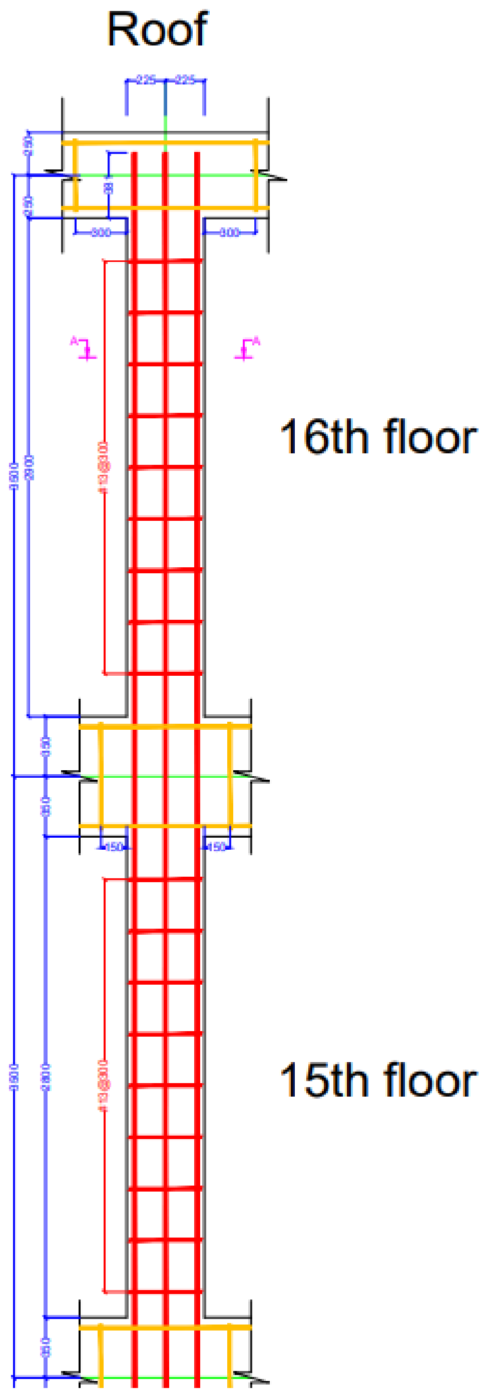


Figure 1-35. 15-16th floors column design detailings

vi. Joint

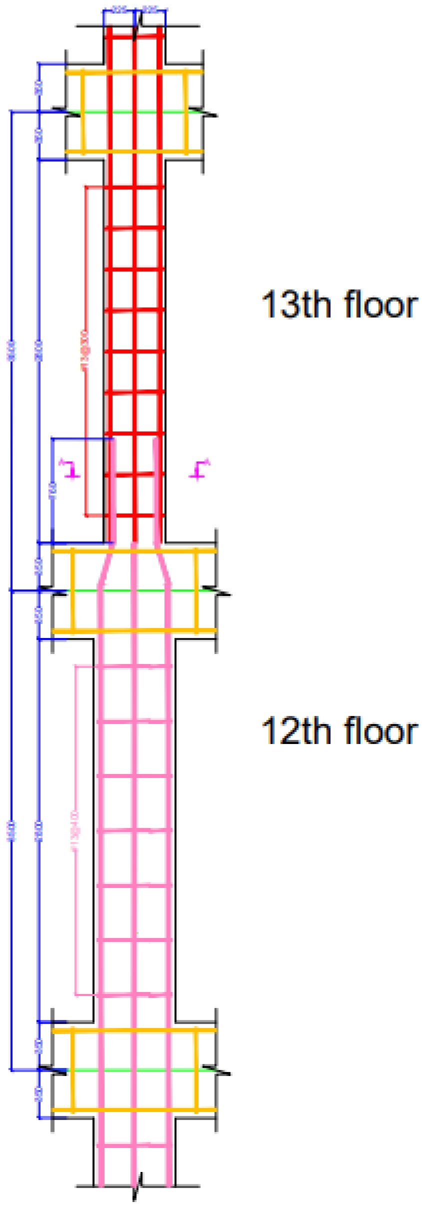


Figure 1-36. 12-13th floors joint design detailings

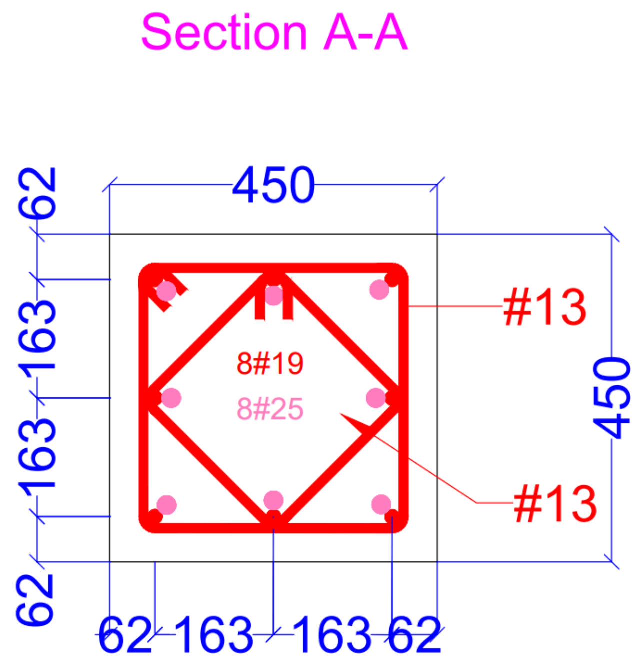


Figure 1-37. 12-13th floors joint design cross section

1.8. Structural serviceability design

In major beam structural member design

1.8.1. Vertical deflection

In structural member design, as per ACI Code Sections 7.3.1.1, a major beam structural member was designed based on the minimum thickness for one-way flexural members and one-way slabs. These values apply to members not supporting or connected to partitions or other structures that could be affected by significant deflections. Permissible deflection is influenced by various factors, including the building type, the structure's appearance, the presence of plastered ceilings and partitions, the potential damage from excessive deflection, and the type and amount of live load.

1.8.2. Lateral drift

Lateral drift was calculated for two frames : 3B and 9A in two directions. For the seismic load case, lateral drifts from the hand calculations and SAP 2000 are highlighted in Table 1-42. The allowable inter-story drift in the case of seismic loading is 2% according to the ACI 319 code. Drifts from the hand calculations and SAP 2000 for all floors passed the check.

Table 1-42. Drift Results for 3B and 9A frames for Seismic Loading

Floor / Drift (%)	Frame 3B		Frame 9A		Allowable Drift Limit (%)	Check
	Hand Calculation	SAP 2000	Hand Calculation	SAP 2000		
16	0.04	0.16	0.11	0.16	2	✔
15	0.07	0.16	0.20	0.16	2	✔
14	0.11	0.16	0.31	0.16	2	✔
13	0.10	0.16	0.28	0.31	2	✔
12	0.07	0.31	0.18	0.16	2	✔
11	0.06	0.16	0.18	0.16	2	✔
10	0.07	0.31	0.20	0.31	2	✔
9	0.06	0.31	0.18	0.31	2	✔
8	0.05	0.31	0.14	0.16	2	✔
7	0.05	0.16	0.13	0.31	2	✔
6	0.05	0.31	0.14	0.31	2	✔
5	0.04	0.31	0.13	0.16	2	✔
4	0.04	0.31	0.12	0.31	2	✔
3	0.04	0.31	0.13	0.31	2	✔
2	0.05	0.16	0.14	0.16	2	✔
1	0.05	0.16	0.16	0.16	2	✔

For the wind load case, lateral displacement from the hand calculations and SAP 2000 are highlighted in Table 1-43. The allowable absolute displacement in the case of wind loading according to the ACI 319 code calculated with the formula below:

$$Allowable\ Absolute\ Displacement\ \ Limit\ = H/400$$

Absolute displacements from the hand calculations and SAP 2000 for all floors passed the check.

Table 1-43. Absolute Displacement Results for 3B and 9A frames for Wind Loading

Floor / Absolute Displacement(mm)	Frame 3B		Frame 9A		Allowable Absolute Displacement (mm)	Check
	Hand Calculation	SAP 2000	Hand Calculation	SAP 2000		
16	10.0	45	52.2	118.9	143.5	✔
15	9.7	44	50.7	111.4	133.0	✔
14	9.3	43	48.6	105.1	122.5	✔
13	8.7	41	45.5	96.6	113.8	✔
12	8.0	38	42.4	89.2	105.0	✔
11	7.5	36	39.7	81.6	96.3	✔
10	6.9	31	36.9	73.5	87.5	✔
9	6.3	28	33.7	67	78.8	✔
8	5.7	25	30.5	58.8	70.0	✔
7	5.1	21	27.5	52.6	61.3	✔
6	4.5	18	24.4	45.3	52.5	✔
5	3.9	14	21.1	37.6	43.8	✔
4	3.3	10	17.7	29.5	35.0	✔
3	2.6	6	14.4	23.1	26.3	✔
2	1.9	4	10.6	15.3	17.5	✔
1	1.0	2	5.6	8.1	8.8	✔

1.8.3. Crack width

To estimate the maximum width of cracks that will occur in the tension faces of flexural members Gergely-Lutz equation:

$$w = 0.0113\beta_h f_s \sqrt[3]{d_c A}$$

Where w is the estimated cracking width in thousandths of mm, β_h is the ratio of the distance to the neutral axis to the centroid of the tensile concrete fiber to the distance from the neutral axis to the centroid of the tensile steel (for beams it is typically equal to 1.20), f_s is the steel stress at service load equal to $0.6f_y$ for normal structures, d_c is the cover of the outermost bar measured from the extreme tension fiber to the center of the closest bar or wire equal to 80mm, A is the effective tension area of concrete around the main reinforcement divided by the number of bars.

$$A = \frac{350mm*80mm}{4} = 7000mm^2$$

$$w = 0.0113 * 1.20 * (0.6 * 413.68MPa) * \sqrt{40mm * 7000mm^2} = 0.22mm$$

Permissible crack width under humidity, moist air exposure condition is 0.30 mm. From Table 1-44 results the allowable crack width is checked for major beams (for all 4 bars are designed) and minor beams (for floors 1-8, 5 bars and for 9-16 floors 4 bars are designed)

Table 1-44. The results of beam crack width calculations

	A, mm2	β_h	w, mm	allowable w, mm
major beam	7000	1.2	0.2235	0.3
minor beam 1-8 floors	3200	1.2	0.1722	0.3
minor beam 9-16 floors	4000	1.2	0.1855	0.3

The ACI 319 Code requires that flexural tensile reinforcement be well distributed within the zones of maximum tension so that the center-to-center spacing of the deformed bars and wires closest to a tension surface is not greater than the value computed with the following expression:

$$s = 15\left(\frac{40,000}{f_s}\right) - 2.5c_c \leq 12\left(\frac{40,000}{f_s}\right)$$

$$s = 15\left(\frac{40,000}{0.6*60000}\right) - 2.5 * 1.57 \leq 12\left(\frac{40,000}{0.6*60000}\right) = 12.74in\ (323mm) \leq 13.33in(338mm)$$

Thus a bar spacing not more than 323 mm would be required. The spacing in beams

300mm satisfies the requirement.

1.8.4. Durability

Mix design

In Louisiana observed sulfate level percent exceedance and the humidity level is high (2022 Louisiana Water Quality Integrated Report, 2022).

Type V Portland cement is specially designed to withstand sulfate attacks from soil and groundwater rich in sulfates. It has a reduced amount of tricalcium aluminate (C3A) compared to other cement types, making it less prone to sulfate attack. Lower water-cement (w/c) ratios enhance the sulfate corrosion resistance of concrete and lead to thinner corrosion layers. As sulfate corrosion progresses, models of corrosion thickness for different w/c ratios have been established based on the degradation of cubic compressive strength in corroded concrete samples (Du J.M, et al., 2016). In magnesium-rich sulfate conditions, concrete mixtures with silica fume outperform those with fly ash and blast-furnace slag, the latter providing the poorest performance (Al-Amoudi, O. S. B., 1998). For concrete exposed to sea or brackish water, or that contacts significant amounts of chlorides, the water-cement ratio should be kept as low as possible, ideally under 0.40 (Guide to Durable Concrete).

2. Architectural part

2.1. LEED

LEED (Leadership in Energy and Environmental Design) is a globally acknowledged certification program for buildings that aims to improve energy efficiency and promote sustainable design. Projects seeking certification must meet specific criteria, such as adhering to LEED minimum construction program requirements and meeting LEED mandatory standards. They also need to earn at least 40 points across various categories.

The project achieved a total score of 69, reflecting its focus on energy efficiency and basic environmental design standards. Although the score is good, there is potential for further improvement to reach a higher certification level.

The building's location was strategically chosen to minimize its environmental footprint by being situated in an established, populated area that promotes walkability and decreases reliance on cars for access to essential services. The area also includes several bus stops for convenient travel to other regional locations, and there are bicycle parking facilities available (Figure 2-1). Green landscapes help to reduce the size of the parking area.

Table 2-1. Location and transportation

Yes/No	**Location and Transportation**	***16***	13
yes	Neighborhood Development location	1	1
no	Sensitive Land protection	1	
no	High priority site	2	
yes	surrounding density and diverse uses	5	5
yes	access to Quality transit	5	5
yes	Bicycle facilities	1	1
yes	reduced parking footprint	1	1
no	green vehicles	1	

The chosen site undergoes careful evaluation to guarantee environmentally sustainable construction practices. The parking lot features green space that promotes a connection with nature. Further details on rainwater management calculations will be covered in Capstone 2, and the parking area allows for unobstructed views of the night sky.

Table 2-2. Sustainable sites

Yes/No	**Sustainable Sites**	***10***	8
	Construction Activity Pollution Prevention	required	
yes	Site assessment	1	1
no	site development protect or restore habitat	2	
yes	open spaces	1	1
yes	rainwater management	3	3
yes	heat island reduction	2	2
yes	light pollution reduction	1	1

Outdoor water use is minimized by using reclaimed water solely for irrigation purposes. Indoor water use will be reduced by replacing fresh water with treated greywater for flushing toilets. Additional measures to further decrease water consumption will be investigated.

Table 2-3. Water efficiency

Yes/No	**Water Efficiency**	***11***	9
	outdoor water use reduction	required	
	indoor water use reduction	required	
	building level water metering	required	
yes	outdoor water use reduction	2	2
yes	indoor water use reduction	6	6
no	cooling tower water use	2	
yes	water metering	1	1

The principles of the waste management hierarchy are applied to reduce landfill waste disposal, with the goal of optimizing performance in this aspect.

Table 2-4. Materials and resources

Yes/No	**Materials and Resources**	***13***	2
	storage and collection of recyclables	required	
	construction and demolition waste management planning	required	
no	building life cycle impact reduction	5	
no	building product disclosure and optimization-environmental product declaration	2	
no	building product disclosure and optimization - sourcing and raw materials	2	
no	building product disclosure and optimization - material ingredients	2	
yes	construction and demolition waste management	2	2

Energy consumption will be evaluated to identify opportunities for reduction, with an emphasis on achieving energy efficiency that is both environmentally and economically sustainable. This will help decrease greenhouse gas emissions.

Table 2-5. Energy and atmosphere

Yes/No	**Energy and Atmosphere**	***33***	22
	fundamental commissioning and verification	required	
	minimum energy performance	required	
	building level energy metering	required	
	fundamental refrigerant management	required	
no	enhanced commissioning	6	
yes	optimize energy performance	18	18
yes	advanced energy metering	1	1
yes	demand response	2	2
no	renewable energy production	3	
yes	enhanced refrigerant management	1	1
no	green power and carbon offsets	2	

The hotel's indoor environmental quality adheres to established standards, providing comfortable access to natural light, quality views, and soothing acoustics. Air quality and temperature will be regularly monitored for consistency.

Table 2-6. Indoor environmental quality

Yes/No	**Indoor Environmental Quality**	***16***	15
	minimum indoor air quality performance	required	
	environmental tobacco smoke control	required	
yes	enhanced indoor air quality strategies	2	2
yes	low emitting materials	3	3
yes	construction indoor air quality management plan	1	1
yes	indoor air quality assessment	2	2
yes	thermal comfort	1	1
yes	interior light	2	2
yes	daylight	3	3
yes	quality views	1	1
no	acoustic performance	1	

2.2. Life Cycle Cost Analysis

The life cycle cost of one slab on the 1st floor was conducted resulting in construction cost, barrier cost, repair cost, and life-cycle cost in USD per m^2 (Figure 2-1). According to the previous analysis of durability and mix design assumption the overall concrete mix will include water/cement ratio of 0.40 and 10% of silica fume supplementary cementitious material and sealer barrier. The Reinforcement is black steel (Figure 2-2). The price of stainless steel is typically more expensive than black steel. Here to prevent it from corrosion the vapor control layer sealer was used. The service life of the slab was estimated to be 80 years.

In result of estimation in software Life 365 the construction cost would br 14.13USD per m^2, the barrier cost is 7USD per m^2, the repair cost is 225.8USD per m^2 and the overall life-cycle cost is 246.93USD per m^2 for the slab of 2 m^3.

Life-Cycle Costs

Name	Construction Cost	Barrier Cost	Repair Cost	Life-Cycle Cost
Base case	$14.13 per sq. m.	$7.00 per sq. m.	$225.80 per sq. m.	$246.93 per sq. m.

Figure 2-1. Life-cycle cost of the slab

Concrete Mixes

Alt name	User?	w/cm	SCMs	Inhib.	Barrier	Reinf.
Base case		0.4	Silica Fume (10%);		Sealer	Black Steel

Figure 2-2. Concrete mix of the slab

2.3. Corrosion

2.3.1. The corrosion prevention methods.

Use of a waterproofing layer is needed to prevent water from penetrating the concrete surface. According to Rodrigues, R. (2021), the main influencing factors are the water content and the pore structure at the steel–concrete interface layer. Based on the earlier assessment of durability and mix design considerations, the concrete mixture will consist of Type V Portland cement, a water-to-cement ratio of 0.40, 10% silica fume as a supplementary cementitious material. The reinforcement concrete of the whole frame has barrier material - sealer.

2.3.2. Corrosion monitoring methods.

Depending on the RC material's degree of damage and age of structure different corrosion monitoring methods can be applied. According to Fan, L., & Shi, X. (2021), on the initiation and corrosion onset stage, the spectrometer and electrochemical sensors for chloride ions and pH measurements should be conducted; then later on structure's service life until the loss of steel cross section and concrete cracking or spalling occurs, wireless sensors inserted with metal link and potentiostat, or electrochemical sensors like linear polarization resistance, galvanic pulse method, electrochemical impedance spectroscopy are used, otherwise at its end of service life rehabilitation would be needed. A combination of measuring techniques is recommended.

3. Environmental part

3.1. Background

The Integrated Solid Waste Management Plan for Louisiana State was reported in 1994. In the USA, different states have their own approach regarding Solid Waste Management. Louisiana compared to the states as Ohio, New Jersey and Missouri did not have issues regarding landfill capacity. In the report, it was mentioned that landfill capacity for Louisiana state was estimated as ten years of remaining landfill capacity. However, comparing the reports regarding landfill capacity in 2011 and 2021, it can be seen that it rises from 4 millions wet-tons in 2011 to 9.5 millions wet-tons in 2021 of municipal solid waste. In ten years, the solid waste quantity in landfills has doubled. From this information the conclusion that initial estimation made in 1994 SWM plan was not efficient can be drawn. The SWM plan should be based on the ISWM hierarchy, which is shown in Figure 3-1.

Figure 3-1. Integrated Solid Waste Management Hierarchy

So, the first priority in the hierarchy is reducing solid waste generation and reusing items to decrease the quantity of SW. In report 1994, the priority was given for recycling, claiming that it can reduce solid waste disposed in landfills by 25%. Other SWM practices are not mentioned in the reports regarding SWM in Louisiana. However, it was mentioned that making a voluntary planning program for SWM is welcomed.

3.2. Waste management

According to these decisions, it was estimated that 100% of recyclable material will be recycled, and other compounds of SW that can be composted and incinerated will go through appropriate practices. The more detailed data was shown in Table 3-1.

Table 3-1. Waste compounds and recovery type

Compound	Generated amount, kg/year	Recovery type	Proportion to recovery, %	Recovered amount, kg/year	Remaining amount to landfill, kg/year
Paper	86236.5	Recycling	68 (*The Paper Recycling Coalition*)	58640.8	27595.7
Glass	28186.3	Recycling	42 (*Glass Half Full | Glass recycling, coastal restoration*)	11838.2	16348.1
Metal	14764.2	Recycling	44 (Statista, 2023)	6496.2	8268.0
Plastic	35568.4	Recycling	5 (Sire, 2023)	1778.4	33790.0
Electronics	671.1	-	-	-	671.1
Organics	134555.8	Composting	31 (Pinkerton, 2023)	41712.3	92843.5
Construction	35232.8	Recycling	78 (*Construction & Demolition Waste Management & Recycling | USA Waste & Recycling*)	27481.6	7751.2
Household hazardous	335.6	-	-	-	335.6
Sum	335550.7				187603.1

3.2.1. Methane Generation

Methane generation from the disposed waste in the landfill will be calculated to identify the impact on the environment. Moisture percentage of the compounds are assumed to be their absorption capacity. And the dry mass was computed according to it. Table 3-2 depicts the details.

Table 3-2. Dry mass of compounds

Compound	Amount, kg/year	Moisture, %	Moisture amount, kg/year	Dry mass, kg/year
Paper	27595.7	98 (*[Absorption capacity of different brands of paper points]*, 1989)	27043.8	551.9
Plastic	33790.0	36 (Kober *et al.*, 2007)	12164.4	21625.6
Organics	92843.5	63 (Kassa *et al.*, 2024)	58491.4	34352.1
Sum				56529.6

From the dry mass, every compound's chemical composition was calculated based on their chemical properties. The results are shown in Table 3-3.

Table 3-3. Chemical composition of waste compounds

	Element	C	H	O	N	Other
Atomic weight, g/moles		12	1	16	14	-
Dry mass %	Paper	43.4	5.8	44.3	0.3	6.2
	Plastic	60	7.2	22.8	0	10
	Organics	48	6.4	37.6	2.6	5.4
Dry mass, kg/year	Paper	239.5	32.0	244.5	1.7	-
	Plastic	12975.4	1557.0	4930.6	0.0	-
	Organics	16489.0	2198.5	12916.4	893.2	-
Number of moles	Paper	19960.9	32011.0	15281.1	118.3	-
	Plastic	1081279.4	1557042.3	308164.6	0	-
	Organics	1374083.8	2198534.1	807274.2	63796.7	-

The reaction of methane generation is :

$$C_aH_bO_cN_d + nH_2O \rightarrow mCH_4 + sCO_2 + dNH_3$$

The number of moles of methane is calculated by the following formula:

$$m = (4a + b - 2c - 3d)/8$$

The calculated methane mass generated from waste compounds is written in Table 3-4.

Table 3-4. Generated methane mass

Compound	Chemical formula	Number of methane moles	Methane mass, kg
Paper	$C_{19960.9}H_{32011}O_{15281.1}N_{118.3}$	10117.2	161.9
Plastic	$C_{1081279.4}H_{1557042.3}O_{308164.6}N_{0}$	658228.8	10531.7
Organics	$C_{1374083.8}H_{2198534.1}O_{807274.2}N_{63796.7}$	736116.3	11777.9

3.3. Waste Collection Plan

For the waste collection plan the generation rate of the solid waste in peak days was selected to avoid the cases of overloading. During peak days the waste generation $993.705\ kg/day$. According to this data, waste collection is planned, shown in Table 3-5.

Table 3-5. Waste Collection Plan Details

Container	Capacity	726 kg (*Container sizes*)
	Dimensions	190.5 x 182.9 x 208.3 cm^3 (*Container sizes*)
	Quantity	2
Collection	Frequency	2 times per week
Collection Transport	Total body length	6 - 7 m (Btr, 2020)
	Width	2.4 m (Btr, 2020)
	Height	2.4 m (Btr, 2020)
	Compaction rate	593.3 kg per cubic meters (Btr, 2020)
	Pack cycle	17-19 sec, reloading 6-7 sec (Btr, 2020)
	Capacity	3.03 cubic meters (Btr, 2020)

The route of the waste collection transport is illustrated in Figure 3-2.

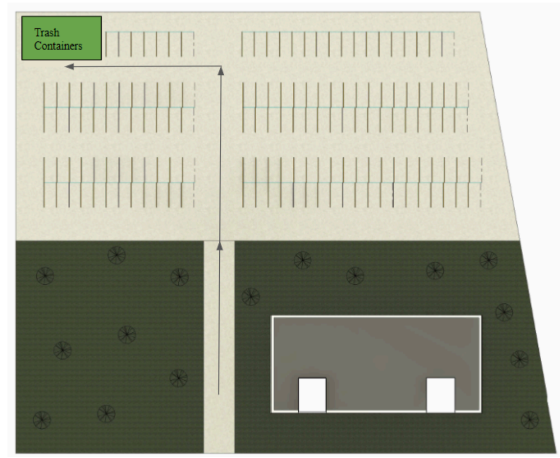


Figure 3-2. The route of waste collection truck

3.4 Integrated Solid Waste Management Plan

The Integrated Solid Waste Management Plan for the hotel is depicted in the form of flow chart in Figure 3-3.

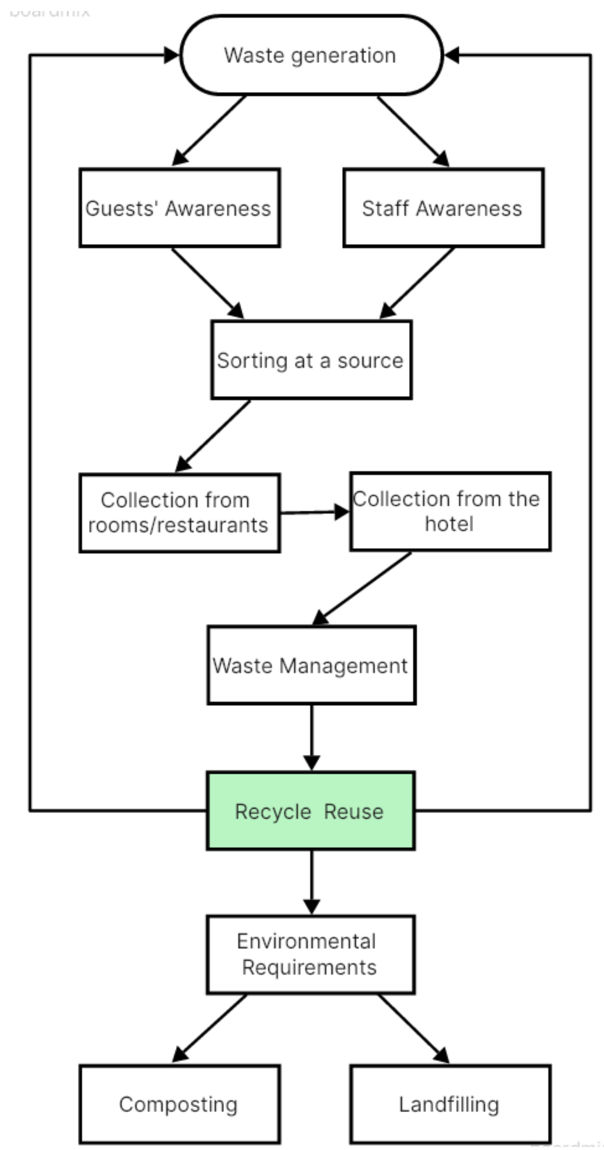


Figure 3-3. Solid Waste Management Plan

4. Geotechnical part

4.1 Site description

The chosen site is in Jefferson Parish, Louisiana, found at 3680 Pritchard Road, marked with a red placemark in Figure 4.1. Jefferson Parish is in a unique spot between the Mississippi River and the Gulf of Mexico. It has a varied landscape with bayous, marshlands, and a vibrant cultural history (USAFacts, 2023).

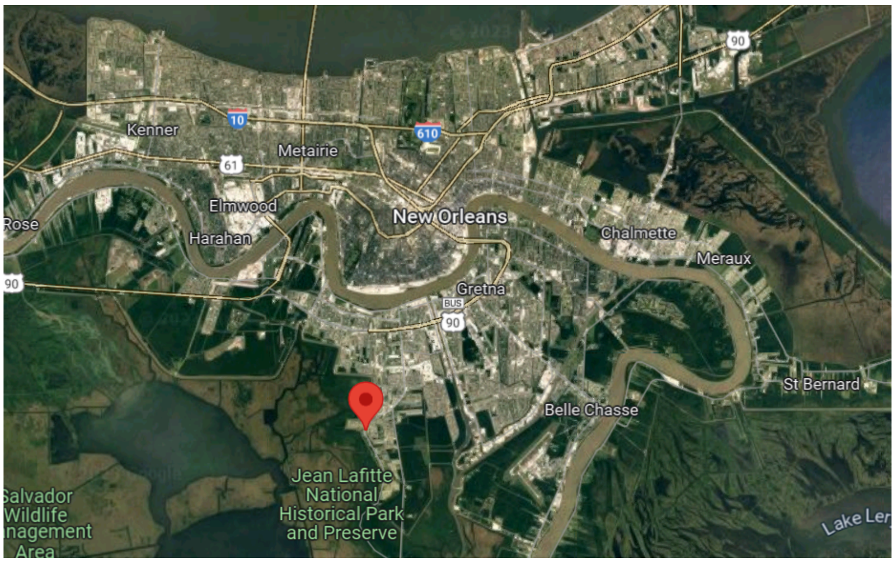


Figure 4.1. Regional map

Figure 4.2. The site location in Jefferson Parish, LA, USA

A thorough analysis of the construction site indicates its nearness to the intersection of Pritchard Road and Barataria Boulevard, as shown in Figure 4.2. To evaluate the soil conditions at this chosen spot, the geotechnical site investigation report was referred from nearby regions. For comparison, the soil profile of borehole WWHC-41 UPT was selected, that is located 300 meters from the construction site, assuming similar soil composition due to the short distance. This reference is in Figure 4.3.

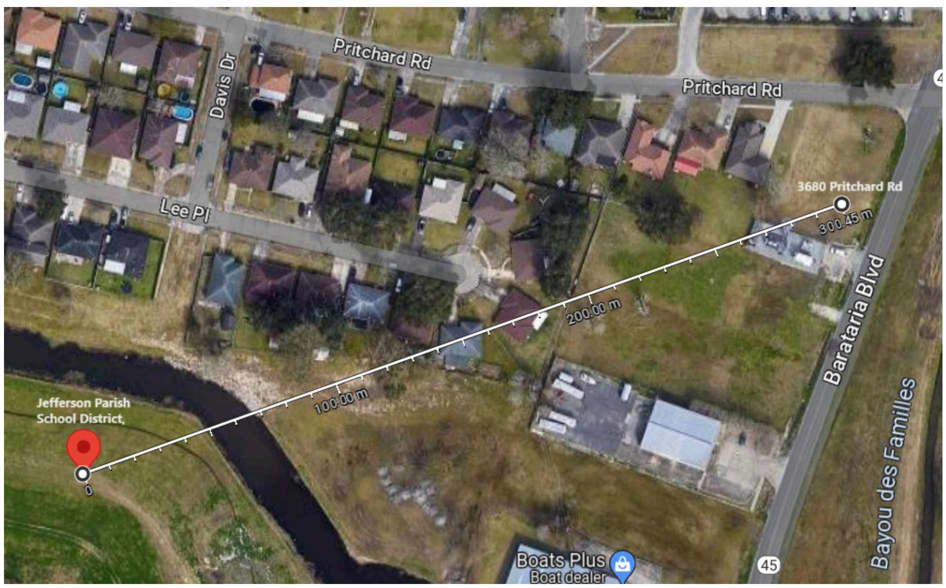


Figure 4.3. The distance between the construction site and the borehole (29°50'38.00"N 90°07'23.32"W)

Currently, 3680 Pritchard Road functions as a parking lot, but there are plans to clear the existing pavement for an upcoming construction project. The proposed development entails building a 16-story hotel, as depicted in the layout shown in Figure 2.1. This ambitious project is aimed at promoting the progress and improvement of the Jefferson Parish area.

4.2 Field investigation

In total, 32 new boreholes were drilled for soil investigation, providing undisturbed samples. Depths of these new boreholes range from 21.37m to 27.43m. Additionally, there are 22 boreholes drilled before the initial expansion, with depths ranging from 15.24m to 30.48m. The new boreholes labeled with "WWHC-" were drilled between February and March of 2007, while those labeled with (HL-) and (HLC-) were drilled in 1987 and 1990, respectively. The locations of all boreholes are indicated in Figures 4.4-4.5.

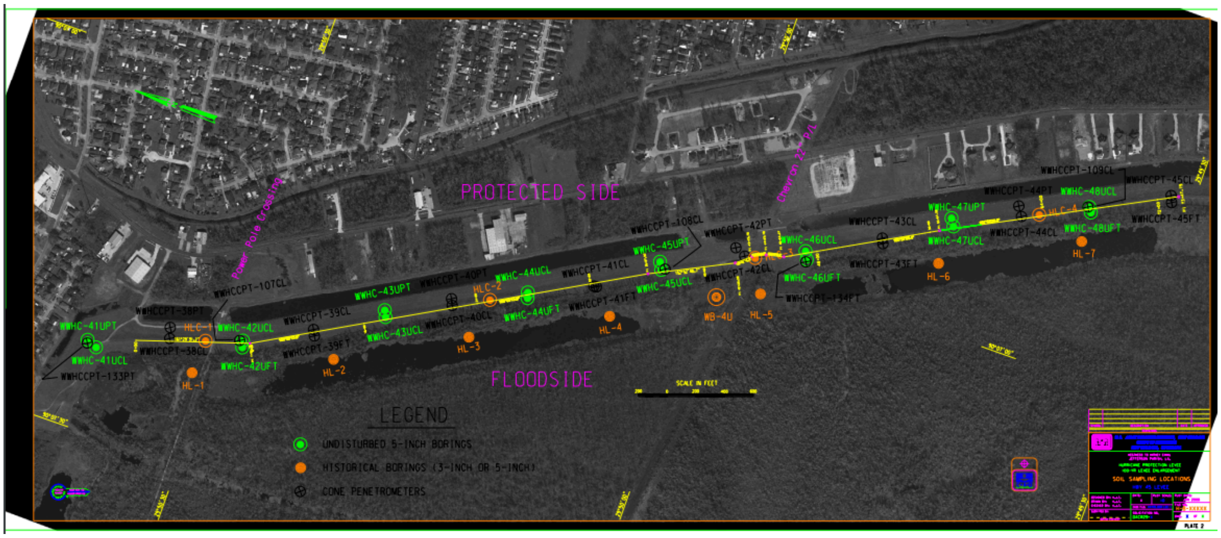


Figure 4.4. Location of boreholes

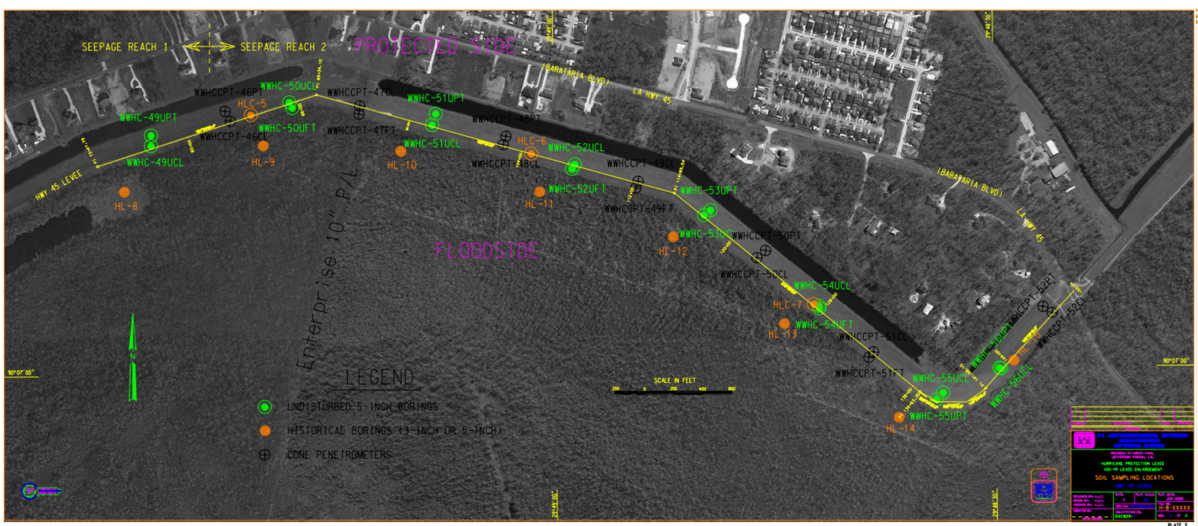


Figure 4.5. Location of boreholes

All soil samples collected from the undisturbed boreholes were classified using the Unified Soil Classification System (USCS). This classification involved visual assessment and sieve analysis, along with determining moisture content, organic matter content, and Atterberg limits. Additionally, Cone Penetration Tests (CPT) were conducted, as well as Shear Strength tests, including Unconfined Compression (UCT) tests and Unconsolidated Undrained (Q) tests.

4.3 Soil profile

Creating an accurate soil profile of the construction site is crucial, so a thorough geotechnical investigation was conducted and detailed in Table 4.1. The soil primarily consists of lean and fat clay, with their sequence varying with depth. The upper soil layer is lean clay, characterized by low-to-medium plasticity, with a thickness of 0.24 meters. Below that, from 0.24 meters to 1.22 meters, lies fat clay, which is cohesive, compressible, and highly plastic. Based on the Atterberg Limit test results, the soil samples show low to medium plasticity, indicating a potential for soil expansion. Moreover, no bedrock was encountered during the investigation down to a depth of 21 meters.

Table 4.1. Generalized soil profile from the borehole WWHC-41UPT (29°50'38.00"N 90°07' 23.32"W)

Layer no.	**Depth range, m**	**Depth, m**	**Soil type (USCS)**	**Description**	**Dry unit weight,** kN/m^3	**Friction angle, °**	**Elastic Modulus, MPa**	**Poisson's ratio**	**Saturated unit weight,** kN/m^3	**Cohesion,** kPa
1	0.0-0.24	0.24	Lean clay (CL)	Clay of low-to-medium plasticity	-	-	15	0.417	-	-
2	0.24-1.22	0.98	Fat clay (CH)	Cohesive and compressible clay of high plasticity	12.41	-	10	0.4	16.97	60.23
4	1.22-1.74	0.52	Lean clay (CL)	Clay of low-to-medium plasticity	-	-	15	0.417		-
5	1.74-2.99	1.25			14.14	-			18.85	18.67
6	2.99-4.18	1.19			13.67	0			18.38	25.7
7	4.18-5.39	1.21	Fat clay (CH4)	Cohesive and compressible clay	11.9	0	10	0.4	17.44	23.7
8	5.39-6.34	0.95			11.6	0			16.80	40.3

9	6.34-6.61	0.27		of high plasticity	10.99	0			16.65	26.0
10	6.61-8.11	1.5			8.48	0			15.08	-
11	8.11-8.53	0.42	Lean clay (CL4)	Clay of low-to-medium plasticity	9.7	-	15	0.417	16.02	23.4
12	8.53-8.78	0.25	Fat clay (CH2)	Cohesive and compressible clay of high plasticity	-	-	10	0.4	-	-
13	8.78-9.05	0.27	Lean clay (CL6)	Clay of low-to-medium plasticity			15	0.417		
14	9.05-10.27	1.22	Fat clay (CH3)	Cohesive and compressible clay of high plasticity	10.99	-	10	0.4	16.80	19.06
15	10.27-11.2	0.93	Fat clay (CH4)		11.78	-			17.28	29.44
16	11.2-11.5	0.3			9.58	0			15.87	26.57
17	11.5-12.7	1.2			9.42	-			15.55	-
18	12.7-13.65	0.95			10.21	-			16.49	33.66
19	13.65-14.9	1.25			10.05	0			16.18	33.89
	14.9-16.4	1.5			10.68	-			16.49	30.36
20	16.4-17.6	1.2			11.15	-			16.80	30.16
21	17.6-18.8	1.2			10.37	0			16.02	47.2

22	18.8-20.0	1.2	Sandy clay (CL4-S)	Sandy clay	10.05	-		0.470	16.02	29.78
23	20.0-21.24	1.24	Fat clay (CH4)	Cohesive and compressible clay of high plasticity	13.66	-	10	0.4	18.07	33.56
24	21.24-21.5	0.26			10.21	-			16.34	64.35

4.3.1. Location of groundwater table

In geotechnical design, it's crucial to take into account the depth of groundwater, as it impacts the overall bearing capacity of a foundation. Free water was discovered approximately 2.21 meters (7.25 feet) below mean sea level (MSL) (West Bank H.P.P., 2008). Figure 2 illustrates the gauges, observation well, piezometer, and levee systems in Greater New Orleans, Louisiana. As shown in Figure 1, the levee system in Jefferson Parish is located near the construction site.

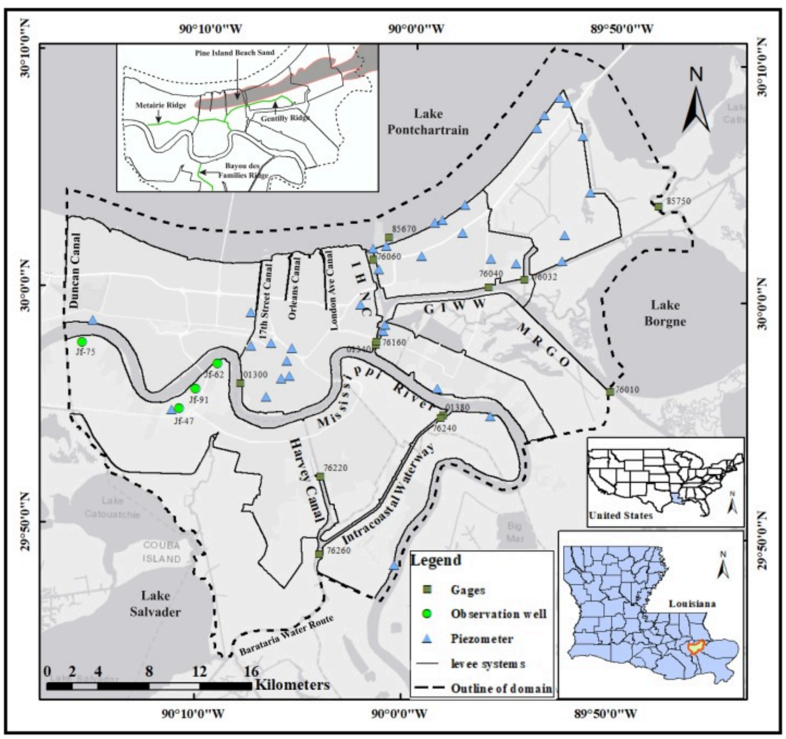


Figure 4.6. The illustration of the levee systems in Greater New Orleans, Louisiana (Yang & Tsai, 2020)

Precipitation is abundant throughout the year, with the majority falling between March and June, while snow is rare (People Stats, 2023). Comparatively, the average annual rainfall in the US is 38 inches, whereas it's 64 inches in Jefferson Parish, with almost no snowfall. The climate in Jefferson Parish, Louisiana, is humid and subtropical.

4.4 Foundation design

4.4.1. Foundation type: Shallow Foundation

The shallow foundation design was done to support the external loads. The foundation was designed based on the General Bearing Capacity equation, proposed by Meyerhof (1963):

$$q_u = c'N_c F_{cs} F_{cd} F_{ci} + qN_q F_{qs} F_{qd} F_{qi} + \frac{1}{2}\gamma B N_\gamma F_{\gamma s} F_{\gamma d} F_{\gamma i} \tag{4.1}$$

where

c' - cohesion;

q - effective stress at the level of the bottom of the foundation;

γ - unit weight of soil;

B - width of foundation;

N_c, N_q, N_γ - bearing capacity factors;

F_{cs}, F_{qs}, $F_{\gamma s}$ - shape factors;

F_{cd}, F_{qd}, $F_{\gamma d}$ - depth factors;

F_{ci}, F_{qi}, $F_{\gamma i}$ - load inclination factors.

The bearing capacity factors, N_c and N_q are determined by Prandtl (1921) and Reissner (1924), respectively. The relation for N_γ was derived by Caquot and Kerisel (1953) and Vesic (1973). The bearing capacity factors, N_c, N_q and N_γ are found as follows:

$$N_c = (N_q - 1)cot\varphi' \tag{4.2}$$

$$N_q = tan^2(45 + \frac{\varphi'}{2})e^{\pi tan\varphi'} \tag{4.3}$$

$$N_\gamma = 2(N_q + 1)tan\varphi' \tag{4.4}$$

Also, the bearing capacity factors can be found with soil friction angles.

DeBeer (1970), Hansen (1970), Meyerhof (1963); Meyerhof and Hanna (1981) proposed the equations below to determine shape, depth and inclination factors:

$$F_{cs} = 1 + (\frac{B}{L})(\frac{N_q}{N_c}) \tag{4.5}$$

$$F_{qs} = 1 + (\frac{B}{L})tan\varphi' \tag{4.6}$$

$$F_{\gamma s} = 1 - 0.4(\frac{B}{L}) \tag{4.7}$$

The depth factor factors for $\frac{D_f}{B} \leq 1$ and $\varphi = 0$ are found as follows:

$$F_{cd} = 1 + 0.4(\frac{D_f}{B}) \tag{4.8}$$

$$F_{qd} = 1 \tag{4.9}$$

$$F_{\gamma i} = 1 \tag{4.10}$$

The load inclination factors are determined as follows:

$$F_{ci} = F_{qi} = (1 - \frac{□°}{90°}) \tag{4.11}$$

$$F_{\gamma i} = (1 - \frac{□°}{\varphi'}) \tag{4.12}$$

where

□ - inclination of the load on the foundation with respect to the vertical.

Given the groundwater table's proximity to the surface, it's crucial to adjust the general bearing capacity accordingly. The water table is assumed to be situated at $0 \leq d \leq B$. The effective surcharge, q, is computed as follows:

$$q = \bar{\gamma} D_f \tag{4.13}$$

$$\bar{\gamma} = \gamma' + \frac{d}{B}(\gamma - \gamma') \tag{4.14}$$

$$\gamma' = \gamma_{sat} - \gamma_w \tag{4.15}$$

where

d - distance between the bottom of the foundation and the water level;

B - width of foundation;

$\bar{\gamma}$ - weighted average value of the effective unit weight;

D_f - embedded depth;

γ' - difference between the saturated unit weight of soil and unit weight of water;

γ_{sat} - saturated unit weight of soil;

γ_w - unit weight of water.

Then, it is important to reduce the general bearing capacity by a factor of safety, FS, as follows:

$$q_{all} = \frac{q_u}{FS} \tag{4.16}$$

The factor of safety serves to prevent structural collapse under critical conditions. Its magnitude depends on various factors such as assumptions regarding loading characteristics, the heterogeneous nature of soil material, and the limited understanding of soil properties like shear strength parameters (Jumikis, n.d.). According to Alfreds & Jumikis (n.d.), typical factors of safety values for many soil engineering structures range from 1.5 to 2.5. Moreover, due to uncertainties surrounding shear strength parameters, a relatively conservative factor of safety of 3 is often employed in shallow foundation design (Das & Sivakugan, 2019). Opting for a higher factor of safety of 3 accounts for potential hazards such as strong winds and proximity to the water table, ensuring safety from potential risks.

4.4.1.1. Spread Footing

The further data and assumptions, given in Table 4.2, were used to design a spread footing.

Table 4.2. Data and assumptions for the design of the spread footing

Embedded Depth, D_f, m	1	Assumption	Based on Louisiana Building Code, 29654, minimum depth is 304.8 mm
Shape	BxB	Assumption	Due to location of columns
Factor of Safety, FS	3	Assumption	Braja M.Das & Nagaratnam Sivakugan, Principles of Foundation Engineering, Ninth Edition, SI Edition, 2017
Soil Layer	Fat clay, cohesive and compressible clay of high plasticity	Given	
Cohesion, c', kN/m^3	25.71	Given	
Dry Unit Weight, γ, kN/m^3	13.67	Given	
Saturated Unit Weight of soil, γ_{sat}, kN/m^3	18.38	Given	
Unit Weight of water, γ_w, kN/m^3	9.81	Given	
The distance from the bottom of the foundation to the water level, d, m	1.21	Given	
Angle of Friction Angle, φ'	0	Given	
Maximum Load on Column, $Q_{designed}$, kN	6696	Calculated	

First of all, the effective surcharge, q is found as follows:

$$\bar{\gamma} = (18.38 - 9.81) + \frac{1.21}{B}(13.67 - (18.38 - 9.81))$$

$$\bar{\gamma} = 8.57 + \frac{6.171}{B}$$

$$q = \bar{\gamma}D_f = (8.57 + \frac{6.171}{B}) \times 1 = 8.57 + \frac{6.171}{B}$$

For $\varphi' = 0$, the bearing capacity factors are:

$$N_c = 5.14$$

$$N_q = 1$$

$$N_\gamma = 0$$

It is assumed that the shape of the spread footing is a square, so that the width = the length. The shape factors are calculated by following procedure:

$$F_{cs} = 1 + \frac{B}{B} \times \frac{1}{5.14} = 1.19455$$

$$F_{qs} = 1 + \frac{B}{B}tan0 = 1$$

$$F_{\gamma s} = 1 - 0.4(\frac{B}{B}) = 0.6$$

The depth factors are as following:

$$F_{cd} = 1 + 0.4(\frac{1}{B}) = 1 + \frac{0.4}{B}$$

$$F_{qd} = 1$$

$$F_{\gamma i} = 1$$

It is assumed that the inclination of the load is 0, so the inclination factors are as follows:

$$F_{ci} = F_{qi} = F_{\gamma i} = 1$$

The general bearing capacity is determined as follows:

$$q_u = 25.71 \times 5.14 \times 1.19455\times (1 + \frac{0.4}{B}) \times 1 + (8.57 + \frac{6.171}{B}) \times 1 \times 1 \times 1 \times 1$$

$$q_u = (166.429399 + \frac{69.3147595}{B})kN/m^2$$

The allowable bearing capacity was calculated as the general bearing capacity divided by a factor of safety:

$$q_{all} = \frac{q_u}{FS} = \frac{1}{3}(166.429399 + \frac{69.3147595}{B}) = 55.47646633 + \frac{23.10491983}{B}$$

Then the allowable bearing capacity is found from the following equation by equalization:

$$q_{all} = \frac{Q_{all}}{B^2} = \frac{6473}{B^2}$$

$$55.47646633 + \frac{23.10491983}{B} = \frac{6473}{B^2}$$

$$55.47646633B^2 + 23.10491983B = 6473$$

$$B = 10.59\, m \approx 10.6m$$

As a result, the ultimate bearing capacity is determined as following:

$$q_u = (166.429399 + \frac{69.3147595}{B}) = 166.429399 + \frac{69.3147595}{10.6} = 172.97kN/m^2$$

Consequently, the ultimate bearing load is as follows:

$$Q_u = q_u \times A = 172.97 \times 10.6 \times 10.6 = 19419\, kN$$

The allowable load is determined as follows:

$$Q_{all} = \frac{Q_u}{FS} = \frac{19419}{3} = 6473\, kN$$

The analysis indicates that the spread footing should be a square measuring 10.6 meters on each side. However, due to the column spacing of 5 meters (across 40 meters) and 6 meters (across 18 meters) in the structure, a spread footing of 10.9 meters width is not feasible. Consequently, it is crucial to revise the design of the mat foundation.

4.4.1.2. Mat Foundation

The footprint of the mat foundation is the same shape of the structure, as in Figure 4.7.

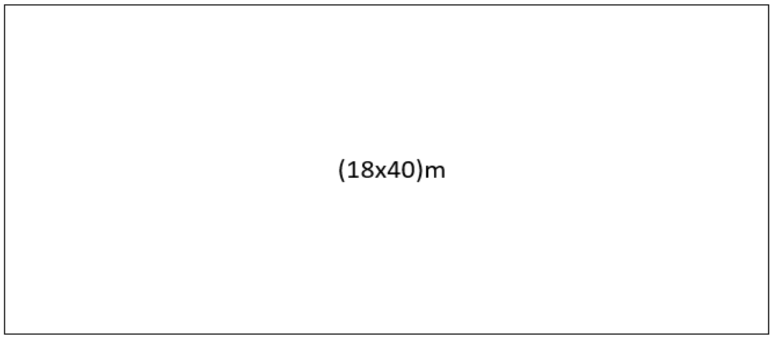


Figure 4.7. The layout of the mat foundation

The following data and assumptions were used to design the mat foundation:

Table 4.3. Data and assumptions for the design of the mat foundation

Embedded Depth, D_f , m	1.5	Assumption	According to Louisiana Building Code, 29654, minimum depth is 304.8 mm
Width, m	18	Given	
Length, m	40	Given	
Undrained shear strength, c_u, kN/m^3	25.71	Given	$c = c_u$ (According to Braja M.Das & Nagaratnam Sivakugan, Principles of Foundation Engineering, Ninth Edition, SI Edition, 2017)
The distance from the bottom of the foundation to the water level, d, *m*	0.71	Given	
Factor of Safety	3	Assumption	According to Braja M.Das & Nagaratnam Sivakugan, Principles of Foundation Engineering, Ninth Edition, SI Edition, 2017
Maximum Load, $Q_{designed}$, kN	233028	Calculated	36 Columns

The net ultimate bearing capacity of a mat foundation on a clay is found as follows:

$$q_u = q_{u(net)} + q \tag{4.17}$$

$$q_{u(net)} = 5.14c_u[1 + \frac{0.195B}{L}][1 + 0.4\frac{D_f}{B}] \tag{4.18}$$

$$q_{u(net)} = 5.14 \times 25.71 \times [1 + \frac{0.195\times18}{40}][1 + 0.4\frac{1.5}{18}] = 135.6457233\ kN/m^2$$

To calculate the net ultimate bearing capacity, considering a mat foundation width of 18 meters, which exceeds 2 meters, the reduction factor, r_γ, proposed by Bowles (1996) should be applied:

$$r_\gamma = 1 - 0.25log(\frac{B}{2}) \tag{4.19}$$

$$r_\gamma = 1 - 0.25log(\frac{18}{2}) = 0.7614393726$$

The corrected net ultimate bearing capacity is determined by the following equation:

$$Corrected\ q_{u(net)} = q_{u(net)} \times r_\gamma = 135.6457233 \times 0.7614393726 = 103.2859945\ kN/m^2$$

The effective surcharge is found :

$$\bar{\gamma} = (18.38 - 9.81) + \frac{0.71}{18}(13.67 - (18.38 - 9.81))$$

$$\bar{\gamma} = 8.771\ kN/m^3$$

$$q = \bar{\gamma}D_f = 8.771 \times 1.5 = 13.156\ kN/m^2$$

Then the ultimate bearing capacity is calculated by Equation 17:

$$q_u = 103.2859945 + 13.156 = 116.4427445\ kN/m^2$$

Then, the ultimate bearing load is as follows:

$$Q_u = q_u \times A = 116.4427445 \times 18 \times 40 = 83838.77603\ kN$$

The allowable load is found as follows:

$$Q_{all} = \frac{Q_u}{FS} = \frac{83838.77603}{3} = 27946.25868\ kN$$

The calculations reveal that the final allowable load of 27946.25868 kN falls short of the designed load of 233028 kN, indicating that the designed mat foundation is unsuitable for the structure. Further investigation into deep foundation options was undertaken as a result.

4.4.2 Foundation selection

During Capstone 1, it was determined that a deep foundation is suitable to withstand the load from the structure. The shallow foundation types such as spread footing and mat foundation are also designed in the section of shallow foundations, but they are not proposed as they do not satisfy the design requirements. Deep foundation is a type of foundation that supports the loads from the columns and transfers them from weak, compressible soils to harder, less compressible soils (Das & Sivakugan, 2019). Different types of piles are utilized in construction, depending on factors like the expected load, soil conditions, groundwater levels, and installation methods. The primary materials for pile construction are steel, concrete, and timber. The precast concrete driven pile (reinforced) was selected for several reasons:

1. Widely recognized for use in soft clay conditions.
2. Capable of withstanding high bending and axial stress.
3. Provides enhanced durability when constructed with appropriate concrete cover and grade.
4. Requires less time and cost compared to bored piles (Prasad, 2020).

For soft clay conditions, the non-displacement pile installation method was chosen because:

1. It minimizes disturbance to the surrounding soil.
2. It reduces the risk of settlement issues (University of the West of England, n.d.).

4.4.3 Foundation design of deep foundation under axial loading

4.4.3.1 Hand calculation of axial bearing capacity

Estimation of ultimate bearing capacity has shown that single piles are not suitable for our structure, as they are not capable of carrying loads of internal columns. Therefore, it was decided to use groups of piles to increase bearing capacity and make the project more economically efficient. In Capstone Ⅰ we used center-to-center pile spacing as 3D. However, distance between columns was not enough for such spacing, it means pile groups would overlap each other, so we decreased d to 2D.

To determine bearing capacity of pile groups two methods were used:.

Method Ⅰ

$$\Sigma Q_u = n_1 n_2 (Q_p + Q_s) \quad (4.20)$$

$$Q_p = A_p [9c_{u(p)}] \quad (4.21)$$

$$Q_s = \Sigma \alpha p c_u \Delta L \quad (4.22)$$

$c_{u(p)} = \ undrained\ cohesion\ of\ the\ clay\ at\ the\ pile\ tip$

$n_1 n_2 = number\ of\ piles\ in\ a\ group$

The ultimate load-bearing capacity of pile group can be calculated by the following equation

$$\Sigma Q_u = n_1 n_2 (A_p 9c_{u(p)} + \Sigma \alpha p c_u \Delta L) \quad (4.23)$$

Method ⅠⅠ

In these estimations it was considered that pile group is acting as a block with dimensions $L_g \times B_g \times L$

$$\Sigma Q_u = L_g B_g c_{u(p)} N_c^* + \Sigma 2(L_g + B_g) c_u \Delta L \quad (4.24)$$

$N_c^* = bearing\ capacity\ factor\ (see\ Appendix\ A4.22)$

$$L_g = (n_1 - 1)d + 2(D/2) \quad (4.25)$$

$$B_g = (n_1 - 1)d + 2(D/2) \quad (4.26)$$

Calculations were done for pile groups with diameters of 0.4 m, 0.5 m and configurations of 3×3 and 4×4. As two methods were used, lower results' value was taken as final ΣQ_u.

Table 4.4. Calculation of allowable bearing capacity

D = 0.4m				**D = 0.5m**			
L, m	n_1	n_2	$\Sigma Q_{g(all)}$, **kN**	**L, m**	n_1	n_2	$\Sigma Q_{g(all)}$, **kN**
10	3	3	1295.99	10	3	3	1657.98
15			1863.96	15			2368.74
20			2610.82	20			3316.77
25			3414.81	25			4337.03
10	4	4	2303.99	10	4	4	2947.52
15			3313.71	15			4211.09
20			4641.45	20			5896.49
25			6070.78	25			7710.28

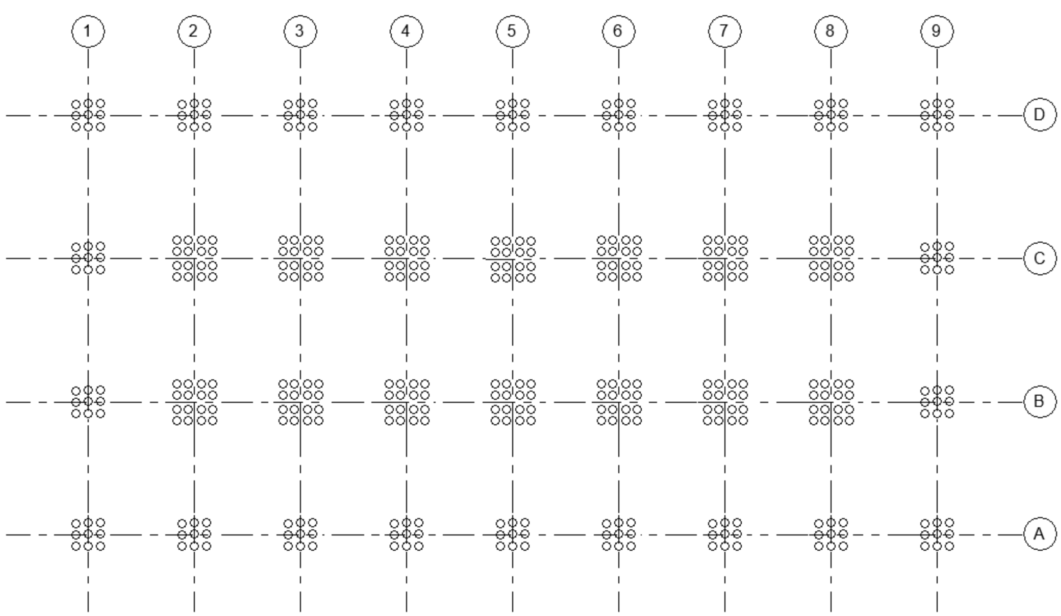


Figure 4.8. Preliminary piles and group piles location layout

4.4.3.2. Hand calculation of settlement

4.4.3.2.1. Elastic settlement

Elastic settlement for pile group was calculated by the following equation:

$$s_{g(e)} = \sqrt{\frac{B_g}{D}} * s_e \tag{4.27}$$

where

$s_{g(e)} = elastic\ settlement\ of\ group\ piles$

$B_g = width\ of\ group\ pile\ section$

$D = diameter\ of\ each\ pile\ in\ the\ group$

$s_e = elastic\ settlement\ of\ each\ pile\ at\ comparable\ working\ load$

Table 4.5. Elastic settlement of pile groups

D=0.5m				
L, m	B_g	$n \times n$	s_e, mm	$s_{g(e)}, mm$
10	2.5	3×3	7.010	12.34
15			9.474	17.32
20			14.864	28.01
25			21.889	42.42
10	3.5	4×4	7.010	19.83
15			9.474	26.79
20			14.864	42.03
25			21.889	61.91

4.4.3.2.2. Consolidation settlement

Consolidation settlement for a group of piles was calculated by 2:1 stress distribution method with an assumption that transmitted load Q_g starts at a depth of 2/3L.

$$\Delta\sigma'_i = \frac{Q_g}{(B_g+z_i)(L_g+z_i)} \tag{4.28}$$

$\Delta\sigma_i'$- *increase in effective stress*

B_g, L_g- *width and length of pile group*

z_i- *distance from z=0 to the middle of clay layer*

$$\Delta s_{c(i)} = \left[\frac{\Delta e_{(i)}}{1+e_{0(i)}}\right] H_i \qquad (4.29)$$

$\Delta s_{c(i)}$- *consolidation settlement of layer*

$\Delta e_{(i)}$- *change of void ratio*

$e_{0(i)}$- *initial void ratio*

H_i- *thickness of layer*

Total consolidation settlement of soil is

$$\Delta s_{c(g)} = \Sigma \Delta s_{c(i)} \qquad (4.30)$$

Chosen length for the pile group is 25m. Therefore, 2/3L = 16.7m

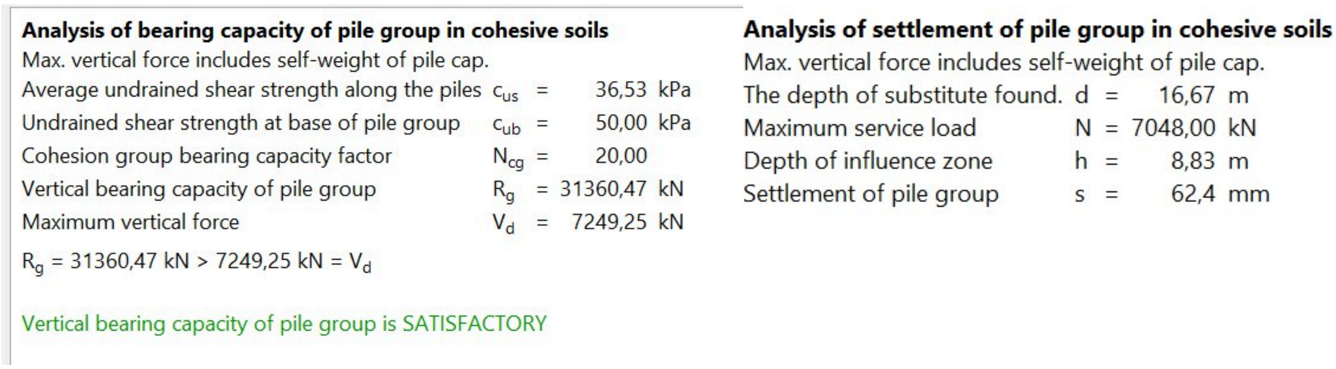
For consolidation settlement we considered piles with a diameter of 0.5m and configurations of 3x3, 4x4.

Table 4.6. Consolidation settlement of pile group

D=0.5m		
mm	**3x3**	**4x4**
$\Delta s_{c(1)}$	68.0	70.7
$\Delta s_{c(2)}$	21.1	24.5
$\Delta s_{c(3)}$	20.6	24.9
$\Delta s_{c(g)}$	109.7	120.1

4.4.2.3 Software calculation of axial bearing capacity and settlement

Firstly, the axial bearing capacity of the deep foundation was verified by GEO5, and then the settlement was calculated. In Figure 4.9 can be seen that the vertical bearing capacity of the pile group is satisfactory, these values are further used in PLAXIS 3D.


Analysis of bearing capacity of pile group in cohesive soils

Max. vertical force includes self-weight of pile cap.

Average undrained shear strength along the piles	c_{us}	=	36,53	kPa
Undrained shear strength at base of pile group	c_{ub}	=	50,00	kPa
Cohesion group bearing capacity factor	N_{cg}	=	20,00	
Vertical bearing capacity of pile group	R_g	=	31360,47	kN
Maximum vertical force	V_d	=	7249,25	kN

R_g = 31360,47 kN > 7249,25 kN = V_d

Vertical bearing capacity of pile group is SATISFACTORY

Analysis of settlement of pile group in cohesive soils

Max. vertical force includes self-weight of pile cap.

The depth of substitute found.	d	=	16,67	m
Maximum service load	N	=	7048,00	kN
Depth of influence zone	h	=	8,83	m
Settlement of pile group	s	=	62,4	mm

Figure 4.9. GEO5 verification of axial bearing capacity and settlement

Then the settlement of a group of piles (4x4, 3x3) was simulated in Plaxis 3D. The maximum load from the structure was 6696 kN from the internal column. For this load, the 4x4 group piles were constructed in Plaxis 3D. The other loads are shown below in Figure 4.10.

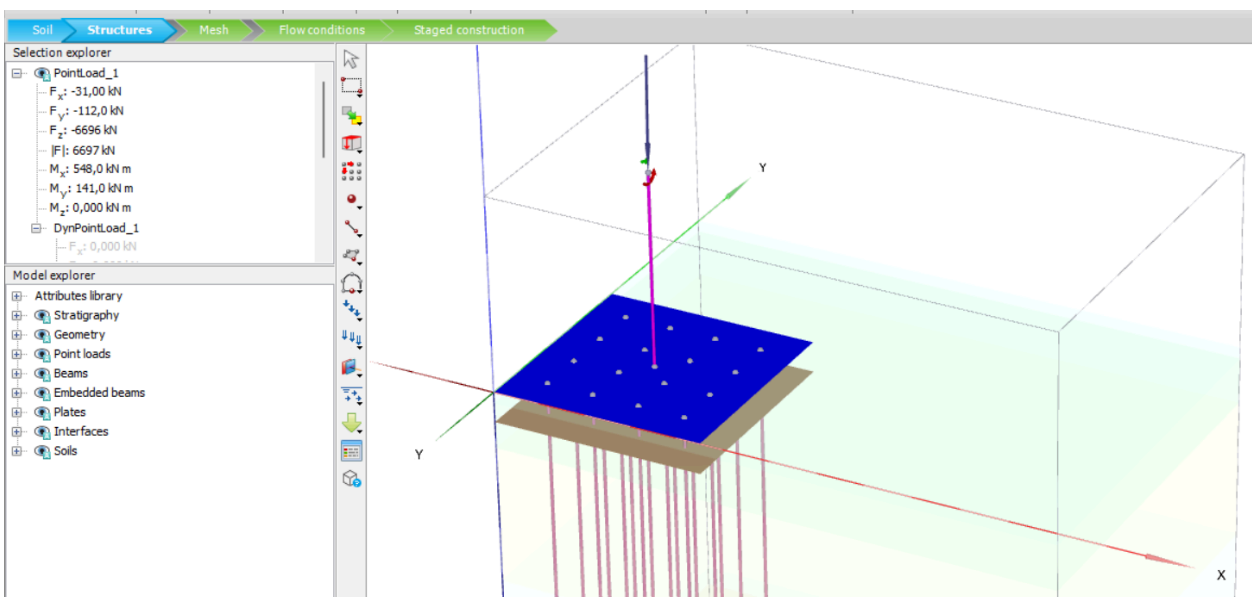


Figure 4.10. The load and moment values from the internal column to 4x4 group pile

Figure 4.11 shows the soil profile and 4x4 group pile with loads applied and moments. The settlement of the soil can be seen in Figure 4.12. The settlement value is 3.783 mm, which is less than allowable settlement of 25 mm. Moreover, the settlement for the 3x3 group pile is also within the limit. The settlement results can be seen in Table 4.7.

Table 4.7. Summary of the settlement results obtained from the simulation and hand calculations

Group piles	Number of piles	GEO5, mm	PLAXIS 3D, mm	Hand calculation, mm
Exterior	3	47.6	5.26	151.7
Corner	3	47.6	5.26	151.7
Internal	4	62.4	3.78	182

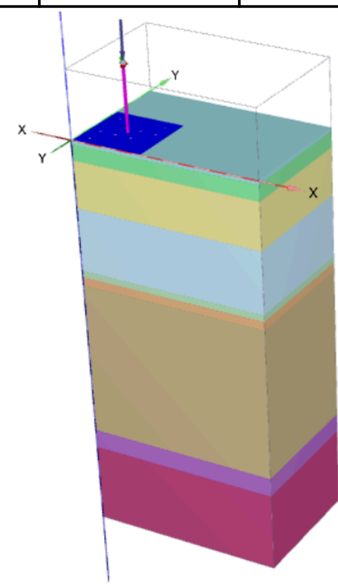


Figure 4.11. The soil profile and 4x4 group pile in PLAXIS 3D

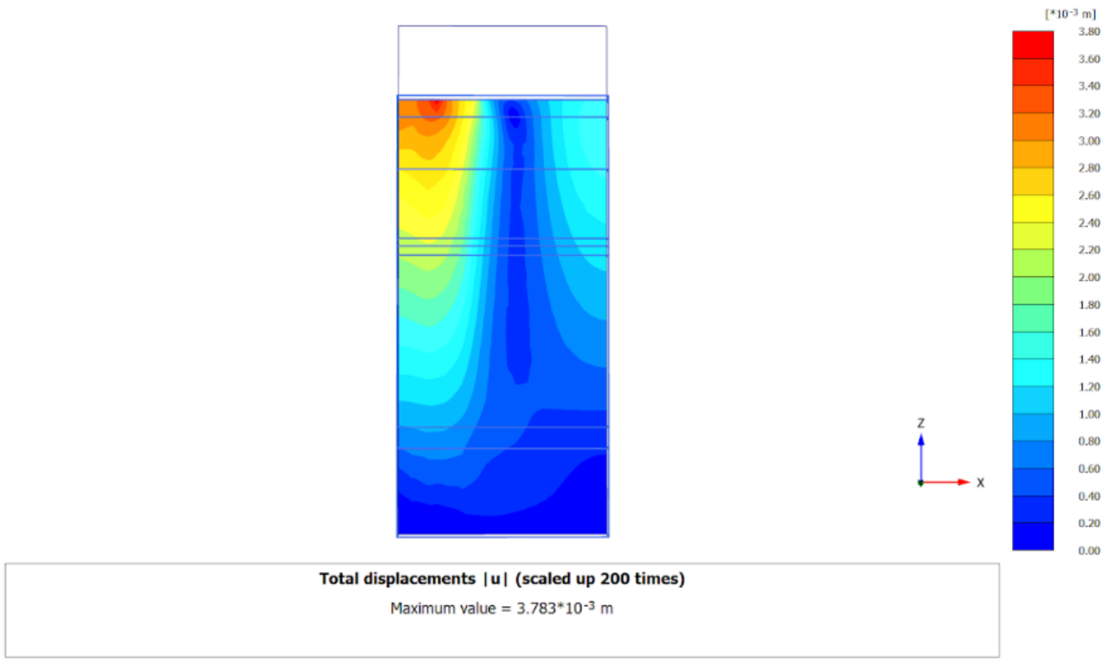
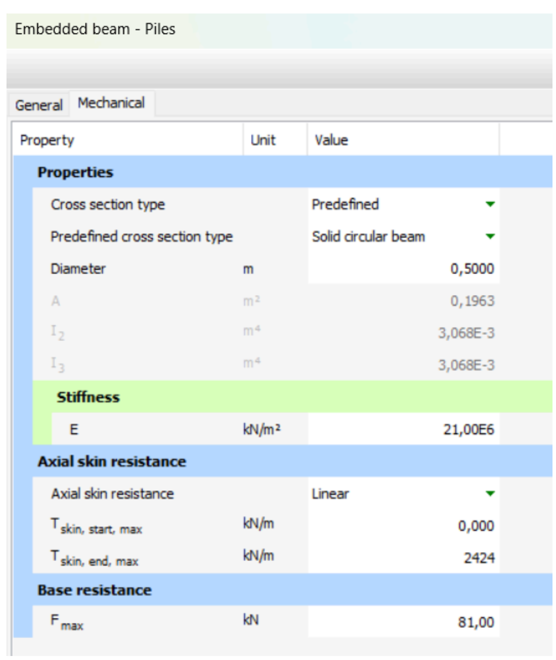


Figure 4.12. The total settlement of soil for 4x4 group piles in PLAXIS 3D

The staged construction consists of 3 phases, showing the stages of construction .The final stage consists of applied load, column, group of pile, pile cap. Figure 4.13 below shows the properties set for a single pile. The point bearing capacity is 81 kN, frictional resistance capacity is 2424 kN.



Embedded beam - Piles

General | Mechanical

Property	Unit	Value
Properties		
Cross section type		Predefined
Predefined cross section type		Solid circular beam
Diameter	m	0,5000
A	m²	0,1963
I_2	m⁴	3,068E-3
I_3	m⁴	3,068E-3
Stiffness		
E	kN/m²	21,00E6
Axial skin resistance		
Axial skin resistance		Linear
$T_{skin, start, max}$	kN/m	0,000
$T_{skin, end, max}$	kN/m	2424
Base resistance		
F_{max}	kN	81,00

Figure 4.13. The properties of a single pile in PLAXIS 3D

4.4.4 Group pile design

To withstand the bending and shear, the reinforcement of pile and pile cap is designed according to the method by Ray S.S.(1994).

4.4.4.1 Pile cap reinforcement

Step 1. Pile cap size determination

Based on the axial load analysis, pile group configuration and its sizes were determined. The pile cap sizing for various configurations is shown in Table 4.8.

Table 4.8. Pile cap sizing for group configurations

Pile group configurations	Lg, mm	Bg, mm	H, mm
1	1500	1500	1000
2x2	2500	2500	1000
3x3	3500	3500	1000
4x4	4500	4500	1000

Step 2. Estimation of load on pile.

It was chosen to have a rigid pile cap with backfill of 0.5m.

Table 4.9. Loads

Variables	Value	Unit
Dead Load, DL	199.4	kN/m^2
Live Load, LL	44.3	kN/m^2
Height of slab	1000	mm
Pile cap Dead Load	24	kN/m^2
Weight of pile cap, $W_{pile\ cap}$	$\gamma_{concrete} \times h = 24$	kN/m^2
Weight of backfill, $W_{backfill}$	$\gamma_{soil} \times h = 15$	kN/m^2

Weight of surcharge, $W_{surcharge}$	$DL + LL = 243.7$	kN/m^2

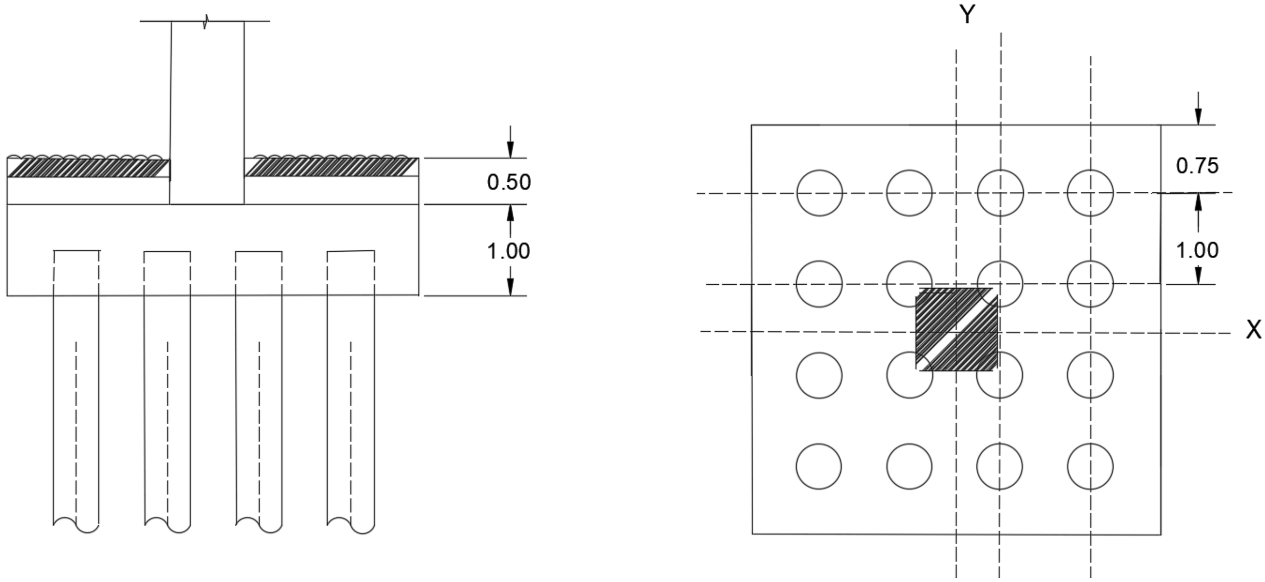


Figure 4.14. Backfill on pile cap

$$LC1 = 1.4DL + 1.6LL \quad (4.31)$$

$$LC2 = 1.2DL + 1.2LL + 1.2WL \quad (4.32)$$

$$LC3 = 1.4DL + 1.4WL \quad (4.33)$$

Factored weight of pile cap is $Wpc = 24 * 1.4 = 33.6 kN/m^2$

$$Wpc + Wsurcharge + Wbackfill = 24 + 243.7 + 15 = 282.7\ kN/m^2.$$

Applying load factor for dead load from weights:

$$1.2 * 282.7\ kN/m^2 = 339.2\ kN/m^2$$

Since $L_g = B_g$, the pile cap is symmetrical M_{11}'and M_{22}' are the same.

Sections 1-1 and 2-2 were taken at the face of the column.

Column size = 900mm x 900 mm.

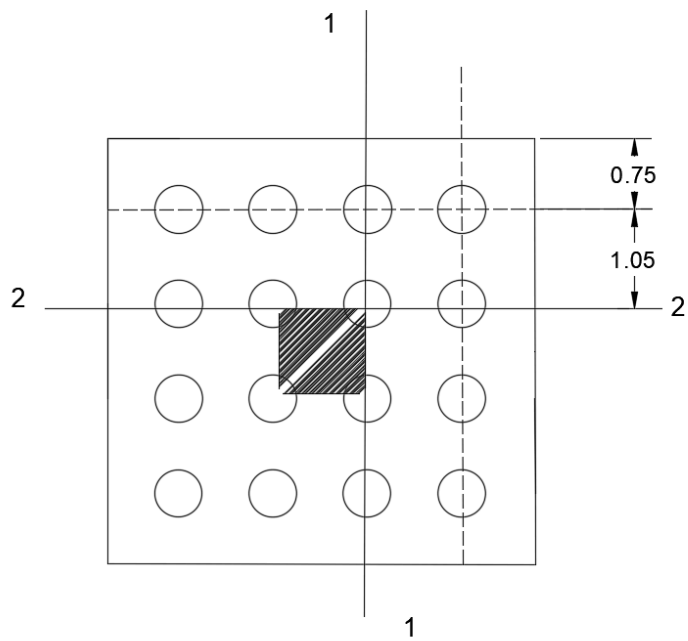


Figure 4.15. Critical sections for bending

$$M_{11}' = M_{22}' = \frac{4.5*339.2*1.8^2}{2} = 2478.6kNm$$

To estimate reaction forces Q_{1-4} following equations were used:

$$Q = \frac{P}{R} \pm \frac{M_{xx}y}{I_{xx}} \pm \frac{M_{yy}x}{I_{yy}} \quad (4.34)$$

P - vertical load on a group of piles

$$P = N + Wpc + Ws + Wb \quad (4.35)$$

M_{xx}- moment about x axis

$$M_{xx} = M_x + Ne_y + H_y h + M_x^* \quad (4.36)$$

M_{yy}- moment about y axis

$$M_{yy} = M_y + Ne_x + H_x h + M_y^* \quad (4.37)$$

$$I_{xx} = \Sigma y^2 \text{(about x axis)} \quad (4.38)$$

$$I_{yy} = \Sigma x^2 \text{(about y axis)} \quad (4.39)$$

$$Q_2 = \frac{Q_1+Q_3}{2} \quad (4.40)$$

$$M_{11}'' = 1.4(Q_3 + Q_4) \tag{4.41}$$

$$M_{22}'' = 0.5(Q_1 + Q_2 + Q_3) \tag{4.42}$$

$$M_{11} = M_{11}' + M_{11}'' \tag{4.43}$$

$$M_{22} = M_{22}' + M_{22}'' \tag{4.44}$$

Table 4.10. Loads on pile group

	Q_1	Q_2	Q_3=Q_4	M_{11}'=M_{22}'	M_{11}''	M_{22}''	M_{11}	M_{22}
LC1	570.57	571.00	571.42	2478.60	1599.9	856.50	4078.60	3335.10
LC2(x)	447.23	482.01	516.79	2478.60	1447.0	723.02	3925.63	3201.62
LC2(y)	349.84	482.01	614.18	2478.60	1719.7	723.02	4198.32	3201.62
LC3(x)	421.62	461.83	502.03	2478.60	1405.7	692.75	3884.31	3171.35
LC3(y)	308.00	461.83	615.66	2478.60	1723.8	692.75	4202.45	3171.35

Step 3. Cover to reinforcement.

The geotechnical report does not consist any information about SO_3 and exposure class. Therefore, it was decided to take cover as 75mm that can be used for general purpose.

Step 4. Reinforcement in a pile cap.

Design in x direction.

For the design in x direction a bar with diameter of 28 mm is assumed.

$$d_x = 900 - 75 - 28/2 = 811\, mm$$

$$f_{cu} = 40N/mm^2$$

$$f_y = 460N/mm^2$$

$$K = \frac{M_{11}}{f_{cu}bd^2} = 0.035$$

$$z = d(0.5 + \sqrt{(0.25 - \frac{K}{0.9})}) = 763mm$$

$$z \leq 0.95d = 770.45$$

$$A_{st} = \frac{M_{11}}{0.87f_y z} = 13629mm^2$$

$$A_b = 615.4mm^2$$

$$A_{st}/A_b = 22.14 \approx 23$$

23 bars with diameter of 28 mm with spacing of 161 mm will be used.

Design in y direction.

For the design in y direction a bar with diameter of 24 mm is assumed.

$$d_x = 900 - 75 - 24 - 24/2 = 783\ mm$$

$$f_{cu} = 40N/mm^2$$

$$f_y = 460N/mm^2$$

$$K = \frac{M_{11}}{f_{cu}bd^2} = 0.03$$

$$z = d(0.5 + \sqrt{(0.25 - \frac{K}{0.9})}) = 744mm$$

$$z \leq 0.95d = 770.45$$

$$A_{st} = \frac{M_{11}}{0.87f_y z} = 11203mm^2$$

$$A_b = 615.4mm^2$$

$$A_{st}/A_b = 18.2 \approx 19$$

19 bars with diameter of 28 mm with spacing of 198 mm will be used.

Step 5. Check the shear stress in a pile cap.

Since $L_g = B_g$, the pile cap is symmetrical V_{33}'and V_{44}' are the same.

Sections 3-3 and 4-4 were taken at d/5 for the closest pile to the column.

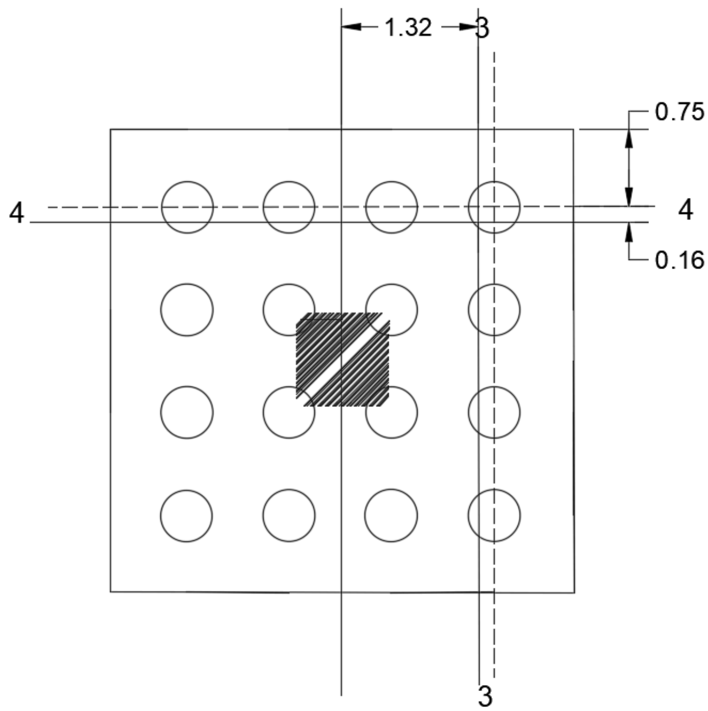


Figure 4.16. Critical sections for shear stress

$$V_{33}' = V_{44}' = \frac{4.5*339.2*1.4^2}{2} = 2142kNm$$

$$V_{33}'' = Q_3 + Q_4 \quad (4.45)$$

$$V_{44}'' = 0.5(Q_1 + Q_2 + Q_3) \quad (4.46)$$

$$V_{33} = V_{33}' + V_{33}'' \quad (4.47)$$

$$V_{44} = V_{44}' + V_{44}'' \quad (4.48)$$

Table 4.11. Shear forces on pile group

	V_{33}'=V_{44}'	V_{33}''	V_{44}''	V_{33}	V_{44}
LC1	2142	1142.86	1713.01	3284.86	3855.01
LC2(x)	2142	1033.59	1446.05	3175.59	3588.05
LC2(y)	2142	1228.37	1446.05	3370.37	3588.05
LC3(x)	2142	1004.08	1385.50	3146.08	3527.50
LC3(y)	2142	1231.32	1385.50	3373.32	3527.50

3-3 section: shear stress check

$$a_v = 4500/2 - 750/2 - 1320 = 555mm$$

$$a_v \leq 1.5d_x = 1216.5\ (enhancement\ is\ allowed)$$

$$v = \frac{V_{33}}{bd} = 0.92N/mm^2$$

$$p = \frac{100A_s}{bd} = 0.236\%$$

$$v_c = 0.45N/mm^2 \leq 0.92$$

External piles should be moved towards the center to achieve a_v= 535mm.

$$\frac{2d}{a_v} = 3.03$$

$$v_{c2} = v_c * \frac{2d}{a_v} = 1.36N/mm^2 \geq 0.92N/mm^2 \rightarrow OK$$

4-4 section: shear stress check

$$a_v = 4500/2 - 750/2 - 1320 = 555mm$$

$$a_v \leq 1.5d_y = 1174.5\ (enhancement\ is\ allowed)$$

$$v = \frac{V_{44}}{bd} = 1.09N/mm^2$$

$$p = \frac{100A_s}{bd} = 0.279\%$$

$$v_c = 0.46N/mm^2 \leq 1.09$$

External piles should be moved towards the center to achieve a_v= 535mm.

$$\frac{2d}{a_v} = 2.82$$

$$v_{c2} = v_c * \frac{2d}{a_v} = 1.29N/mm^2 \geq 1.09N/mm^2 \rightarrow OK$$

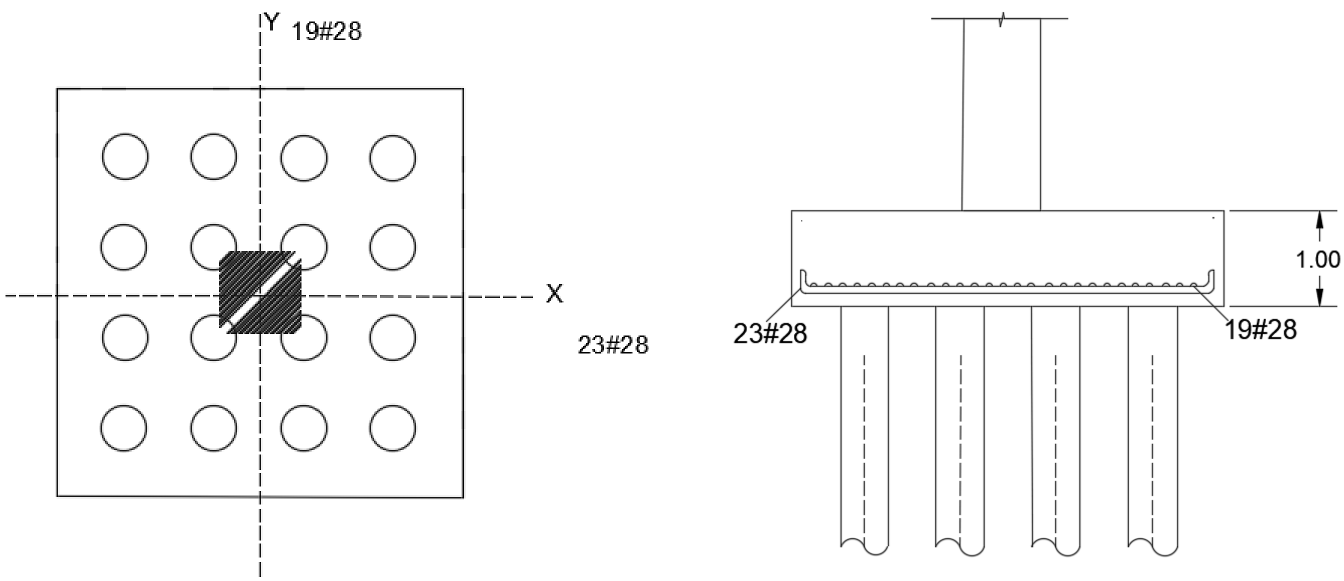


Figure 4.17. Preliminary design of a pile cap

Step 6. Check the punching shear stress in a pile cap.

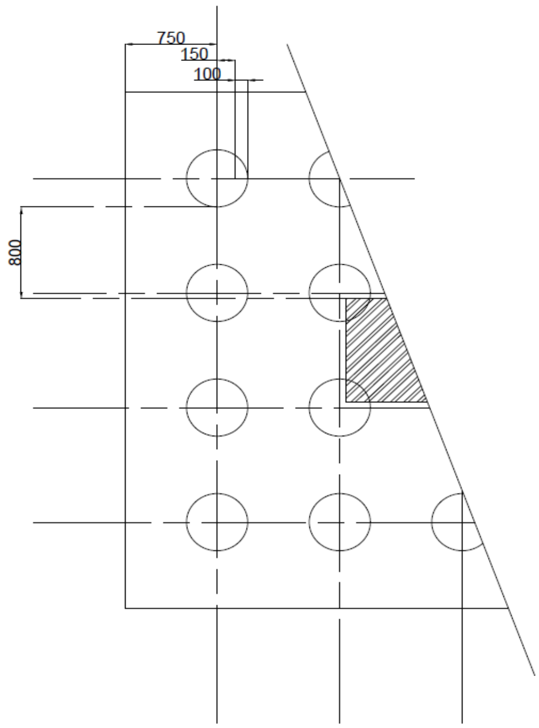


Figure 4.18. Punching shear stress in a pile cap

$U_1 = perimeter\ of\ column\ = 2(900 + 900) = 3600\ mm$

Pile spacing is designed as 1000 mm, which is less than 3 times the diameter of pile, 1500 mm, therefore the punching shear stress at critical perimeter for column is not checked.

$U_2 = perimeter\ on\ punching\ shear\ critical\ for\ pile\ load\ = 4 \times 1800 = 7200\ mm$

From calculations of steps above:

- Ultimate maximum column load, N = 8740.256 kN
- Ultimate maximum pile load, Q = 571.428 kN

Next, column punching shear stress and at perimeter of pile and pile punching shear stress are calculated as follows:

$$Column\ punching\ shear\ stress\ = \frac{N}{U_1 d} = \frac{8740.256\times10^3}{3600\times0.5\times(915+899)} = 2.677\ N/mm^2 < 5\ N/mm^2\ OK$$

$$Punching\ shear\ stress\ at\ perimeter\ of\ pile\ = \frac{571.428\times10^3}{\pi\times500\times915} = 0.398\ N/mm^2 < 5\ N/mm^2\ OK$$

$$Pile\ punching\ shear\ stress\ =\ \frac{Q}{U_2 d} = \frac{571.428\times10^3}{7200\times0.5(915+899)} = 0.088\ N/mm^2$$

Minimum ν_c for Grade 40 concrete = 0.40 N/mm^2 is okay.

Step 7. Check the area of reinforcement.

$$l_e = \beta l_0 \qquad ()$$

Where,

$l_e = unsupported\ length\ of\ a\ pile$;

$\beta\ =\ 1.2\ for\ piles\ with\ fixed\ pile\ cap.$

The unsupported length of the pile is designed as 0. Therefore, the pile is considered as a short column. Minimum cover is assumed as 75 mm.

$$e\ = \frac{M}{N} = \frac{117.52}{571.428} = 0.205$$

$$\frac{e}{R} = \frac{0.205}{0.25} = 0.823$$

$$\frac{N}{h^2} = \frac{571.428\times10^3}{500^2} = 2.286\ N/mm^2$$

$$k\ = \frac{h_s}{h} = \frac{0.426}{0.5} = 0.852$$

Minimum reinforcement ratio is determined as p = 1.0% ($f_{cu} = 40\ N/mm^2$).

$$A_{sc} = p\pi R^2 = 0.01\ \times\ \pi\ \times\ 250^2 = 1963.49\ mm^2$$

Minimum reinforcement is used (6 bars).

$$A_{st}(for\ a\ single\ bar)\ = \frac{1963.49}{6} = 327.2\ mm^2$$

As a result, bar #24 (A_{st} = 452 mm^2) is used.

Step 8. Chear shear capacity of pile.

No shear check is necessary if $M_{pu}/N_u \leq 0.6h$.

$$\frac{M_{pu}}{N_u} = \frac{117.52\times10^6}{308\times10^3} = 381.5\,mm > 0.6 \times 500 = 300\,mm$$

Hence, shear check is necessary.

$$A_c = 0.25\pi h^2 = 0.25 \times \pi \times 500^2 = 196349.5\,mm^2$$

$$\nu = \frac{H_{pu}}{0.75A_c} = \frac{47.39}{0.75\times196349.5} = 0.000322\,N/mm^2$$

$$p = \frac{100A_s}{1.5A_c} = \frac{100\times1200}{1.5\times196349.5} = 0.407$$

From Figure X, $\nu_c = 0.55$.

$$v_c' = v_c + \frac{0.6N_u \times H_{pu} \times h}{A_c \times M_{pu}} = 0.55 + \frac{0.6\times308\times0.047\times0.5}{0.196\times117.5} = 0.74\,N/mm^2 < 5\,N/mm^2$$

As $v_c' > \nu$, shear reinforcement is not required.

Step 9. Check minimum reinforcement in the pile.

$$\frac{100A_{sc}}{A_c} = \frac{100\times1963.5}{196349.5} = 1.0 < 6 \quad OK$$

Step 10. Containment of reinforcement in the pile.

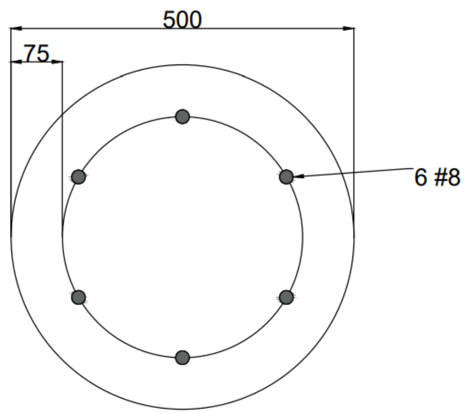
$$minimum\ diameter\ of\ links = 0.25 \times 24 = 6\,mm \geq 6\,mm \quad OK$$

$$maximum\ spacing\ of\ links = 12 \times 6 = 72\,mm \quad OK$$

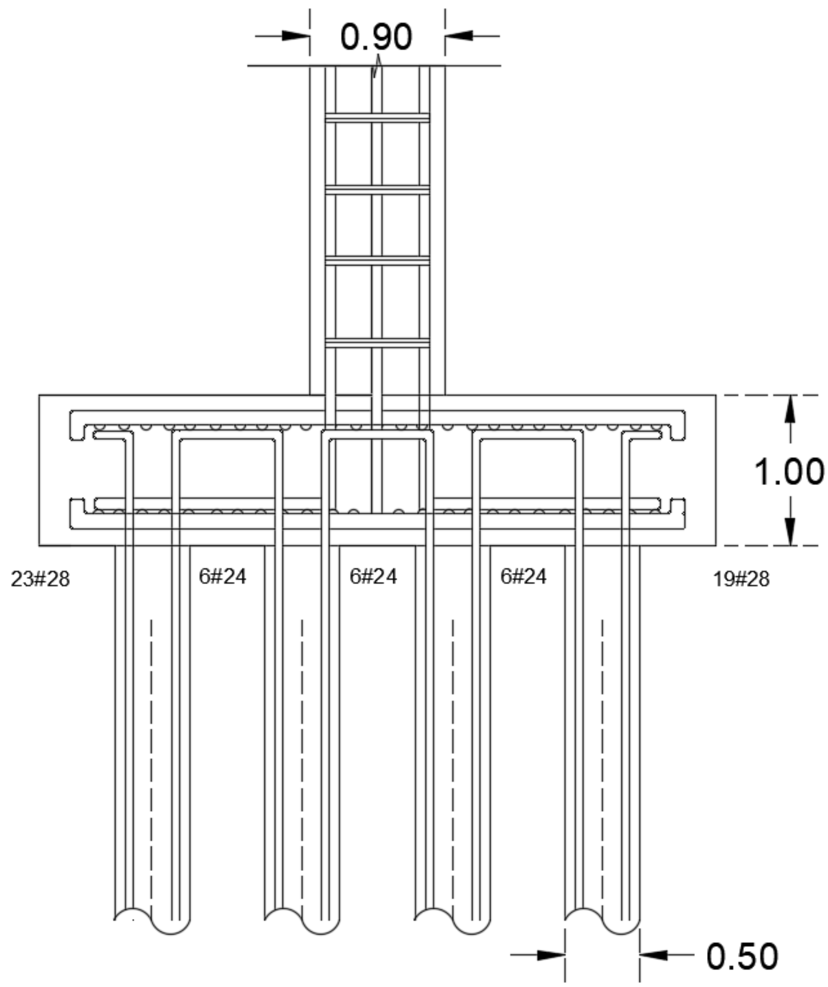
Step 11. Minimum tension reinforcement in pile cap.

$$A_{s\,(min)} = 0.0013bh = 0.0013 \times 4500 \times 1000 = 5850\,mm^2 \quad OK$$

Figure 4.19 below shows the design for a pile and pile cap.



(a)



(b)

Figure 4.19. (a) pile and (b) pile cap design

4.4.4.2. Software analysis of reinforcement,

To make a reinforcement analysis we designed a single pile in GEO 5. There is an option in software that demonstrates the reinforcement details. Additionally, it shows bending moment and shear force diagrams. Reinforcement details, such as number and diameter of bars, cover are illustrated in Figure 4.20. Also, it shows that bending, compression, shear and reinforcement ratio results are satisfactory. It can be seen that the reinforcement details in GEO5 are almost the same with hand calculations.

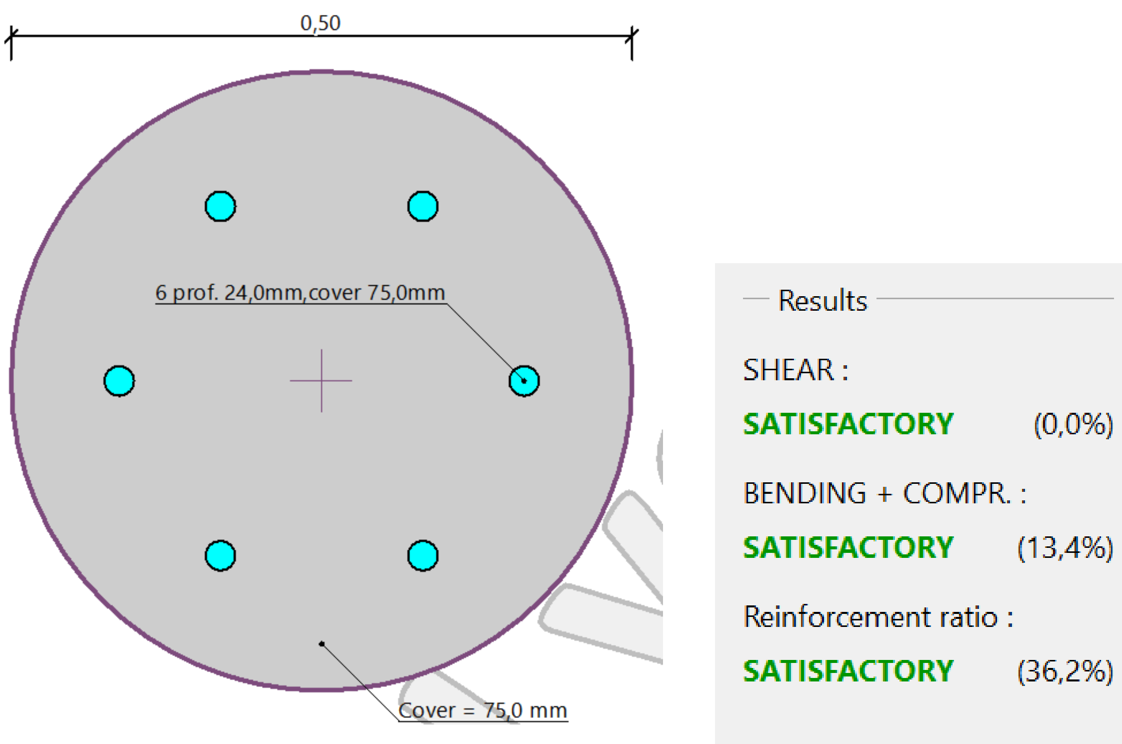


Figure 4.20. (a) pile reinforcement and (b) GEO5 results

4.5. Foundation construction procedure.

Any construction of the building requires a substructure construction procedure. For instance, during installation of piles it is necessary to control quality and if there are unsatisfactory results, steps should be redone to reach a sufficient level of reliability and safety. The flow chart with construction steps is shown in Figure 4.21.

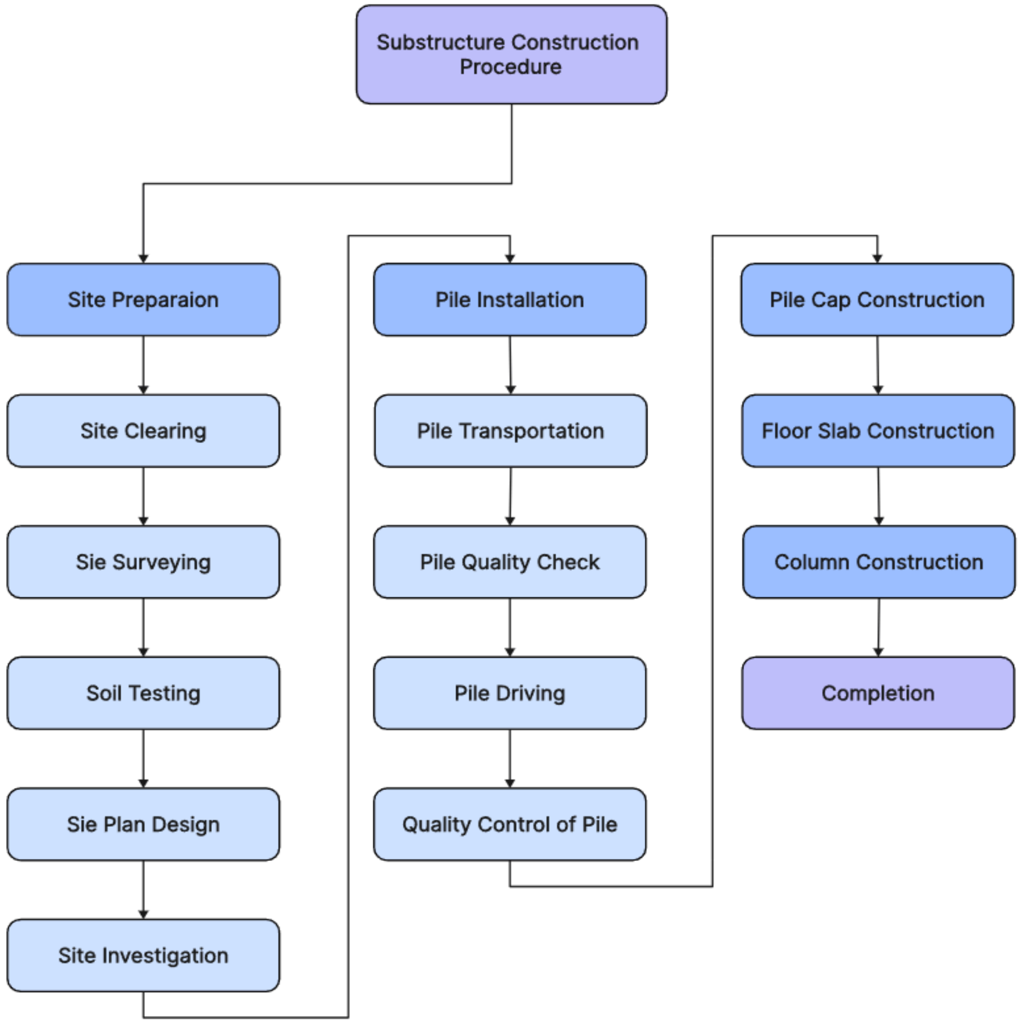


Figure 4.21. Substructure construction procedure

4.5.1. Site preparation.

First of all, preparation starts with the site clearing. Clearing and grading are needed to divide construction area into working zones, such division allows to speed up logistics and processing. Next is site surveying in order to determine dimensions, location of public utility and elevations. The construction is going to be in an urban area. Therefore, it is important to do a proper site layout to avoid constraints with drainage, natural gas or water systems. After clearing and surveying, site investigation is needed. To determine the soil profile and soil properties several boreholes should be placed, as soil layers can be distributed non uniformly. Conduction of SPT and CPT tests are needed for identification of the soil profile, while testing of samples from the boreholes allows to investigate physical, mechanical and chemical properties. Since the construction site is not seismically active, tests and surveys for seismicity are not required. Additionally, construction area should be leveled to provide uniform alignment.

4.5.2. Pile installation.

For this project it was decided to install a precast concrete driven pile. After delivery of piles it is necessary to check their quality level, in terms of their dimensions, concrete strength and prestressing forces.

The installation of piles should be done properly as soil response, bearing capacity and settlement are influenced by placement of piles, while placement can be affected by type of equipment. To choose the right one we compared diesel and hydraulic hammers in terms of their reliability and efficiency.

Although a hydraulic hammer is heavier, it provides 15 percent higher energy per blow. In addition, in case of concrete piles hydraulic hammers are more preferred for concrete piles and create less noise (Justason, n.d). Therefore, it was decided to use a hydraulic hammer instead of a diesel for this project.

After checking piles' integrity and setting the equipment the process of installation can be finished. The pile cap for a group of concrete driven piles is with a height of 1m.

4.5.3. Construction control.

Throughout the whole construction process proper supervision is required. It provides safety not only for the project but also surrounding buildings. Actual conditions at the site may not be exactly the same as the analyzed results. Therefore, obtaining data in real time and maintaining consistency with design specifications are crucial construction parts. Additionally, continuous monitoring allows analysis and evaluation of soil and structure behavior.

5. Construction Management part

5.1. Project charter

The project charter is defined as a document issued by the project sponsor that formally authorizes the existence of a project and provides the project manager with the authority to apply organizational resources to project activities (Project Management Institute, 2017). Figure 5.1 represents the project charter of this project. The main goal is to construct a new 16 story hotel in Louisiana, USA. The time frame of this project is 2 years (October 2023-October 2025). The total cost of the project is $12 million dollars (estimated by RSMeans).

Louisiana Economic Development (LED) can be a sponsor of this project, since they are responsible for providing economic growth and development in Louisiana. The general contractor is Woodward which is one of the largest construction companies in Louisiana.

Permits are taken from Jefferson Parish Permit Division which is a local authority in Southeast Louisiana. The project steering committee includes Chief Executive Officer, Supply Chain Manager and Chief Finance Officer.

Project Charter			
Project Name	Design of a 16 story hotel building in the Southeast Louisiana, USA		
Project Scope	The project aims to construct a new 16 story hotel with 111 guest rooms		
Project Manager	Project Sponsor	Project Approval Date	Last Revision Date
Bauyrzhan Toleubekov	Daulet Nagimetov	05.10.2023	06.10.2023
Project Description	Woodward Design+Build intends to design and build the following		
	1. A 16 story hotel building with 111 guest rooms		
	2. Parking area for 200 cars		
	3. Restaurant with panoramic view		
Time frame	2 Years		
Budget	$12,000,000		
	Name	Title	Organization
Stakeholders	Ward Richard	Project Sponsor	Louisiana Economic Development (LED)
	Tom Abernathy	Contactor	Woodward Design+Build
	Danny Ferrara	Local authority	Jefferson Parish Permit Division
Project Steering Committee	CEO		
	Supply Chain Manager		
	Chief Finance Officer		
Project Sponsor:______________		Project Manager:______________	
Date:______________		Date:______________	

Figure 5.1. Project charter

5.2. Feasibility study

Feasibility study is a part of the project initiation phase which determines the potential success of the project. 3 case scenarios were considered for the feasibility study of this project (Figure 5.2): worst, optimal and best.

In the worst case scenario occupation of economy class rooms will be 50%, comfort 20% which will generate about 2 million dollars annually. The payback period in the worst case is 6 years. In the optional case scenario occupation of economy class rooms will be 80%, comfort 70% which will generate about 4 million dollars annually. The payback period in the optimal case is 3.5 years. In the best case scenario, rooms will be booked for 95% (economy) and 85% (comfort). The payback period will be 3 years.

Worst	**Optimal**	**Best**
Economy 50%	Economy 80%	Economy 95%
Comfort 20%	Comfort 70%	Comfort 85%
5550$/day	10,305$/day	12,308$/day
166,500$/month	309,150$/month	369,225$/month
1,998,000$/year	3,709,800$/year	4,4307,00$/year
6 years	**3.5 years**	**3 years**

Figure 5.2. Feasibility study

5.3. Cost/benefit analysis

Cost estimation helps in developing an accurate budget for the project. A well-defined budget ensures that the project has sufficient funds allocated to complete all planned activities. Figure 5.3 shows cost estimation made by using RSMeans software. The cost of a 16 story hotel in Louisiana is $10,925,023. Cost per S.F is $1409.

$10,925,023.31	New Orleans, LA	16	No
Building cost	Location	Stories	Basement
$1,409.68	7,750	11.77	$2,279,646.06
Cost per S.F.	Floor Area	Story Height	Additive Cost

Figure 5.3. RSMeans Cost Estimation

Figure 5.3 shows more details about cost estimation of substructure which includes foundation of the hotel. It needs about $200,000 which is 2.35% of the total cost.

			Quantity	% of Total	Cost per S.F.	Cost
A		Substructure		2.35%	$24.56	$190,314.35
A1010		Standard Foundations			$4.64	$35,943.21
		Foundation wall, CIP, 4' wall height, direct chute, .148 CY/LF, 7.2 PLF, 12" thick	419.1		$3.88	$30,049.47
		Pile caps, 12 piles, 11' - 6" x 8' - 6" x 49", 40 ton capacity, 19" column size, 900 K column	0.55		$0.26	$2,006.18
		Pile caps, 14 piles, 11' - 6" x 10' - 9" x 55", 80 ton capacity, 29"column size, 2155 K column	0.77		$0.50	$3,887.56
A1020		Special Foundations			$19.62	$152,016.98
		Steel H piles, 100' long, 800K load, end bearing, 12 pile cluster	1.32		$11.55	$89,474.31
		Grade beam, 30' span, 52" deep, 14" wide, 12 KLF load	381		$8.07	$62,542.67
A1030		Slab on Grade			$0.30	$2,290.69
		Slab on grade, 4" thick, non industrial, reinforced	484.37		$0.30	$2,290.69
A2010		Basement Excavation			$0.01	$63.47
		Excavate and fill, 30,000 SF, 4' deep, sand, gravel, or common earth, on site storage	484.37		$0.01	$63.47

Figure 5.4. Substracture

Figure 5.5 provides cost estimation details regarding the shell. It includes costs of floor construction, concrete columns, beams, exterior walls, doors and roof. This section needs the largest amount of resources. The construction of the shell has cost estimation of $4,397,937 which is 54,41% of the total cost.

		Quantity	% of Total	Cost per S.F.	Cost
B	**Shell**		**54.41%**	**$567.48**	**$4,397,937.01**
B1010	Floor Construction			$284.31	$2,203,391.03
	Cast-in-place concrete column, 12", square, tied, minimum reinforcing, 150K load, 10'-14' story height, 135 lbs/LF, 4000PSI	499.11		$3.18	$24,656.03
	Cast-in-place concrete column, 16", square, tied, minimum reinforcing, 300K load, 10'-14' story height, 240 lbs/LF, 4000PSI	499.11		$4.43	$34,349.50
	Cast-in-place concrete column, 20", square, tied, minimum reinforcing, 500K load, 10'-14' story height, 375 lbs/LF, 4000PSI	499.11		$6.38	$49,415.38
	Concrete I beam, precast, 18" x 36", 790 PLF, 25' span, 6.44 KLF superimposed load	6431.28		$255.71	$1,981,774.82
	Precast concrete double T beam, 2" topping, 24" deep x 8' wide, 50' span, 30 PSF superimposed load, 120 PSF total load	7265.62		$13.66	$105,868.37
	Precast concrete double T beam, 2" topping, 24" deep x 8' wide, 50' span, 75 PSF superimposed load, 165 PSF total load	484.37		$0.95	$7,326.93
B2010	Exterior Walls			$225.97	$1,751,270.58
	Brick wall, composite double wythe, standard face/CMU back-up, 8" thick, perlite core fill, 3" XPS	58521.6		$225.97	$1,751,270.58
B2020	Exterior Windows			$53.82	$417,096.80
	Windows, aluminum, awning, insulated glass, 4'-5" x 5'-3"	636.10		$53.82	$417,096.80
B2030	Exterior Doors			$0.28	$2,159.68
	Door, aluminum & glass, without transom, narrow stile, with panic hardware, 3'-0"x 7'-0"opening	0.17		$0.07	$568.82

Figure 5.5. Shell

Figures 5.6-5.7 provides details about the interiors. It includes finishing works of walls, floors and doors. The cost estimation for interiors is $357.001.

B3010	Roof Coverings			$2.27	$17,570.70
	Roofing, single ply membrane, EPDM, 60 mils, loosely laid, stone ballast	484.37		$0.09	$718.53
	Insulation, rigid, roof deck, extruded polystyrene, 40 PSI compressive strength, 4" thick, R20	484.37		$0.25	$1,908.67
	Roof edges, aluminum, duranodic, .050" thick, 6" face	381		$1.20	$9,262.43
	Flashing, aluminum, no backing sides, .019"	381		$0.21	$1,656.34
	Gravel stop, aluminum, extruded, 4", mill finish, .050" thick	381		$0.52	$4,024.73
B3020	Roof Openings			$0.83	$6,448.22
	Roof hatch, with curb, 1" fiberglass insulation, 2'-6" x 3'-0", galvanized steel, 165 lbs	6		$0.83	$6,448.22
C	**Interiors**		**4.42%**	**$46.06**	**$357,001.37**
C1010	Partitions			$14.16	$109,741.91
	Concrete block (CMU) partition, light weight, hollow, 6" thick, no finish	688.88		$0.72	$5,607.21
	Metal partition, 5/8"fire rated gypsum board face, no base,3 -5/8" @ 24" OC framing, same opposite face, sound attenuation insulation	6200		$3.68	$28,521.86
	Gypsum board, 1 face only, exterior sheathing, fire resistant, 5/8"	58521.6		$6.23	$48,303.73
	Add for the following: taping and finishing	58521.6		$3.52	$27,309.11
C1020	Interior Doors			$12.96	$100,460.67
	Door, single leaf, kd steel frame, hollow metal, commercial quality, flush, 3'-0" x 7'-0" x 1-3/8"	86.11		$12.96	$100,460.67

Figure 5.6. Interiors

			Quantity	% of Total	Cost per S.F.	Cost
C2010		Stair Construction			$2.12	$16,458.73
		Stairs, steel, pan tread for conc in-fill, picket rail,16 risers w/ landing	1.06		$2.12	$16,458.73
C3010		Wall Finishes			$8.35	$64,699.18
		Painting, interior on plaster and drywall, walls & ceilings, roller work, primer & 2 coats	11022.22		$0.96	$7,433.39
		Painting, interior on plaster and drywall, walls & ceilings, roller work, primer & 2 coats	58521.6		$5.09	$39,466.97
		Ceramic tile, thin set, 4-1/4" x 4-1/4"	2755.55		$2.30	$17,798.82
C3020		Floor Finishes			$5.36	$41,573.25
		Carpet tile, nylon, fusion bonded, 18" x 18" or 24" x 24", 35 oz	6200		$4.05	$31,414.78
		Vinyl, composition tile, maximum	775		$0.27	$2,125.40
		Tile, ceramic natural clay	775		$1.04	$8,033.07
C3030		Ceiling Finishes			$3.11	$24,067.63
		Gypsum board ceilings, 5/8" fire rated gypsum board, painted and textured finish,1-5/8" metal stud furring, 24" OC support	7750		$3.11	$24,067.63
D		**Services**		**10.62%**	**$110.80**	**$858,729.26**
D1010		Elevators and Lifts			$6.61	$51,241.45
		Traction geared freight, 4000 lb., 15 floors, 10' story height, 200FPM	0.01		$1.24	$9,632.39
		Traction, geared passenger, 3500 lb,15 floors, 10' story height, 2 car group, 350 FPM	0.08		$5.37	$41,609.06

Figure 5.7. Interiors

Figures 5.8-5.9 presents details about services which include elevators, energy supply, water supply, cooling system, draining system, etc. It will cost about $858,729 which is 10,62% of the total cost.

D2010		Plumbing Fixtures			$17.01	$131,834.55
		Water closet, vitreous china, bowl only with flush valve, wall hung	19.37		$7.70	$59,645.94
		Urinal, vitreous china, wall hung	0.43		$0.06	$499.53
		Lavatory w/trim, vanity top, PE on CI, 20" x 18"	19.37		$2.76	$21,397.65
		Kitchen sink w/trim, countertop, stainless steel, 33" x 22" double bowl	0.13		$0.04	$275.97
		Service sink w/trim, PE on CI,wall hung w/rim guard, 22" x 18"	0.51		$0.22	$1,709.90
		Bathtub, recessed, PE on CI, mat bottom, 5' long	19.37		$6.03	$46,751.88
		Shower, stall, baked enamel, terrazzo receptor, 36" square	0.43		$0.14	$1,099.45
		Water cooler, electric, wall hung, wheelchair type, 7.5 GPH	0.25		$0.06	$454.23
D2020		Domestic Water Distribution			$0.20	$1,549.84
		Gas fired water heater, commercial, 100< F rise, 500 MBH input, 480 GPH	0.05		$0.20	$1,549.84
D2040		Rain Water Drainage			$0.21	$1,613.45
		Roof drain, CI, soil,single hub, 5" diam, 10' high	0.13		$0.05	$356.83
		Roof drain, CI, soil,single hub, 5" diam, for each additional foot add	19.28		$0.16	$1,256.62
D3010		Energy Supply			$2.27	$17,554.08
		Commercial building heating system, fin tube radiation, forced hot water, 1mil SF, 10 mil CF, total 5 floors	8525		$2.27	$17,554.08
D3030		Cooling Generating Systems			$11.38	$88,184.93

Figure 5.8. Services

		Packaged chiller, water cooled, with fan coil unit, medical centers, 60,000 SF, 140.00 ton	7750		$11.38	$88,184.93
D4010		Sprinklers			$3.16	$24,510.35
		Wet pipe sprinkler systems, steel, light hazard, 1 floor, 50,000 SF	5425		$1.58	$12,209.72
		Wet pipe sprinkler systems, steel, light hazard, each additional floor, 50,000 SF	7207.5		$1.54	$11,923.37
		Standard High Rise Accessory Package 16 story	0.01		$0.05	$377.26
D4020		Standpipes			$7.79	$60,380.34
		Wet standpipe risers, class III, steel, black, sch 40, 6" diam pipe, 1 floor	0.05		$0.09	$709.31
		Wet standpipe risers, class III, steel, black, sch 40, 6" diam pipe, additional floors	7.23		$3.47	$26,911.98
		Fire pump, electric, with controller, 5" pump, 100 HP, 1000 GPM	1		$3.77	$29,189.90
		Fire pump, electric, for jockey pump system, add	1		$0.46	$3,569.15
D5010		Electrical Service/Distribution			$47.95	$371,647.74
		Underground service installation, includes excavation, backfill, and compaction, 100' length, 4' depth, 3 phase, 4 wire, 277/480 volts, 2000 A	2		$11.58	$89,751.60
		Feeder installation 600 V, including RGS conduit and XHHW wire, 60 A	100		$0.21	$1,601.04
		Feeder installation 600 V, including RGS conduit and XHHW wire, 200 A	100		$0.51	$3,976.50
		Feeder installation 600 V, including RGS conduit and XHHW wire, 2000 A	400		$21.25	$164,688.00
		Switchgear installation, incl switchboard, panels & circuit breaker, 277/480 V, 2000 A	2		$14.40	$111,630.60
D5020		Lighting and Branch Wiring			$10.27	$79,575.77

Figure 5.9. Services

Security in our hotel is a priority, therefore to provide a safe environment for clients, fire detectors, alarm systems, and cameras are installed. This costs about $30.000.

		Receptacles incl plate, box, conduit, wire, 10 per 1000 SF, 1.2 W per SF, with transformer	7905		$3.00	$23,284.34
		Wall switches, 5.0 per 1000 SF	7750		$0.98	$7,578.57
		Miscellaneous power, to .5 watts	7750		$0.11	$884.43
		Central air conditioning power, 4 watts	9455		$0.60	$4,621.51
		Motor installation, three phase, 460 V, 15 HP motor size	10		$2.74	$21,247.05
		Motor feeder systems, three phase, feed to 200 V 5 HP, 230 V 7.5 HP, 460 V 15 HP, 575 V 20 HP	500		$0.56	$4,331.57
		Motor connections, three phase, 200/230/460/575 V, up to 5 HP	1		$0.01	$94.95
		Motor connections, three phase, 200/230/460/575 V, up to 100 HP	1		$0.06	$439.02
		Fluorescent fixtures recess mounted in ceiling, 0.8 watt per SF, 20 FC, 5 fixtures @32 watt per 1000 SF	7750		$2.21	$17,094.33
D5030		Communications and Security			$3.63	$28,158.09
		Communication and alarm systems, fire detection, addressable, 100 detectors, includes outlets, boxes, conduit and wire	0.19		$1.65	$12,798.05
		Fire alarm command center, addressable with voice, excl. wire & conduit	0.05		$0.08	$603.94
		Communication and alarm systems, includes outlets, boxes, conduit and wire, intercom systems, 100 stations	0.06		$0.86	$6,661.49
		Communication and alarm systems, includes outlets, boxes, conduit and wire, master TV antenna systems,100 outlets	0.04		$0.67	$5,212.10

Figure 5.10. Communication and security

Comfort is our next priority, so we need to ensure that furnishing and equipment in our rooms will satisfy clients' requirements. Fast elevators, restaurant furniture, beds, TV systems and other equipment will cost $2,279,646.06 which is 28,2% of the total cost.

		Internet wiring, 2 data/voice outlets per 1000 S.F.	5.81		$0.37	$2,882.51
D5090		Other Electrical Systems			$0.32	$2,478.67
		Generator sets, w/battery, charger, muffler and transfer switch, diesel engine with fuel tank, 500 kW	11.65		$0.32	$2,478.67
E		**Equipment & Furnishings**		**28.20%**	**$294.15**	**$2,279,646.06**
E1090		Other Equipment			$294.15	$2,279,646.06
E1090D10101	⊕	2.00-Traction gearless elevators, passenger, 3000 lb, 10 floors, 200 FPM	2.00		$105.14	$814,814.00
E1090D10101	⊕	2.00-Traction gearless elevators, passenger, 5000 lb, 10 floors, 200 FPM	2.00		$107.37	$832,130.00
E1090125416:	⊕	2.00-Restaurant furniture, bar, built-in, front bar	2.00		$0.11	$871.20
E1090112173:	⊕	2.00-Folders, blankets and sheets, king-size with automatic stacker	2.00		$19.24	$149,072.00
E1090112173:	⊕	2.00-Laundry equipment, combination washer/extractor, 50 lb. capacity	2.00		$3.90	$30,201.60
E1090132416!	⊕	2.00-Saunas, prefabricated, cedar on cedar, 10' x 12' x 7' h, incl. heater & controls	2.00		$3.87	$30,008.00
E1090125416:	⊕	2.00-Restaurant furniture, bar, built-in, back bar	2.00		$0.09	$696.96
E1090112173:	⊕	2.00-Ironers, institutional, single roll, 110"	2.00		$10.22	$79,182.40
E1090112173:	⊕	2.00-Laundry equipment, combination washer/extractor, 125 lb. capacity	2.00		$9.74	$75,504.00
E1090274133	⊕	130.00-TV systems, VHF reception & distribution, 30 outlets	130.00		$4.71	$36,539.10
E1090284611:	⊕	300.00-Detection system, heat detector, smoke detector, ceiling type, excl. wires & conduit	300.00		$9.14	$70,857.00

Figure 5.11. Equipment and Furnishings

Finally, the total cost of the building is $8,083,628 but there are also contractor's fee, architectural fees which makes the total cost of the project about $11,000,000.

E1090284611:	⊕	300.00-Detection system, heat detector, smoke detector, ceiling type, excl. wires & conduit	300.00		$9.14	$70,857.00
E1090282313	⊕	100.00-Closed circuit television system (CCTV), surveillance, one station (camera & monitor)	100.00		$18.42	$142,780.00
E1090275119	⊕	10.00-Sound system, amplifier, 250 W, excl rough-in wires, cables & conduits	10.00		$2.19	$16,989.80
F		**Special Construction**		**0%**		
G		**Building Sitework**		**0%**		
		SubTotal		**100%**	**$1,043.05**	**$8,083,628.05**
		Contractor Fees (GC,Overhead,Profit)		**25.0%**	**$260.76**	**$2,020,907.01**
		Architectural Fees		**6.0%**	**$78.23**	**$606,272.10**
		User Fees		**2.0%**	**$27.64**	**$214,216.14**
		Total Building Cost			**$1,409.68**	**$10,925,023.31**

Figure 5.12. Total cost

5.4. WBS

Work Breakdown Structure is a fundamental project management tool which is decomposing project scope into smaller and manageable elements. Figure 5.13 shows WBS of this project which was prepared based on the PMBOK guide. It has main components like hierarchy and work packages.

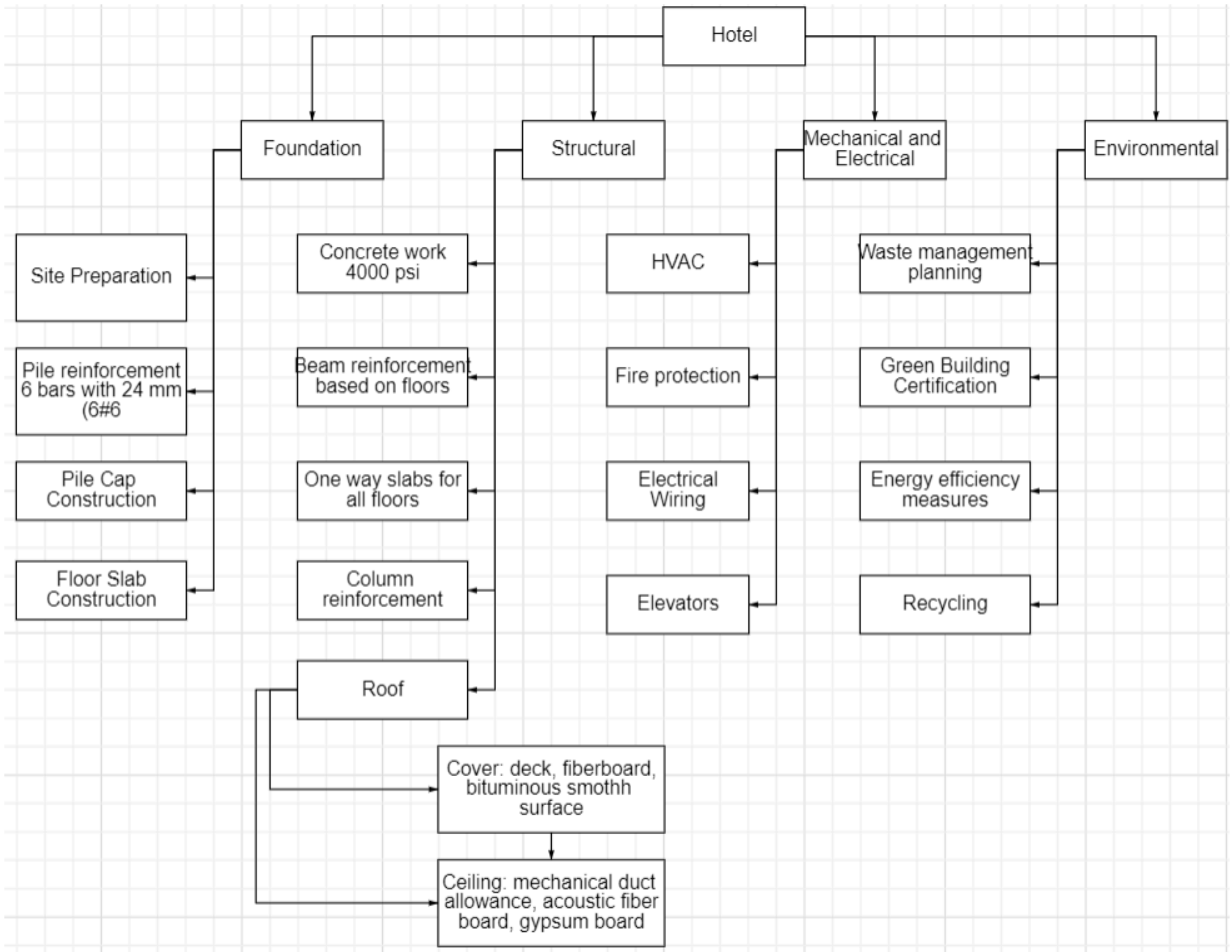


Figure 5.13. Work Breakdown Structure

WBS dictionary provides detailed information about each component or work package included in the WBS. **Table X** shows a WBS dictionary for this project which includes WBS level, code, name and description.

Table 5.1. WBS dictionary

WBS Level	WBS Code	WBS Name	WBS Description
1	1	Hotel	The main element of the WBS
2	1.1	Foundation	Includes all foundation elements
2	1.2	Structural	Includes all structural members of the project
2	1.3	Mechanical and Electrical	Includes all mechanical and electrical components of the project
2	1.4	Environmental	Environmental side of the project
3	1.1.1	Site preparation	Initial activities required to prepare the construction site
3	1.1.2	Pile reinforcement	Installation of steel or concrete piles to provide structural support for foundation
3	1.1.3	Pile cap	Construction of a reinforced concrete cap to distribute loads from piles
3	1.1.4	Floor slab	Installation of reinforced concrete slabs to form the building's floors
3	1.2.1	Concrete work	General concrete construction activities
3	1.2.2	Beam reinforcement	Installation of steel reinforcement bars within concrete beams
3	1.2.3	One way slabs	Construction of reinforced concrete slabs
3	1.2.4	Column reinforcement	Installation of steel reinforcement bars within concrete columns
3	1.2.5	Roof	Construction of the building's roof structure
3	1.3.1	HVAC	Heating, ventilation, and air conditioning systems
3	1.3.2	Fire protection	Implementation of fire detection

3	1.3.3	Electrical wiring	Installation of electrical wiring
3	1.3.4	Elevators	Installation of elevators
3	1.4.1	Waste management	Implementation of strategies and systems for proper collection
3	1.4.2	Green building certification	Certification from programs such as LEED
3	1.4.3	Energy efficiency measures	Implementation of technologies and practices to reduce energy consumption
3	1.4.4	Recycling	Implementation of recycling programs to minimize the environmental impact
4	1.2.5.1	Cover	Installation of covering materials
4	1.2.5.2	Ceiling	Installation of ceiling coverings or finishes

5.5. Scheduling

Project scheduling helps in effectively managing project timelines by defining when specific tasks and activities need to be completed. This ensures that the project stays on track and is completed within the allotted time frame. Figure 5.14 shows the time schedule of our project which is made in Primavera software. All activities have ID, start date, end date, duration and late start date, finish date. The project started on 5th of October 2023 and will finish on 8th of October 2025.

Activity ID	Activity Name	Original Duration	Early Start	Early Finish	Late Start	Late Finish
A1000	Project initiation	0	05-Oct-23	05-Oct-23	12-Aug-21	12-Aug-21
A1010	Create project team and assign responsib	10	05-Oct-23	20-Oct-23	15-Sep-23	28-Sep-23
A1020	Conduct initial stakeholder meeting	5	23-Oct-23	30-Oct-23	29-Sep-23	05-Oct-23
A1030	Finilize project design	10	05-Oct-23	20-Oct-23	13-Aug-21	26-Aug-21
A1040	Obtain permits and approvals	10	23-Oct-23	06-Nov-23	27-Aug-21	09-Sep-21
A1050	Bidding process	10	07-Nov-23	22-Nov-23	10-Sep-21	23-Sep-21
A1060	Initiate construction site	5	23-Nov-23	29-Nov-23	24-Sep-21	30-Sep-21
A1070	Site cleatring and excavation	15	30-Nov-23	25-Dec-23	01-Oct-21	21-Oct-21
A1080	Soil compaction	15	26-Dec-23	15-Jan-24	22-Oct-21	11-Nov-21
A1090	Retaining walls	15	16-Jan-24	05-Feb-24	12-Nov-21	02-Dec-21
A1100	Concrete work	15	06-Feb-24	26-Feb-24	03-Dec-21	23-Dec-21
A1110	Core walls and columns	15	27-Feb-24	18-Mar-24	24-Dec-21	13-Jan-22
A1120	Steel framing	30	19-Mar-24	29-Apr-24	14-Jan-22	24-Feb-22
A1130	Slab and floor construction	30	30-Apr-24	10-Jun-24	25-Feb-22	07-Apr-22
A1140	Roof structure	15	11-Jun-24	01-Jul-24	08-Apr-22	28-Apr-22
A1150	Exterior walls	30	02-Jul-24	12-Aug-24	29-Apr-22	09-Jun-22
A1160	Electrical wiring	60	13-Aug-24	04-Nov-24	10-Jun-22	06-Sep-22
A1170	HVAC	90	05-Nov-24	10-Mar-25	07-Sep-22	30-Jan-23
A1180	Fire protection systems	15	11-Mar-25	31-Mar-25	31-Jan-23	22-Feb-23
A1190	Power systems	20	01-Apr-25	28-Apr-25	23-Feb-23	30-Mar-23
A1200	Elevators	20	29-Apr-25	26-May-25	06-Sep-23	05-Oct-23
A1210	Windows and exterior doors	20	29-Apr-25	26-May-25	31-Mar-23	02-May-23
A1220	Facade finishes	30	27-May-25	07-Jul-25	22-Aug-23	05-Oct-23
A1230	Interior works	60	27-May-25	18-Aug-25	03-May-23	08-Aug-23
A1240	Final inspections	30	19-Aug-25	29-Sep-25	09-Aug-23	26-Sep-23
A1250	Hand over the project to owner	7	30-Sep-25	08-Oct-25	27-Sep-23	05-Oct-23

Figure 5.14. Schedule

Figure 5.15 shows a Gantt chart which is helpful to visualize the project schedule.

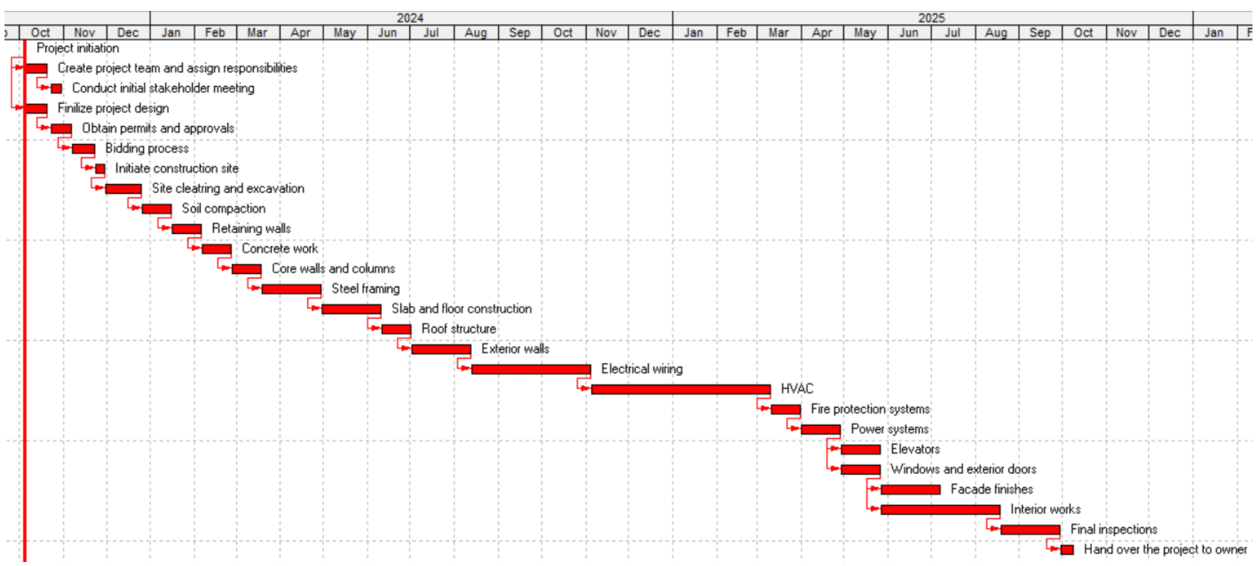


Figure 5.15. Gantt chart

5.6. Risk management

Risk management planning is deciding how to approach, plan and execute the risk management activities for a project (PMBOK, 3rd edition). Proper planning is important to avoid unexpected accidents. Risk assessment includes risk matrix, risk rating and risk mitigation measures. Figure 5.16 shows a risk matrix which assesses every risk based on probability rating and severity (impact rating). Probability rating has a score range from 1-very low to 5-very high. Impact rating has scores from 1-very low to 7-high. Risk level is the product of probability rating and impact rating.

Risk matrix					
Probability Rating	5-Very High	5	10	20	35
	4-High	4	8	16	28
	3-Moderate	3	6	12	21
	2-Low	2	4	8	14
	1-Very low	1	2	4	7
		Very low-1	Low-2	Moderate-4	High-7
		Impact Rating			

Figure 5.16. Risk matrix

Figure 5.17 provides details regarding each assigned risk rating. For example, risk has a rating of "very high" risk if there is more than 20% cost increase of the project or more than 6 month delay of the project.

Rating--------------->	Very low	Low	Moderate	High	Very High
Cost Impact of Threat	Insignificant cost increase	<5% cost increase	5-10% cost increase	10-20% cost increase	>20% cost increase
Cost Impact of Opportunity	Insignificant cost reduction	<1% cost decrease	1-3% cost decrease	3-5% cost decrease	>5% cost decrease
Schedule Impact of Threat	Insignificant slippage	<1 month slippage	1-3 months slippage	3-6 months slippage	>6 months slippage
Schedule Impact of Opportunity	Insignificant improvement	<1 month improvement	1-2 months improvement	2-3 months improvement	>3 month improvement
Probability	1-9%	10-19%	20-39%	40-59%	60-99%

Figure 5.17. Risk rating

Table 5.2 provides a template which is used to respond to risks. It includes ID, risk source, probability, severity, risk level and mitigation measures.

Table 5.2. Risk mitigation measures

ID	Risk Source	Probability	Severity	Risk level	Mitigation measures
Budget risks					
B1	Budget Overrun	3	4	12	-Monitor project expenses -Develop a detailed budget plan with clear cost estimates
B2	Fluctuating Material Costs	4	2	8	-Monitor market trends and forecast material costs -Order material in advance and fix the lowest cost
B3	Unexpected project expenses	2	7	14	-Monitor project expenses
Safety risks					
S1	Fall Hazards	3	7	21	-Use fall protection systems such as guardrails, safety nets
S2	Electrical Hazards	3	7	21	-Conduct regular inspections and maintenance of electrical equipment and wiring
S3	Trenches	3	7	14	-Use shoring system
Environmental risks					
E1	Soil Contamination	2	4	8	-Conduct soil testing and analysis prior to excavation or ground
E2	Air pollution	2	4	8	-Implement dust control measures such as water spraying, dust barriers
E3	Water pollution	2	4	8	-Implement erosion control measures to prevent sediment runoff into water
Time risks					

T1	Weather Delays	2	4	8	-Plan for indoor work or alternative activities during inappropriate weather
T2	Permitting and Regulatory Approvals	1	4	4	-Start the permitting process early and engage with regulatory authorities proactively
T3	Labor Shortages	2	7	14	-Invest in training and development programs to upskill workers

Then, risk with their IDs can be filled into the risk matrix. Figure 5.18 shows an updated risk matrix with risk IDs.

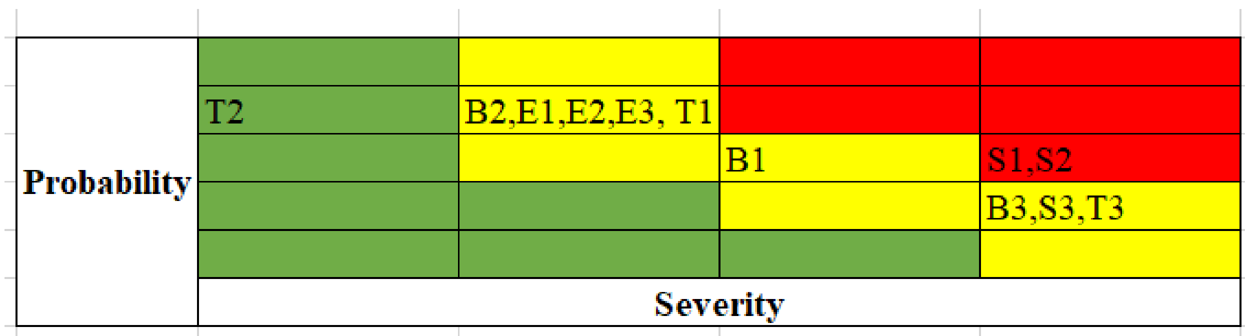


Figure 5.18. Risk matrix with IDs

5.7. Quality management

Quality management is an important part of project management, since it ensures the high quality of all processes in the project. Quality is assessed by checklists, metrics and quality control measures are needed to achieve desired results.

Figure 5.19 shows the quality management process in the project. Quality should satisfy governmental and organizations' standards, rules and regulations. After quality planning, quality assurance should be done based on the quality metrics, checklists and management plan. Any changes in the quality management should be done through integrated change control.

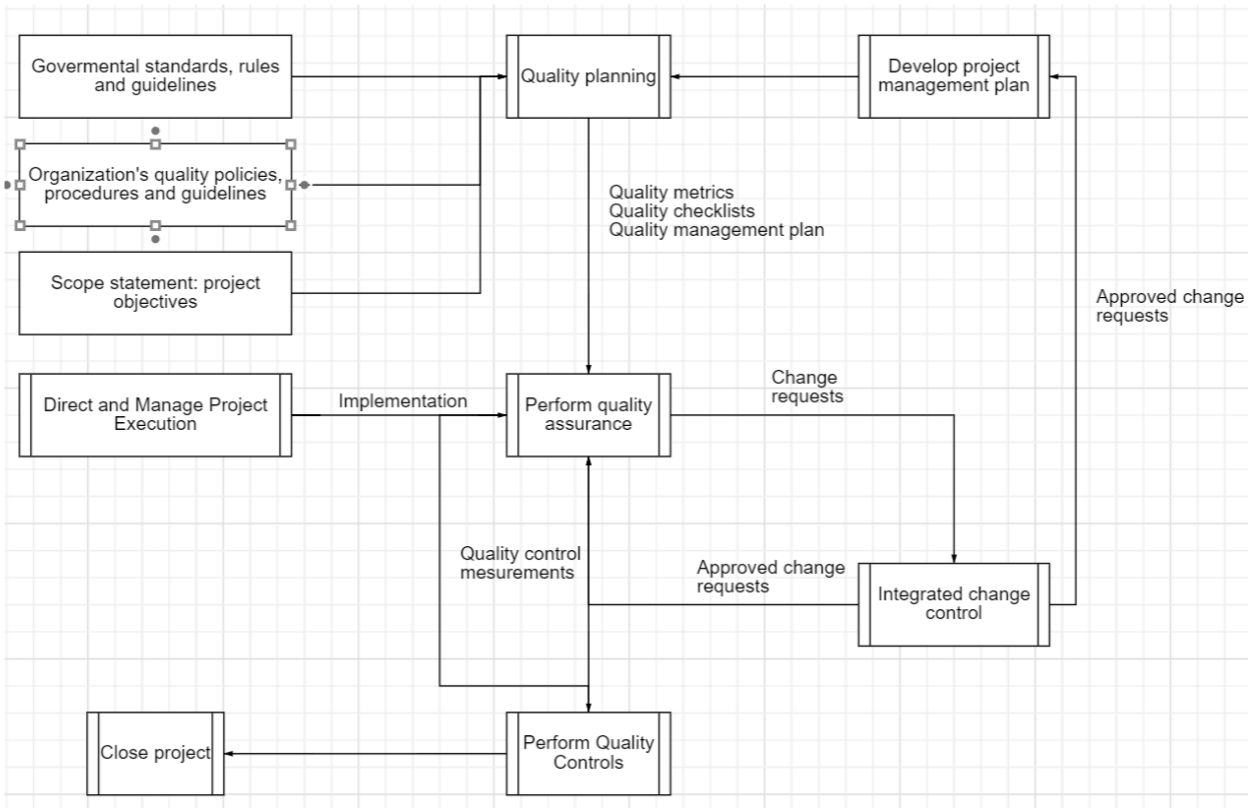


Figure 5.19. Quality management process

Quality checklists are lists of criteria, requirements used to systematically evaluate project deliverables, processes, or activities against established quality standards. Figure 5.20 shows a quality checklist template with an example of concrete. This template has document number, project name, date, requirements, checklists and boxes for signature. To verify the quality of the concrete, the inspector should verify the type, grade, dosage and other requirements for the concrete.

Document number:	
Project name:	
Date:	

Quality control checklist: Concrete					
#	Requirements	Yes	No	N/A	Comments
1	Verify the type and grade of cement used				
2	Inspect aggregates for cleanliness, gradation, and moisture content				
3	Verify the type, dosage, and compatibility of admixtures.				
4	Verify that the concrete mix proportions meet design specifications				
5	Verify the accuracy of materials and proportions during batching				
6	Ensure timely delivery of concrete to the construction site				
7	Ensure proper consolidation of concrete using vibrators or other methods				
8	Implement curing methods such as water curing, curing compounds, or curing blankets				
9	Conduct compression tests on concrete cylinders or cores				
10	Perform non-destructive tests such as ultrasonic testing or rebound hammer tests				
11	Maintain records of concrete mix designs, test results				

Checked by:	
Approved by:	

Figure 5.20. Quality checklist

Quality metric is an operational definition that describes, in very specific terms, what something is and how the quality process measures it. Figure 5.21 shows an example of a quality metric for piles. Quality metrics template should have ID, category, item. Piles have concrete and reinforcing steel as the items. Then these items have descriptions like “C35 cast in place concrete”. One of the most important aspects is the method of measurement and metrics. For example, a compressive test for concrete should be done in 3,7,14,28 days and it should reach 3000 psi strength.

Quality metrics		
	Template ID:	
	Date:	

ID	Category	Item	Description	Method of measurement	Metrics	Reference	Statistical Sampling
FP1	Bored piles	Concrete	C35 cast in place concrete	Compressive Test	3,7,14,28 days: 3000 psi	Structural Specification	3 sets of cylinder: 10 piles/day
				Slump	100 mm (max)	Structural Specification	For each batch of concrete
		Reinforcing Steel	Reinforcement #10 bars	Tensile Test	60 ksi	Structural Specification	Applied load for sample: 2,500 kg

Figure 5.21. Quality metric template

Quality measures are activities which ensure the quality of the project. Measures like inspection, testing, quality audits, benchmarking and cost estimation are used to enhance the quality. Their descriptions and frequencies are presented in the Table 5.3 below. Also, there are responsible people on each quality control measure.

Table 5.3. Quality measures

#	Quality Control Measures	Description	Frequency	Performed by
1	Inspection	Inspection involves examining project deliverables, processes, or work items to ensure they meet specified requirements.	Everyday	On site supervisors
2	Testing	Testing involves systematically evaluating project deliverables to verify their functionality, performance, or compliance with requirements.	Everyday	On site supervisors and Laboratory employers
3	Quality audits	Quality audits are systematic examinations of project processes and activities to ensure compliance with organizational policies, procedures, and standards.	Unexpected, 1-2/month	Directors and Governmental organizations
4	Benchmarking	Benchmarking involves comparing project performance or quality metrics against industry standards or best practices.	1/month	Directors
5	Cost estimation	Cost estimation involves predicting the monetary resources required to complete project activities.	1/quarter	Directors, accountants, economists

5.8. Procurement planning

Procurement management plan is a component of the project or program management plan that describes how a project team will acquire goods and services from outside of the performing organization (PMBOK, 3rd edition). Project planning is an important part of project management because it ensures that resources are allocated effectively by determining what goods and services need to be acquired from sellers. Also, it plays a key role in cost management by estimating the costs associated with procuring goods and services.

Figure 5.22 shows the procurement process which occurs in our project during purchasing products or services.

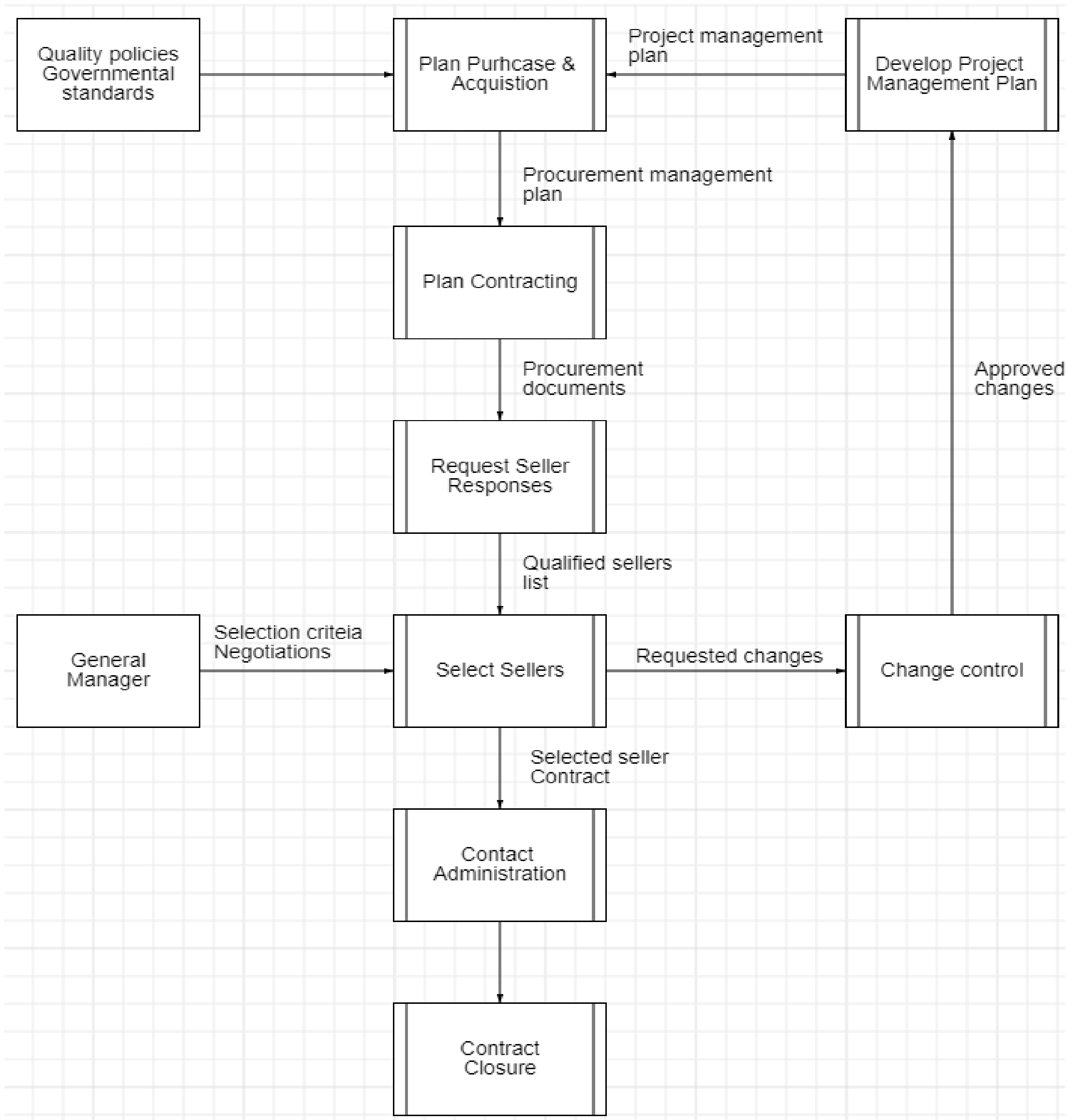


Figure 5.22. Procurement process

Plan Purchase & Acquisition is determining what to purchase or acquire and determining when and how (PMBOK, 3rd edition). It is an essential part of project management, especially procurement planning. Quality policies and governmental standards are parts of plan purchase and acquisition. Plan contracting documents purchases, products, services and then determines potential sellers. Then seller responses are requested to get a qualified sellers list. Then the general manager and administration selects sellers based on source selection criteria (Table 5.4). Usually, the amount of purchases or prices can be changed through change control. Once there are no more changes, managers contact administration and sign contracts.

Table 5.4. Source selection criteria

Source selection criteria	**Description**
Understanding of need	The product which is planned to be purchased should be really needed in the project.
Fair cost	The product should have a market price or lower the average price.
Intellectual property rights	The seller should have proper certifications.
Technical capability	The seller should be technically capable to finish a specific work at a project.
References from customers from the past	It is better to ask their previous customers about quality.

Procurement documents clearly state the requirements, specifications, and terms of the procurement process. They provide a common understanding between the buyer and the seller regarding the goods or services being procured, the desired quality standards, delivery schedules, and payment terms. The procurement documents for this project are Invitation for bid, Request for proposal, Request for quotation, Tender notice, Invitation for negotiation and Contractor initial response. Description of these documents are provided in Table 5.5.

Table 5.5. Procurement documents

Procurement document	Description
Invitation for bid	Request for commercial proposal
Request for proposal	Requesting proposals from prospective sellers
Request for quotation	Requesting price quotations from sellers
Tender notice	Formal announcement issued by a buyer (such as a project owner or organization) to invite potential suppliers or contractors to submit bids or proposals for the provision of goods
Invitation for negotiation	Formal communication issued by a buyer to potential suppliers to initiate negotiations for the procurement of goods
Contractor initial response	Initial communication or response provided by a potential contractor in response to a procurement invitation

Qualified sellers list can be also known as vendor list which involves identifying and selecting suppliers, contractors, and service providers who can supply the necessary materials, equipment, and services needed for the project. Usually, it includes the vendor's name, category and their contact information. An example of a vendor list for this project is presented in Table 5.6.

Table 5.6. Vendors list

Vendor Category	Vendor name	Contact Information
General Contractor	Turner Construction	Phone: (504) 555-1234 Email: info@turnerconstruction.com
Subcontractor	Cajun Industries	Phone: (225) 555-5678 Email: info@cajunindustries.com
Material Suppliers	Boh Bros. Construction	Phone: (504) 555-9876 Email: sales@bohbros.com
Equipment Rental	H&E Equipment Services	Phone: (504) 555-4321 Email: info@he-equipment.com
Engineering and Design	CSRS	Phone: (225) 555-8765 Email: info@csrsinc.com
Transportation and Logistics	Dupré Logistics	Phone: (337) 555-1098 Email: info@duprelogistics.com
Waste management	Waste management	Phone: (504) 555-2109 Email: info@wm.com

5.9. Construction safety

Construction safety is one of the most important aspects of project management. It ensures the safety of workers, visitors, and the surrounding environment on a construction site. Therefore, safety on the construction site must be assessed and controlled properly. Health and safety in the workplace are primarily regulated by the Louisiana Occupational Safety and Health Administration (OSHA) program, which operates under the Louisiana Workforce Commission (LWC). Louisiana OSHA does inspections to assess construction safety in our project. Figure 5.23 shows OSHA Construction Safety Template which will be used for daily inspections of general safety on the site. This template must have an inspector's name, date and signature. General safety includes activities such as housekeeping, fall protection, emergency exits, fire protection. Inspector checks the boxes with variants "yes" (means safety assessment is successful), "no" (means activity is not safe) and "N/A" (means not available-no data). Also, the inspector writes some important observations, notes, and corrective measures in the provided space.

OSHA Construction Safety Inspection

Construction Site Name/ Location			
Date of Inspection		Inspector's Name	

INSTRUCTIONS:
This checklist is designed to conduct a comprehensive safety inspection of the construction site to identify potential hazards and ensure compliance with OSHA regulations. Carefully review each item and mark the corresponding checkbox to indicate compliance or note any observations and areas for improvement. Use the "Notes/Observations" section to provide additional details, corrective actions, and any required follow-up.

GENERAL SAFETY

Housekeeping: Inspect for cleanliness and clutter-free work areas.	☐ Yes	☐ No	☐ NA
Fall Protection: Verify the use of fall protection systems (e.g., guardrails, harnesses).	☐ Yes	☐ No	☐ NA
Emergency Exits: Ensure emergency exits are accessible and unblocked.	☐ Yes	☐ No	☐ NA
Fire Extinguishers: Check fire extinguishers for accessibility and inspection tags.	☐ Yes	☐ No	☐ NA

Observations / Notes / Corrective actions, if any:

Figure 5.23. OSHA Construction Safety Inspection Template-General safety

Figure 5.24 is a continuation of the template provided by OSHA. It assesses safety regarding the scaffolding and trenching activities. Scaffolding is a temporary structure that is used to support construction workers, inspectors, cleaners, and others who need to work at height. It should be stable and surrounded by guardrails (Figure 5.25). Inspector should verify that there is a safe access to scaffolding platforms

SCAFFOLDING

Scaffolding Construction: Inspect the construction and stability of scaffolding.	☐ Yes	☐ No	☐ NA
Scaffolding Access: Verify safe access to scaffolding platforms.	☐ Yes	☐ No	☐ NA
Guardrails: Ensure guardrails are installed on all open sides of scaffolding.	☐ Yes	☐ No	☐ NA

Observations / Notes / Corrective actions, if any:

EXCAVATIONS AND TRENCHES

Shoring and Support: Check shoring and support systems in excavations.	☐ Yes	☐ No	☐ NA
Access and Egress: Ensure safe access and egress from trenches.	☐ Yes	☐ No	☐ NA
Sloping and Benching: Verify proper sloping and benching of excavations.	☐ Yes	☐ No	☐ NA

Observations / Notes / Corrective actions, if any:

Figure 5.24. OSHA Construction Safety Inspection Template-Scaffolding, trenches

Figure 5.25. Scaffolding guardrails

Regarding the trenches, the inspector must check shoring and support systems. A shoring system is a temporary structure used to support and stabilize excavations, trenches during construction (Figure 5.26).

Figure 5.26. Shoring system

The next important aspect of construction safety is ensuring fall and electrical protection. Figure 5.27 shows checklists for these activities which include personal fall arrest systems, safety nets, electrical wire inspection.

FALL PROTECTION

Personal Fall Arrest Systems: Verify the use of personal fall arrest systems when working at heights.	☐ Yes	☐ No	☐ NA
Guardrails: Inspect guardrails on elevated platforms and edges.	☐ Yes	☐ No	☐ NA
Safety Nets: Ensure the presence and proper installation of safety nets.	☐ Yes	☐ No	☐ NA

Observations / Notes / Corrective actions, if any:

ELECTRICAL SAFETY

Electrical Wiring: Inspect electrical wiring for damages or exposed wires.	☐ Yes	☐ No	☐ NA
Electrical Panels: Ensure electrical panels are properly labeled and accessible.	☐ Yes	☐ No	☐ NA
Ground Fault Circuit Interrupters (GFCIs): Check for the presence of GFCIs in wet areas.	☐ Yes	☐ No	☐ NA

Observations / Notes / Corrective actions, if any:

Figure 5.27. OSHA Construction Safety Inspection Template-Fall protection, electrical safety

Figure 5.28 shows a personal fall arrest system which ensures that workers will not fall when they work on height.

Figure 5.28. Personal fall arrest system

The most important safety measure is to have safety gear (Figure 5.29) which includes hard hat, safety glasses, work shirt, construction vest, work pants, construction gloves, leather boots and fall protection. Inspector must ensure that nobody enters the construction site without proper equipment.

Figure 5.29. Safety gear

Nowadays, ensuring that everyone on site has safety gear can be accomplished by using modern technologies. For example, Figure 5.30 shows implementation of face ID technology on construction site. Camera scans the worker for a helmet, construction vest, gloves and then turnstiles open.

Figure 5.30. Face identification for safety gear

Generally, safety in the workplace is protected by Health and Safety in the Workplace Policy (LA), law stated as of 19 Dec 2023. Law requires that employers provide a safe workplace for workers.

Any risks should be mentioned in the template by the inspector. Additionally, all risks should be registered in the risk register. It is a document used to track information about identified risks throughout the project lifecycle. Table 5.7 shows two examples of registered risks. Risk ID ensures that risk is registered and it has its unique number. Current status can be “in progress” and “closed”. Also, the inspector should mention issue description, issue type, identification date. Then, he assigns a responsible person to mitigate the risk, usually supervisors are working on it. Every risk should have a response and expected resolution date.

Table 5.7. Risk register example

ID	Current status	Issue description	Issue Type	Identified date	Assigned to	Risk response	Expected resolution date	Final resolution and comments
1	In Progress	Instability of foundation	Structural	19.04.2024	Supervisor	Conduct site assessment, provide additional reinforcement	26.04.2024	Risk is eliminated, additional tests can be done
2	Closed	Elevator failure	Mechanical	11.04.2024	Supervisor	Call mechanical engineer, identify cause and propose solutions	13.04.2024	Elevator is fixed

5.10. Construction site planning

Construction site planning is systematic organization and coordination of all important elements of a construction project. It provides visual representation of the site which is helpful for navigation through the construction project. For example, access roads show the path how construction transports should move in the site. Also, a construction site plan is important for resource allocation which includes steel, stone depots. Figure 5.31 shows the construction site plan of our project which includes all important elements such as building foundation, access roads, excavated ground, etc.

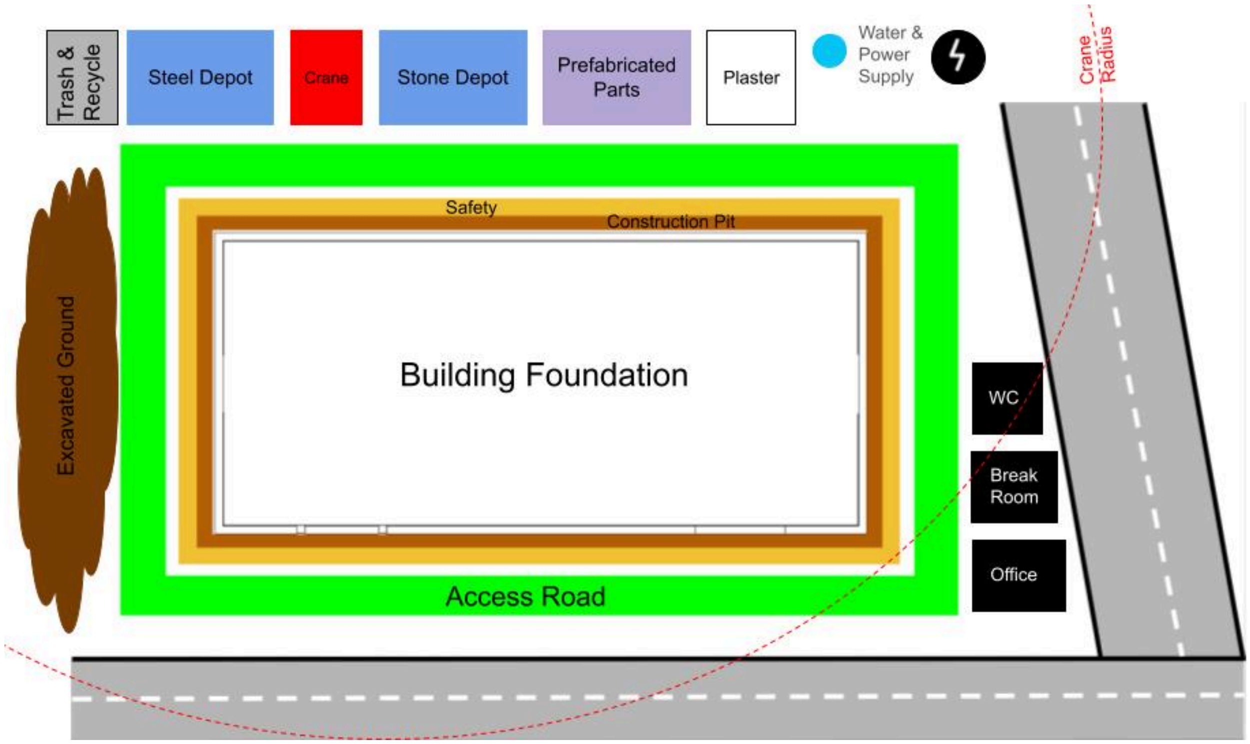


Figure 5.31. Construction Site Plan

Table 5.8 provides definitions for all elements presented on Figure 5.31.

Table 5.8. Definitions of elements on construction site plan

Element	**Description/Definition**
Building Foundation	Foundation of the hotel
Construction Pit	Excavated space for the foundation of a building
Excavated Ground	Soil removed from the ground during excavation activity
Trash and Recycle	Place of waste materials generated during construction activities
Steel Depot (Material stockpile)	Area where steel materials like reinforcement bars are stored
Stone Depot (Material stockpile)	Area where stone materials gravel, crushed stone, or decorative stones are stored
Crane	A large, tall machine used to lift and lower heavy materials and equipment on a construction site
Crane Radius	The maximum distance from the center of a crane's rotation to the tip
Prefabricated Parts	Building components that are manufactured off-site in a factory and then transported to the construction site for installation. For example, concrete panels
Plaster	A mixture of lime, sand, and water that is applied to interior walls and ceilings to create a smooth, durable surface for painting or

	decoration.
Water and Power Supply	Infrastructure required to provide water and electrical power to a construction site
WC	Restroom facility
Break room	A designated area on a construction site where workers can take breaks, eat meals, and relax during break time.
Office	A temporary facility used as a workspace for project managers, engineers, supervisors, and administrative staff
Access Road	A pathway providing entry and exit for vehicles and equipment to access a construction site

Conclusion

In conclusion, the comprehensive design process undertaken for the construction of the 16-story hotel in a high wind environment encompassed various disciplines including architectural, structural, geotechnical, and environmental considerations. Through meticulous literature review, analytical analysis, and calculations, the project aimed not only to meet the immediate goal of constructing a functional hospitality facility but also to adhere to sustainable principles and mitigate potential environmental impacts.

The structural and architectural aspects of the project incorporated LEED principles for sustainable building design, alongside a thorough evaluation of internal forces, drift, and deflections to ensure structural integrity. Material selection was guided by both project requirements and sustainability standards, with a keen focus on longevity and environmental impact.

On the environmental front, waste management was carefully considered, with plans developed for waste collection, disposal, and overall solid waste management. Geotechnical analyses were conducted to determine foundation suitability, ensuring the structural stability of the high-rise hotel in the face of varying loads and settlement concerns.

Additionally, construction management strategies were employed to ensure the successful execution of the project within budget, time, and resource constraints. Risk assessment and mitigation measures were implemented, alongside efforts to maintain a safe working environment and uphold the highest standards of material quality and construction practices.

In essence, the culmination of these efforts resulted in a comprehensive design that not only meets the functional requirements of the hospitality facility but also reflects a commitment to sustainability, structural integrity, and efficient project management. This project stands as a testament to the multidisciplinary approach required for the successful realization of complex construction endeavors in challenging environments.

References

[Absorption capacity of different brands of paper points] (1989). https://pubmed.ncbi.nlm.nih.gov/2639457/.

ACI 318-14, Building Code Requirements for Structural Concrete and Commentary

A Guide to the Project Management Body of Knowledge Third Edition (PMBOK ® Guide). (n.d.).

Al-Amoudi, O. S. B. (1998). Sulfate attack and reinforcement corrosion in plain and blended cements exposed to sulfate environments. In Building and Environment (Vol. 33, Issue 1, pp. 53–61). Elsevier BV. https://doi.org/10.1016/s0360-1323(97)00022-x

Btr (2020) 'What are the Rear Loader Garbage Truck Specifications?,' *Big Truck Rental*, 6 October. https://bigtruckrental.com/rear-loader-garbage-truck-rental-service/what-are-the-rear-loader-specifications/#:~:text=Total%20body%20length%3A%20235%20inches,Height%20above%20frame%3A%2096%20inches.

Comparison of Diesel and Hydraulics Hammers for Pile Driving . (n.d.). Scribd. Retrieved April 20, 2024, from https://ru.scribd.com/document/427082600/260552214-comparison-of-diesel-and-hydraulics-hammer-pdf-pdf

Construction & Demolition Waste Management & Recycling | USA Waste & Recycling (no date). https://www.usarecycle.com/commercial/construction-and-demolition.

Container sizes (no date). https://valleywastesolutions.com/container-sizes/.

Du, J. M., Han, X. L., Li, Z. T., Li, G., & Ji, Y. S. (2016). Effects of Water-Cement Ratio on Concrete Sulfate Corrosion Rate. In Key Engineering Materials (Vol. 711, pp. 295–301). Trans Tech Publications, Ltd. https://doi.org/10.4028/www.scientific.net/kem.711.295

Fan, L., & Shi, X. (2021). Techniques of corrosion monitoring of steel rebar in reinforced concrete structures: A review. In Structural Health Monitoring (Vol. 21, Issue 4, pp. 1879–1905). SAGE Publications. https://doi.org/10.1177/14759217211030911

Glass Half Full | Glass recycling, coastal restoration (no date). https://glasshalffull.co/.

Guide to Durable Concrete. (n.d.) ACI Committee 201.ACI 201.2R-01. https://icemeltnow.com/reference/aci_201.2r01_guide_to_durable_concrete.pdf

Hassoun M.N, and Al-Manaseer A. (2020). Structural Concrete: Theory and Design. 7th Edition. Published by John Wiley & Sons, Inc., Hoboken, New Jersey.

Kassa, A. *et al.* (2024) 'Insights into water absorption characteristics of various waste-based inorganic additives and their application for soil stabilization,' *Journal of Cleaner Production*, p. 141470. https://doi.org/10.1016/j.jclepro.2024.141470.

Kober, E. *et al.* (2007) 'Modification of water absorption capacity of a plastic based on bean protein using gamma irradiated starches as additives,' *Radiation Physics and Chemistry*, 76(1), pp. 55–60. https://doi.org/10.1016/j.radphyschem.2006.05.017.

Lycett T. (2022). Concrete strength explained: Cor-TUF. *cor-tuf.com.* https://cor-tuf.com/2022-everything-you-need-to-know-about-concrete-strength/#:~:text=The%20bare%20minimum%20is%20usually,bridges

McCormac J.K, and Brown R.H. (2015). Design of reinforced Concrete. 10th Edition. USA

Pinkerton, D. (2023) *BioCycle Nationwide Survey: Full-Scale food waste composting infrastructure in the U.S. | BioCycle*. https://www.biocycle.net/us-food-waste-composting-infrastructure/.

Prasad. (2020, August 7). *Driven pile foundations design and construction*. Structural Guide. https://www.structuralguide.com/driven-pile/

Ray. S.S. (1994). Reinforced concrete: design and analysis. Blackwell Science.

Rodrigues, R., Gaboreau, S., Gance, J., Ignatiadis, I., & Betelu, S. (2021). Reinforced concrete structures: A review of corrosion mechanisms and advances in electrical methods for corrosion monitoring. In Construction and Building Materials (Vol. 269, p. 121240). Elsevier BV. https://doi.org/10.1016/j.conbuildmat.2020.121240

Sire, D.F.A. (2023) 'Only 5% of plastic in the U.S. is recyclable. What should you do with the rest? | Here & Now,' *WBUR.org*, 5 September. https://www.wbur.org/hereandnow/2023/09/05/unrecyclable-plastic-pollution.

Statista (2023) *Recycling rate of metals in the U.S. 2021, by type*. https://www.statista.com/statistics/251345/percentage-of-recycled-metals-in-the-us-by-m etal/#:~:text=In%202021%2C%20about%2069%20percent,rate%20of%20roughly%2044%20percent.

Sumanth, B. H. & K, V. (2020). Effect of w/c ratio on the corrosion of rebar embedded in high strength concrete. Journal of Corrosion Science and Engineering, 23. https://www.researchgate.net/publication/340427898_Effect_of_wc_ratio_on_the_corrosion_of_rebar_embedded_in_high_strength_concrete

The Paper Recycling Coalition (no date). https://www.paperrecyclingcoalition.com/#:~:text=In%202022%2C%20nearly%2068%20percent,over%20the%20past%2020%20years.

University of the West of England. (n.d.). *Foundations*. Uwe.ac.uk. Retrieved April 18, 2024, from http://environment.uwe.ac.uk/geocal/foundations/fountype.htm

2022 Louisiana Water Quality Integrated Report. (2022). Louisiana Department of Water quality. https://www.deq.louisiana.gov/assets/docs/Water/Integrated_Report/2022_Integrated_Report/22_IR1_Master_Text_FINAL_For_ATTAINS_3-30-22.pdf

Appendix

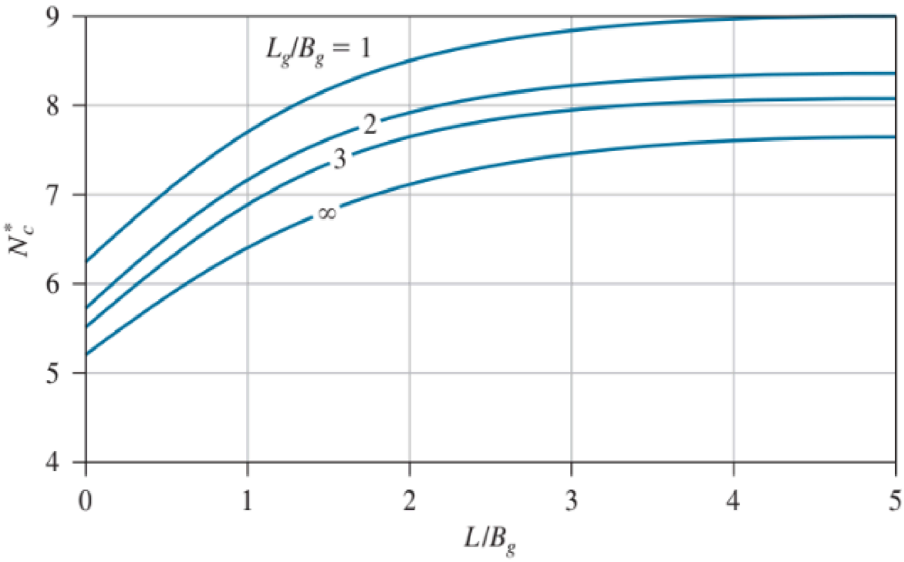


Figure A4.22. Values of N_c^*

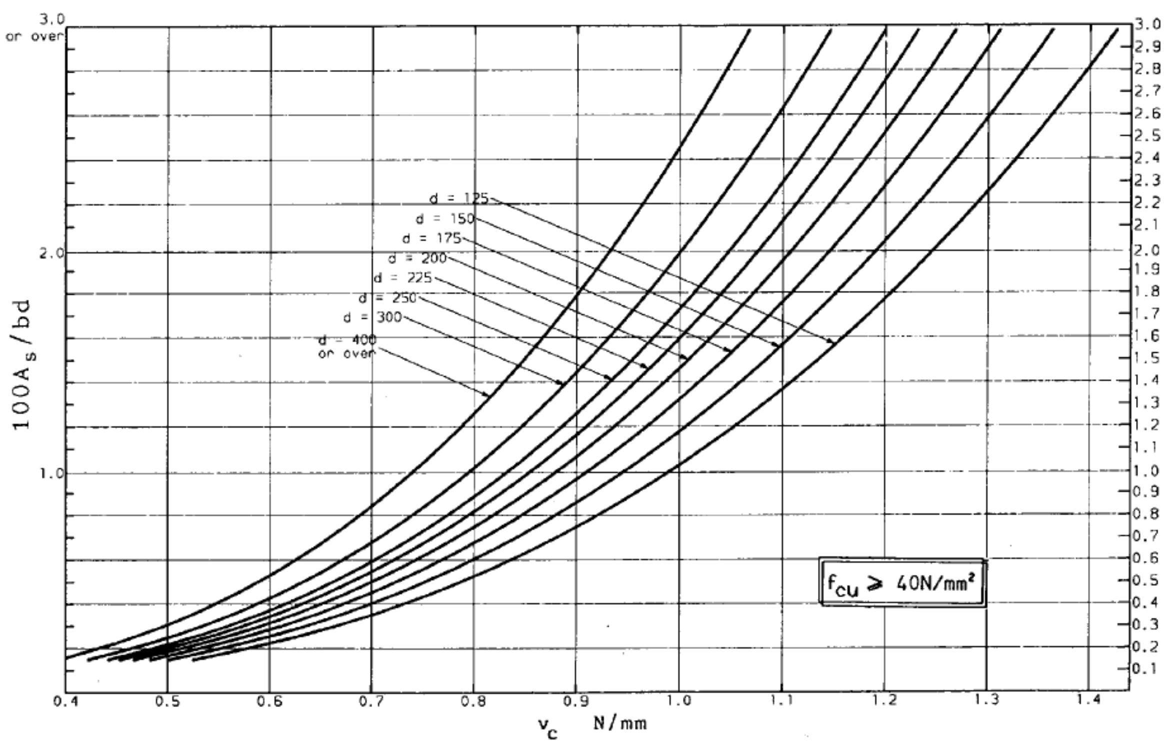


Figure A4.23. Values of $\nu_c - design\ concrete\ shear\ stress.$

Table A4.12. Area of steel reinforcement for various spacing.

Diameter (mm)	Perimeter (mm)	Area (mm^2)	Weight (kg/m)	Area of steel reinforcement for different spacings (mm^2/m)							
				50 mm	75 mm	100 mm	125 mm	150 mm	200 mm	250 mm	300 mm
6	18.8	28.3	0.222	566	377	283	226	188	141	113	94
8	25.1	50.3	0.395	1006	670	503	402	335	251	201	167
10	31.4	78.5	0.616	1570	1046	785	628	523	392	314	261
12	37.7	113.1	0.888	2262	1508	1131	904	754	565	452	377
16	50.3	201.1	1.579	4022	2681	2011	1608	1340	1005	804	670
20	62.8	314.2	2.466	6284	4189	3142	2513	2094	1571	1256	1047
25	78.5	490.9	3.854	9818	6544	4909	3926	3272	2454	1963	1636
32	100.5	804.2	6.313	–	10722	8042	6433	5361	4021	3216	2680
40	125.7	1256.6	9.864	–	–	12566	10052	8377	6283	5026	4188
50	157.1	1963.5	15.413	–	–	–	15708	13090	9817	7854	6545

Table A4.13. Circular columns - $f_{cu} = 40\ N/mm^2,\ k = 0.85$.

e/R	p=0.4		p=1.0		p=2.0		p=3.0		p=4.0		p=5.0		p=6.0	
	N/R²	z/R	N/R²	z/R	N/R²	z/R	N/R²	z/R	N/R²	z/R	N/R²	z/R	N/R²	z/R
0.10	54.26	0.698	60.30	0.687	70.45	0.673	80.71	0.662	91.04	0.653	101.44	0.645	111.90	0.639
0.20	47.04	0.796	52.28	0.785	61.02	0.770	69.78	0.759	78.56	0.751	87.37	0.744	96.19	0.739
0.30	39.95	0.890	44.69	0.872	52.41	0.853	60.06	0.840	67.69	0.831	75.30	0.824	82.91	0.818
0.40	33.40	0.974	37.97	0.946	45.06	0.920	51.91	0.903	58.68	0.892	65.39	0.883	72.09	0.877
0.50	27.74	1.046	32.39	1.005	39.06	0.971	45.31	0.951	51.41	0.938	57.43	0.928	63.41	0.921
0.60	22.71	1.113	27.91	1.051	34.23	1.010	40.00	0.987	45.57	0.972	51.04	0.962	56.45	0.954
0.70	18.19	1.175	24.07	1.093	30.37	1.040	35.72	1.015	40.84	0.999	45.84	0.987	50.78	0.979
0.80	14.70	1.222	20.46	1.136	27.24	1.063	32.23	1.037	36.96	1.020	41.57	1.008	46.12	0.999
0.90	12.12	1.256	17.51	1.169	24.40	1.088	29.33	1.054	33.74	1.037	38.01	1.024	42.22	1.015
1.00	10.07	1.287	15.21	1.194	21.72	1.116	26.91	1.068	31.02	1.050	35.00	1.037	38.92	1.028
1.20	7.15	1.334	11.95	1.228	17.51	1.154	22.42	1.106	26.70	1.071	30.21	1.056	33.64	1.048
1.40	5.43	1.363	9.79	1.250	14.57	1.179	18.92	1.134	22.88	1.099	26.56	1.073	29.62	1.063
1.60	4.34	1.381	8.28	1.265	12.45	1.196	16.24	1.153	19.88	1.123	23.21	1.096	26.46	1.074
1.80	3.60	1.393	7.10	1.279	10.86	1.208	14.21	1.167	17.46	1.138	20.54	1.114	23.47	1.093
2.00	3.07	1.402	6.16	1.292	9.62	1.217	12.63	1.178	15.53	1.149	18.39	1.128	21.06	1.108
2.20	2.67	1.408	5.43	1.303	8.63	1.225	11.36	1.186	13.99	1.158	16.57	1.138	19.09	1.120
2.40	2.37	1.413	4.85	1.311	7.83	1.231	10.32	1.193	12.72	1.166	15.08	1.145	17.42	1.129
2.60	2.12	1.417	4.38	1.317	7.16	1.235	9.45	1.198	11.66	1.172	13.83	1.152	15.98	1.136
2.80	1.93	1.420	4.00	1.323	6.60	1.239	8.72	1.203	10.76	1.177	12.77	1.157	14.76	1.142
3.00	1.76	1.423	3.67	1.327	6.12	1.243	8.09	1.206	9.99	1.181	11.86	1.162	13.71	1.146
3.25	1.59	1.425	3.33	1.332	5.61	1.246	7.42	1.210	9.17	1.185	10.89	1.166	12.60	1.151
3.50	1.45	1.428	3.05	1.336	5.18	1.249	6.85	1.214	8.48	1.189	10.07	1.170	11.65	1.156
3.75	1.33	1.430	2.81	1.339	4.81	1.252	6.37	1.217	7.88	1.192	9.36	1.174	10.83	1.159
4.00	1.23	1.431	2.60	1.342	4.48	1.254	5.95	1.219	7.36	1.195	8.74	1.177	10.12	1.162
4.25	1.15	1.432	2.43	1.344	4.20	1.256	5.58	1.221	6.90	1.197	8.21	1.179	9.50	1.165
4.50	1.07	1.434	2.27	1.346	3.95	1.258	5.25	1.223	6.50	1.199	7.73	1.181	8.94	1.167
4.75	1.00	1.435	2.14	1.348	3.73	1.259	4.96	1.225	6.14	1.201	7.30	1.183	8.45	1.169
5.00	0.95	1.436	2.02	1.349	3.54	1.261	4.70	1.226	5.82	1.203	6.92	1.185	8.01	1.171
5.50	0.85	1.437	1.81	1.352	3.20	1.263	4.25	1.229	5.27	1.205	6.27	1.188	7.26	1.174
6.00	0.77	1.438	1.64	1.354	2.92	1.265	3.88	1.231	4.81	1.208	5.73	1.190	6.63	1.177
6.50	0.70	1.439	1.50	1.356	2.69	1.266	3.57	1.233	4.43	1.210	5.27	1.193	6.11	1.179
7.00	0.65	1.440	1.39	1.358	2.49	1.267	3.31	1.234	4.10	1.211	4.89	1.194	5.66	1.181
7.50	0.60	1.441	1.29	1.359	2.32	1.269	3.08	1.235	3.82	1.213	4.55	1.196	5.27	1.182
8.00	0.56	1.442	1.20	1.360	2.16	1.270	2.88	1.237	3.58	1.214	4.26	1.197	4.93	1.184
8.50	0.52	1.442	1.12	1.361	2.03	1.271	2.71	1.238	3.36	1.215	4.00	1.198	4.64	1.185
9.00	0.49	1.443	1.06	1.362	1.91	1.273	2.55	1.238	3.17	1.216	3.77	1.199	4.37	1.186
9.50	0.46	1.443	1.00	1.363	1.80	1.274	2.42	1.239	3.00	1.217	3.57	1.200	4.14	1.187
10.00	0.44	1.443	0.95	1.364	1.71	1.275	2.29	1.240	2.85	1.217	3.39	1.201	3.93	1.188
11.00	0.40	1.444	0.85	1.365	1.55	1.277	2.08	1.241	2.58	1.219	3.08	1.202	3.56	1.189
12.00	0.36	1.445	0.78	1.366	1.41	1.279	1.90	1.242	2.36	1.220	2.81	1.203	3.26	1.190
13.00	0.33	1.445	0.72	1.367	1.30	1.280	1.75	1.243	2.18	1.221	2.59	1.204	3.01	1.191
14.00	0.31	1.445	0.66	1.367	1.20	1.281	1.63	1.243	2.02	1.221	2.41	1.205	2.79	1.192
15.00	0.29	1.446	0.62	1.368	1.12	1.282	1.52	1.244	1.88	1.222	2.24	1.206	2.60	1.193
17.50	0.24	1.446	0.53	1.369	0.96	1.284	1.30	1.245	1.61	1.223	1.92	1.207	2.22	1.195
20.00	0.21	1.447	0.46	1.370	0.83	1.285	1.13	1.246	1.41	1.224	1.68	1.208	1.94	1.196