



NAZARBAYEV
UNIVERSITY

Department of Civil and Environmental Engineering

Capstone Project II

**DESIGN OF 4 STORY SHOPPING MALL “KHAN TENGRI MALL” IN LOS
ANGELES, USA**

Group 5

Adina Serikkyzy 202077035

Adelina Nurgaliyeva 202067099

Rassul Ibrayev 202013545

Zhanbolat Sadratdinov 202287414

Baidibek Mussayev 202178264

Nurislam Nuraddinov 202097326

Alibi Yeskazy 202068459

Declaration

We declare that this report, titled “Design of 4 story Shopping Mall “Khan Tengri Mall” in Los Angeles, USA” is the product of our own project work, with all quotations and citations appropriately referenced. We further confirm that it has not been submitted previously or simultaneously for any other degree at Nazarbayev University.

Adina Serikkyzy



Alibi Yeskazy



Zhanbolat Sadratdinov



Baidibek Mussayev



Rassul Ibrayev



Adelina Nurgaliyeva



Nurislam Nuraddinov



Acknowledgements

We would like to thank all our professors who have helped us in our academic process, they are Mert Guney, Dichuan Zhang, Chang-Seon Shon, Alfredo Satyanaga, Sung-Woo Moon, Ferhat Karaca, Abid Nadeem and Jong Kim. Their support, motivation, and efforts have been very useful to our personal and academic development.

It is also our personal gratitude to the whole body of faculty at the Civil Engineering and Environmental Department that has been as reliable as ever in our course work and the capstone project. They have been an inspiration in terms of their experience, mentoring as well as their dedication to excellence and we owe them a great deal of gratitude in learning and skills they have imparted in us.

Abstract

The present paper is a design and construction proposal of a 4-story shopping mall at the site of 130 W College St, Los Angeles, California, USA. The location is in a dynamic commercial hub and with the rising urbanization and the rising pressure to develop mixed-use projects, there is a need to come up with innovative and effective construction practices. These issues have been addressed through a holistic method that entails incorporation of architectural, structural, material, and geotechnical factors.

The architectural design of the building complies with the rules of the International Building Code (IBC). The designs of the structures are guided by the American Concrete Institute (ACI) and American Society of Civil Engineers (ASCE). The SAP 2000 software has been utilized to carry out the structural analysis in detail and model the response of the building to the gravity and lateral loads.

Reinforced concrete was the choice of a major construction material as this material is durable, versatile, and cost-effective. A comparative analysis of the foundation options and geotechnical investigations suggested that the best solution to be used to ensure that the superstructure is supported is the use of a pile foundation.

This project designs a solid waste management system for a shopping mall, focusing on waste separation into recyclables, organics, and residuals. The system is based on local regulations and waste composition data. It includes an analysis of recyclable materials. The design ensures compliance with sustainability goals and adaptability to future changes and to LEED standards to make certain that the construction process is safe, sustainable and environmentally friendly

This project shows a comprehensive approach to urban commercial construction, and that is innovative design, sustainability, and adherence not only to international but also local building codes.

Keywords: design of architectures, design of structures, geotechnical design, waste management design, construction design, sustainable construction.

Table of Contents

1. Introduction	22
2. Architectural Design	23
2.1 Overview	23
2.2 Site Selection and Site Analysis	24
2.3 Design Concept	33
2.4 Geometry Design and Site Layout	34
2.5 Leadership in Energy and Environmental Design (LEED) for Green Building Design, Construction, Operations and Performance	41
2.6 Literature Review of Design Codes	56
a. Maximum exit travel distance.	57
b. Number of Elevators and Stairs	57
c. Area for the shopping mall	60
d. Door Dimensions	61
e. Window Dimensions	62
f. Corridor Dimensions	63
g. Allowable Clear Building Height	64
h. Traffic Flow of Parking	64
i. Roof Slope Design	65
j. Parking Ramp Design	65
2.7 Design of Non-structural Elements	66
3. Structural Design	67
3.1 Material Selection	67
3.2 Analytical Model Development	68
3.2.1 Model Geometry	68
3.2.2 Material Properties	69
3.2.3 Frame Sections	70
3.2.4 Element Types, Connections	70
3.2.5 Boundary Connections	70
3.3 GLRS	70
3.3.1 Dead Load Calculations	70
3.3.2 Live Load Calculations	74
3.3.3 Snow Load Calculations	82
3.3.4 Estimation of member sizes	82
3.3.4.1. Determination of structural layout	82
3.3.4.2. Calculation of size dimensions of columns	86
3.3.5. Assigning Gravity Forces to the SAP 2000 model	90
3.4 Analysis and Design of Lateral Force Resisting System (LFRS)	104

3.4.1. Wind load with torsional effect	104
3.4.2. Seismic Load Calculation	112
3.4.2.1 Seismic Loads Including Torsional Effect	120
3.4.3 Assigning Lateral Forces to SAP 2000	123
3.5. Lateral Drift Analysis and Check under Wind and Seismic Loads	128
3.5.1. Wind and Seismic Drifts Calculations	128
3.5.2. Comparison of interstory drifts	133
3.5.3. P- Drift Calculation	138
3.5.4. Stability Check	140
3.5.5. Torsional Irregularity Check	141
3.6. Internal Force Analysis	142
3.6.1. Dead Load Internal Force Calculation	142
3.6.2. Wind Load Internal Force Calculation	150
3.6.3. Load Combinations	162
3.7. Structural Member Design	163
3.7.1. Major and Minor Beam Design	164
3.7.2. Prestressed Beam Design	174
3.7.2.1. Geometry Definition and Section Properties	174
3.7.2.2. Material Properties	174
3.7.2.3. Load Definition	176
3.7.2.4. Design Parameters and Allowable Stresses	176
3.7.2.5. Calculation of Demand Values	177
3.7.2.6. Post Tensioning	178
3.7.2.7. Serviceability Check	179
3.7.2.8. Design for Flexure, Shear, and Torsion	180
3.7.3. Column Design	181
3.7.3.1. Slenderness Ratio	181
3.7.3.2. Longitudinal Rebars	182
3.7.3.3. Shear Reinforcement	183
3.7.3.4. Biaxial Bending	183
3.7.4. Two-way slab	183
3.7.4.1. Flexural Analysis	184
3.7.5. Joint Selection	186
3.7.6. Bar Selection	189
3.7.7. Development Length	190
3.7.8. Lap splices	192
3.8. Special seismic provisions for reinforcement detailing	192
3.8.1. Beams	192

3.8.2. Columns	193
3.8.3. Joints	193
3.9. Structural serviceability design	193
3.9.1. Minimum Deflection	193
3.9.2. Crack width	194
4. Geotechnical Design	196
4.1 Site description	196
4.2 Geotechnical field investigation	198
4.3 Soil profile	199
4.4 Seismicity analysis	201
a. Liquefaction analysis	201
b. Shear wave velocity	204
4.5 Shallow foundation design	204
a. Bearing capacity: Shallow foundation, Footing Foundation	205
b. Bearing capacity: Shallow foundation, Mat Foundation	209
4.6 Shallow Foundation settlement	210
4.7 Deep Foundation Design of a Single Pile	211
4.7.1 Point Resistance (Q_p)	212
4.7.2 Shaft Resistance (Q_s)	216
a. Shaft Resistance in Sand	216
b. Shaft Resistance in Clay	222
4.7.3 Negative skin friction	229
4.8 Deep foundation settlement of a single pile	231
4.8.1 Elastic settlement of a single pile	232
4.8.2 Consolidation settlement of a single pile	235
4.9 Group Pile Design	237
4.9.1 Hand calculation of Group Pile Design	237
4.9.2. Software Calculation and Comparison of Group Piles with Hand Calculations	242
4.9.3 Plaxis Assessment of Seismicity	245
4.10 Group Pile Settlement	248
4.10.1 Hand Calculation of Group Pile's Settlement	248
4.10.2 Software Calculation of Group Pile's Settlement with Plaxis	252
4.10.3 Software Calculation of Foundation Settlement with Plaxis	255
4.11 Foundation Design Under Lateral Loading	260
4.11.1 Hand calculation of lateral bearing capacity	261
4.12 Lateral Active and Passive Pressures	266
4.13 Retaining Wall Design	270

4.13.1 Hand Calculation of Retaining Wall	270
a. Overturning Failure Check:	270
b. Sliding Failure Check	272
c. Bearing Capacity Failure Check	273
4.13.2 Software Calculation of Retaining Wall	274
4.14 Construction Procedure	277
4.15 Group pile cap reinforcement	277
5 Construction Management	282
5.1 Project Charter	282
5.2 Feasibility Study	285
5.3 Work Breakdown Structure	288
5.4 Scheduling	289
5.5 Cost/Benefit Analysis	290
5.5.1 Costs Estimation	290
5.5.2 Cash Inflow	301
5.6 Risk Management	303
5.6.2 Risk Management Process	305
5.6.3 Risk Ownership	307
5.7 Quality Management	308
5.7.1 Quality Metrics	309
5.7.2 Quality Checklist	310
5.8 Procurement Planning	313
5.8.1 Procurement Process	313
5.8.2 Procurement Documents	315
5.8.3 Source selection criteria	316
5.8.4 Vendor List	317
5.9 Construction safety	319
5.9.1 Hazard Identification and Risk Assessment	319
5.9.2 Hierarchy of controls	320
5.9.3 Safety Measures	321
5.9.4 Working Training	322
5.9.5 Inspections	322
5.10 Construction Site Planning	323
6. Environmental engineering	325
6.1 Waste management options	325
6.1.1 Recycling potential	325
6.1.2 Composting potential	326
6.1.3 Incineration potential	326

6.2 Waste management plan	328
6.2.1 Central bin allocation	328
6.2.2 Floor-level bin placement	330
6.2.3 Collection schedule	331
6.2.4 Bin quantity verification	331
6.3 Waste collection plan	332
6.3.1 Internal logistics and custodial routing	332
6.3.2 Contamination management at the dock	332
6.3.3 Waste flow and personnel responsibilities	333
6.3.4 Staff training requirements	333
6.4 Integrated Solid Waste Management (ISWM) plan	333
6.4.1 System goal and elements	333
6.4.2 Plan modifications based on ISWM goal	334
6.4.3 ISWM flowchart	334
6.5 Operational finances of waste management plan	335
2.5.2 Operational costs (OPEX)	336
2.5.3 Projected revenue	337
Reference List	339
Appendix	344

List of Tables

Table 1.1 Job Distribution	22
Table 3.1.1 Reinforced concrete Vs Steel Comparison	67
Table 3.2.2.1. Properties of reinforced concrete	69
Table 3.2.2.2. Properties of steel rebar	69
Table 3.2.3.1. Cracking moment of inertia for each structural member	70
Table 3.3.1.1. Uniformly Distributed Dead Load for Floors	71
Table 3.3.1.2. Uniformly Distributed Dead Load for Ceiling	71
Table 3.3.1.3. Uniformly Distributed Dead Load for Roof	71
Table 3.3.1.4. Uniformly Distributed Dead Load for Interior Walls	72
Table 3.3.1.5. Uniformly Distributed Dead Load for Exterior Walls	72
Table 3.3.1.6. Uniformly Distributed Dead Load for Roof Parapets	73
Table 3.3.1.7. Dead Load of Interior Walls to Slabs	73
Table 3.3.1.8. Total Dead Loads to Slabs	73
Table 3.3.2.1. Floor Live Load Calculation for the basement	74
Table 3.3.2.2. Floor Live Load Calculation for the first floor	75
Table 3.3.2.3. Floor Live Load Calculation for the second floor	76
Table 3.3.2.4. Floor Live Load Calculation for the third floor	77
Table 3.3.2.5. Floor Live Load Calculation for the fourth floor	79
Table 3.3.2.6. Roof Live Loads Calculations	81
Table 3.3.2.7. Load Summary	81
Table 3.3.4.2.1. Load calculation for interior columns	86
Table 3.3.4.2.2. Load calculation for interior columns including large span beam self-weight	87
Table 3.3.4.2.3. Load calculation for interior columns including typical and minor beam self-weight	87
Table 3.3.4.2.4. Iterative method for minimum interior column size estimation	88
Table 3.3.4.2.5. Minimum column sizes	89
Table 3.3.4.2.5. Final column sizes	90
Table 3.4.1.1. Exposure Category Constants	105
Table 3.4.1.2. Height Effect Coefficient Calculations	105
Table 3.4.1.3. External Pressure Coefficients	106
Table 3.4.1.4. Wind Forces for Case 1, Longitudinal Direction (Y-axis)	108
Table 3.4.1.5. Wind Forces for Case 1, Transverse Direction (X-axis)	109
Table 3.4.1.6. Wind Forces for Case 2, Longitudinal Direction (Y-axis)	109
Table 3.4.1.6. Wind Forces for Case 2 Transverse Direction (X-axis)	109
Table 3.4.1.7. Wind Force Calculations for third floor in both directions including the torsional effect (Case 2)	109

Table 3.4.1.9. Wind Force Calculations for Case 3	110
Table 3.4.1.10. Wind Force Calculations for Case 4	111
Table 3.4.1.11. Wind Force Calculations for third floor in both directions including the torsional effect (Case 4)	111
Table 3.4.2.1. Effective Seismic Weight	119
Table 3.4.2.2 Seismic force for each floor.	120
Table 3.4.2.3 Direct forces on each frame including torsion	122
Table 3.4.2.4 Seismic load calculations for third floor including torsional effect	122
Table 3.5.1.1. Lateral Shear Drift due to Wind Load in X-direction	129
Table 3.5.1.2. Lateral Flexural Drift due to Wind Load in X-direction	130
Table 3.5.1.3. Lateral Shear Drift due to Wind Load in Y-direction	131
Table 3.5.1.4. Lateral Flexural Drift due to Wind Load in Y-direction	131
Table 3.5.1.5. Lateral Shear Drift due to Seismic Load in X-direction	132
Table 3.5.1.6. Lateral Flexural Drift due to Seismic Load in X-direction	132
Table 3.5.1.7. Lateral Shear Drift due to Seismic Load in Y-direction	132
Table 3.5.1.8. Lateral Flexural Drift due to Seismic Load in Y-direction	133
Table 3.5.2.1. Comparison of interstory displacements on LFRS, Frame 1	133
Table 3.5.2.2. Comparison of interstory displacements on LFRS, Frame A	134
Table 3.5.3.1. Amplified drift for Wind load, Frame A (P-D drift)	139
Table 3.5.3.2. Amplified drift for Wind load, Frame 1 (P-D drift)	139
Table 3.5.3.3. Amplified drift for Seismic load, Frame A (P-D drift)	139
Table 3.5.3.4. Amplified drift for Seismic load, Frame 1 (P-D drift)	140
Table 3.5.4.1. Stability Coefficient Calculation for Frame 1	140
Table 3.5.4.2. Stability Coefficient Calculation for Frame A	141
Table 3.5.5.1. Torsional Irregularity Check	141
Table 3.6.1.1. Comparison of Axial Forces under Dead Load	148
Table 3.6.1.2. Comparison of Shear Forces under Dead Load	148
Table 3.6.1.3. Comparison of Moments under Dead Load	149
Table 3.6.2.1. Comparison of internal forces under wind load (2D, 3D, and hand calculations)	161
Table 3.7.1.1.1 Detailed Major Beam Reinforcement Design for Floor 4 in the Exterior Frame	168
Table 3.7.1.1.2 Detailed Minor Beam Reinforcement Design for Floor 4	171
Table 3.7.2.2.1 Material Properties for Concrete, Prestressing Steel, and Non-Prestressed Reinforcement	175
Table 3.7.2.4.1 Allowable Stresses	177
Table 3.7.2.5.1 Actions Due to Dead and Live Loads	177
Table 3.7.2.7.1. Stress check at critical points	180
Table 3.7.2.7.2. Deflection Check Results for Prestressed Beam	180
Table 3.7.2.8.1. Final Structural Detailing for Prestressed Beam for Floor 1-4	181
Table 3.7.4.1. Two-Way Slab Design Procedure for Exterior panel	185
Table 3.7.4.2. Two-Way Slab Design Procedure for Interior panel	186
Table 3.7.6.1. Final selected bars.	189

Table 3.7.8.1. Column Lap Splices	192
Table 4.3.1 Soil profile	199
Table 4.3.2 Soil properties	200
Table 4.4.1 Liquefaction values	201
Table 4.5.1 Soil properties for bearing capacity, Terzaghi's method	205
Table 4.5.2 Soil properties for bearing capacity, Meyerhof method	206
Table 4.5.3 Footing foundation results	208
Table 4.5.4 Mat foundation results	209
Table 4.6.1 Settlement of mat foundation	210
Table 4.7.1.1 Undrained Shear Strength values for each layer	214
Table 4.7.1.2 Point bearing capacity in clay.	214
Table 4.7.2.1 Shaft resistance values using effective stress method	217
Table 4.7.2.2 Frictional resistance in sand using Meyerhof's, Briaud et al., and Coyle and Castello methods.	220
Table 4.7.2.2 Alpha () Method Q_s values	223
Table 4.7.2.3 Effective vertical stress areas	224
Table 4.7.2.4 Shaft resistance using λ method	225
Table 4.7.2.5 Shaft resistance using method	227
Table 4.7.3. Ultimate Bearing Capacity of a single pile with $L=16m$	229
Table 4.7.1 Lateral Active Pressure	230
Table 4.7.2 Negative Skin Friction and Downward Drag Force	231
Table 4.8.1.1 Elastic shortening of a single 16m pile with different diameters	233
Table 4.8.1.2 Settlement due to working load on the tip of the 16m pile	234
Table 4.8.1.3 Settlement due to working load along the shaft of the 16m pile	234
Table 4.8.1.4 Total elastic settlement of the 16m pile	235
Table 4.8.2.1 Consolidation settlement of the 16m pile	236
Table 4.8.2.2 Total settlement of 16m pile	236
Table 4.9.1.1. Load Distribution for Different Column Types	239
Table 4.9.1.2. Load Distribution for Different Column Types	240
Table 4.9.1.3. Dimensions of Pile Caps	240
Table 4.9.2.1 Comparison of Group Piles on Software values and Hand Calculation values	242
Table 4.10.1.1. Settlement of Group Piles	250
Table 4.10.3.1 Hand and Plaxis 3D Settlement Comparison	259
Table 4.11.1 Lateral loads	261
Table 4.11.1.2 Modulus of subgrade reaction	261
Table 4.11.1.3 Determination of pile type	262
Table 4.11.1.4 Rankine's passive earth pressure constant for each layer	263
Table 4.11.1.5 Lateral resistance values	264
Table 4.11.1.6 Comparing lateral forces with pile lateral resistance	265
Table 4.12.1 Lateral Active Pressure	268
Table 4.12.2 Lateral Passive Pressure	269

Table 4.13.1.1 Calculation of MR	271
Table 4.13.1.2 Calculation of MO Pa, i	272
Table 4.15.1 Pile Cap Reinforcement Design for Each Column Type	279
Table 5.1.1 Project Charter	282
Table 5.5.1.1 Cost Estimation report by RSMeansOnline.	291
Table 5.5.1.2 Hand Calculation according to RSMeansOnline.	298
Table 5.5.1.3 Full hand calculations with assumed values.	299
Table 5.5.1.4 Cost Estimation of the project.	300
Table 5.5.2.1 Cash inflow of the project.	301
Table 5.6.1 Risk Matrix 3x3	304
Table 5.6.2 Risk Score Table	305
Table 5.6.3 Risk Matrix	308
Table 5.7.2. Quality Checklists.	311
Table 5.8.3. Source Selection Criteria.	316
Table 5.8.4. Vendor List.	317
Table 5.9.1. Risk Matrix Example.	319
Table 6.1.1. Realistic recycling potential and revenue	325
Table 6.1.2. Composting potential	326
Table 6.1.3 Incineration potential	327
Table 6.1.2. Central waste enclosure bins	329
Table 6.2.2. Floor-level bin placement	330
Table 6.2.3. Collection schedule	331
Table 6.2.4. Minimum required collection space calculation	332
Table 9.1. Load calculation for exterior columns in typical span	346
Table 9.2. Load calculation for exterior columns in typical span including large span beam self-weight	347
Table 9.3. Load calculation for exterior columns in typical span including typical span beam self-weight	347
Table 9.4. Load calculation for exterior columns in typical span including typical and minor beam self-weight	347
Table 9.5. Iterative method for minimum exterior columns in typical span size estimation	348
Table 9.6 Load calculation for exterior columns in large span	349
Table 9.7. Load calculation for exterior columns in large span including large span beam self-weight	349
Table 9.8. Load calculation for exterior columns in large span including typical span beam self-weight	350
Table 9.9. Load calculation for exterior columns in large span including typical and minor beam self-weight	350
Table 9.10. Iterative method for minimum exterior columns in large span size estimation	351
Table 9.11 Load calculation for corner columns in typical span	352

Table 9.12. Load calculation for corner columns including large span beam self-weight	352
Table 9.13. Load calculation for corner columns including typical span beam self-weight	353
Table 9.14. Load calculation for corner columns including typical and minor beam self-weight	353
Table 9.15. Load calculation for corner columns including exterior wall self-weight in typical span	353
Table 9.16. Load calculation for corner columns including exterior wall self-weight in large span	354
Table 9.17. Iterative method for minimum corner columns size estimation	354
Table 9.18. Wind Force Calculations for first floor in both directions including the torsional effect (Case 2)	355
Table 9.19. Wind Force Calculations for second floor in both directions including the torsional effect (Case 2)	356
Table 9.20. Wind Force Calculations for fourth floor in both directions including the torsional effect (Case 2)	357
Table 9.21. Wind Force Calculations for first floor in both directions including the torsional effect (Case 4)	358
Table 9.22. Wind Force Calculations for second floor in both directions including the torsional effect (Case 4)	359
Table 9.23. Wind Force Calculations for fourth floor in both directions including the torsional effect (Case 4)	360
Table 9.24. Stiffness Center Calculations	361
Table 9.25. Seismic Load Calculations for first floor in both directions including the torsional effect	362
Table 9.26. Seismic Load Calculations for second floor in both directions including the torsional effect	362
Table 9.27. Seismic Load Calculations for fourth floor in both directions including the torsional effect	363
Table 9.28 Detailed Major Beam Reinforcement Design for Roof in the Exterior Frame	364
Table 9.29 Detailed Major Beam Reinforcement Design for Floor 3 in the Exterior Frame	365
Table 9.30 Detailed Major Beam Reinforcement Design for Floor 2 in the Exterior Frame	365
Table 9.31 Detailed Major Beam Reinforcement Design for Floor 1 in the Exterior Frame	366
Table 9.32 Detailed Major Beam Reinforcement Design for Roof in the Interior Frame	366
Table 9.33 Detailed Major Beam Reinforcement Design for Floor 4 in the Interior Frame	367
Table 9.34 Detailed Major Beam Reinforcement Design for Floor 3 in the Interior Frame	367
Table 9.35 Detailed Major Beam Reinforcement Design for Floor 2 in the Interior Frame	368
Table 9.36 Detailed Major Beam Reinforcement Design for Floor 1 in the Interior Frame	368
Table 9.37 Detailed Minor Beam Reinforcement Design for Roof	369
Table 9.38 Detailed Minor Beam Reinforcement Design for Floor 4	369
Table 9.39 Detailed Minor Beam Reinforcement Design for Floor 3	370

Table 9.40 Detailed Minor Beam Reinforcement Design for Floor 2	370
Table 9.41 Detailed Minor Beam Reinforcement Design for Floor 1	371
Table 9.42. Final Structural Detailing for Prestressed Beam in the Floor 1-4	371

List of Figures

Figure 2.1.1 Rendered Site 3D View in Revit	24
Figure 2.2.1. Los Angeles temperature history in 2024 (WeatherSpark)	27
Figure 2.2.2. Los Angeles temperature history in 2023 (WeatherSpark)	27
Figure 2.2.3. Monthly climate change in Los Angeles	27
Figure 2.2.4. Sun direction	28
Figure 2.2.5. The wind directions on the site	29
Figure 2.2.6. View to downtown area from the site (Google earth)	29
Figure 2.2.7. Site shadowing view	30
Figure 2.2.8. Public transportation map (Google Maps)	31
Figure 2.2.9. Bushfire risks (Los Angeles Fire Department, 2025)	31
Figure 2.2.10. Stormwater drainage map (LA County Storm Drain System, 2025)	32
Figure 2.2.11. Sewer system map (Los Angeles GeoHub, 2023)	33
Figure 2.3.1 First and last versions of the shape of our building.	33
Figure 2.4.1 Floor Plan for Level 1	34
Figure 2.4.2 Floor Plan for Level 2	35
Figure 2.4.3 Floor Plan for Level 3	35
Figure 2.4.4 Floor Plan for Level 4	35
Figure 2.4.5 Emergency Plan for Level 1	36
Figure 2.4.6 Emergency Plan for Level 2	36
Figure 2.4.7 Emergency Plan for Level 3	37
Figure 2.4.8 Emergency Plan for Level 4	37
Figure 2.4.9. Site Layout	38
Figure 2.4.10. Basement Plan (Underground Parking)	38
Figure 2.4.11 Side View Plan	40
Figure 2.4.12 Section Cut 1	40
Figure 2.4.13 Section Cut 2	41
Figure 2.5.1. LEED Scorecard	42
Figure 2.5.2. L&T Score	43
Figure 2.5.3. Sustainable Sites Score	44
Figure 2.5.4. Water Efficiency Score	45
Figure 2.5.5. E&A Score	47
Figure 2.5.6. Materials & Resources Score	48
Figure 2.5.7. Indoor Environmental Quality Score	49
Figure 2.5.8 Regional Priority	50
Figure 2.5.9. Cost Parameters window from Life-365	51
Figure 2.5.10. Structure Type & Dimensions window from Life-365	51

Figure 2.5.11. Temperature Cycle window from Life-365	52
Figure 2.5.12. Base Case Mixture	53
Figure 2.5.13 Alternative 1 Mixture	53
Figure 2.5.14. Alternative 2 Mixture	54
Figure 2.5.15. Life Cycle Costs	54
Figure 2.5.16. LCC results	55
Figure 2.6.1 College Street Zoning Map	56
Figure 2.6.2 Maximum Travel distance	57
Figure 2.6.4 Stair Dimensions	59
Figure 2.6.5 Riser and Tread Depth of Stairs	59
Figure 2.6.6 Passenger Elevator dimensions	60
Figure 2.6.7 Rooms in Shopping Mall with areas	61
Figure 2.6.8 Main Door Dimensions	62
Figure 2.6.9 Other Doors Dimensions	62
Figure 2.6.10 Window Dimensions	63
Figure 2.6.11 General Corridor and Narrowest Corridor Dimensions	64
Figure 2.6.12 Traffic Flow Diagram	65
Figure 2.6.13 Drainage system on the roof of the scupper (Wolnitzek, 2012)	65
Figure 2.6.13 Exterior and Interior Wall Compositions	66
Figure 2.6.14 Floor Composition	67
Figure 3.2.1.1. Building Model Geometry	69
Figure 3.3.4.1.1. Structural layout of the building	86
Figure 3.3.5.1. Chosen Frame for Load Assignment	91
Figure 3.3.5.2. Material Property Data and Joint Restraints	92
Figure 3.3.5.3. Structural Element Section Properties	96
Figure 3.3.5.4. 2D Load Assignment for Dead Loads	97
Figure 3.3.5.4. 2D Load Assignment for Live Load	97
Figure 3.3.5.4. 2D Load Assignment for Roof Live Load	98
Figure 3.3.5.5. 3D Uniform Dead load distributed to Area (Slab)	99
Figure 3.3.5.6. 3D Uniform Dead load distributed to Frames	100
Figure 3.3.5.7. 3D Uniform Live load distributed to Area (Slab)	101
Figure 3.3.5.8. 3D Uniform Live load distributed to Frames	102
Figure 3.3.5.9. 3D Uniform Roof Live load distributed to Area (Slab)	103
Figure 3.3.5.10. 3D Uniform Roof Live load distributed to Frames	104
Figure 3.4.1.1. Wind Load Cases (American Society of Civil Engineers, 2022)	107
Figure 3.4.2.1. ASCE Hazard Tool Seismic Design Data	113
Figure 3.4.2.2. Site Class Table	113
Figure 3.4.2.3. Seismic Design Category Table (American Society of Civil Engineers, 2017)	114

Figure 3.4.2.4. Design Coefficients and Factors for Seismic Force-Resisting System (American Society of Civil Engineers, 2017)	115
Figure 3.4.2.5. Importance Factor Table (American Society of Civil Engineers, 2022)	115
Figure 3.2.4.6. Assigning the Mass Source for Modal Analysis	116
Figure 3.2.4.7. Defining Modal Analysis case	117
Figure 3.2.4.8. Mode 1 Results	117
Figure 3.2.4.9. Mode 2 Results	118
Figure 3.2.4.10. Mode 3 Results	118
Figure 3.4.3.1. Seismic Load Assignment for Frame C	124
Figure 3.4.3.2. 3D Seismic Load Assignment	126
Figure 3.4.3.3. Case 1 Wind Load Assignment for Frame C	126
Figure 3.4.3.4. 3D Case 1 Wind Load Assignment	128
Figure 3.5.1.1. Structural layout with the chosen Frame A and Frame 1	128
Figure 3.5.2.1 Inter-Story Wind Drift Comparison between Hand Calculations and SAP 2000 for Frame A, X - Direction	135
Figure 3.5.2.2 Inter-Story Wind Drift Comparison between Hand Calculations and SAP 2000 for Frame 1, Y - Direction	135
Figure 3.5.2.3. Inter-Story Seismic Drift Comparison between Hand Calculations and SAP 2000 for Frame A, X - Direction	136
Figure 3.5.2.4. Inter-Story Seismic Drift Comparison between Hand Calculations and SAP 2000 for Frame 1, Y - Direction	136
Figure 3.5.2.5. Amplified Inter-Story Seismic Drift Comparison between Hand and SAP 2000 Calculations with the Allowable Limit for Frame 1, Y - Direction	137
Figure 3.5.2.6. Amplified Inter-Story Seismic Drift Comparison between Hand and SAP 2000 Calculations with the Allowable Limit for Frame A, X - Direction	138
Figure 3.6.1.1 Axial Force from Dead Load in SAP2000 3D frame model	143
Figure 3.6.1.2 Shear Force from Dead Load in SAP2000 3D frame model	143
Figure 3.6.1.3 Moment Diagram of Dead Load in SAP2000 3D frame model	144
Figure 3.6.1.4 Axial Force from Dead Load in SAP2000 2D frame model	144
Figure 3.6.1.5 Shear Force from Dead Load in SAP2000 2D frame model	145
Figure 3.6.1.6 Moment Diagram of Dead Load in SAP2000 2D frame model	145
Figure 3.6.1.8 Analyzed Vertical Load Frame	146
Figure 3.6.1.9 Hand Calculation Results of Axial Force from Dead Load	147
Figure 3.6.1.10 Hand Calculation Results of Shear Force from Dead Load	147
Figure 3.6.1.10 Hand Calculation Results of Moments from Dead Load	147
Figure 3.6.1.11. Dead Load Axial Force Comparison between Hand Calculations and SAP2000	148
Figure 3.6.1.12. Dead Load Shear Force Comparison between Hand Calculations and SAP 2000	149
Figure 3.6.1.13. Dead Load Moment Comparison between Hand Calculations and SAP 2000	150

Figure 3.6.2.1. Portal Method Calculations for Wind Load Internal Force	153
Figure 3.6.2.2 Hand Calculation Results of Axial Force from Wind Load	154
Figure 3.6.2.3 Hand Calculation Results of Shear Force from Wind Load	154
Figure 3.6.2.4 Hand Calculation Results of Moments from Wind Load	154
Figure 3.6.2.5 Axial Force Diagram of Wind Load in SAP2000 3D frame model for Columns	155
Figure 3.6.2.6 Axial Force Diagram of Wind Load in SAP2000 3D frame model for Beams	155
Figure 3.6.2.7 Shear Force Diagram of Wind Load in SAP2000 3D frame model for Columns	156
Figure 3.6.2.8 Shear Force Diagram of Wind Load in SAP2000 3D frame model for Beams	156
Figure 3.6.2.9 Moment Diagram of Wind Load in SAP2000 3D frame model for Columns	157
Figure 3.6.2.10 Moment Diagram of Wind Load in SAP2000 3D frame model for Beams	158
Figure 3.6.2.11 Axial Force Diagram of Wind Load in SAP2000 2D frame model for Columns	158
Figure 3.6.2.12 Axial Force Diagram of Wind Load in SAP2000 2D frame model for Beams	159
Figure 3.6.2.13 Shear Force Diagram of Wind Load in SAP2000 2D frame model for Columns	159
Figure 3.6.2.14 Shear Force Diagram of Wind Load in SAP2000 2D frame model for Beams	160
Figure 3.6.2.15 Moment Diagram of Wind Load in SAP2000 2D frame model for Columns	160
Figure 3.6.2.16 Moment Diagram of Wind Load in SAP2000 2D frame model for Beams	160
Figure 3.6.2.2. Wind Load Axial Force Comparison between Hand Calculations and SAP 2000	162
Figure 3.6.2.3. Wind Load Shear Force Comparison between Hand Calculations and SAP 2000	162
Figure 3.6.2.4. Wind Load Moment Comparison between Hand Calculations and SAP 2000	162
Figure 3.6.3.1. Load Combinations	163
Figure 3.7.0.1. Capacity Check Verification	164
Figure 3.7.1.1.1 Major Beam Reinforcement Design for Negative Moment	165
Figure 3.7.1.1.2 Major Beam Reinforcement Design for Positive Moment	167
Figure 3.7.1.1.4 Minor Beam Reinforcement Design for Negative Moment	169
Figure 3.7.1.1.5 Minor Beam Reinforcement Design for Positive Moment	170
Figure 3.7.1.1.6. Determination of theoretical cutoff points from moment envelop	173
Figure 3.7.2.6.1. Geometry and actions of a parabolic tendon	179
Figure 3.7.4.1.1. Two-Way Slab Design	184
Figure 3.7.5.1 Joint design between the column and beam (ACI, 2019)	187
Figure 3.7.5.2. Reinforcement details at the beam-column joint.	187
Figure 3.7.7.1. Utilized formulas for development Length Calculations	191
Figure 4.1.1 Location map	196
Figure Figure 4.1.2 Satellite photo of existing lot	197
Figure 4.4.2 Shear wave velocity graph	204
Figure 4.9.1. Group Pile Foundation (Das & Sivakugan, 2019)	237

Figure 4.9.1.1. Length and Width of Group Piles and Cap (Das & Sivakugan, 2019)	239
Figure 4.9.1.2. Group Piles Layout	241
Figure 4.9.1.3. (a) 4x3, (b) 3x3, (c) 3x2 Group Piles	242
Figure 4.9.2.1 4x3 Piles Group Bearing Capacity Check	243
Figure 4.9.2.2 3x3 Pile Group Bearing Capacity	244
Figure 4.9.2.3 3x2 Pile Group Bearing Capacity	244
Figure 4.9.2.4 Group Pile Location in Plaxis	245
Figure 4.9.3.1 Response Spectra Plot	246
Figure 4.9.3.2 Fourier Spectra Plot	246
Figure 4.9.3.3 Arias Intensity Plot	247
Figure 4.9.3.4 Signal Plot	248
Figure 4.10.2.1 Results of Plaxis 3D analysis (a) general view (b) contour plot of vertical settlement	252
Figure 4.10.2.2 Total displacement in Plaxis 3D (a) before excavation and applying load (b) after excavation without applying load (c) after excavation with load	253
Figure 4.10.2.3 Properties of Pile (Embedded beam)	254
Figure 4.10.2.4 Properties of soil layer as example	255
Figure 4.10.3.1 Construction Procedure	257
Figure 4.10.3.2 Deformation Analysis in Plaxis 3D	258
Figure 4.10.3.3 Deformation Analysis in Plaxis 3D	260
Figure 4.10.3.4 Cross Section of Total Displacement in Plaxis 3D	260
Figure 4.11.1.1 Cross Section of Total Displacement in Plaxis 3D	264
Figure 4.11.1.2 Pile Deflection	266
Figure 4.12.1 Lateral Active Pressure	269
Figure 4.12.2 Lateral Passive Pressure	270
Figure 4.13.2.1 Cantilever Retaining Wall Design in 3D	275
Figure 4.13.2.2 Cantilever Retaining Wall Dimensions	275
Figure 4.13.2.3 Retaining Wall Overturning Failure Check	276
Figure 4.13.2.4 Retaining Wall Sliding and Bearing Capacity checks	276
Figure 4.15.1 Single pile reinforcement	280
Figure 4.15.2 I-1 Column Reinforcement of Foundation	281
Figure 5.3.1. Work breakdown structure from Primavera P6.	288
Figure 5.4.1. Gantt Chart (Microsoft Project).	290
Figure 5.5.1. Cost estimation by RSmeans (2018)	290
Figure 5.7.1 Quality Metrics.	310
Figure 5.9.2. Percentage of implemented measures per area.	321
Figure 5.9.3.1. Fall protection equipment.	321
Figure 5.9.3.2. Personal Protective Equipment.	321

Figure 5.9.5. OSHA Construction Safety Inspection Template. Adapted from Safetymint. OSHA Construction Safety Inspection Checklist.	323
Figure. 5.10. Construction Site Plan.	324
Figure 6.2.2. Bin placement for one of the floors	331
Figure 6.4.3. ISWM flowchart	335
Figure 9.2 Load Assignment for Case II Wind Load (Frame C)	344
Figure 9.3 Load Assignment for Case III Wind Load (Frame C)	345
Figure 9.4 Load Assignment for Case IV Wind Load (Frame C)	346

1. Introduction

The presented report contains a proposal of design and construction of the Khan Tengri Mall, which is a 4-story shopping center that will be situated at the following address: 130 W College St, Los Angeles, California, USA. The total floor area of the mall is 4,000 m² with a supermarket located on the first floor, a stock electronics and home appliance shop on the second floor, a food court on the third floor, and a cinema located on the fourth floor. In the report, the findings of a thorough literature survey are summarized, the design assumptions are outlined and the calculations that would justify the suggested shopping mall design are given. It includes the architectural, structural, geotechnical, environmental and construction management outcomes of the construction as well as the design of the project. These five core components are described as in Table 1.1 that shows the overview of the project disciplines.

Table 1.1 Job Distribution

Project Disciplines	Percentage Weight	Responsible Person
Structural and Architectural	35%	Adina Serikkyzy Rassul Ibrayev
Geotechnical	35%	Zhanbolat Sadratdinov Alibi Yeskazy Adelina Nurgaliyeva
Construction Management	15%	Baidibek Mussayev
Environmental	15%	Nurislam Nuraddinov

In order to serve the project requirements, the architectural and structural components will be subject to the application of the International Building Code (IBC), ASCE 7, and ACI 318-19. Structural analysis of the mall will be done on both the gravity and lateral load including dead, live, wind, and seismic loads. In addition, the construction of the Khan Tengri Mall is designed to meet the requirements of LEED (Leadership in Energy and Environmental Design) so that it is sustainable. Geotechnical study will consider the alternative foundation systems to decide the most appropriate system to be used on the four story commercial building. The project deliverables will also be identified within the construction management scope and an elaborate schedule will be prepared. On the environmental factor, the emphasis will be on the development

of an effective solid waste collection and management system that will be specific to the work of a modern shopping mall.

This project is meant to be a design of a contemporary multi-story shopping mall that focuses on safety, sustainability and functionality. This report is a detailed evaluation process that will guarantee that the facility is structurally sound, environmentally friendly and under proper management throughout its service life. The key objectives are:

- To conduct an elaborate feasibility study of the proposed development;
- To come up with an architectural design that can conform to the IBC standards and that balances beauty and functionality;
- In determining geotechnical conditions in order to arrive at the most suitable foundation system to use in the construction of the four story building;
- To create an effective solid waste collection and management strategy that fits the shopping mall functioning;
- In order to set up a construction management plan with scope, schedule and cost;
- To implement LEED regulations to realize sustainable and energy saving design.

2. Architectural Design

2.1 Overview

The four story commercial complex is the Khan Tengri Mall construction project, at 130 W College St, Los Angeles, California, USA. The basement level is an area that includes underground parking, and the rest of the site is outside recreational areas where visitors get to interact with one another. The mall has a total floor space of about 4,000 m². Every floor has a boutique retail store as well as a key anchor space: on the first floor, there is a supermarket, the second floor a large electronics and home appliances store, the third floor a food court with a variety of dining establishments, and the fourth floor a modern cinema complex. With this retail, entertainment and communal mix, the mall has become a multi-purpose venue in the locality. Figure 1 demonstrates the 3D site view that has been rendered of the proposed development.

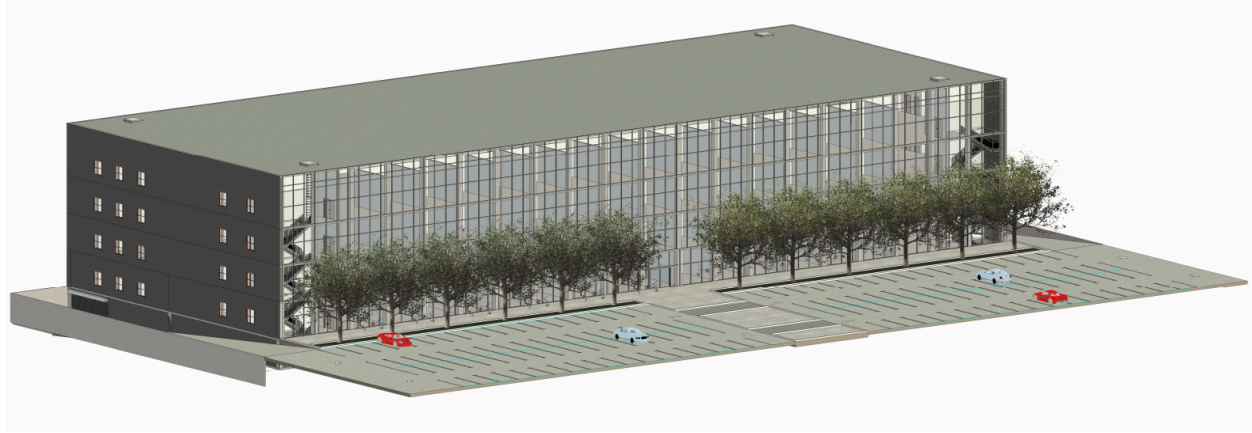


Figure 2.1.1 Rendered Site 3D View in Revit

2.2 Site Selection and Site Analysis

The proposed shopping mall will be built at 130 W College Street, Los Angeles, CA 90012, USA, which is the center of the Chinatown area. The place was selected with great care as it was constructed to be located centrally and culturally and at the same time be able to host a sizable commercial development. The fact that it is located close to downtown Los Angeles enables the project to blend with an already existing commercial and residential center besides helping in the current urban renewal of Chinatown.

The infrastructure in civil engineering at this location should be in a manner that considers the critical environmental hazards, which are typical of Southern California. The seismic activity and strong wind are the two most significant conditions that present significant challenges to the structural safety.

1. **High Seismic Zone.** The site has a short-period spectral response acceleration value of $S_s = 2.28\text{ g}$ as determined by the ASCE Hazard Tool and this value is significantly greater than the minimum of 1.0 g to establish high seismic zones. The value of this is equivalent to a 2,475 years period of return and the project falls under the category of extreme seismic risk. The design implication is no light matter: the structural system should have the capacity to dissipate seismic energy, collapse prevention, and integrity of a large-span multi-story commercial structure. The design that will not collapse in case of an earthquake is not just a regulation but a vital component that will determine the survival of the shopping mall in the long run (ASCE Hazard Tool, n.d.).

2. Strong Wind Zone. Besides seismic risks, there are also extreme wind conditions in the location. The ASCE Hazard Tool defines 700 year MRI (Mean Recurrence Interval) of 3 seconds of a gust wind speed of 95 mph (~ 42 m/s) to be used in this site. This is very high compared to the 40 m/s mark of the strong wind areas that should be well taken into account when developing structural and non-structural building elements like facades, roofing, and signage. In addition, the Los Angeles area is highly affected by the Santa Ana winds, which is a seasonal occurrence that entails strong downslope and offshore winds. These winds come when the dry air masses move down the Great Basin to the coast of Southern California becoming adiabatically heated as they move over the mountain ranges to the Los Angeles Basin. They are commonly linked to the very low humidity, unnatural heat waves, and high-risk fire. The winds in the Santa Ana events may reach speeds of more than 30 m/s, which introduces one more layer of risk to wind design aspects (Gershunov et al., 2021).

High seismic activity and a high level of the wind hazard are the reasons to emphasize the significance of good solutions through the engineering of the shopping mall. The structural systems should be engineered to withstand both the lateral seismic forces and the extreme wind loads and have to be functional and safe to the occupants. Furthermore, the site design should include external and internal parking areas, which should meet the standards of the urban requirements and should have adequate parking space to accommodate the visitors.

Civil engineering infrastructures tend to be exposed to the environmental conditions which can lead to corrosion and the construction project of the Khan Tengri Mall is not an exception. The level which is utilized as a parking garage is especially susceptible to moisture accumulation, automobile emissions and potential chloride pollution by road salt introduced to the basement due to cars. The exterior ground level recreational spaces also subject the structural sections and facade systems to the fluctuations in temperature and humidity as such typical of the Los Angeles weather. These factors can be the cause of corrosion by carbonation and chloride and corrosion within the life of the building that can determine the performance and longevity of reinforced concrete and steel parts. Corrosion analysis, i.e. identifying exposure classes,

durability standard and mitigation strategies, have still not been performed but would be addressed in the next semester as part of the on-going development of this project.

Historical background.

The location is located in Chinatown, one of the culturally diverse and historical parts of Los Angeles. Chinatown is a residential and commercial divide and is in the stages of redevelopment and urban renewal. The location chosen in this region will not only be a commercial success, but also a cultural inclusion and development of the city. The new shopping mall will enhance the presence of Chinatown as a regional destination and balance out its already existing cultural and economic environment.

Climate and site-specific variables.

Los Angeles has a Mediterranean climatic condition known to have hot, dry summers and milder winters that are wet. Such climatic conditions present one with a special risk and prospect of building design. The average monthly temperature is around 8 degC in the month of January and 29 degC in the month of August, which implies that the building should be ready during both cool and hot seasons of the year (WeatherSpark, 2025).

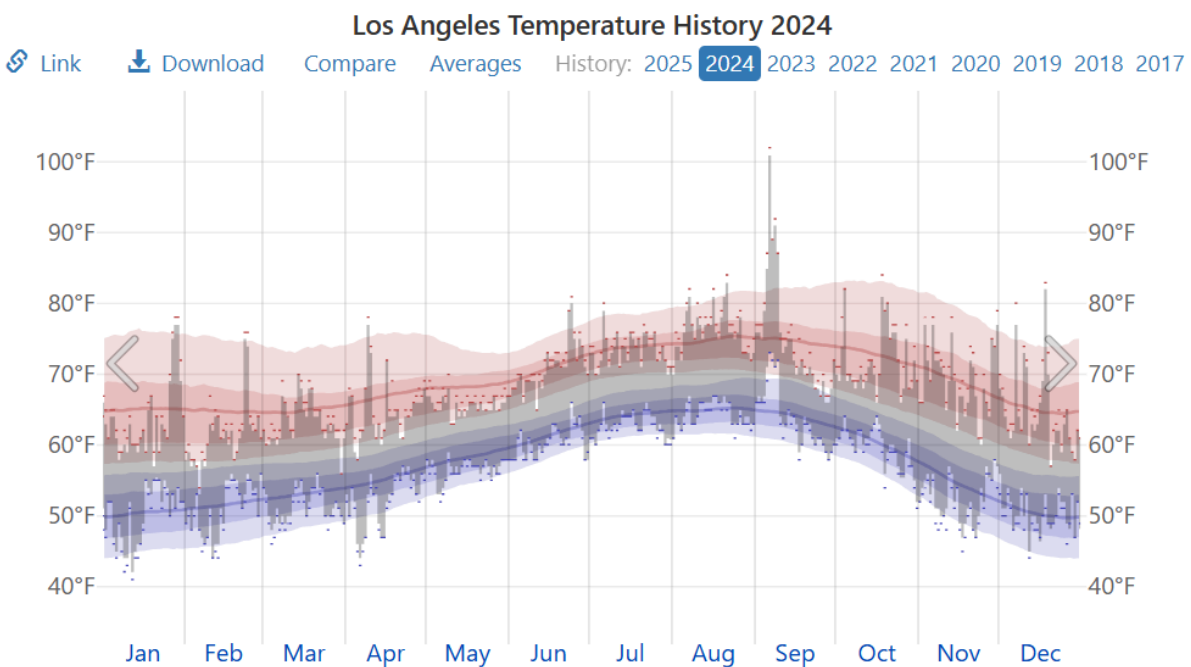


Figure 2.2.1. Los Angeles temperature history in 2024 (WeatherSpark)

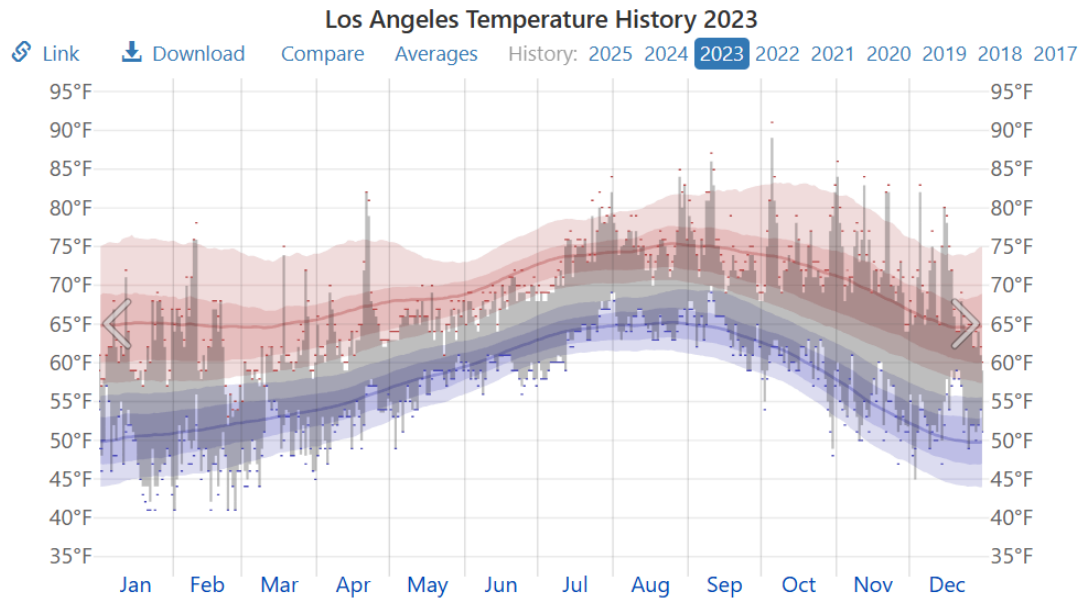


Figure 2.2.2. Los Angeles temperature history in 2023 (WeatherSpark)

The average annual precipitation is 380 mm and most of the rainfall is received during November and March (WRCC, n.d.). This is because the amount of rainfall received is relatively low compared to other big cities and its water management systems would have to focus on efficiency and collection of stormwater due to the short but hectic wet seasons. The climate even explains why energy-efficient HVAC, facade shading, and natural ventilation measures need to be incorporated in order to minimize the use of mechanical cooling.

LOS ANGELES DWTN USC CAMPUS, CALIFORNIA (045115)

Period of Record Monthly Climate Summary

Period of Record : 07/01/1877 to 06/09/2016

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual
Average Max. Temperature (F)	66.4	67.3	68.8	71.0	72.9	76.9	82.3	83.1	81.9	77.6	72.8	67.4	74.0
Average Min. Temperature (F)	48.3	49.5	51.1	53.5	56.5	59.7	63.2	63.8	62.6	58.7	53.3	49.1	55.8
Average Total Precipitation (in.)	3.20	3.38	2.40	1.01	0.25	0.06	0.01	0.05	0.27	0.48	1.25	2.41	14.77
Average Total SnowFall (in.)	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Average Snow Depth (in.)	0	0	0	0	0	0	0	0	0	0	0	0	0

Percent of possible observations for period of record.

Max. Temp.: 99.5% Min. Temp.: 99.5% Precipitation: 99.5% Snowfall: 41.6% Snow Depth: 41.6%

Figure 2.2.3. Monthly climate change in Los Angeles

Orientation and solar access.

The location is on College Street, with south and west exposures of the site as the primary.

Natural daylighting is a good opportunity that is created by this orientation but it also brings its challenges of heat gain especially during the summer period when the sun radiates the most. To eliminate this, facade treatments and shade devices will be required to minimize cooling loads. The shadow studies conducted on SunCalc indicate that overshadowing is insignificant due to the fact that neighboring buildings are within the size of 3 to 5 stories, which does not create significant problems to the site in terms of solar accessibility (SunCalc, 2025). This makes passive solar strategies of daylighting and solar energy capture viable designs.

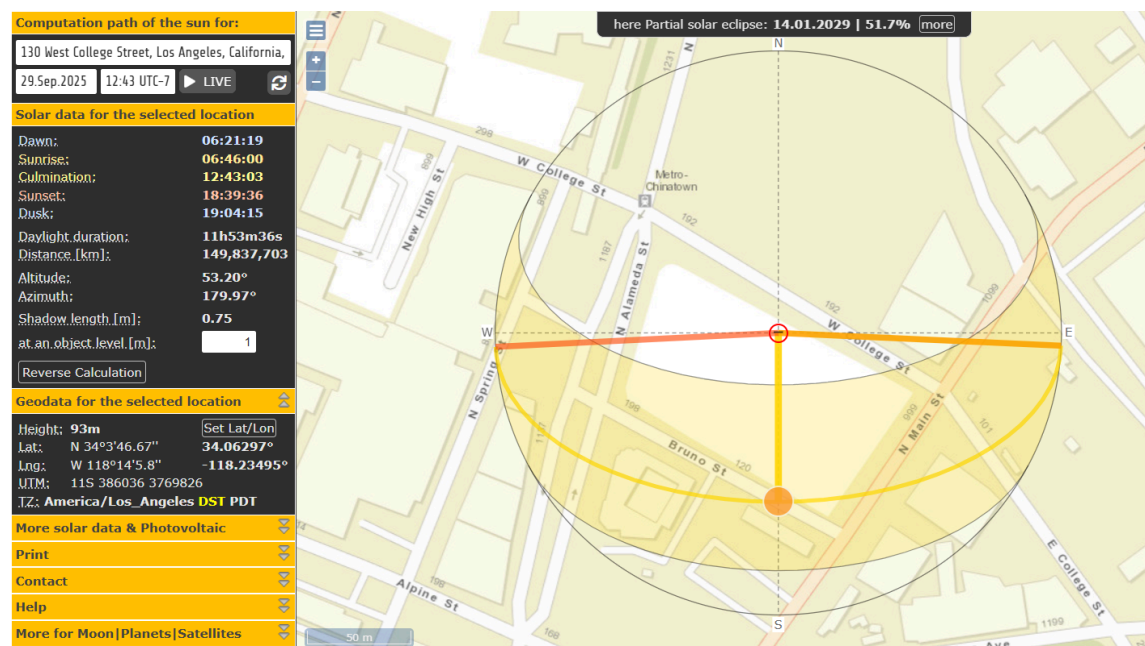


Figure 2.2.4. Sun direction

Cool breeze access.

Climate-responsive design should take into consideration the natural ventilation potential. The Los Angeles area has the predominant wind blowing southwesterly which is cool and may be used to advantage through building orientations and ventilation. Nevertheless, the Santa Ana winds make this trend more difficult as they are seasonal and come in with hot and dry air and severe winds of the northeast. Even though these winds are dangerous, they also highlight the significance of resilience-based building envelope and structural designs (Gershunov et al., 2021).



Figure 2.2.5. The wind directions on the site

Views.

The site has had an advantage in terms of visibility in urban terms, both of the immediate Chinatown streetscape, and a wider view of downtown Los Angeles. Such perceptions make the site more appealing, and this is in line with the idea that it is a shopping mall and both beauty and visibility add to the business.



Figure 2.2.6. View to downtown area from the site (Google earth)

Site survey.

1. Overshadowing: low, because the nearby buildings were medium in height.
2. Slope: The site is on flat ground and there will be no extensive grading to do and the foundation design will be easy.



Figure 2.2.7. Site shadowing view

Access and transport.

The facility is also very reachable through roads and buses. The large streets to which it is bordered such as the College Street, Main Street, and Spring Street facilitate access to automobiles. Notably, the site can be reached within 200 m of Chinatown Metro Station that belongs to the Los Angeles Metro A Line and is also served by several buses and bike trails. This multimodal availability can sustainability, as well as integrate urban attributes (Google Maps, 2025).

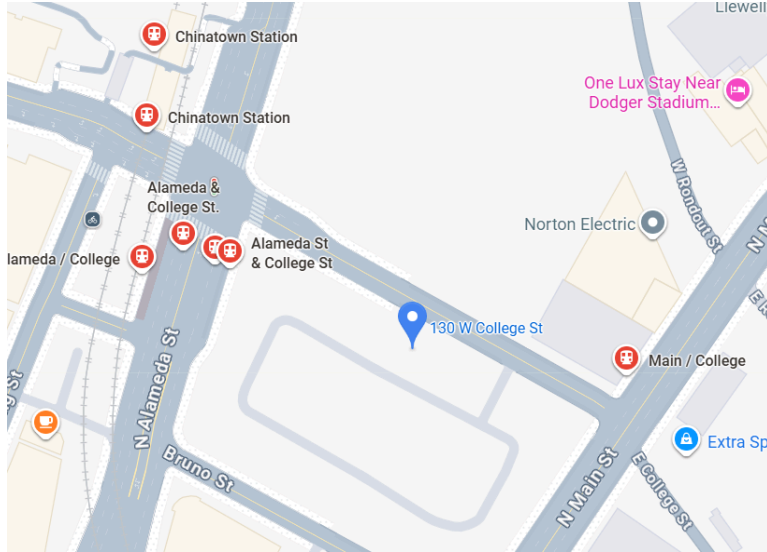


Figure 2.2.8. Public transportation map (Google Maps)

Bushfire risks.

Though Los Angeles County is characterized by high risks of wildfires during the Santa Ana wind events, the LAFD Fire Zone Map reveals that the project location does not occupy a specified zone of high level of fire hazard severity (LAFD, 2025). This will mitigate the risk of direct bush fires but not the indirect implications of poor air quality, emergency access requirements or regional inconvenience during large scale fire incidents.

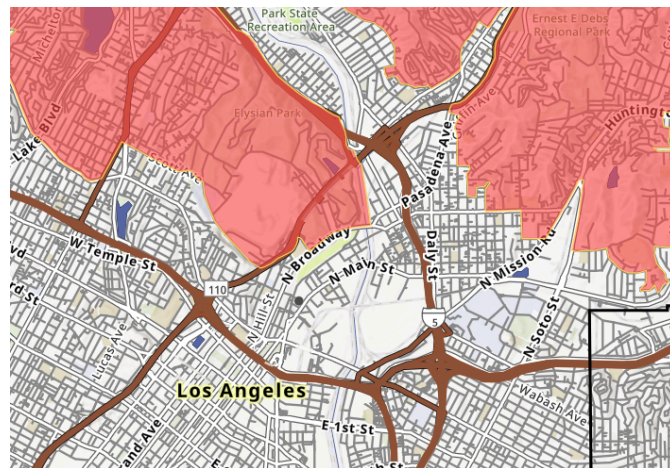
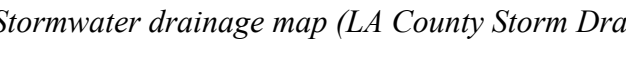


Figure 2.2.9. Bushfire risks (Los Angeles Fire Department, 2025)

Stormwater drainage.

Los Angeles County Storm Drain System has the existing infrastructure of stormwater management in the area, such as catch basins and discharge facilities at the site (LA County

A map of the Mission Junction area in San Francisco. The map shows several streets including Pacific Avenue, Mission Avenue, and W. Embarcadero. A green area is labeled 'Mission Junction'. A blue line indicates a route or boundary. The map is part of a larger document discussing the impact of the 2008 financial crisis on the local economy.



Water supply: Municipal water: Los Angeles Department of Water and Power

2. See

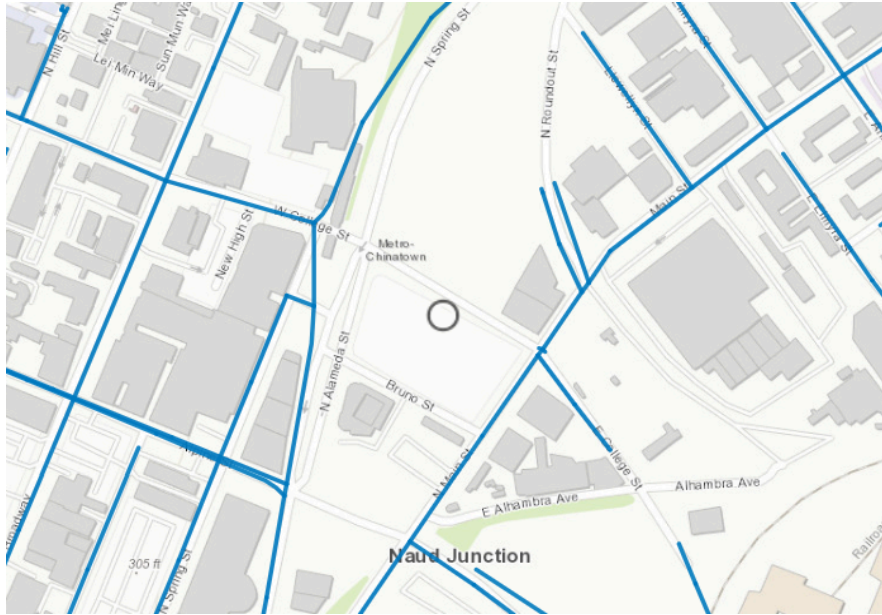


Figure 2.2.11. Sewer system map (Los Angeles GeoHub, 2023)

2.3 Design Concept

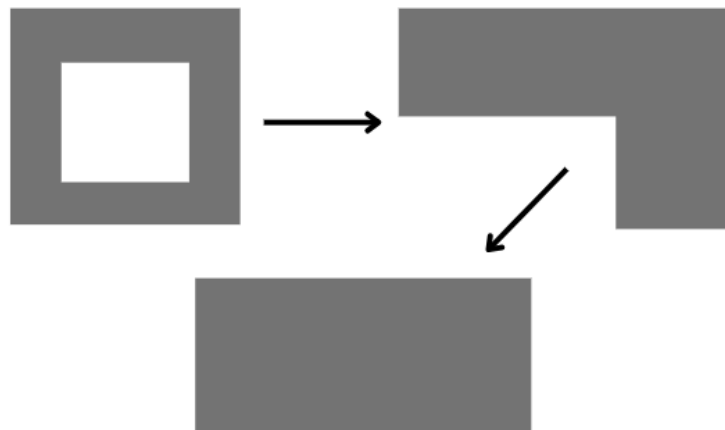


Figure 2.3.1 First and last versions of the shape of our building.

In the beginning, our team wanted to design a building that was distinct, attractive, and welcoming, and we have intentionally shunned the traditional architectural designs. Nevertheless, we changed our mindset to functionality after a long period of deliberation, where

we decided to come up with a design that is aesthetically pleasing besides having adequate space to fit all the retail units in the shopping mall.

Moreover, in choosing the site of the project, we put into consideration some critical issues, which included accessibility of transport, central and filled location, and sufficient terrain to accommodate huge tenant areas.

2.4 Geometry Design and Site Layout

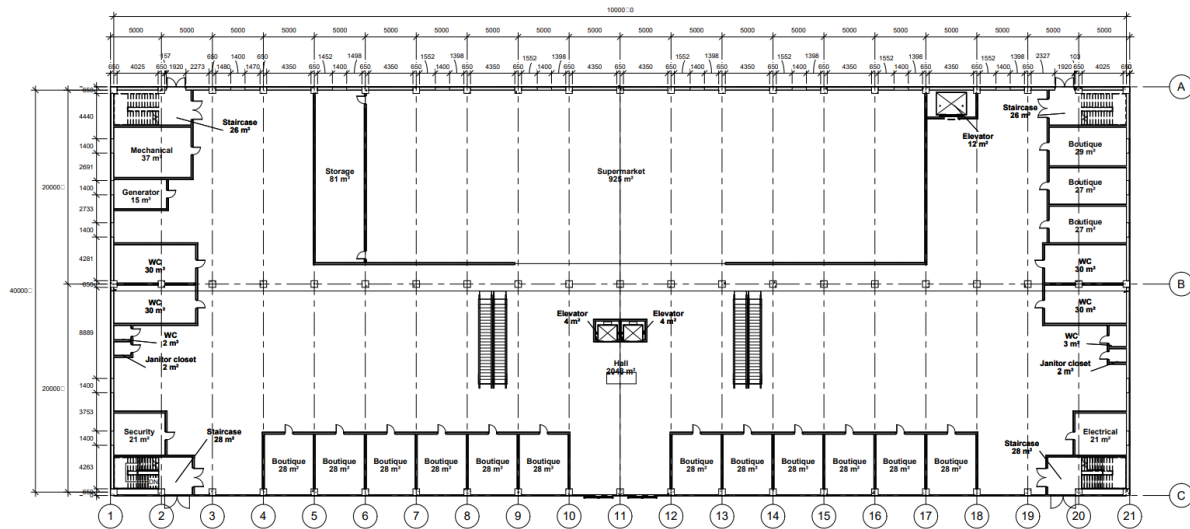


Figure 2.4.1 Floor Plan for Level 1

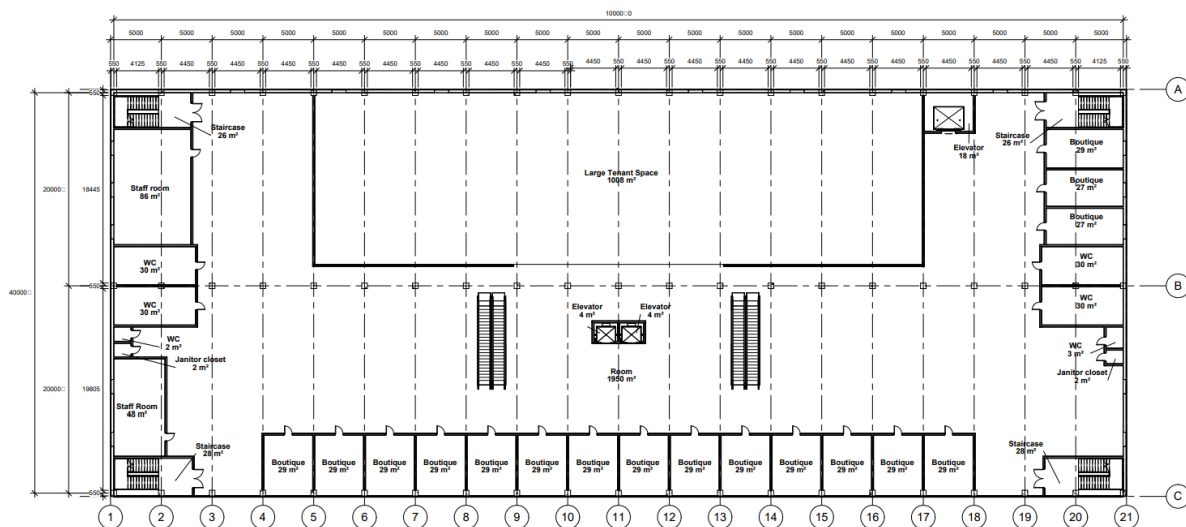


Figure 2.4.2 Floor Plan for Level 2

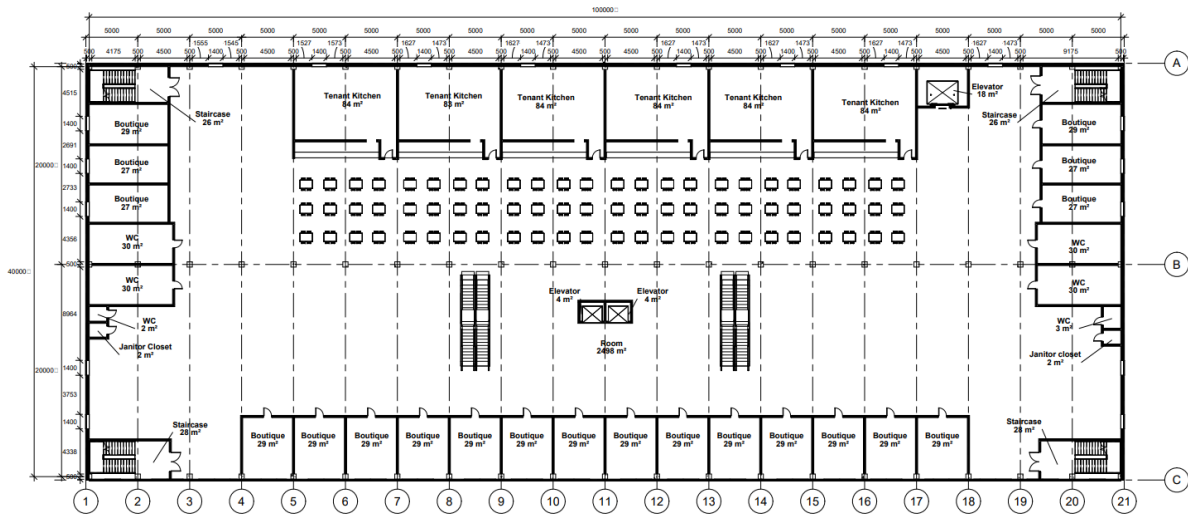


Figure 2.4.3 Floor Plan for Level 3

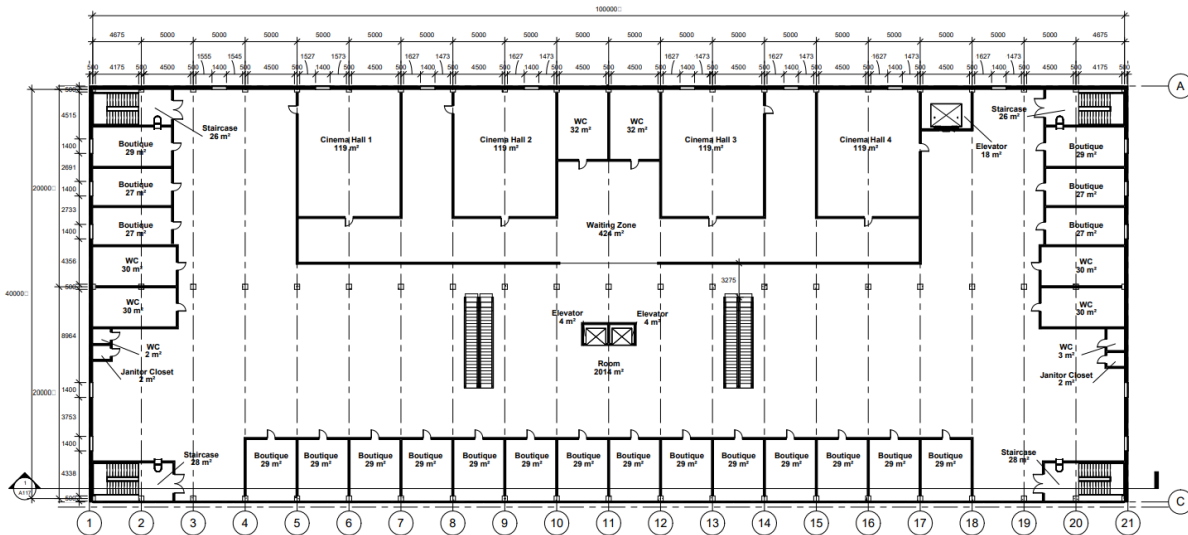


Figure 2.4.4 Floor Plan for Level 4

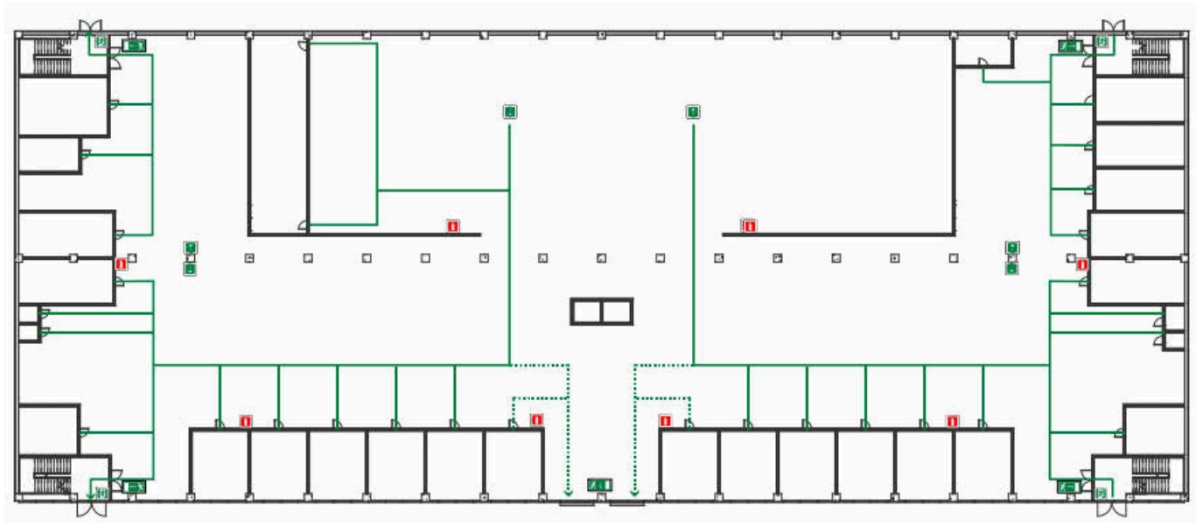


Figure 2.4.5 Emergency Plan for Level 1

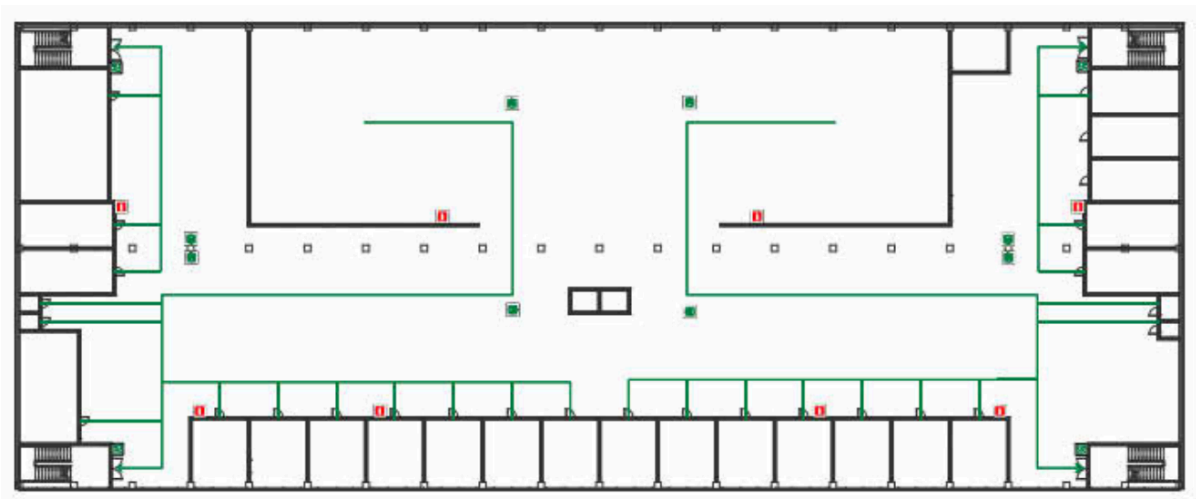


Figure 2.4.6 Emergency Plan for Level 2

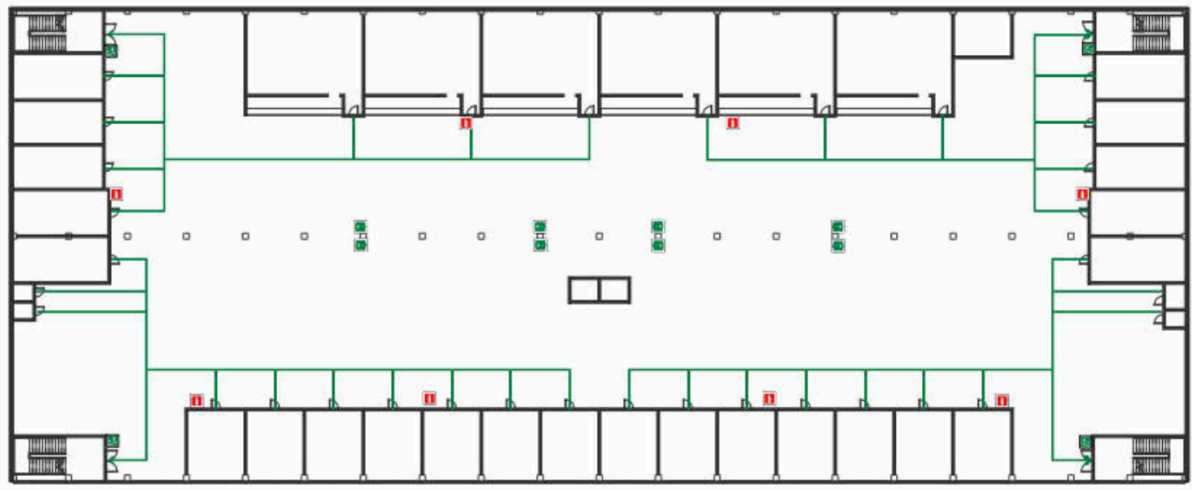


Figure 2.4.7 Emergency Plan for Level 3

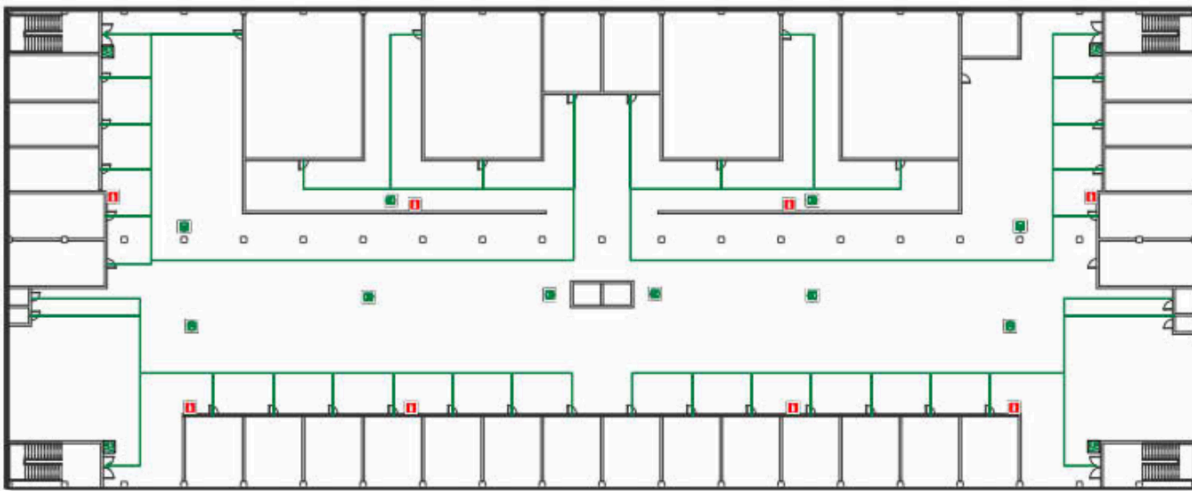


Figure 2.4.8 Emergency Plan for Level 4

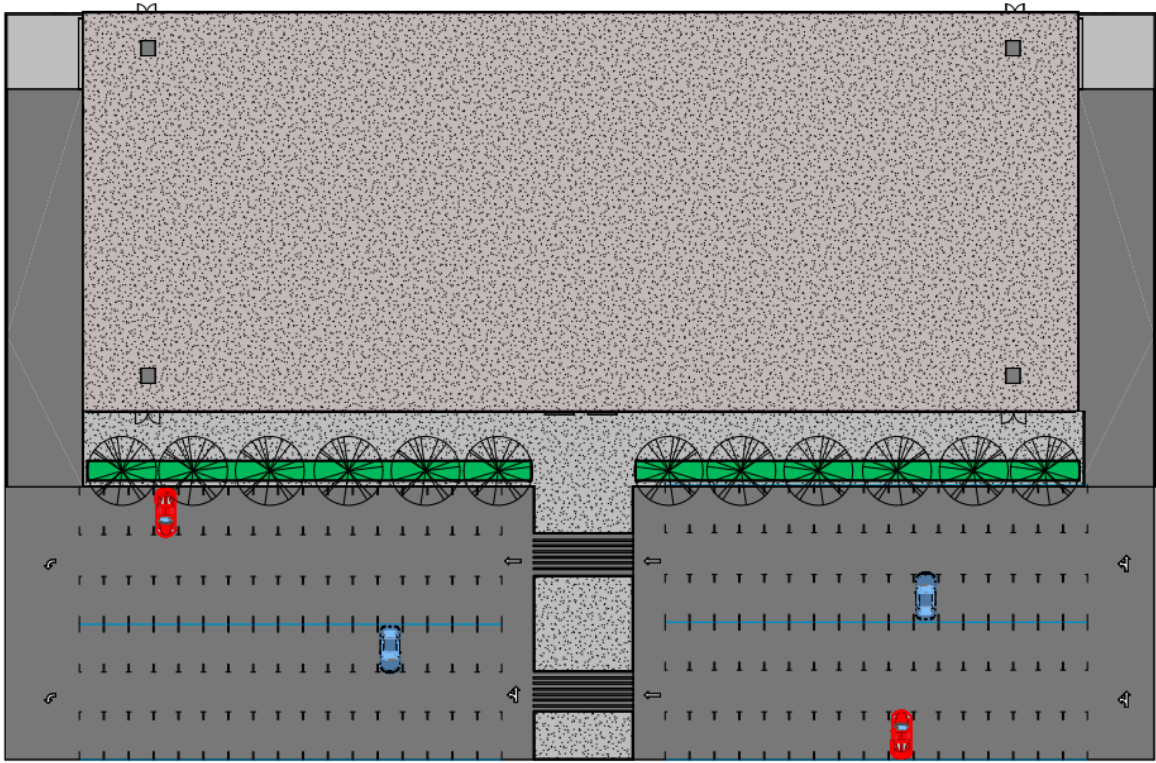


Figure 2.4.9. Site Layout

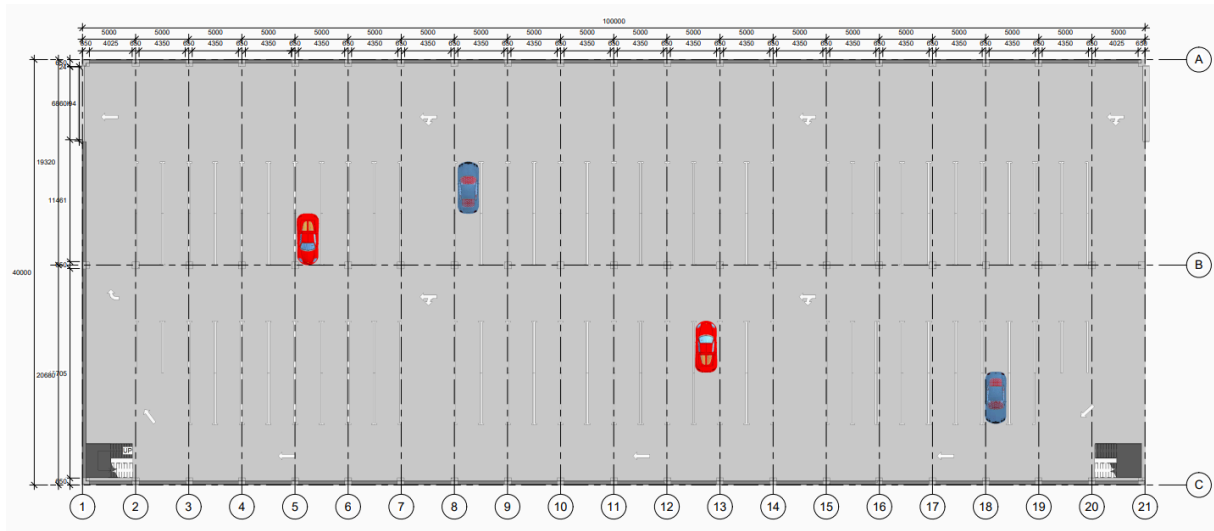


Figure 2.4.10. Basement Plan (Underground Parking)

Parking capacity

The total number of parking spaces was calculated by first determining the usable area after deducting circulation losses. The general formula is

$$A_{usable} = A_{gross} \times (1 - L)$$

where A_{gross} is the gross floor area and L is the loop/circulation loss factor. For the underground garage, with $A_{gross} = 4000 \text{ m}^2$ and $L = 0.15$, we obtain

$$A_{surf,usable} = 4000 * 0.85 = 3400 \text{ m}^2$$

For the surface lot, $A_{gross} = 1500 \text{ m}^2$ and $L = 0.15$, the calculation is

$$A_{surf,usable} = 2000 * 0.85 = 1700 \text{ m}^2$$

The number of spaces is then given by

$$N = \frac{A_{usable}}{A_{space}}$$

with $A_{space} = 12.5 \text{ m}^2$ per stall including a share of aisles. Thus, underground capacity is

$$N_{und} = \frac{3400}{12.5} = 272 \text{ spaces}$$

while surface capacity is

$$N_{sur} = \frac{1700}{12.5} = 136 \text{ spaces}$$

Finally, the total is obtained by summation:

$$N_{total} = N_{und} + N_{sur} = 272 + 136 = 408 \text{ spaces}$$

In addition, the project will rely on the availability of nearby off-site parking facilities, which further increases the effective capacity and ensures full accommodation of visitor and tenant parking needs.



Figure 2.4.11 Side View Plan



Figure 2.4.12 Section Cut 1

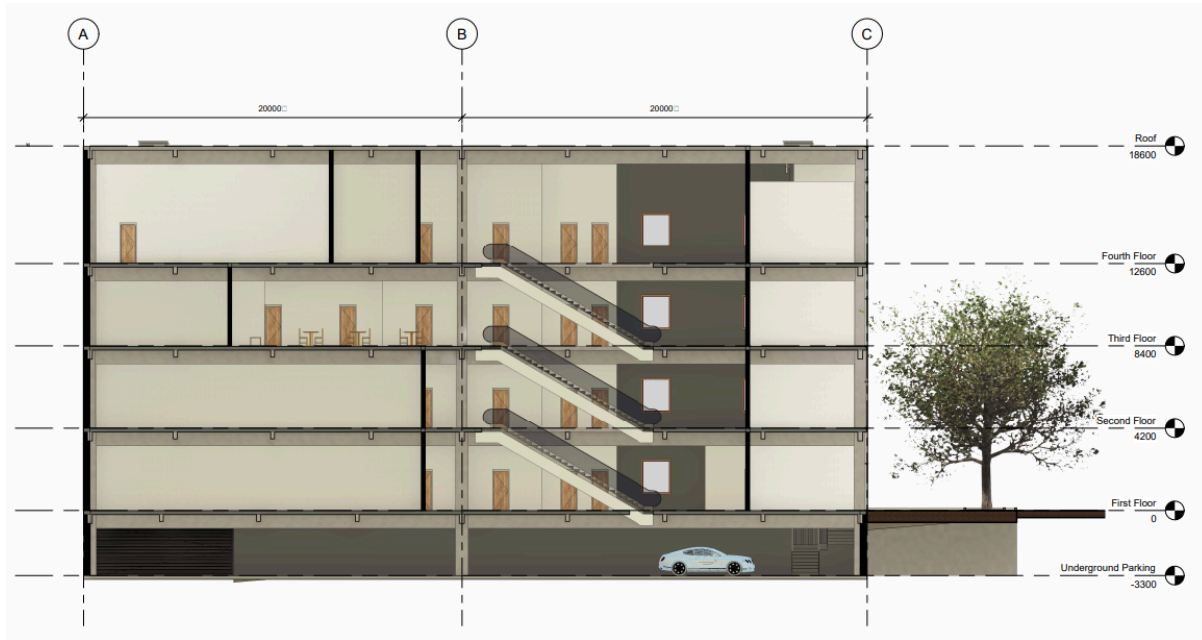


Figure 2.4.13 Section Cut 2

2.5 Leadership in Energy and Environmental Design (LEED) for Green Building Design, Construction, Operations and Performance

LEED (Leadership in Energy and Environmental Design) is the most popular world-wide green building rating system. It was developed by the U.S. Green Building Council (USGBC), and it offers a blueprint of healthy, highly efficient and cost-effective green buildings. In this project, the 4-story shopping mall in Los Angeles, CA was to be designed to be highly sustainable, targeting to minimize carbon emissions, resource conservation and health promotion of visitors and employees. Project certification is on the Gold level. According to the credits recorded in the scorecard below, the project scored 77 points in total which is well within the range of the Gold certification (60-79 points).



LEED Scorecard

Gold 77/110

▼ INTEGRATIVE PROCESS	0 / 1	
▼ LOCATION AND TRANSPORTATION	13 / 16	
▼ SUSTAINABLE SITES	8 / 10	
▼ WATER EFFICIENCY	6 / 11	
▼ ENERGY & ATMOSPHERE	25 / 33	
▼ MATERIALS & RESOURCES	9 / 13	
▼ INDOOR ENVIRONMENTAL QUALITY	12 / 16	
▼ INNOVATION	0 / 6	
▼ REGIONAL PRIORITY CREDITS	4 / 4	

Figure 2.5.1. LEED Scorecard


Location and Transportation (13/16)


☒ **Sensitive Land Protection**
1pt This credit is intended to cultivate community resilience, avoid the development of environmentally sensitive lands that provide critical ecosystem services and reduce the environmental impact from the location of a building on a site. [Learn more about this credit.](#)

☐ **High Priority Site and Equitable Development**
2pts This credit is intended to build the economic and social vitality of communities, encourage project location in areas with development constraints and promote the ecological, cultural, and community health of the surrounding area while understanding the needs and goals of existing residents and businesses. [Learn more about this credit.](#)

☒ **Surrounding Density and Diverse Uses**
5pts This credit is intended to conserve land and protect farmland and wildlife habitat by encouraging development in areas with existing infrastructure. It is also intended to support neighborhood and local economies, promote walkability, and low or no carbon transportation, and reduce vehicle distance traveled for all. Furthermore, it is intended to improve public health by encouraging daily physical activity. [Learn more about this credit.](#)

☒ **Access to Quality Transit**
5pts This credit is intended to encourage development in locations shown to have multimodal transportation choices or otherwise reduced motor vehicle use, thereby reducing greenhouse gas emissions, air pollution, and other environmental and public health harms associated with motor vehicle use. [Learn more about this credit.](#)

☒ **Bicycle Facilities**
1pt This credit is intended to promote bicycling and transportation efficiency and reduce vehicle distance traveled. It is also intended to improve public health by encouraging utilitarian and recreational physical activity. [Learn more about this credit.](#)

☒ **Reduced Parking Footprint**
1pt This credit is intended to minimize the environmental harms associated with parking facilities, including automobile dependence, land consumption, and rainwater runoff. [Learn more about this credit.](#)

☐ **Electric Vehicles**
1pt This credit is intended to reduce pollution by promoting alternatives to conventionally fueled automobiles. [Learn more about this credit.](#)

Figure 2.5.2. L&T Score

The project scored 13 out of 16 points in the Location and Transportation category.

- Sensitive Land Protection (1 pt): The project was built on an already developed land and it did not encroach sensitive habitats and farm land as stated in the checklist.
- Surrounding Density and Diverse Uses (5 pts): The mall lies within a dense urban environment in Los Angeles, with an existing infrastructure and access to various services, which promotes walking.
- Availability of Quality Transit (5 pts): LA public transportation networks serve the site well, minimizing the use of personal vehicles by shoppers and employees.

- Bicycle Facilities & Reduced Parking Footprint (2 pts): The underground parking design reduces parking footprint on the surface to encourage alternative transportation and there are specific bicycle storage and changing facilities.

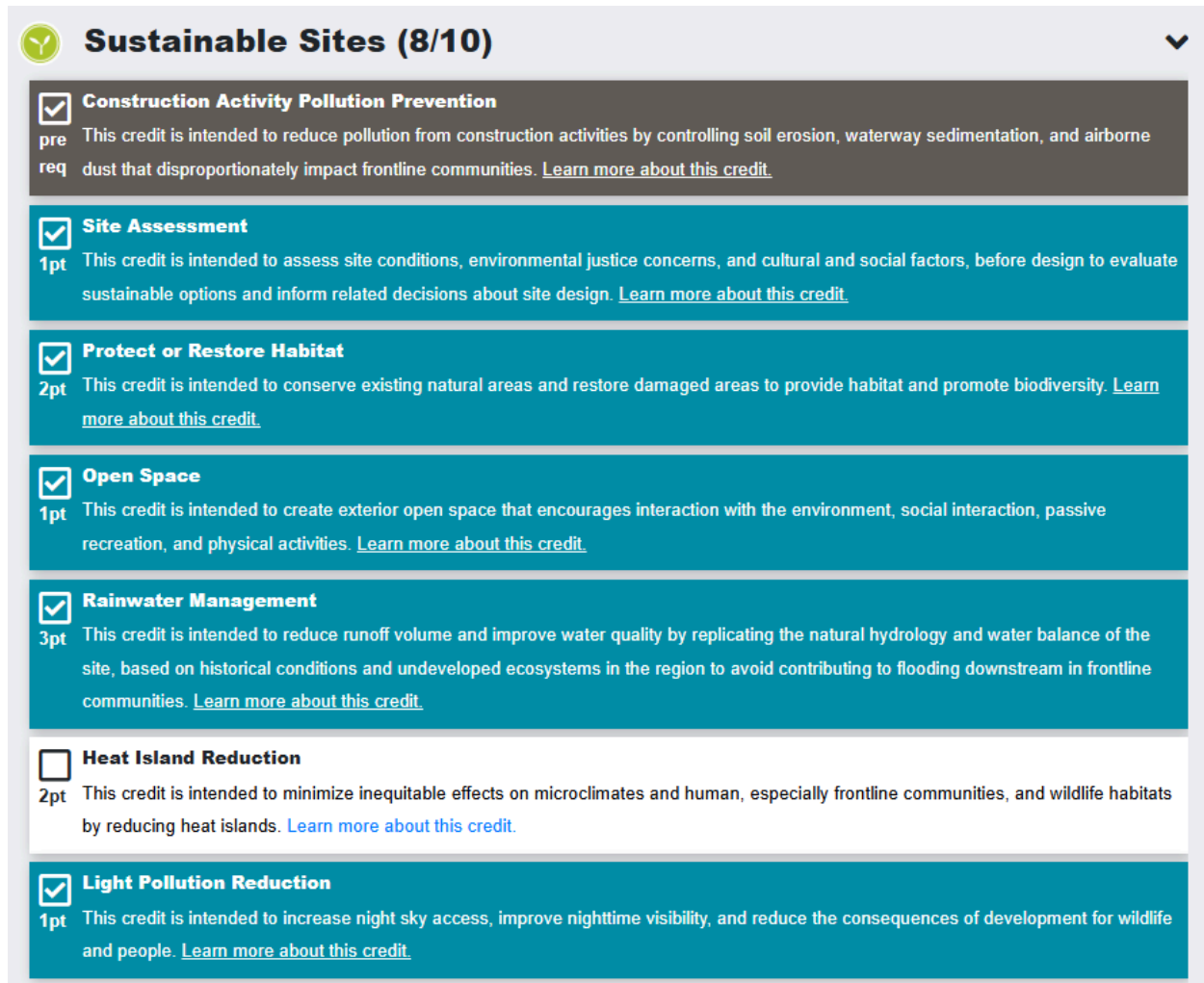


Figure 2.5.3. Sustainable Sites Score

The project has obtained 8 out of 10 points in the category of Sustainable Sites which is aimed at reducing the impact of the building on the local ecosystem.

- Construction Activity Pollution Prevention (Prerequisite): Soil erosion and airborne dust were contained by strict measures that were taken during construction to prevent the pollution of the nearby LA neighborhood.

- Heat Island Reduction (2 pts): The project employs high-reflectance roofing material and a large area of parking is located underground. This minimizes heat uptake common in big shopping malls, countering the urban heat island effect.
- Rainwater Management (3 pts): Stormwater management systems are provided in the site design to collect and treat runoff to simulate natural hydrology and avoid flooding in the urban drainage system.
- Light Pollution Reduction (1 pt): Exterior lighting is planned to be designed to reduce backlight, uplight, and glare, as well as to conserve the night sky and to minimize light intrusion into adjacent residential zones.


Water Efficiency (6/11)


☒ **Outdoor Water Use Reduction**
pre This credit is intended to reduce outdoor potable water consumption and preserve no and low-cost potable water resources. [Learn more](#)
req [about this credit.](#)

☒ **Indoor Water Use Reduction**
pre This credit is intended to reduce indoor potable water consumption and preserve no and low cost potable water resources. [Learn more](#)
req [about this credit.](#)

☒ **Building-Level Water Metering**
pre This credit is intended to conserve low cost potable water resources and support water management and identify opportunities for
req additional water savings by tracking water consumption. [Learn more about this credit.](#)

☐ **Outdoor Water Use Reduction**
2pt This credit is intended to reduce outdoor potable water consumption and preserve no and low-cost potable water resources. [Learn more](#)
[about this credit.](#)

☒ **Indoor Water Use Reduction**
6pt This credit is intended to reduce indoor potable water consumption and preserve no and low cost potable water resources. [Learn more](#)
[about this credit.](#)

☐ **Optimize Process Water Use**
2pt This credit is intended to conserve low cost potable water resources used for mechanical processes while controlling corrosion and scale
in the condenser water system. [Learn more about this credit.](#)

☐ **Water Metering**
1pt This credit is intended to conserve low cost potable water resources and support water management and identify opportunities for
additional water savings by tracking water consumption. [Learn more about this credit.](#)

Figure 2.5.4. Water Efficiency Score

The project scored 6 out of 11 points in Water Efficiency through emphasizing on the minimization of potable water consumption.

- Reduce Indoor Water Use: The choice of high-efficiency plumbing fixtures was made in all the mall restrooms and food court areas. This involves low flow faucets, dual flush toilets,

and water efficient urinals which will reduce the operational water base of the building significantly.

- Outdoor Water Use Reduction: Landscaping employs the use of drought-tolerant native plants that are adapted to the Southern California climate, and do not require permanent irrigation systems.

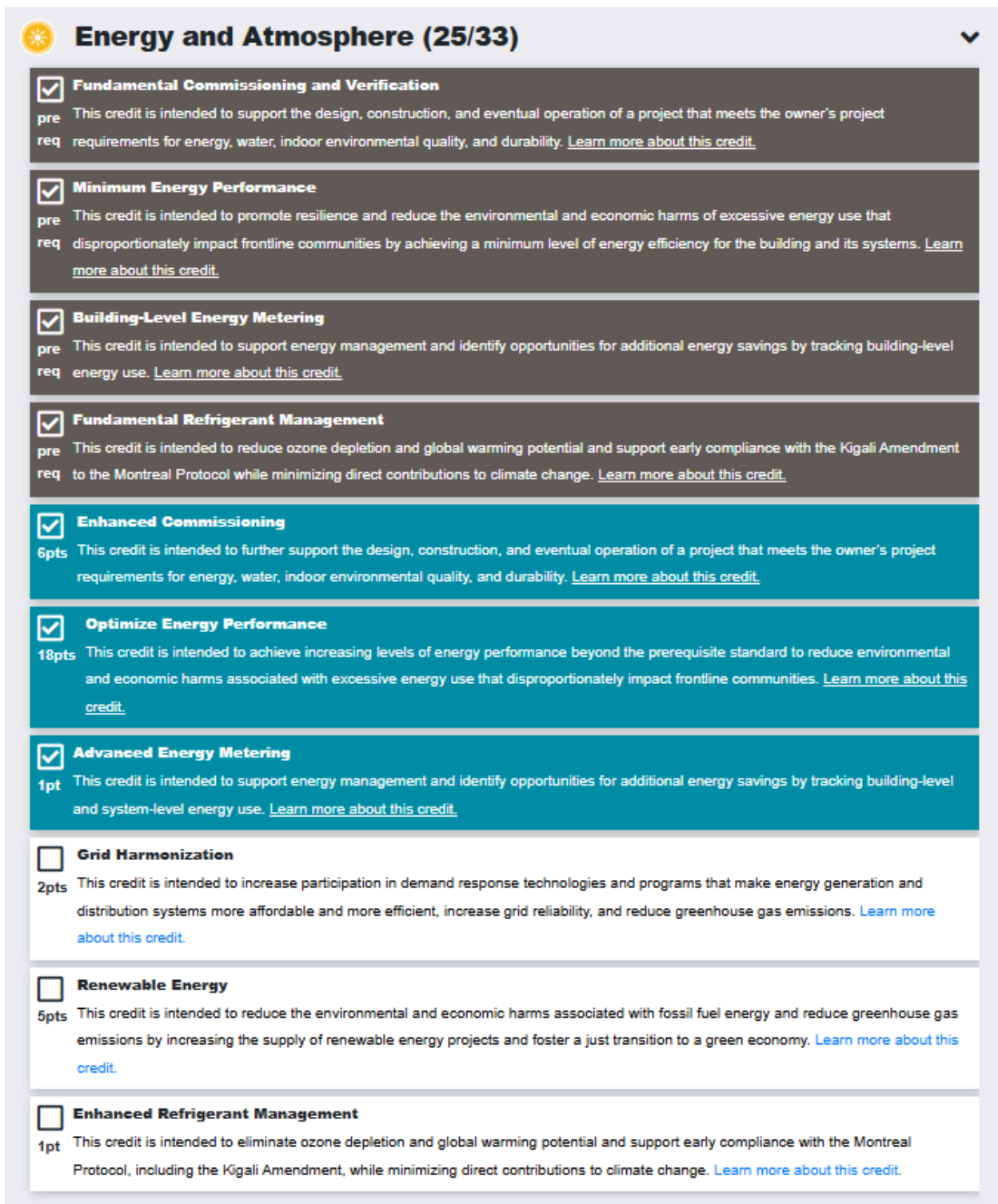


Figure 2.5.5. E&A Score

This category demonstrate the project's strongest performance, scoring 25 out of 33 points.

- Optimize Energy Performance (18 pts): The shopping mall has been equipped with a high-performance building envelope and state-of-the-art HVAC system. Energy modeling proves to be very saving in contrast to the ASHRAE baseline, which is very essential in the case of the retail spaces that are energy intensive.
- Basic and Advanced Commissioning (6 pts): A commissioning authority was hired to ensure that all energy systems (HVAC, lighting and controls) were installed and adjusted to perform efficiently as they were intended.
- Advanced Energy Metering (1 pt): The facility has advanced meters to monitor energy use in real-time so that facility managers can detect and rectify inefficiencies as they run the facility.


Materials and Resources (9/13)


☒ **Storage and Collection of Recyclables**
pre req This credit is intended to reduce the disproportionate burden of landfills and incinerators that is generated by building occupants' waste hauled to and disposed of in landfills and incinerators through reduction, reuse and recycling service and education, and to conserve natural resources for future generations. [Learn more about this credit.](#)

☒ **Building Life-Cycle Impact Reduction**
5pts This credit is intended to encourage adaptive reuse and optimize the environmental performance of products and materials. [Learn more about this credit.](#)

☐ **Environmental Product Declarations**
2pts This credit is intended to encourage the use of products and materials for which life-cycle information is available and that have environmentally, economically, and socially preferable life-cycle impacts. It is also intended to reward project teams for selecting products from manufacturers who have verified improved environmental life-cycle impacts. [Learn more about this credit.](#)

☒ **Sourcing of Raw Materials**
2pts This credit is intended to encourage the use of products and materials for which life cycle information is available and that have environmentally, economically, and socially preferable life cycle impacts. It is also intended to reward project teams for selecting products verified to have been extracted or sourced in a responsible manner. [Learn more about this credit.](#)

☐ **Material Ingredients**
2pts This credit is intended to encourage the use of products and materials for which life-cycle information is available and that have environmentally, economically, and socially preferable life-cycle impacts. It is also intended to reward project teams for selecting products for which the chemical ingredients in the product are inventoried using an accepted methodology and for selecting products verified to minimize the use and generation of harmful substances. Furthermore, it is intended to reward raw material manufacturers who produce products verified to have improved life-cycle impacts. [Learn more about this credit.](#)

☒ **Construction and Demolition Waste Management**
2pts This credit is intended to reduce construction and demolition waste disposed of in landfills and incineration facilities through waste prevention and by reusing, recovering, and recycling materials, and conserving resources for future generations. Furthermore, it is intended to delay the need for new landfill facilities that are often located in frontline communities and create green jobs and materials markets for building construction services. [Learn more about this credit.](#)

Figure 2.5.6. Materials & Resources Score

The project scored 9 out of 13 points by focusing on responsible material sourcing and waste reduction.

- Construction and Demolition Waste Management (2 pts): An extensive waste management strategy was adopted, which redirected a large proportion of construction waste (concrete, steel, wood) off landfills, by recycling and reusing.
- Environmental Product Declarations (EPDs) (2 pts): Materials used were chosen based on the manufacturers that offer EPDs, which provide transparency on the environmental life-cycle impact of the building materials.
- Sourcing of Raw Materials (2 pts): A preference was made towards sourcing materials in a responsible and local way where possible, with an aim of reducing transportation emissions.

Indoor Environmental Quality (12/16)

☒	Minimum Indoor Air Quality Performance	pre req	This credit is intended to contribute to the comfort and well-being of all building occupants by establishing minimum standards for indoor air quality (IAQ). Learn more about this credit.
☒	Environmental Tobacco Smoke Control	pre req	This credit is intended to prevent or minimize exposure of building occupants, indoor surfaces, and ventilation air distribution systems to environmental tobacco smoke. Learn more about this credit.
☒	Enhanced Indoor Air Quality Strategies	2pts	This credit is intended to promote occupants' comfort, well-being, and productivity by improving indoor air quality. Learn more about this credit.
☒	Low-Emitting Materials	3pts	This credit is intended to reduce concentrations of chemical contaminants that can damage air quality and the environment, and to protect the health, productivity, and comfort of installers and building occupants. Learn more about this credit.
☒	Construction Indoor Air Quality Management Plan	1pt	This credit is intended to promote the well-being of construction workers and building occupants by minimizing indoor air quality problems associated with construction and renovation. Learn more about this credit.
☐	Indoor Air Quality Assessment	2pts	This credit is intended to establish better quality indoor air in the building after construction and during occupancy to protect human health, productivity, and well-being. Learn more about this credit.
☒	Thermal Comfort	1pt	This credit is intended to promote occupants' productivity, comfort, and well-being by providing quality thermal comfort. Learn more about this credit.
☒	Interior Lighting	2pts	This credit is intended to promote occupants' productivity, comfort, and well-being by providing high-quality lighting. Learn more about this credit.
☒	Daylight	3pts	This credit is intended to connect building occupants with the outdoors, reinforce circadian rhythms, and reduce the use of electrical lighting by introducing daylight into the space. Learn more about this credit.
☐	Quality Views	1pt	This credit is intended to give building occupants a connection to the natural outdoor environment by providing quality views. Learn more about this credit.
☐	Acoustic Performance	1pt	This credit is intended to provide workspaces and classrooms that promote occupants' well-being, productivity, and communications through effective acoustic design. Learn more about this credit.

Figure 2.5.7. Indoor Environmental Quality Score

The project received 12 out of 16 points, putting emphasis on the comfort and health of the shoppers and tenants.

- Low-Emitting Materials (3 pts): Paints, coating, adhesives and sealants applied to the interior are kept to strict standards of Volatile Organic Compounds (VOCs), which ensures high indoor air quality.
- Interior Lighting (2 pts): The mall is fitted with LED lighting that has advanced controls to offer high-quality lighting and minimize the energy loads.
- Thermal Comfort (1 pt): The HVAC system will be set to offer individual thermal comfort control to certain areas, so that even during the Los Angeles heat, a comfortable shopping experience can be achieved.
- Daylight (3 pts): The architectural design includes the use of skylights and massive windows to bring the natural daylight into the common areas and atrium, to connect the visitors with the outdoors and minimize the use of artificial lighting in the daytime.

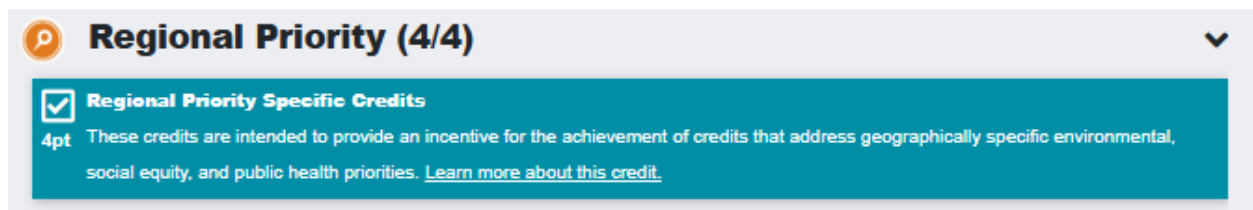


Figure 2.5.8 Regional Priority

The Life-365 is a tangible service life forecasting software which assists in estimating service life of concrete constructions that are exposed to chloride conditions. Parameters of costs were considered as the average cost of Los Angeles, California. The project environments were set to US Units with the concentration of chloride in the form of the percentage of weight concentration. Cost parameters utilized included: concrete, black steel, epoxy coated steel, and stainless steel, which were 115\$/cub.yd, 0.65\$/lb, 0.85\$/lb and 3.50\$/lb respectively. In the case of barrier systems, the prices of membrane, sealer and inhibitor were 4.50\$/ sq.ft, 0.85\$/ sq.ft and 7.00\$/ gal respectively. In case of repair, the cost was determined as 50.00\$/ sq.ft with 10% repair area and fixed repair period of 10 years.

Cost Parameters		
Concrete & Steel	Barriers & Inhibitors	Repairs
Concrete (\$/cub. yd.) *	Membrane (\$/sq. ft.) *	Repair (\$/sq. ft.) *
115,00	4,50	50,00
Black Steel (\$/lb) *	Sealer (\$/sq. ft.) *	Area to repair (%) *
0,65	0,85	10,00
Epoxy Coated Steel (\$/lb) *	Inhibitor (\$/gal) *	Fixed repair intervals (yrs) *
0,85	7,00	10
Stainless Steel (\$/lb) *		
3,50		

Figure 2.5.9. Cost Parameters window from Life-365

The type of the structure, Slabs and Walls (1-D) was chosen, its slab thickness is 7.87 in (200 mm), depth of reinforcement is 2.00 in, and total area is 43,055.64 sq.ft (4,000 m²).

Select Structure Type & Dimensions	
Type Of Structure *	slabs and walls (1-D) ▾
Thickness (in) *	7,87
Reinf. Depth (in) *	2,00
Area (sq. ft.) *	43055,64
Volume Of Concrete : 1045.83 cub. yd. Chloride concentration units : % wt. conc.	
	
Define Economic Parameters	
Base Year (e.g. 2026) *	Analysis Period (yrs) *
2024	75
Inflation Rate (%) *	Real Discount Rate (%) *
3,00	3,00

Figure 2.5.10. Structure Type & Dimensions window from Life-365

As the building has an underground parking structure with depth of about 23 km per the ocean, the exposure condition was established to Parking Garages, having a surface chloride concentration 0.6% wt. conc. This is the typical assumption and most conservative assumption of parking structures; vehicles constantly deposit road chlorides on the slab surface. The default weather settings of Los Angeles, California were used and the monthly weather pattern was represented as under:

Temperature Cycle (°F) (Automatically set)

Month	Temperature (°F)
January	56.8
February	57.6
March	57.9
April	60.1
May	62.8
June	65.7
July	69.1
August	70.5
September	70.0
October	66.7
November	61.5
December	56.8

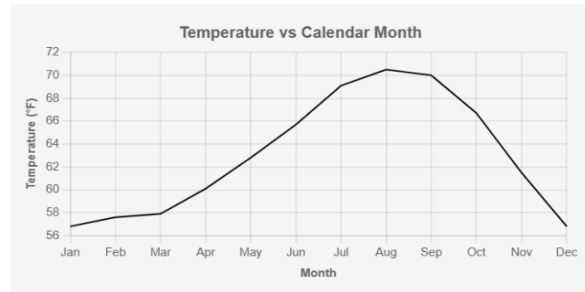


Figure 2.5.11. Temperature Cycle window from Life-365

Concrete Mixtures for Three Cases

W/cm ratio was adjusted to 0.4 to obtain a desired compressive strength of 40 MPa.

Using Abrams' Law:

$$w/cm = \frac{W_{water}}{W_{cement} + W_{slag}}$$

$$f_c = \frac{K_1}{K_2^{w/cm}}; K_1 = 96.5 \text{ MPa}; K_2 = 7$$

$$f_c = \frac{96.5}{7^{0.4}} = 40.7 \text{ MPa}$$

Three mix design cases were examined in order to compare the life cycle costs with the various protection strategies.

Base Case: w/cm = 0.4, 20 percent slag replacement, black steel rebar (1 percent vol. concrete), no barrier, no inhibitor. The 20-percent slag content enhances long-term chloride resistance and lowers permeability, but early-age strength might be slightly lower as a result of lower initial reactivity. Compressive strength meets the 40 Mpa requirement in the long run.

Base case

Mixture	Rebar	Barriers
w/cm *	Rebar Steel Type *	Select Barrier *
0,4	Black Steel	<none>
Class F fly ash (%) *	Rebar % vol. concrete. *	
0	1	
Slag (%) *	Inhibitor	
20	Select Inhibitor *	
Silica Fume (%) *	<none>	
0		

Figure 2.5.12. Base Case Mixture

Alternative 1: w/cm = 0.42, no additional cementitious resources, black steel rebar, default settings with membrane barrier. This option investigates the possibility of a membrane barrier system to offset slightly elevated w/cm ratio and lack of slag.

Alternative 1

Mixture	Rebar	Barriers
w/cm *	Rebar Steel Type *	Select Barrier *
0,42	Black Steel	Membrane
Class F fly ash (%) *	Rebar % vol. concrete. *	☒ Default
0	1	
Slag (%) *	Inhibitor	Initial efficiency (%) *
0	Select Inhibitor *	100
Silica Fume (%) *	<none>	Age at failure (yrs) *
0		20
		#reapplytimes *
		0

Figure 2.5.13 Alternative 1 Mixture

Alternative 2: w/cm = 0.4, no additional cementitious materials, epoxy coated steel rebar, sealer barrier with default settings. This option explores the synergistic action of coated reinforcement and a surface sealer system.

Alternative 2

Mixture	Rebar	Barriers
w/cm *	Rebar Steel Type *	Select Barrier *
0,4	Epoxy Coated	Sealer
Class F fly ash (%) *	Rebar % vol. concrete. *	☒ Default
0	1	Initial efficiency (%) *
Slag (%) *	Inhibitor	90
0	Select Inhibitor *	Age at failure (yrs) *
Silica Fume (%) *	<none>	5
0		#reapplytimes *
		0

Figure 2.5.14. Alternative 2 Mixture

Life-Cycle Cost Results

The figures below indicate the obtained life cycle Cost (LCC) of Life-365 after analysis of the results in 75 years (base year 2024, inflation rate 3 percent, real discount rate 3 percent).

Name	Construction Cost	Barrier Cost	Repair Cost	Life-Cycle Cost
Base case	\$209,300	\$0	\$430,556	\$639,857
Alternative 1	\$209,300	\$193,750	\$645,835	\$1,048,885
Alternative 2	\$236,694	\$36,597	\$430,556	\$703,848

Figure 2.5.15. Life Cycle Costs

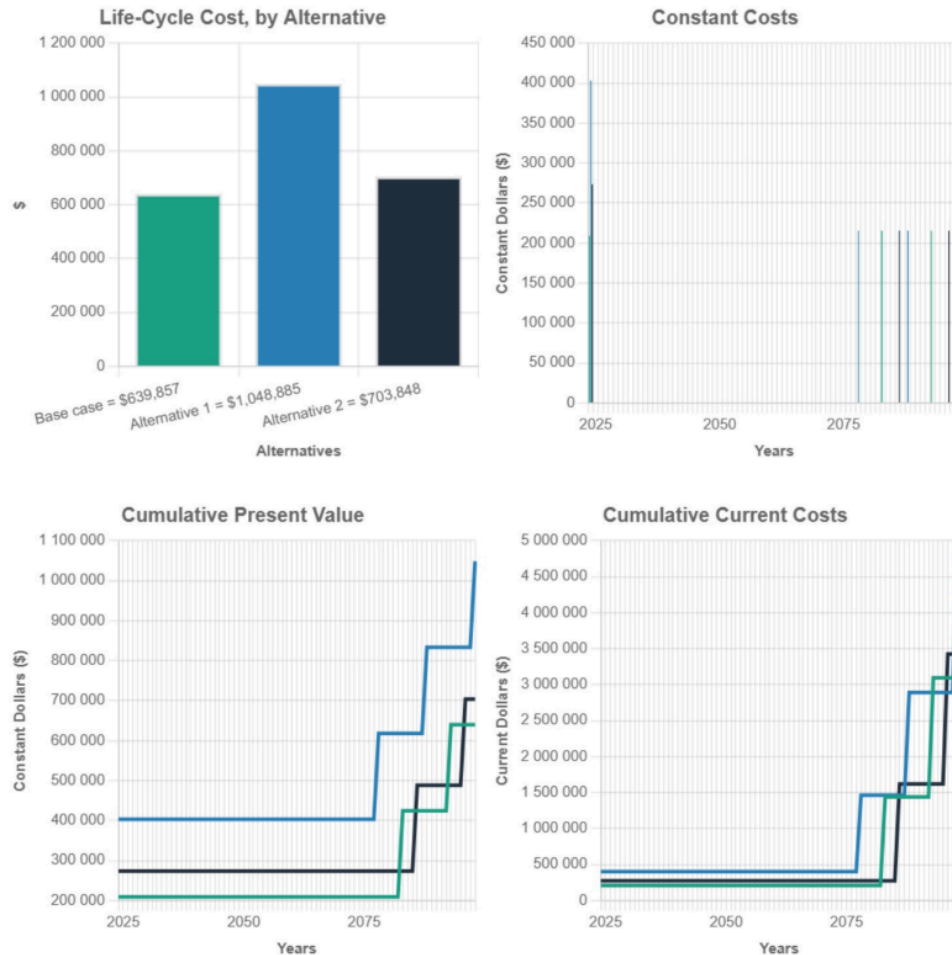


Figure 2.5.16. LCC results

The outcomes reveal clearly that the Base Case is the most economical with the lowest life cycle cost of \$639,857 and thus it is the most economical alternative based on the 75-year service life. Alternative 1, although it has an added cost of a membrane barrier, will lead to the greatest overall cost of \$1,048,885 - a 64 percent increase over the Base Case - due to the high barrier installation cost of \$193,750, as well as the highest cost of repair of the three alternatives. This implies that the membrane barrier was not effective enough to significantly alleviate long-term damage caused by chloride, without a better mix design. Alternative 2 (epoxy coated steel with a sealer) is less expensive than the Base Case (430,556) as it will incur the same repair cost, however, the higher cost of construction will cause the total life cycle cost to be higher (703,848). The results indicate that the most cost-effective long-term durability approach to this parking garage exposure situation is the inclusion of 20% slag in the concrete mix (Base Case) which is more effective than either physical barrier or rebar upgrade options.

2.6 Literature Review of Design Codes

The International Building Code (IBC) of 2009 (Mercantile Group M) under section 309 of the code outlines that shopping centers are occupied in classification (IBC2009), since they are mostly occupied with displaying and selling merchandise. Also, the mall falls under covered mall building provisions (Section 402) that governs buildings with multiple retail tenants, food and beverage and entertainment areas (IBC 2009). As far as Khan Tengri Mall will be designed as a four-story building and will have the total area of 4,000 m², it will meet the general height and area restrictions stated in Chapter 5 of the IBC bearing in mind that such construction type and fire-resistance considerations will be met (IBC 2009).

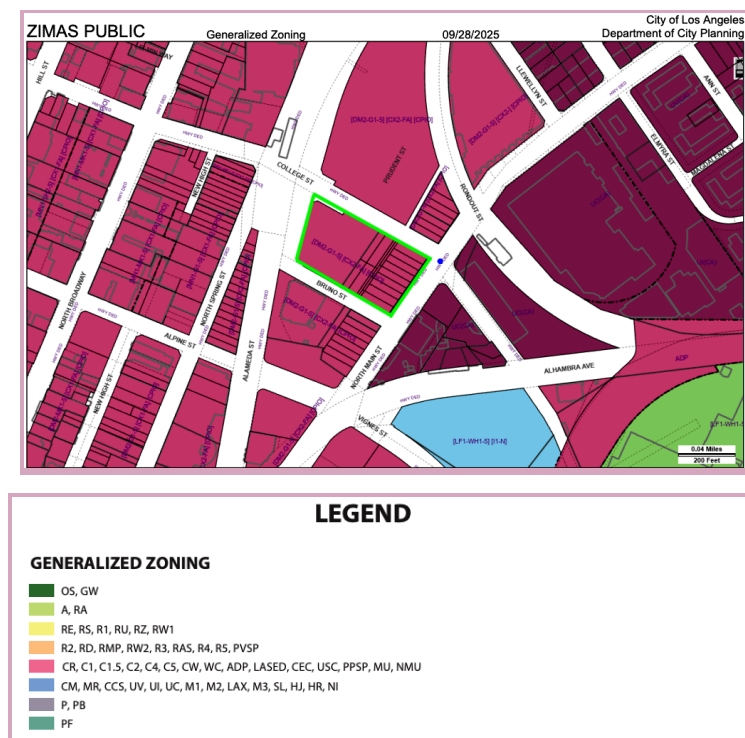


Figure 2.6.1 College Street Zoning Map

Besides, the project aligns with the Los Angeles Zoning Map because the proposed usage of the land falls under the commercial and mixed-use that is allowed on the ground. Specifically, the suggested development is compatible with the zoning designations of CR, C1, C1.5, C2, C4, C5, CW, WC, ADP, LASED, CEC, USC, PPSP, MU, and NMU with all of them permitting commercial retail, entertainment, and mixed-use activities. This affirms that the site and the type

of building of the Khan Tengri Mall is both occupancy and zoning compliant with Los Angeles. The zoning and occupancy of the proposed development site are shown in Figure 2.6.1.

The 2009 International Building Code (IBC 2009) has been wisely implemented to meet the structural and architectural design requirements.

a. Maximum exit travel distance.

Among the essential conditions is the highest distance of exit travel, but it depends on the type of occupancy in the building and whether there is an automatic sprinkler system or not. As the Khan Tengri Mall belongs to Group M (Mercantile Occupancy), the required set of provisions is located in Section 1028.7 (Travel Distance). This part restricts the maximum travelling distance to 250 feet (76.2 meters) in case the building has an automatic sprinkler system. In the planning of the Khan Tengri Mall, any boutique, supermarket, food court, or cinema space, therefore, should not have the farthest exit that is above this distance.

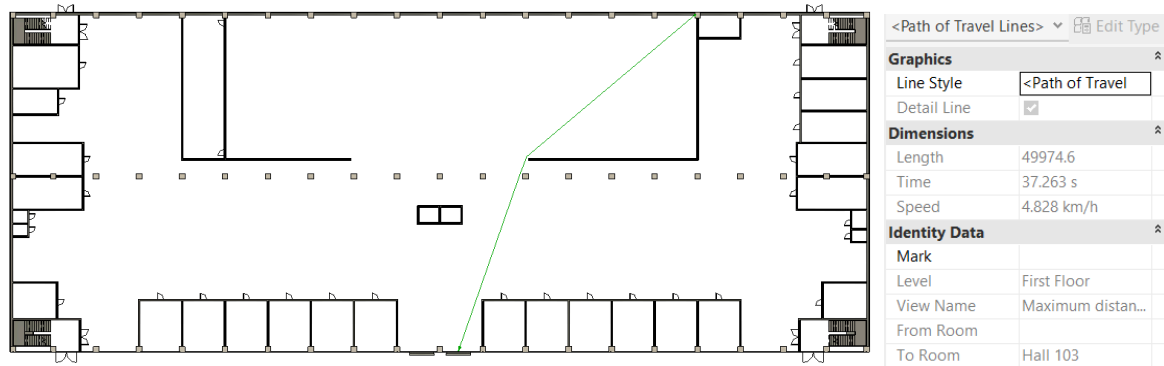


Figure 2.6.2 Maximum Travel distance

The floor plan of the project also meets the set code with the maximum distance traveled to the exit being 49.98 meters.

b. Number of Elevators and Stairs

The occupant load has to be determined to ascertain the minimum number of exits. To calculate the occupant load of the mall design, the occupant-load factors as stated in the IBC Table 1004.1.1 and the covered-mall provisions in the Section 402.4.1.1 are used. The average gross floor-area factor in mercantile (sales) areas is 60ft² per individual, and assembly areas (food

court, cinema) adopt the corresponding assembly factors (such as 15 net ft² per individual in dining areas).

To ensure unit consistency and easy calculation the following relations are utilized:

$$1 \text{ ft}^2 = 0.0929 \text{ m}^2$$

$$\text{Occupant Load} = \frac{OLF}{GLA} = \frac{4000 \text{ m}^2}{0.0929 \text{ ft}^2 \times 60 \text{ ft}^2/\text{person}} = 718 \text{ person}$$

In Section 1015.1.1 (IBC 2009), one occupant load that has a number between 501 and 1000 persons per story should have a minimum of three exits. The minimum exits of three are technically met since the occupant load calculation of the Khan Tengri Mall is about 718 persons on each floor. Nevertheless, with such a large assembly area like the cinema on the fourth floor, the effective occupant load during times of peak should be more. In case of such circumstances, to afford a higher level of safety and smooth evacuation, the design has additional four independent fire exits with stairways which is larger than what the code stipulates.

This building also has a main glass automatic entrance door system on top of these fire exits, which would be the primary entry point to daily operations and can serve as an extra fire exit in case of an emergency. This arrangement is such that the stories of Khan Tengri Mall all satisfy the IBC requirement besides giving a certain level of safety margin to high-density conditions.

In the case of the stairway dimensions, the design of the Khan Tengri Mall is based on the provisions of the IBC 2009 in Section 1009.1. The fire exit stairs are constructed to accommodate the occupant load with each stairway having a clear width of more than 1118 mm, the minimum required width to accommodate the occupancy calculation. A headroom clearance is kept at least 2032 mm (1003.3.1), which means safe and unhindered way out in case of evacuation. The risers of the stair are calculated on the prescribed standard, there being a maximum number of 178 mm, and a minimum of 102 mm, whereas the tread depth is calculated at not less than 275 mm. The landing is proportioned with a minimum length at least equal to the width of the stair, the minimum width being 1219 mm. The general-use stairways extend to the roof as per the requirements of Section 1012.2 so as to ensure full accessibility in terms of fire

protection and emergency operations. The height of handrails is maintained at 900 mm which is within range of 864-965 mm which is the permitted range of handrail height.

Stair dimensions:

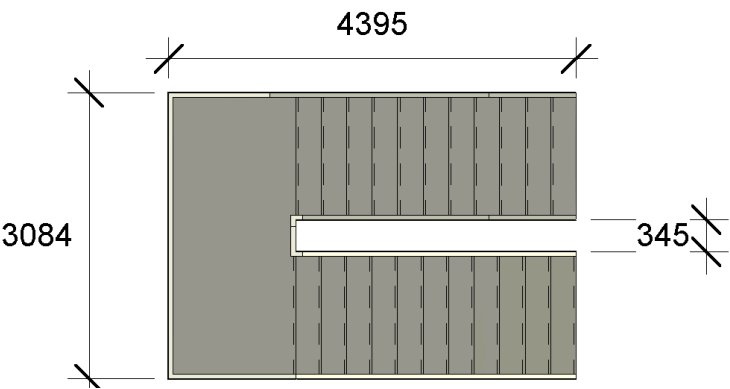


Figure 2.6.4 Stair Dimensions

Dimensions		
Desired Numb...	24	
Actual Numbe...	24	
Actual Riser H...	175.0	
Actual Tread ...	275.0	
Tread/Riser St...	1	

Figure 2.6.5 Riser and Tread Depth of Stairs

As far as the lift design is concerned, the IBC 2009 provisions of Section 3001 have been adopted. As per the code, there should be at least an elevator that would fit an ambulance stretcher with a 610 mm by 2134 mm dimension. In Khan Tengri Mall, there are two passenger lifts serving the public, with a large enough cabin size to transport a stretcher, thus enabling the building to reach the emergency medical access requirements. Also, there is a single service elevator that will facilitate the movement of goods, equipment and staff operations. This will not only meet the minimum requirement stipulated by the code on the accessibility of stretchers, but just improves the overall accessibility and effectiveness of vertical circulation in the mall.

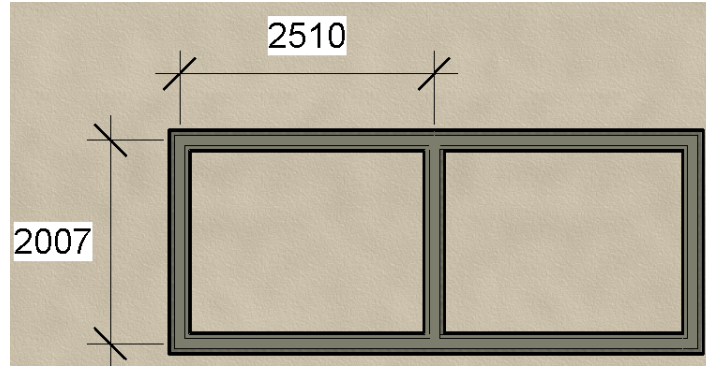


Figure 2.6.6 Passenger Elevator dimensions

c. Area for the shopping mall

Based on Section 602.2, Khan Tengri Mall qualifies as a Type I construction as it will majorly use noncombustible building materials like reinforced concrete and structural steel. Group M (Mercantile), Type I buildings are not limited in building area as per the table of allowable area factors in Section 506.2. Thus, the suggested size of the building 4,000 m² is fully in accordance with the code requirements and is unlimited according to IBC provisions.

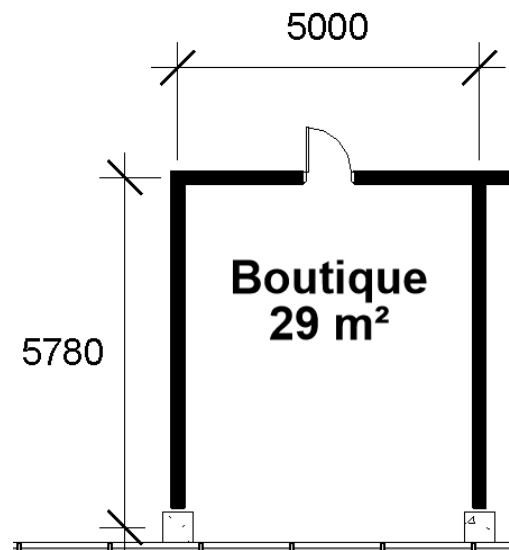


Figure 2.6.7 Rooms in Shopping Mall with areas

d. Door Dimensions

The amount of exit doors is calculated according to Section 1004.5 of IBC 2009, as per which the occupancy load of the area has to be calculated (see Section b of the elaborated calculation of the minimum number of exits per story). Having occupancy of about 718 persons per floor, one of the levels of Khan Tengri Mall has to be equipped with several exit doors, which will ensure safety and efficiency in the evacuation process.

In terms of the size of the door, Section 1010.1.1 provides the following minimum size requirement:

Minimum clear aperture width: 813 mm.

Minimum total door width: 914 mm.

Minimum door height: 2032 mm

Minimum distance between series of doors: 1219 mm.

Under these requirements, the design takes exit and entry doors having a clear opening width of 2857 mm and height of 2400 mm and this is more than the minimum code requirement. There is also compliance with the standard of the spacing between exit doors thus adhering to the provisions of egress safety.

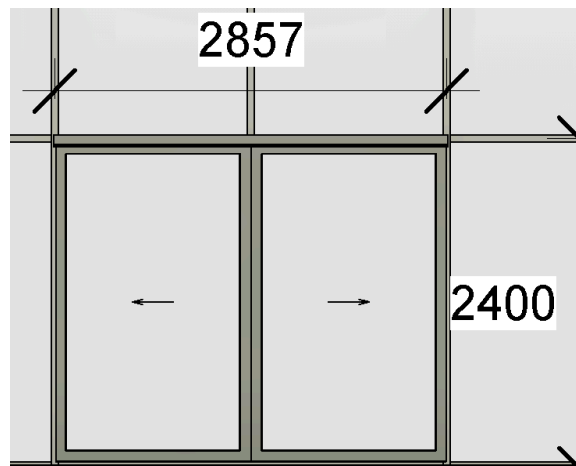


Figure 2.6.8 Main Door Dimensions

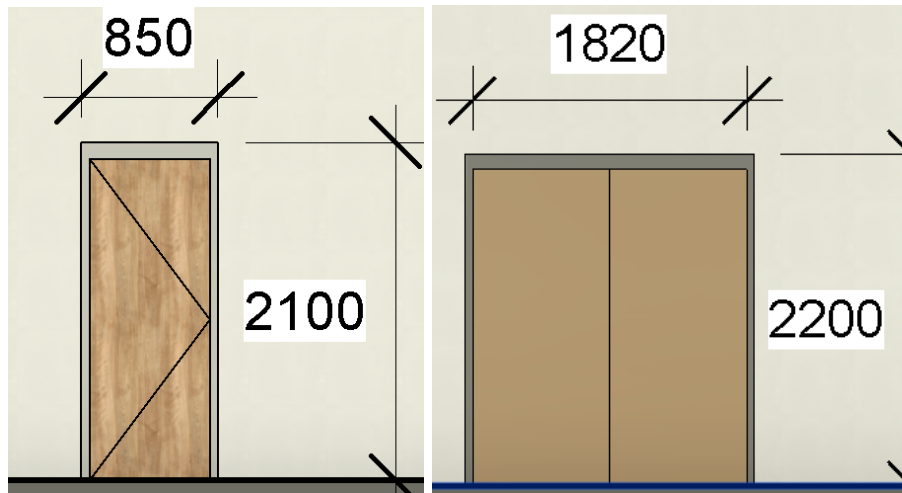


Figure 2.6.9 Other Doors Dimensions

e. Window Dimensions

In case of commercial and assembly occupancy, including the cases of Khan Tengri Mall, windows are mainly controlled in regards to the natural lighting and ventilation requirements. The natural lighting of any regularly occupied space should be arranged in accordance with the provisions of Section 1204.2 of IBC 2009, where the glazed section should not be less than 8 percent of the floor area.

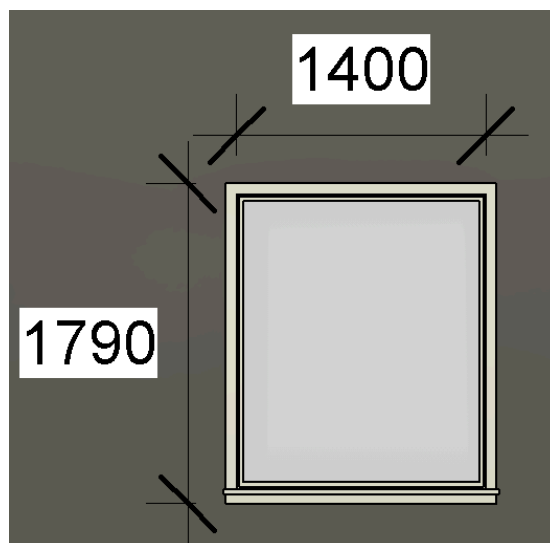


Figure 2.6.10 Window Dimensions

In Khan Tengri Mall, the large storefront windows, the curtain walls on the building facades not only meet this minimum requirement but also guarantee the supermarket, electronics store, food court, and circulation areas plenty of natural daylight. This design strategy adds greater visual comfort to visitors, minimizes the need to use artificial lighting throughout the day, and promotes the achievement of the sustainability objectives of the project.

f. Corridor Dimensions

Under the case of commercial occupancies, IBC 2009 under Section 1018.2 sets the minimum width requirements of corridors. The general minimum clear width in corridors with an occupant load of 50 or above is 1.12 m and thinner corridors with less than 50 occupants are allowed to be 0.91 m wide. In the example of the Khan Tengri Mall, the occupant load per floor (718 persons) is much higher than this limit. All principal circulation routes therefore are laid with a width greater than the minimum, usually between 2.0 m and 3.0 m, to serve the high traffic of visitors in the shopping malls, the food court, and the cinema. This will not only guarantee the full adherence to the provisions of IBC but ensure that the circulation is comfortable, that there is less congestion during the busy time, and that evacuation during emergencies is safe.

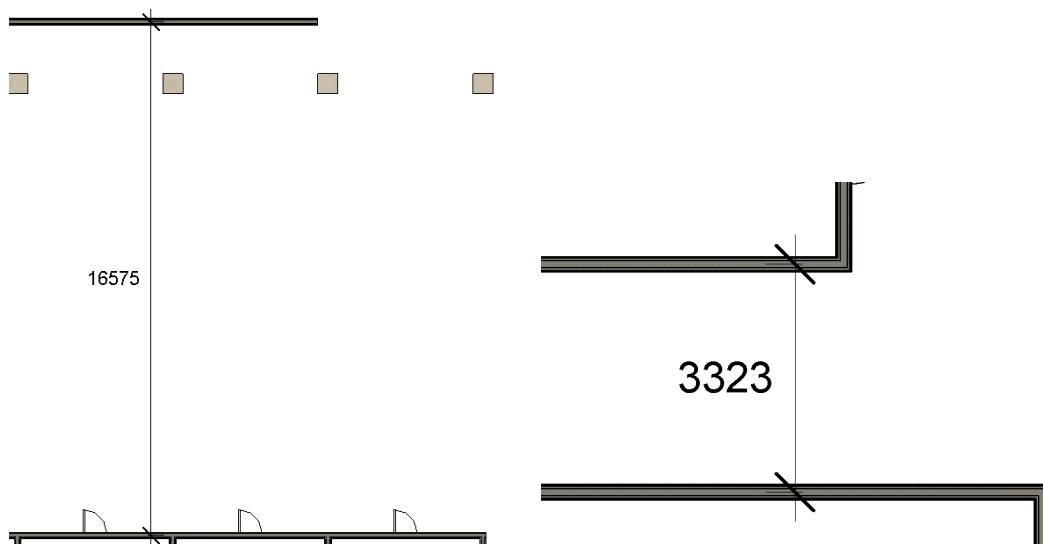


Figure 2.6.11 General Corridor and Narrowest Corridor Dimensions

g. Allowable Clear Building Height

The permitted height of the Khan Tengri Mall is decided based on the availability and type of sprinkler system present. In this project, a complete system of automatic sprinkler system that meets the requirements of NFPA 13 is used and accordingly the building height can be considered unlimited as per the provision of IBC 2009 Section 504.3 of Type I construction. Likewise, the stories are not limited as well in Section 504.4, as the sprinkler system offers a better degree of fire coverage. Therefore, despite the fact that Khan Tengri Mall is a four-story shopping mall with a single parking level under the ground, the project is still within the acceptable boundaries of the code.

h. Traffic Flow of Parking

The circulation of the parking level of Khan Tengri Mall in the basement level is simple, as it is organized in the direction of traffic movement inside the site. Individual crossing and exits are given to alleviate congestion and the design guarantees accessibility of the vehicles and pedestrians. The design is completely in accordance with the basic IBC requirements of parking and exit.

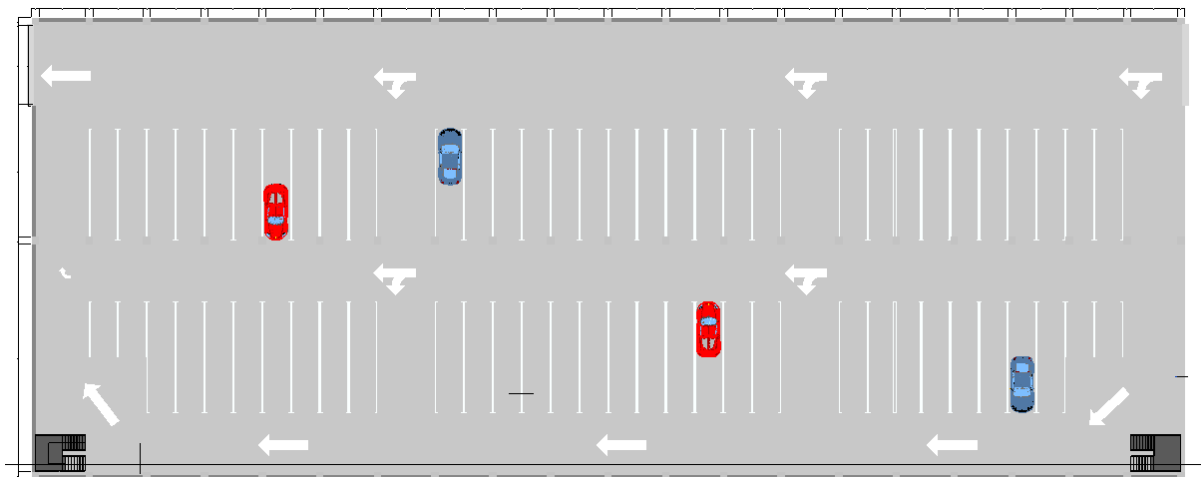


Figure 2.6.12 Traffic Flow Diagram

i. Roof Slope Design

The flat roof of Khan Tengri Mall is constructed according to the recommendations in IBC 2009 under Section 1507.10.1 where the highest allowable slope is set at 2%. Storm water flows into a drainage system which is gravity-based and incorporated in the parapet walls as scuppers and into the primary storm water piping network. With this solution, you are assured of effective elimination of rain on the roof surface and also avoidance of the presence of water on the roof which may compromise the structure or effectiveness in waterproofing.

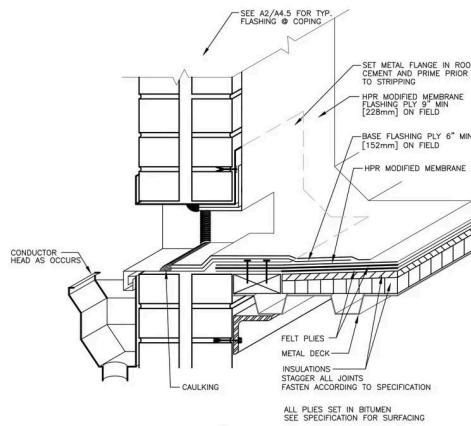


Figure 2.6.13 Drainage system on the roof of the scupper (Wolnitzek, 2012)

j. Parking Ramp Design

The clear floor to floor height of the parking level of the Khan Tengri Mall is 3.3 m. The IBC 2009 Section 406.2.5 provides that the maximum slope of vehicle ramps in the shorter runs should be 1:12 (8.3%). In accordance with this provision the length of the ramp to be provided is computed as:

$$L = \frac{\Delta h}{\text{slope}} = \frac{3.3}{0.083} = 39.8 \text{ m}$$

In this way, the parking ramp will be located with a length of about 40 m, which will provide a safe and code-compliant vehicle access to the basement level.

2.7 Design of Non-structural Elements

In case of the internal partition walls of Khan Tengri Mall, gypsum board partitions were chosen, which is a combination of metal stud framing, as they offer the necessary fire resistance. As per Table 508.4 of the IBC 2009, a minimum of 1 hour fire resistance is provided in partitions between tenant spaces in the mall. Lightweight concrete blocks are applied in service areas and back-of-house corridors, to increase their durability.

In the case of the exterior walls, solid concrete masonry with gypsum insulation cover was selected because of its strength, thermal property and resistance to fire. The outer wall will consist of fiber concrete fill finish, which will offer weather protection and offer a contemporary appearance to the wall. The overall thickness of the usual interior partitions and exterior walls are about 210 mm and 344 mm respectively.

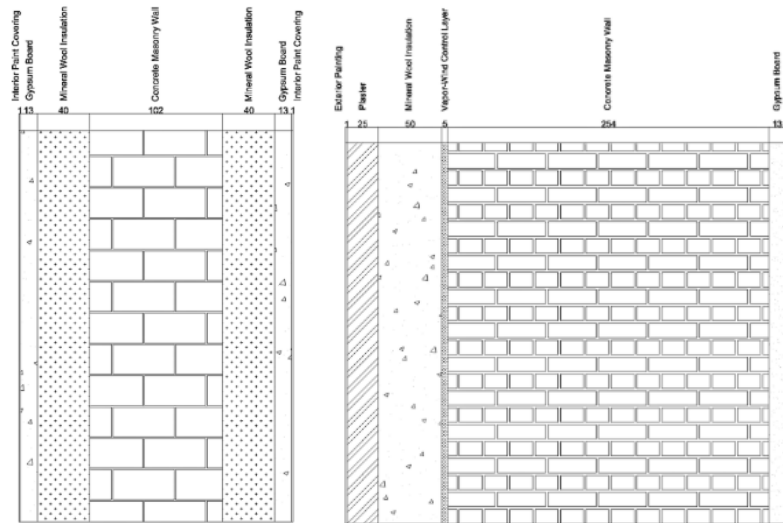


Figure 2.6.13 Exterior and Interior Wall Compositions

In the case of a floor construction, a reinforced concrete slab is adopted with an added layer of acoustic material to reduce noise transmission which is of special concern to the cinema hall and food court. The slab construction comprises a 130 mm RC slab, acoustic fiber board, concrete fill finish, ceramic tile, and water-proofing membrane. The underside is covered with gypsum board ceiling panels to integrate services. The overall average floor thickness is approximately 260 mm.

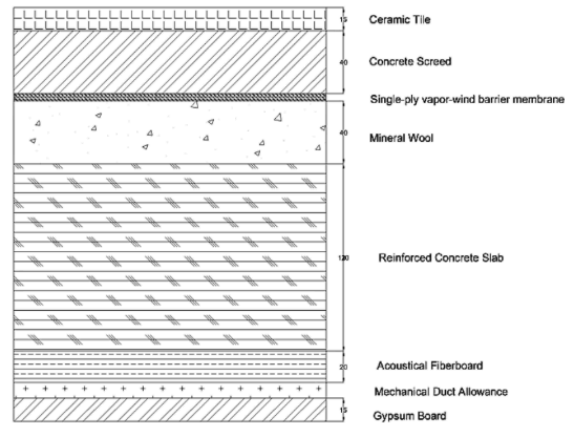


Figure 2.6.14 Floor Composition

3. Structural Design

3.1 Material Selection

The scope of the structural design of Khan Tengri Mall covers the development of a safe and efficient four story shopping mall in a high seismic area. The primary structural material used in the building is reinforced concrete as it is robust, has high seismic performance, and is durable than other materials. A comparative study has been conducted on the reinforced concrete and structural steel whose advantages relating to their strength, fire resistance, cost and seismic resilience have been compared. According to this paper, reinforced concrete was selected as the major material of construction for both structural frame and floor systems. The results of the literature review and comparative evaluation were presented in a tabular manner to help in the design decision.

Table 3.1.1 Reinforced concrete Vs Steel Comparison

Properties	Reinforced Concrete	Structural Steel
Strength	High compressive strength; best when subjected to vertical loads	High stress to weight ratio; best when long spans and flexible structures are needed.
Durability	Very good long-term durability; must be maintained	Susceptible to corrosion; must have protective finishes and frequent

	to prevent rebar corrosion	maintenance.
Fire Resistance	Inherent fire resistance; does not lose strength under fire	Loses strength at high temperatures; needs fireproofing.
Labor / Construction	Widespread skills; traditional construction methods	Needs special welders and skilled labor.
Cost	Reduced initial cost; economical to build large scale construction	Higher initial cost; additional for protection and fabrication.
Seismic Performance	Less ductile; mass is stable, but requires seismic detailing	Very ductile; improved seismic performance
Thermal Properties	Good thermal mass; assists in energy efficiency	High thermal expansion; possibly allows allowances to thermal movement.
Quality Control	On-site quality control; can be more variable	Factory-controlled quality; can produce fabrication that is more precise, with less errors.
Environmental Impact	High CO2 footprint; low recyclability	Low CO2 emissions; high recyclability

The choice of reinforced concrete to construct the Khan Tengri Mall is majorly founded on the geotechnical and environmental nature of the location. Being in a high seismic area, the high ductility of reinforced concrete offers excellent seismic behavior, which guarantees safety and stability of the building. It is also very hardy and therefore it can withstand the local climatic conditions. The large thermal mass of the material also adds to the energy efficiency, but the fact that it is inherently fire resistant and can soak up sound is especially useful when installed in a busy shopping mall setting (GZ Supplies, 2025; CRSI, 2025; SWFunk, 2025; MDPI, 2024).

3.2 Analytical Model Development

3.2.1 Model Geometry

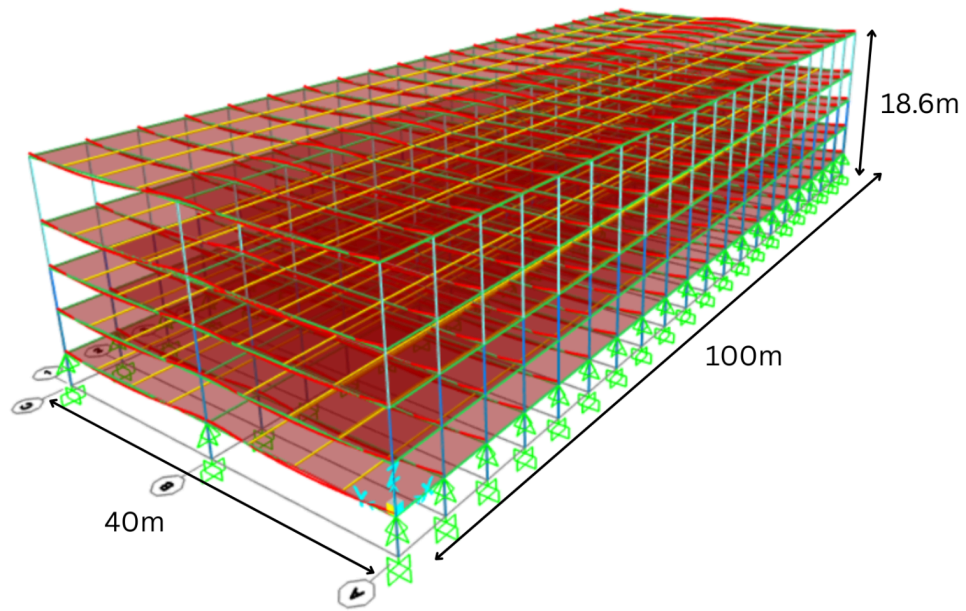


Figure 3.2.1.1. Building Model Geometry

3.2.2 Material Properties

Table 3.2.2.1. Properties of reinforced concrete

Type	f'_c , Mpa	ρ , kN/m^3	E_c , Mpa	ν	G_c , Mpa
4000 psi	27.58	25	24856	0.2	10360
5000 psi	34.47	25	28000	0.2	11600

Table 3.2.2.2. Properties of steel rebar

E , Mpa	f_y , Mpa	ν	ρ , kN/m^3
200000	420	0.3	76.97

3.2.3 Frame Sections

The cracking moments of inertia for the column and beam frame sections, as given in Table 3.2.3.1, were applied in the SAP2000.

Table 3.2.3.1. Cracking moment of inertia for each structural member

Structural Member	Cracking moment of inertia factors
Slabs	0.25
Beams	0.35
Columns	0.7

3.2.4 Element Types, Connections

All slabs were modeled as two-way shell elements, while columns and beams were represented using frame elements. The joints between these elements were assumed to have rigid connections.

3.2.5 Boundary Connections

All basement columns are modeled as fixed supports, meaning their horizontal movement, vertical movement, and rotation are fully restrained. At the ground level, the exterior and corner columns are modeled as pinned supports. Their horizontal and vertical movements are restrained, but rotation is released, allowing the structure to rotate and displace appropriately under lateral loads.

3.3 GLRS

3.3.1 Dead Load Calculations

Both the roof and the floor were calculated using non-structural dead loads as per ASCE 7-10 Chapter 3 (American Society of Civil Engineers, 2010). In contrast to residential buildings, the shopping center has a monotonic flooring finish in all the areas which include corridors, retail space, and common areas. Thus, one representative floor finishing load was taken in account in

the structural design. Tables below will show a calculation of the dead loads of the uniform floor finishing and the roof.

Table 3.3.1.1. Uniformly Distributed Dead Load for Floors

Dead load calculations for all floors.				
Function	Material Type	Thickness, mm	Load, kPa	Total Load, kPa
Cover	Ceramic Tile	15	0.77	5.058
Leveling	Concrete Screed	40	0.023	
Water-proofing	Single-ply vapor-wind barrier membrane	-	0.03	
Insulation	Mineral Wool Slab	40	0.088	
Structural Slab	Reinforced Concrete Slab	130	3.25	

Table 3.3.1.2. Uniformly Distributed Dead Load for Ceiling

Uniformly Distributed Dead Load for Ceiling			
Material Type	Thickness, mm	Load, kPa	Total Load, kPa
Acoustical Fiberboard	20	0.05	0.36
Mechanical Duct Allowance	-	0.19	
Gypsum Board	15	0.008	

Table 3.3.1.3. Uniformly Distributed Dead Load for Roof

Dead load calculations for roof				
Function	Selected Material	Thickness, mm	Load, kPa	Total Load, kPa
Cover	Five-ply and gravel	25	0.29	3.724
Insulation	Fibrous glass	40	0.084	
Water-proofing	Single-ply vapor-wind barrier membrane	-	0.03	
Leveling	Bitomniuous Smooth surface	-	0.07	
Structural Slab	Reinforced Concrete Slab	130	3	

Also, the analysis took into account the loads imposed on interior walls, exterior walls as well as roof parapets. The portion of a partition wall was supposed to be dispersed evenly across the whole floor, but the weight of both exterior wall and roof parapet load was placed directly on edge beams.

Table 3.3.1.4. Uniformly Distributed Dead Load for Interior Walls

Dead load calculations for Interior Wall				
Function	Selected Material	Thickness, mm	Load, kPa	Total Load, kPa
Interior Finish	Interior Paint Covering	1	0.0006	1.4352
Leveling	Gypsum Board	13	0.104	
Insulation	Mineral Wool Insulation	40	0.088	
Autoclaved aerated concrete block	Concrete Masonry Wall	102	1.05	
Insulation	Mineral Wool Insulation	40	0.088	
Leveling	Gypsum Board	13	0.104	
Interior Finish	Interior Paint Covering	1	0.0006	

Table 3.3.1.5. Uniformly Distributed Dead Load for Exterior Walls

Dead load calculations for Exterior Wall				
Function	Selected Material	Thickness, mm	Load, kPa	Total Load, kPa
Exterior finish	Exterior Painting	1	0.06	4.1546
Cement plaster	Plaster	25	0.5	
Insulation	Mineral Wool Insulation	50	0.11	
Air infiltration barrier	Vapor-Wind control Layer	-	0.03	
Autoclaved aerated concrete block	Concrete Masonry Wal	254	3.35	
Leveling	Gypsum Board	13	0.008	
Exterior finish	Paint Covering	1	0.0006	

Table 3.3.1.6. Uniformly Distributed Dead Load for Roof Parapets

Dead load calculations for Roof Parapets				
Function	Selected Material	Thickness, mm	Load, kPa	Total Load, kPa
Exterior finish	Paint Covering	1	0.0006	1.0512
Autoclaved aerated concrete block	Concrete Masonry Wall	102	1.05	
Exterior finish	Paint Covering	1	0.0006	

Table 3.3.1.7. Dead Load of Interior Walls to Slabs

Dead Load from Interior Walls to Slabs					
Location	Height, m	Length, m	Load, kN	Total Area, m2	Floor Load, kPa
Slab 0	3.17	-	-	-	-
Slab 1	4.07	450.5587	2631.83	4000	0.66
Slab 2	4.07	491.147	2868.92	4000	0.72
Slab 3	4.07	464.65	2714.14	4000	0.68
Slab 4	5.87	623.348	5251.47	4000	1.31

Table 3.3.1.8. Total Dead Loads to Slabs

Total Dead Load			
Location	Pressure Load, kPa	Exterior Wall Load, kN/m	Parapets (kN/m)
Slab 0	5.418	13.17	-
Slab 1	6.076	16.91	-
Slab 2	6.135	16.91	-
Slab 3	6.097	16.91	-
Slab 4	6.731	24.39	-
Slab 5	4.084	0	1.12

3.3.2 Live Load Calculations

The minimum uniformly distributed live loads of the shopping mall according to the requirements of the ASCE 7-10 (American Society of Civil Engineers, 2010) are calculated as follows:

- Retail First Floor - 4.79 kN/m²
- Retail Upper Floors - 3.59 kN/m²
- Public Corridors - 4.79 kN/m²
- Stairs and exit ways - 4.79 kN/m²
- Dining room/restaurants - 4.79 kN/m²
- Assembly areas and theaters Fixed seats (fastened to floor) - 2.87 kN/m²
- Lobbies - 4.79 kN/m²
- Passenger vehicles only - 1.92 kN/m²

ASCE 7-10 states that all uniformly distributed loads should have the live load reduction. Calculation of the reduction of floor live loads should be made in the following formula:

$$L = L_o \left(0.25 + \frac{4.57}{\sqrt{K_{LL} A_T}} \right)$$

The element factors of live load, K_{LL} , will be as following:

- Interior columns: $K_{LL} = 4$
- Exterior columns without cantilever slabs: $K_{LL} = 4$
- Corner columns: $K_{LL} = 4$
- Edge beams without cantilever slabs: $K_{LL} = 2$
- Interior beams: $K_{LL} = 2$

Table 3.3.2.1. Floor Live Load Calculation for the basement

Space	A _t (m ²)	K _{LL}	Reduction	L ₀	L (kN/m ²)
-------	----------------------------------	-----------------	-----------	----------------	------------------------

						(kN/m2)	
Basement	Passenger vehicles only	Interior columns	100.00	4.00	-	1.92	-
		Exterior columns	50.00	4.00	-	1.92	-
		Corner columns	25.00	4.00	-	1.92	-
		Typical edge beams	6.25	2.00	-	1.92	-
		Edge beams in large span	43.75	2.00	-	1.92	-
		Interior beams in large span	87.50	2.00	-	1.92	-
		Typical Interior beams	12.50	2.00	-	1.92	-
		minor beams	12.50	2.00	-	1.92	-

Table 3.3.2.2. Floor Live Load Calculation for the first floor

Space			A_t (m2)	K_LL	Reduction	L_0 (kN/m2)	L (kN/m2)
First Floor	Retail First Floor; Public Corridors ; Stairs and exit ways	Interior columns	100	4	0.48	4.79	2.29
		Exterior columns	50	4	0.57	4.79	2.745
		Corner columns	25	4	0.71	4.79	3.39
		Edge beams in large span	43.75	2	0.74	4.79	3.54
		Typical edge	6.25	2	1.54	4.79	4.79

		beams					
		Interior beams in large span	87.5	2	0.595	4.79	2.85
		Typical Interior beams	12.5	2	1.164	4.79	4.79
		minor beams	12.5	2	1.164	4.79	4.79

Table 3.3.2.3. Floor Live Load Calculation for the second floor

Space			A _t (m ²)	K _{LL}	Reduction	L ₀ (kN/m ²)	L (kN/m ²)
Second Floor	Retail Upper Floors	Interior columns	100.00	4.00	0.48	3.59	1.72
		Exterior columns	50.00	4.00	0.57	3.59	2.06
		Minor beam	25.00	2.00	0.90	3.59	3.22
		Edge beams large span	50.00	2.00	0.71	3.59	2.54
		Typical edge beams	12.50	2.00	1.16	3.59	4.18
		Typical Interior beams	25.00	2.00	0.90	3.59	3.22
		Interior beams in large span	100.00	2.00	0.57	3.59	2.06
	Public Corridors ; Stairs and exit ways	Interior columns	100.00	4.00	0.48	4.79	2.29
		Exterior columns	50.00	4.00	0.57	4.79	2.75

		Corner columns	25.00	4.00	0.71	4.79	3.39
		Edge beams in large span	43.75	2.00	0.74	4.79	3.54
		Typical edge beams	6.25	2.00	1.54	4.79	4.79
		Interior beams in large span	87.50	2.00	0.60	4.79	2.85
		Typical Interior beams	12.50	2.00	1.16	4.79	4.79
		minor beams	12.50	2.00	1.16	4.79	4.79

Table 3.3.2.4. Floor Live Load Calculation for the third floor

Space			A _t (m ²)	K _{LL}	Reduction	L ₀ (kN/m ²)	L (kN/m ²)
Third Floor	Retail Upper Floors	Interior columns	100.00	4.00	0.48	3.59	1.72
		Exterior columns	50.00	4.00	0.57	3.59	2.06
		Minor beam	25.00	2.00	0.90	3.59	3.22
		Edge beams large span	50.00	2.00	0.71	3.59	2.54
		Typical edge beams	12.50	2.00	1.16	3.59	4.18
		Typical Interior beams	25.00	2.00	0.90	3.59	3.22
		Interior beams in large span	100.00	2.00	0.57	3.59	2.06

	Public Corridors; Stairs and exit ways;	Interior columns	100.00	4.00	0.48	4.79	2.29
		Exterior columns	50.00	4.00	0.57	4.79	2.75
		Corner columns	25.00	4.00	0.71	4.79	3.39
		Edge beams in large span	43.75	2.00	0.74	4.79	3.54
		Typical edge beams	6.25	2.00	1.54	4.79	4.79
		Typical Interior beams	12.50	2.00	1.16	4.79	4.79
		Interior beams in large span	87.50	2.00	0.60	4.79	2.85
		minor beams	12.50	2.00	1.16	4.79	4.79
	Dining room/restaur ants	Interior columns	100.00	4.00	-	4.79	-
		Exterior columns	50.00	4.00	-	4.79	-
		Edge beams large span	50.00	2.00	-	4.79	-
		Typical edge beams	12.50	2.00	-	4.79	-
		Typical Interior beams	25.00	2.00	-	4.79	-
		Interior beams large span	100.00	2.00	-	4.79	-

		Minor beam	25.00	2.00	-	4.79	-
--	--	------------	-------	------	---	------	---

Table 3.3.2.5. Floor Live Load Calculation for the fourth floor

Space			A _t (m ²)	K _{LL}	Reduction	L ₀ (kN/m ²)	L (kN/m ²)
Fourth Floor	Retail Upper Floors	Interior columns	100.00	4.00	0.48	3.59	1.72
		Exterior columns	50.00	4.00	0.57	3.59	2.06
		Minor beam	25.00	2.00	0.90	3.59	3.22
		Typical edge beams	12.50	2.00	1.16	3.59	4.18
		Edge beams large span	50.00	2.00	0.71	3.59	2.54
		Interior beams in large span	100.00	2.00	0.57	3.59	2.06
		Typical interior beam	25.00	2.00	0.90	2.87	2.57
	Public Corridors; Stairs and exit ways;	Interior columns	100.00	4.00	0.48	4.79	2.29
		Exterior columns	50.00	4.00	0.57	4.79	2.75
		Corner columns	25.00	4.00	0.71	4.79	3.39
		Typical edge beams	6.25	2.00	1.54	4.79	4.79
		Edge beams in large span	43.75	2.00	0.74	4.79	3.54
		Interior beams in	87.50	2.00	0.60	4.79	2.85

		large span					
		Typical Interior beams	12.50	2.00	1.16	2.87	3.34
		minor beams	12.50	2.00	1.16	4.79	4.79
	Assembly areas and theaters Fixed seats (fastened to floor)	Exterior columns	50.00	4.00	0.57	2.87	2.87
		Corner columns	25.00	4.00	0.71	2.87	2.87
		Interior columns	100.00	4.00	0.48	2.87	2.87
		Typical edge beams	12.50	2.00	1.16	2.87	2.87
		Edge beams large span	50.00	2.00	0.71	2.87	2.87
		Interior beams in large span	100.00	2.00	0.57	2.87	2.87
		Typical interior beam	25.00	2.00	0.90	2.87	2.87
		Minor beam	25.00	2.00	0.90	2.87	2.87

In order to derive Roof live load, we used the formula below:

$$L_r = L_0 R_1 R_2 \quad \text{where } 0.58 \leq L_r \leq 0.96$$

The decrease factor R1 may be determined in the following way:

$$1 \quad \text{for } A_T \leq 18.58 \, m^2$$

$$R_1 = 1.2 - 0.011A_T \quad \text{for } 18.58 \, m^2 \leq A_T \leq 55.74 \, m^2$$

$$0.6 \quad \text{for } A_T \geq 55.74 \, m^2$$

In case of normal R2 flat roofs the factor of reduction R2 flat roofs is 1.

Table 3.3.2.6. Roof Live Loads Calculations

Roof Live load						
Structure type	A _t (m ²)	K _{LL}	R1	R2	L0	L
Interior columns	100	4	0.6	1	0.96	0.576
Exterior columns	50	4	0.65	1	0.96	0.624
Corner columns	25	4	0.925	1	0.96	0.888
Edge beams in large span	50	2	0.65	1	0.96	0.624
Typical edge beams	12.5	2	1	1	0.96	0.96
Interior beams in large span	100	2	0.1	1	0.96	0.096
Typical Interior beams	25	2	0.925	1	0.96	0.888
minor beams	25	2	0.925	1	0.96	0.888

Table 3.3.2.7. Load Summary

Load summary			
Dead Load		Live Load	
Floor	Load, kPa	Floor	Load, kPa
Roof	3.91	Roof	0.96
4	6.72	4	4.79
3	6.08	3	4.79
2	6.12	2	4.79
1	6.06	1	4.79

Basement	5.42	Basement	1.92
----------	------	----------	------

3.3.3 Snow Load Calculations

It has been established that the snow loads are not necessary by this project since the site is in Los Angeles, California, which is not prone to snow depositions. Thus, computations of the snow load were not done based on ASCE 7-16 (American Society of Civil Engineers, 2022).

3.3.4 Estimation of member sizes

This project did not conduct a complete comparison between all the three GLRS layout options. Rather a two way slab system has been chosen as the best method to have the necessary structural performance. The main reason is the fact that the building has rather large spans, and the two-way slab is much easier to load and service than the one-way system. Three minor beams were added to achieve sufficient stiffness and make the slab spans manageable to practice-sized dimensions. This design follows common design practice on medium-span residential buildings and satisfies the initial sizing criteria indicated in ACI 318-19.

3.3.4.1. Determination of structural layout

For the beams in typical span, minimum thickness was taken as 1/12 of the length and rounded to the nearest reasonable thickness, and width was determined by half of the thickness.

$$h_{major\ 5} = L/12 = 5/12 = 0.42 \approx 0.45m$$

$$b_{major\ 5} = h/2 = 0.45/2 = 0.225 \approx 0.25m$$

For the beams in large span, prestressed beam design was applied. The sizing of prestressed concrete beam was done preliminarily according to Naaman (2012) and general member heuristic rules based on pretension. In order to have a high degree of serviceability, the practical trial section was chosen to have a depth of 0.70 m and width 0.35 m.

Geometric properties, which were cross-sectional area A_c , moment of inertia I_g , and section modulus S - were calculated directly through dimensions selected. As material parameters, $f_c'=34.47$ Mpa and $E_c=27.6$ Gpa were selected, which are ACI empirical relation:

$$E_c = 4700\sqrt{f'_c}(MPa)$$

The eccentricity of $e_0=0.167$ m (0.25h) was assumed as an initial value: this is a standard e_0 value in the mid-depth tendon. The moment of dead-load that was calculated through SAP2000 which was governing was:

$$M_{DL} = 754.81 \text{ kN} - m$$

The needed prestressing force was determined with the aid of the service stress criterion:

$$F = \frac{\frac{M_s}{S}}{\frac{1}{A} + \frac{e_{mid}}{S}}$$

Giving up a preliminary stress $F_i=3397$ kN (20% loss included). The associated effective prestress was $f_{pe}=1302$ Mpa (30% jacking loss). The quantity of strands needed was determined as:

$$A_{ps} = \frac{F}{f_{pe}}$$

to yield about 1050 mm², or 7 strands of 15.7 mm diameter.

Deflection check and Serviceability check.

The midspan prestress caused camber was determined using:

$$\Delta_{p,mid} = \frac{F_i e_0 L^2}{8E_{ci} I_g}$$

and superimposed to self-weight and live-load deflections. The total deflection that resulted was:

$$\Delta_{total} = 80.09 \text{ mm}$$

This is not exceeding the permitted limit:

$$\Delta_{allow} = L/240 = 20000/240 = 83.33 \text{ mm}$$

The chosen prestressed beam 0.70 m x 0.35 m therefore meets all the serviceability requirements such as tension control at the top fiber, top fiber deflection, and general deflection.

To determine the thickness of slabs needed, the moment of inertia and the elastic modulus of the slab and supporting beam should be calculated at first. These elements have been constructed with concrete grade C40 because they are commonly utilized in the structural elements that carry a greater loading requirement (Base Concrete, 2023). Modulus of elasticity was determined by use of standard empirical expression:

$$f_{cb} = f_{cs} = 40 \text{ MPa}$$

The geometric ratio of the slab to the beam was calculated based on the calculation of the geometric properties:

$$E_b = E_s = 4700\sqrt{f_c} = 29725.41 \text{ MPa}$$

$$I_{b,cr20} = 0.35 \frac{1}{12} b h^3 = 0.35 * \frac{1}{12} * 0.35 * 0.7^3 = 0.0035 \text{ m}^4$$

$$I_{b,cr5} = 0.35 \frac{1}{12} b h^3 = 0.35 * \frac{1}{12} * 0.25 * 0.45^3 = 0.0007 \text{ m}^4$$

$$I_{b,cr\text{ minor}} = 0.35 \frac{1}{12} b h^3 = 0.35 * \frac{1}{12} * 0.2 * 0.35^3 = 0.0003 \text{ m}^4$$

$$I_{s,cr5} = 0.25 \frac{1}{12} L h^3 = 0.25 * \frac{1}{12} * 5 * 0.25^3 = 0.0016 \text{ m}^4$$

Once the members' moments of inertia have been determined, the stiffness ratio should then be calculated:

$$\alpha_{fm} = \frac{E_{cb} I_{b,cr}}{E_{cs} I_{s,cr}}$$

$$\alpha_{f20} = \frac{E_{cb} I_{b,cr}}{E_{cs} I_{s,cr}} = \frac{29725 * 0.0035}{29725 * 0.0016} = 2.151$$

$$\alpha_{f,minor} = \frac{E_{cb} I_{b,cr}}{E_{cs} I_{s,cr}} = \frac{29725 * 0.0003}{29725 * 0.0016} = 0.154$$

$$\alpha_{fm} = \frac{2.151 * 2 + 0.154 * 2}{4} = 1.153 < 2$$

Then the minimum slab thickness was estimated by means of the conventional thickness guideline:

$$h = \frac{l_n (0.8 + f_y / 200000)}{36 + 5\beta(\alpha_{fm} - 0.2)} \geq 5 \text{ in.}$$

and the parameters of contribution can be defined as:

$$L_{yn} = 5 - 0.2 = 4.8 \text{ m} = l_n$$

$$L_{xn} = 5 - 0.35 = 4.65 \text{ m}$$

$$\beta = L_{yn} / L_{xn} = 1.033$$

The span in the calculation is as follows:

$$h = \frac{4.804(0.8 + 60000 / 200000)}{36 + 5 * 1.033(1.153 - 0.2)} = 0.130 \text{ m} \geq 0.127 \text{ m (5 in.)}$$

The slab thickness of $h = 0.13 \text{ m}$ suits the traditional slab system. The resultant concrete quantity is then calculated as:

$$V_{slab} = 5 * 5 * 0.13 = 3.25 \text{ m}^3$$

$$V_{maj \text{ beam } 20} = 0.35 * 0.7 * 20 = 4.9 \text{ m}^3$$

$$V_{maj \text{ beam } 5} = 0.45 * 0.25 * 5 = 0.56 \text{ m}^3$$

$$V_{min\ beam} = 0.35 * 0.2 * 5 = 0.35\ m^3$$

This project was designed to use the two-way slab layout as it is an efficient layout in the distribution of loads and that it is appropriate in the spans. The grid pattern and beam-slab design that we have adopted are shown by Figure 3.3.4.1 below. Blue components are determined as typical major beams, greens are large span beams, and yellows are minor beams.

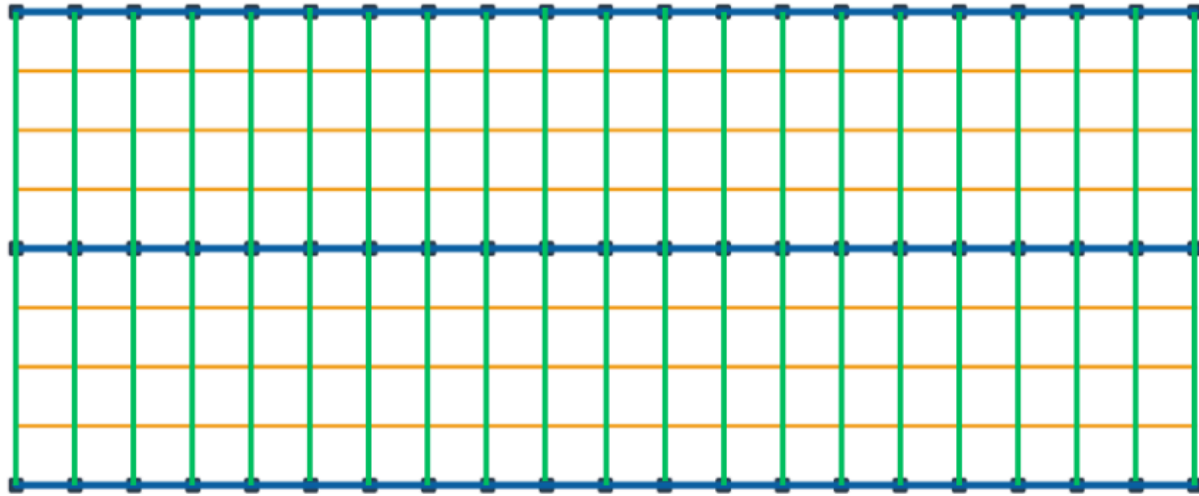


Figure 3.3.4.1.1. Structural layout of the building

3.3.4.2. Calculation of size dimensions of columns

Next we need to determine the size dimensions of columns at each level of the building. It could be done with the help of the self weight of structural members and dead loads and live loads which were previously calculated. Load combination was used to determine the size of columns.

$$w_u = 1.2D + 1.6L + 0.5L_r$$

Table 3.3.4.2.1. Load calculation for interior columns

Interior Column (1 - 4 floor)							
Floor	DL	RL	LL	SL	AT	Factored Load	R(kN)
Basement Slab	6.08	0	2.29	0	100	10.96	1095.84

Floor 1 Slab	6.14	0	2.29	0	100	11.03	1102.95
Floor 2 Slab	6.10	0	2.29	0	100	10.98	1098.31
Floor 3 Slab	6.73	0	2.29	0	100	11.74	1174.43
Floor 4 Slab	4.08	0.576	0.00	0	100	5.19	518.88

Major and minor beam self-weight:

$$SW_{major\ 5} = \rho_{RC} * V_{maj\ beam\ 5} = 25 * 0.56 = 14.14\ kN$$

$$SW_{major\ 20} = \rho_{RC} * V_{maj\ beam\ 5} = 25 * 4.9 = 122.5\ kN$$

$$SW_{minor\ 5} = \rho_{RC} * V_{maj\ beam\ 5} = 25 * 0.35 = 8.65\ kN$$

Table 3.3.4.2.2. Load calculation for interior columns including large span beam self-weight

Interior Column + 20 Beam Self Weight					
Floor	B Self Weight	wb	Rb	Add Rcol	Factored Load
Basement Slab	122.50	6.13	61.25	122.50	147.00
Floor 1 Slab	122.50	6.13	61.25	122.50	147.00
Floor 2 Slab	122.50	6.13	61.25	122.50	147.00
Floor 3 Slab	122.50	6.13	61.25	122.50	147.00
Floor 4 Slab	122.50	6.13	61.25	122.50	147.00

Table 3.3.4.2.3. Load calculation for interior columns including typical and minor beam self-weight

Interior Column + 5 Beam Self Weight + Minor Beam Self Weight						
Floor	B Self Weight	wb	Rb	Add Rcol	Factored Load	Total R col
Basement Slab	14.14	2.83	7.07	14.14	16.97	1290.94

Floor 1 Slab	14.14	2.83	7.07	14.14	16.97	1298.05
Floor 2 Slab	14.14	2.83	7.07	14.14	16.97	1293.41
Floor 3 Slab	14.14	2.83	7.07	14.14	16.97	1369.53
Floor 4 Slab	14.14	2.83	7.07	10.61	12.73	709.74
Minor Beam SW	8.65	1.73	4.32	25.94	31.13	

Also, the calculation of the column sizes of individual stories based on the weight of columns was done through the iteration method.

$$P_u = w_u * A_T$$

$$\phi P_n = 0.8\phi(0.85f'_c A_c + f_y A_s)$$

Table 3.3.4.2.4. Iterative method for minimum interior column size estimation

Interior Column Size Estimation											
Basement											
Iteration	b (mm)	H (mm)	p (kN/m^3)	W1 (kN)	W1+axial	φ	fc (MPa)	fy	Ag	b'	Final
1.00	1000.00	3300.00	25.00	99.00	6060.67	0.65	40.00	420.00	307848.49	554.84	650.00
2.00	553.19	3300.00	25.00	30.30	5992.15	0.65	40.00	420.00	276207.30	525.55	
3.00	523.96	3300.00	25.00	27.18	5989.02	0.65	40.00	420.00	276062.91	525.42	
Floor 1											
Iteration	b (mm)	H (mm)	p (kN/m^3)	W1 (kN)	W1+axial	φ	fc (MPa)	fy	Ag	b'	Final
1.00	1000.00	4200.00	25.00	126.00	4796.73	0.65	40.00	420.00	221104.70	470.22	550.00
2.00	468.51	4200.00	25.00	27.66	4698.59	0.65	40.00	420.00	216580.92	465.38	
3.00	463.65	4200.00	25.00	27.09	4698.02	0.65	40.00	420.00	216554.64	465.35	
Floor 2											
Iteration	b (mm)	H (mm)	p (kN/m^3)	W1 (kN)	W1+axial	φ	fc (MPa)	fy	Ag	b'	Final

			3)								
1.00	1000.00	4200.00	25.00	126.00	3498.68	0.65	40.00	420.00	161271.13	401.59	500.00
2.00	399.67	4200.00	25.00	20.13	3393.00	0.65	40.00	420.00	156399.83	395.47	
3.00	393.52	4200.00	25.00	19.51	3392.39	0.65	40.00	420.00	156371.54	395.44	
Floor 3											
Iteration	b (mm)	H (mm)	p (kN/m^3)	W1 (kN)	W1+axi al	φ	f _c (MPa)	f _y	A _g	b'	Final
1.00	1000.00	4200.00	25.00	126.00	2205.27	0.65	40.00	420.00	101651.59	318.83	400.00
2.00	316.52	4200.00	25.00	12.62	2092.08	0.65	40.00	420.00	96434.03	310.54	
3.00	308.15	4200.00	25.00	11.96	2091.42	0.65	40.00	420.00	96403.72	310.49	
Floor 4											
Iteration	b (mm)	H (mm)	p (kN/m^3)	W1 (kN)	W1+axi al	φ	f _c (MPa)	f _y	A _g	b'	Final
1.00	1000.00	6000.00	25.00	180.00	889.74	0.65	40.00	420.00	41012.43	202.52	300.00
2.00	199.07	6000.00	25.00	7.13	717.12	0.65	40.00	420.00	33055.65	181.81	
3.00	177.94	6000.00	25.00	5.70	715.69	0.65	40.00	420.00	32989.63	181.63	

The same procedure was done for all exterior and corner columns. All the tables can be found in the appendix.

Table 3.3.4.2.5. Minimum column sizes

Final Column Size							
Floor	Interior (m)	Exterior 5 (m)	Exterior 20 (m)	Edge (m)	Exterior 5 Curtain (m)	Height (m)	Density (kN/m³)
Basement	0.650	0.500	0.600	0.450	0.500	3.3	25
1	0.550	0.450	0.550	0.400	0.450	4.2	25
2	0.500	0.400	0.450	0.350	0.400	4.2	25
3	0.400	0.300	0.350	0.300	0.300	4.2	25
4	0.300	0.300	0.300	0.300	0.300	6	25

Considering further drift analysis, it was decided to increase the column sizes and equalize them among the floor. The final choices are seen in the table below.

Table 3.3.4.2.5. Final column sizes

Final Column Size	
Floor	Column size
Basement	0.650
1	0.650
2	0.650
3	0.650
4	0.500

As it can be observed, in Table 3.3.4.2.5, the smallest dimension of the column is 0.5 m, which does not violate the established ACI 318-19 rules of requiring that the minimum dimension of the column should not be less than 12 in or 300 mm. and the dimension across it should be at least 0.4 (ACI, 2019). These requirements are met in all the cases in the column size distribution of this residential complex.

3.3.5. Assigning Gravity Forces to the SAP 2000 model

For Load Assignment Procedure Figure 3.3.51 below was created to clearly demonstrate the chosen frame. Please note that in all the calculations, Figure 3.3.5.1 below was referred to when frame numbers were mentioned. However, due to SAP 2000 technical limitations, the grid line names are swapped.

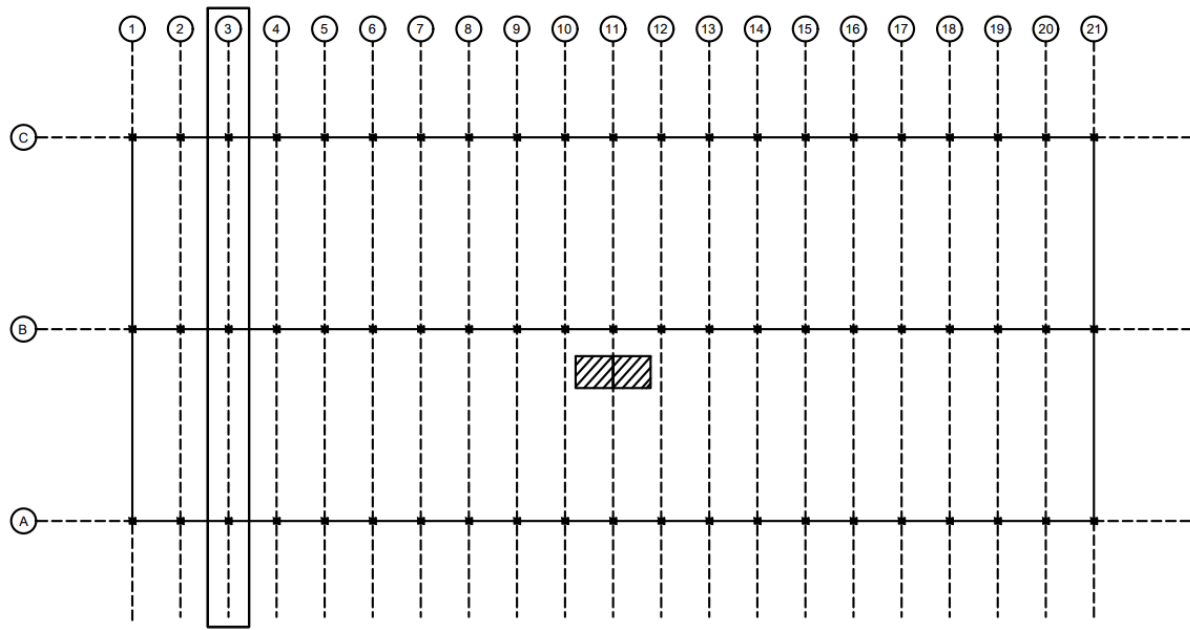


Figure 3.3.5.1. Chosen Frame for Load Assignment

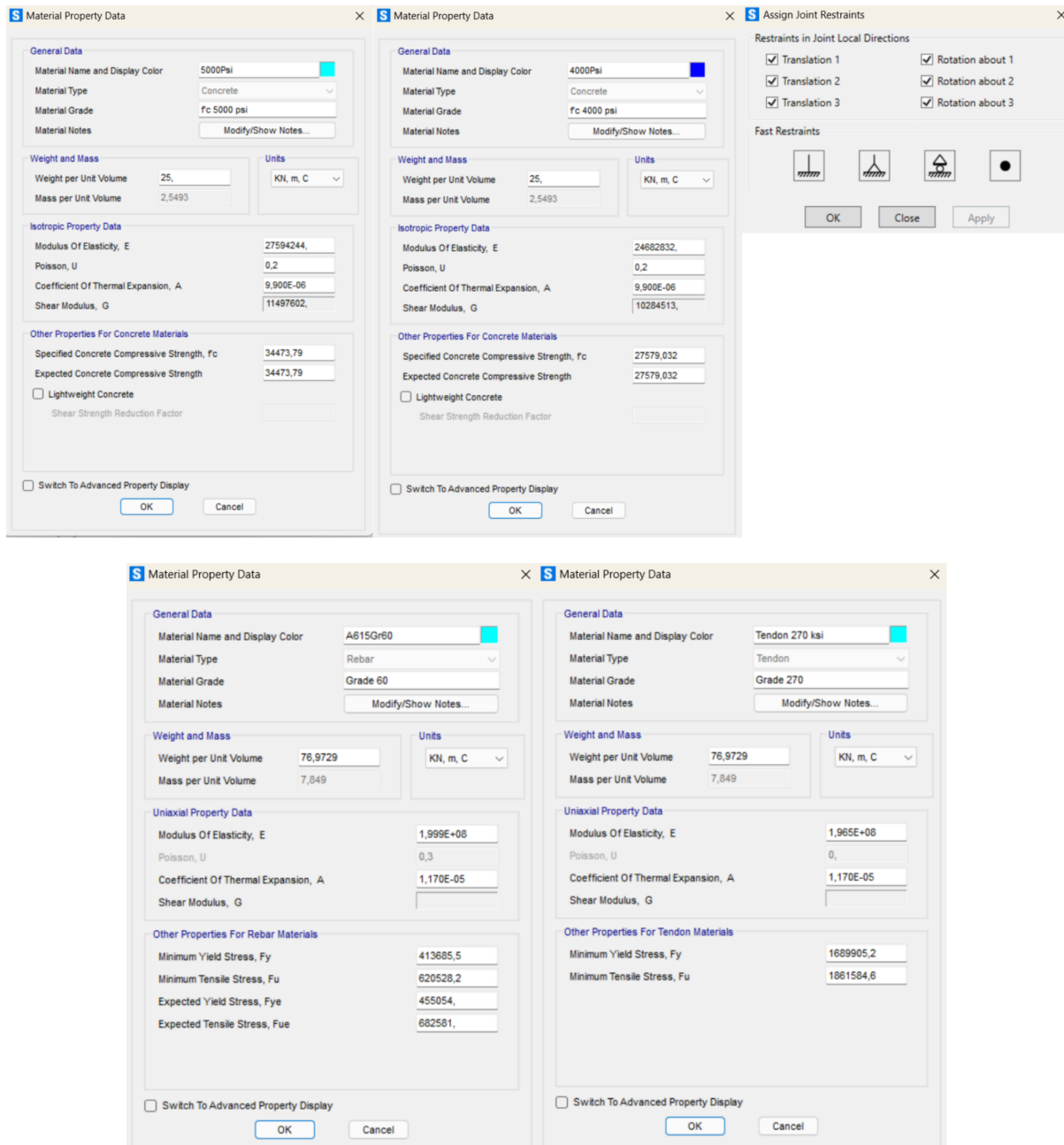


Figure 3.3.5.2. Material Property Data and Joint Restraints

After entering the material property data, the section properties for both the reinforced major beams and the columns were added to the software. As shown in Figure X, the dimensions defined in SAP2000 match exactly with those presented earlier in this section.

S Rectangular Section [X]

Section Name column 650 **Display Color** ☒

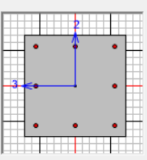
Section Notes

Dimensions

Depth (t3) 0,65

Width (t2) 0,65

Section



Properties

Material

+ 4000Psi

Property Modifiers

S Frame Property/Stiffness Modification Factors [X]

Property/Stiffness Modifiers for Analysis

Cross-section (axial) Area 1

Shear Area in 2 direction 1

Shear Area in 3 direction 1

Torsional Constant 1

Moment of Inertia about 2 axis 0,7

Moment of Inertia about 3 axis 0,7

Mass 1

Weight 1

S Rectangular Section [X]

Section Name column 550 **Display Color** ☒

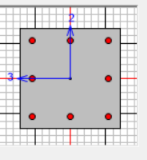
Section Notes

Dimensions

Depth (t3) 0,55

Width (t2) 0,55

Section



Properties

Material

+ 4000Psi

Property Modifiers

S Frame Property/Stiffness Modification Factors [X]

Property/Stiffness Modifiers for Analysis

Cross-section (axial) Area 1

Shear Area in 2 direction 1

Shear Area in 3 direction 1

Torsional Constant 1

Moment of Inertia about 2 axis 0,7

Moment of Inertia about 3 axis 0,7

Mass 1

Weight 1

S Rectangular Section [X]

Section Name **Display Color** ☐

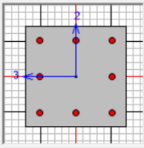
Section Notes

Dimensions

Depth (t3)

Width (t2)

Section



Properties

Material

4000Psi

Property Modifiers

S Frame Property/Stiffness Modification Factors [X]

Property/Stiffness Modifiers for Analysis

Cross-section (axial) Area	1
Shear Area in 2 direction	1
Shear Area in 3 direction	1
Torsional Constant	1
Moment of Inertia about 2 axis	0,7
Moment of Inertia about 3 axis	0,7
Mass	1
Weight	1

S Rectangular Section [X]

Section Name **Display Color** ☐

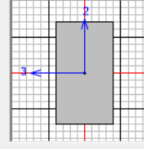
Section Notes

Dimensions

Depth (t3)

Width (t2)

Section



Properties

Material

4000Psi

Property Modifiers

S Frame Property/Stiffness Modification Factors [X]

Property/Stiffness Modifiers for Analysis

Cross-section (axial) Area	1
Shear Area in 2 direction	1
Shear Area in 3 direction	1
Torsional Constant	1
Moment of Inertia about 2 axis	0,35
Moment of Inertia about 3 axis	0,35
Mass	1
Weight	1

S Rectangular Section [X]

Section Name **Display Color** ■

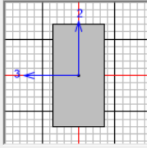
Section Notes

Dimensions

Depth (t3)

Width (t2)

Section



Properties

Material

5000Psi

Property Modifiers

S Frame Property/Stiffness Modification Factors [X]

Property/Stiffness Modifiers for Analysis

Cross-section (axial) Area	1
Shear Area in 2 direction	1
Shear Area in 3 direction	1
Torsional Constant	1
Moment of Inertia about 2 axis	0,35
Moment of Inertia about 3 axis	0,35
Mass	1
Weight	1

S Rectangular Section [X]

Section Name **Display Color** ■

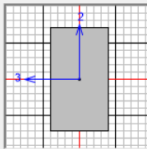
Section Notes

Dimensions

Depth (t3)

Width (t2)

Section



Properties

Material

4000Psi

Property Modifiers

S Frame Property/Stiffness Modification Factors [X]

Property/Stiffness Modifiers for Analysis

Cross-section (axial) Area	1
Shear Area in 2 direction	1
Shear Area in 3 direction	1
Torsional Constant	1
Moment of Inertia about 2 axis	0,35
Moment of Inertia about 3 axis	0,35
Mass	1
Weight	1

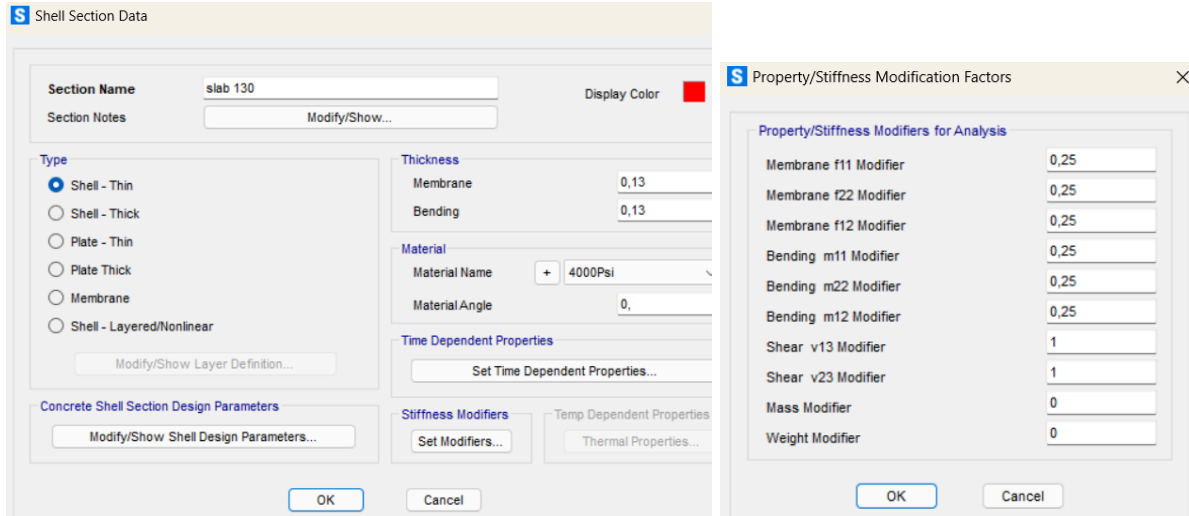


Figure 3.3.5.3. Structural Element Section Properties

With all required parameters entered into SAP2000, the dead, live, and roof live loads were assigned in both the 2D and 3D models, as shown below. In the 3D view, two load distribution configurations were selected: Uniform Distribution to Area and Uniform Distribution to Frame.

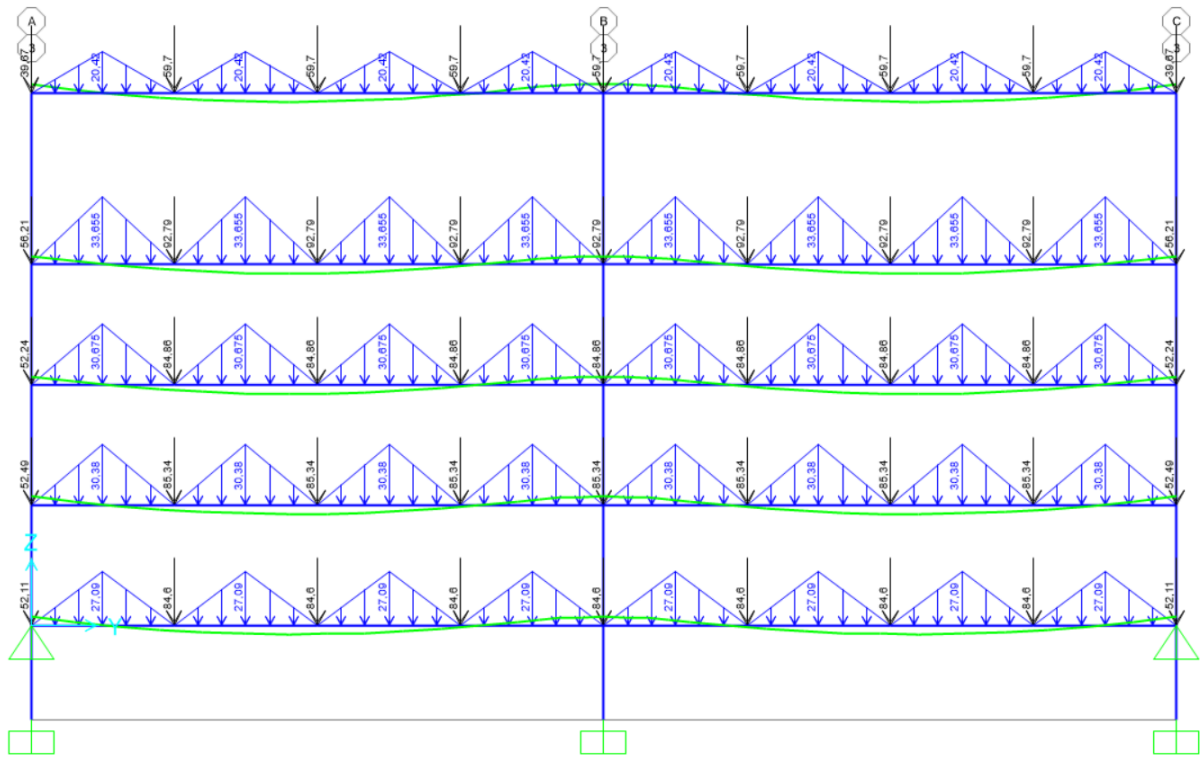


Figure 3.3.5.4. 2D Load Assignment for Dead Loads

The point loads shown with black arrows represent the dead loads transferred from the perpendicular minor and major beams onto the selected frame. This approach provides more accurate and realistic results. The same method is applied to the live load and roof live load shown in the following two figures.

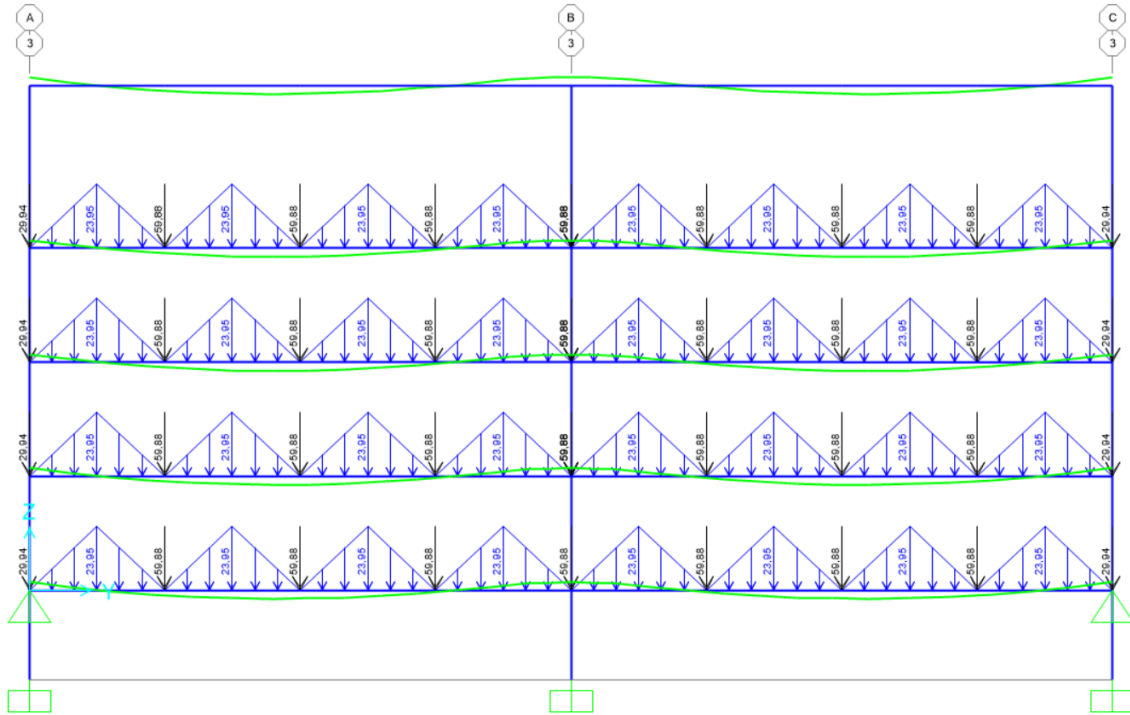


Figure 3.3.5.4. 2D Load Assignment for Live Load

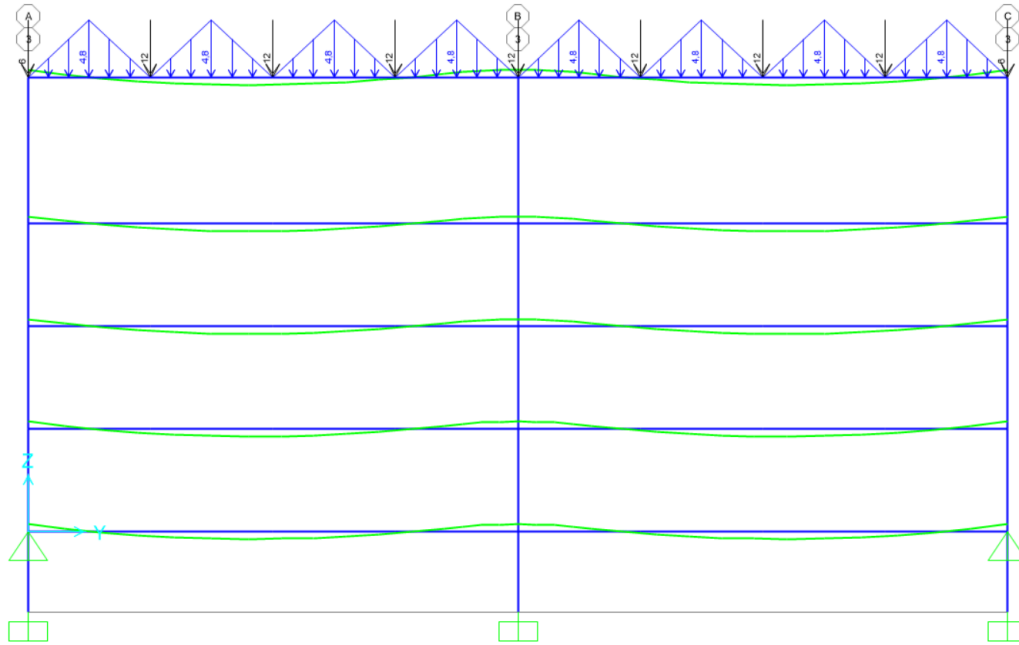


Figure 3.3.5.4. 2D Load Assignment for Roof Live Load

The analysis of the 2D load assignments is provided in the Appendix.

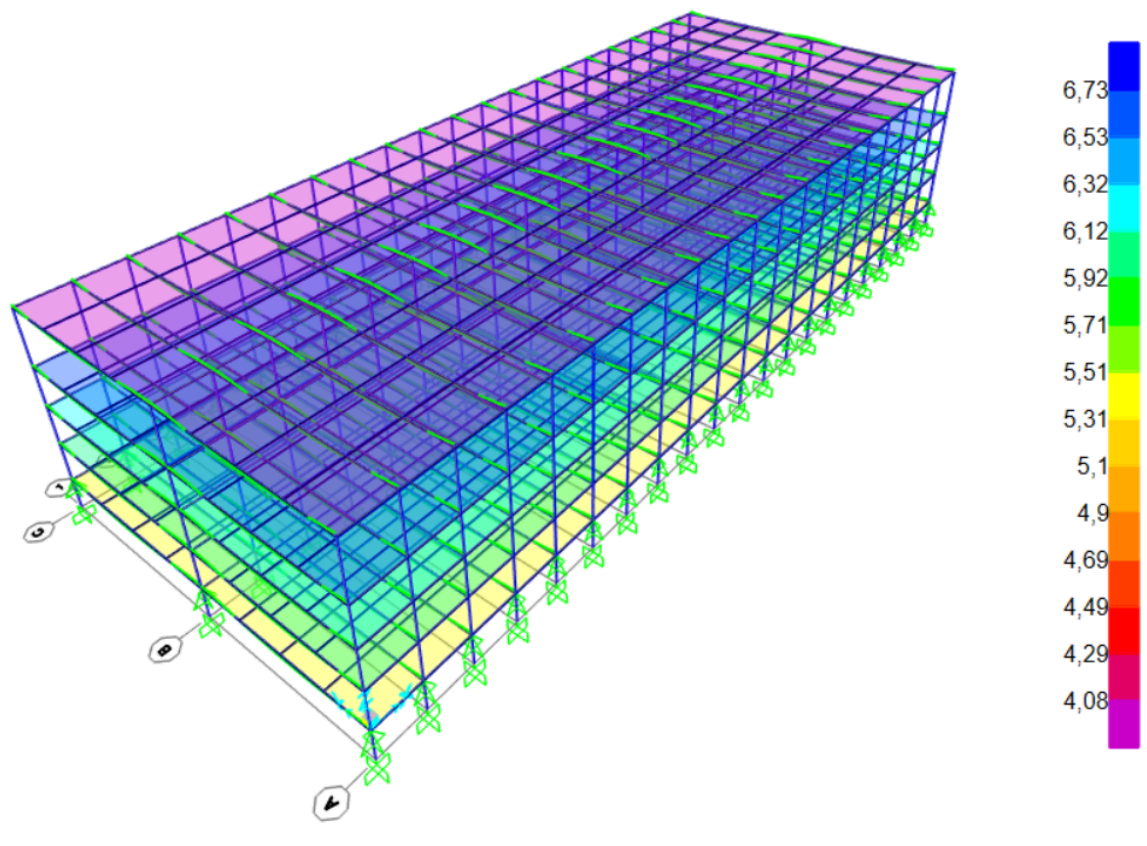


Figure 3.3.5.5. 3D Uniform Dead load distributed to Area (Slab)

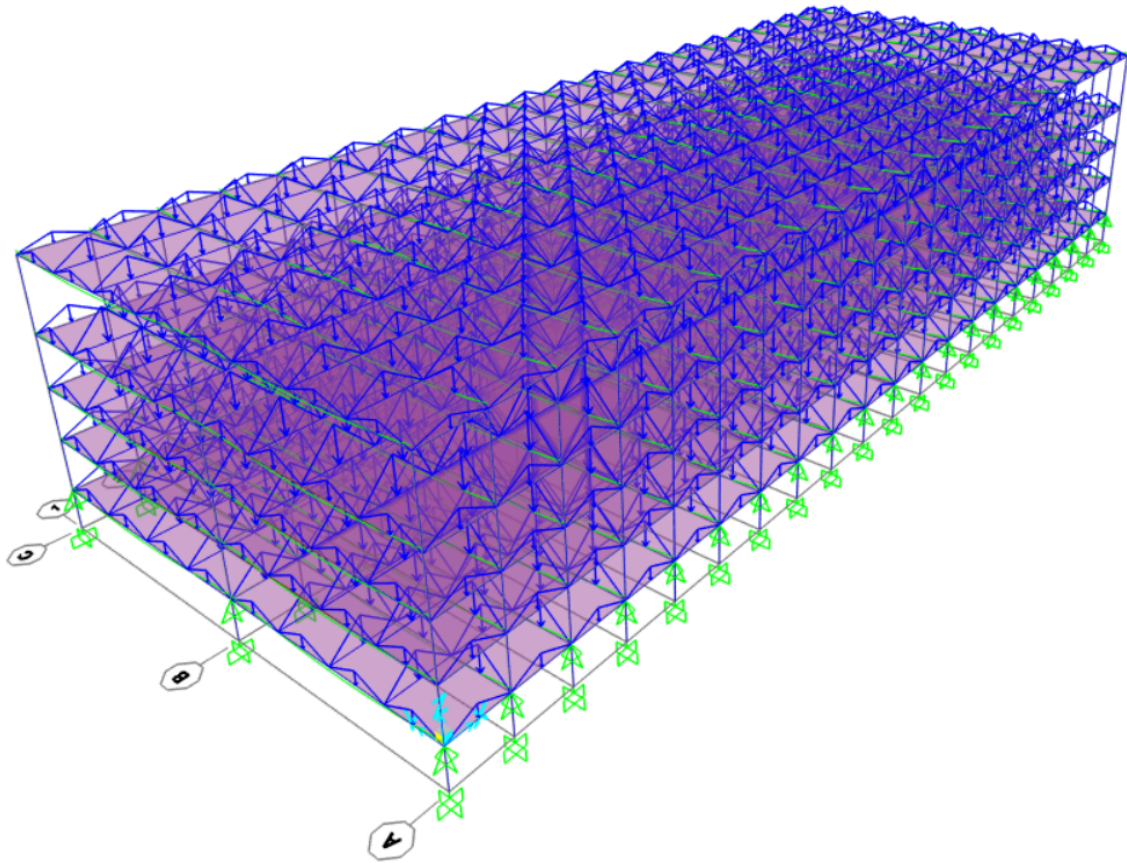


Figure 3.3.5.6. 3D Uniform Dead load distributed to Frames

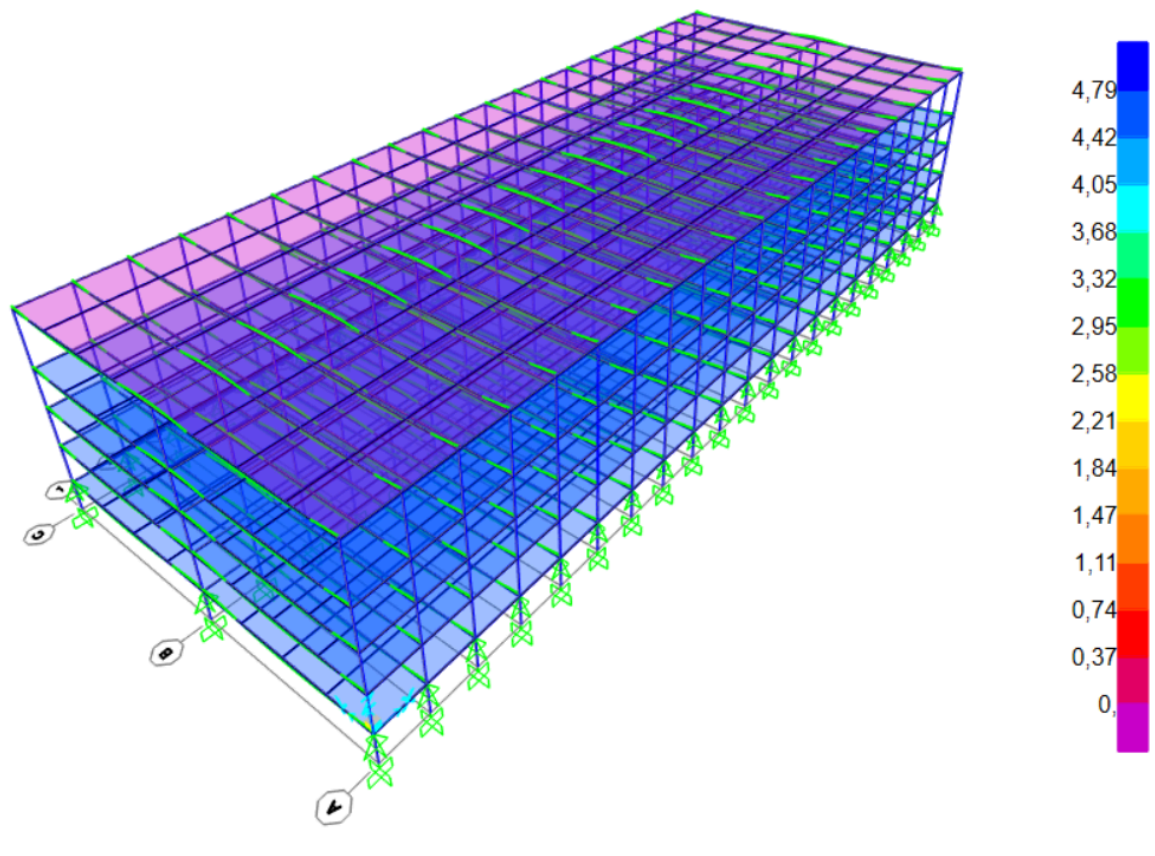


Figure 3.3.5.7. 3D Uniform Live load distributed to Area (Slab)

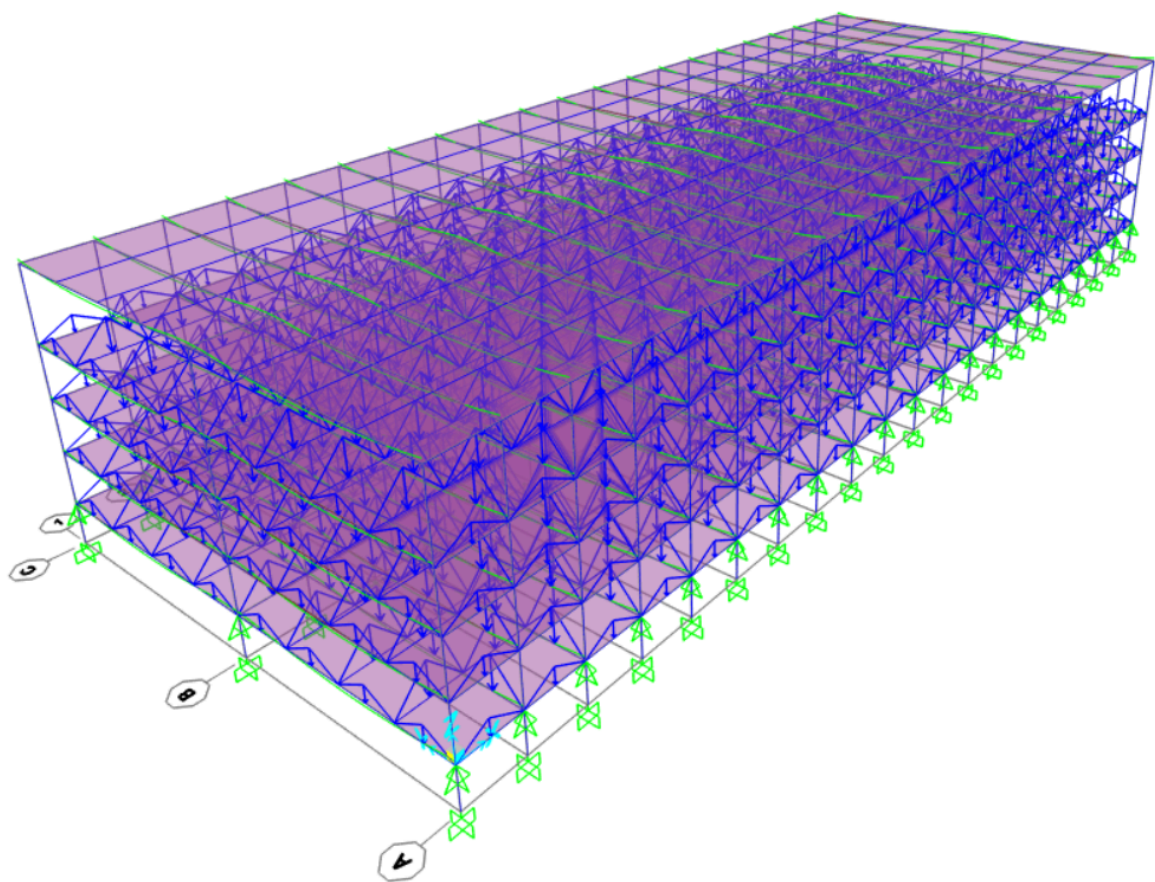


Figure 3.3.5.8. 3D Uniform Live load distributed to Frames

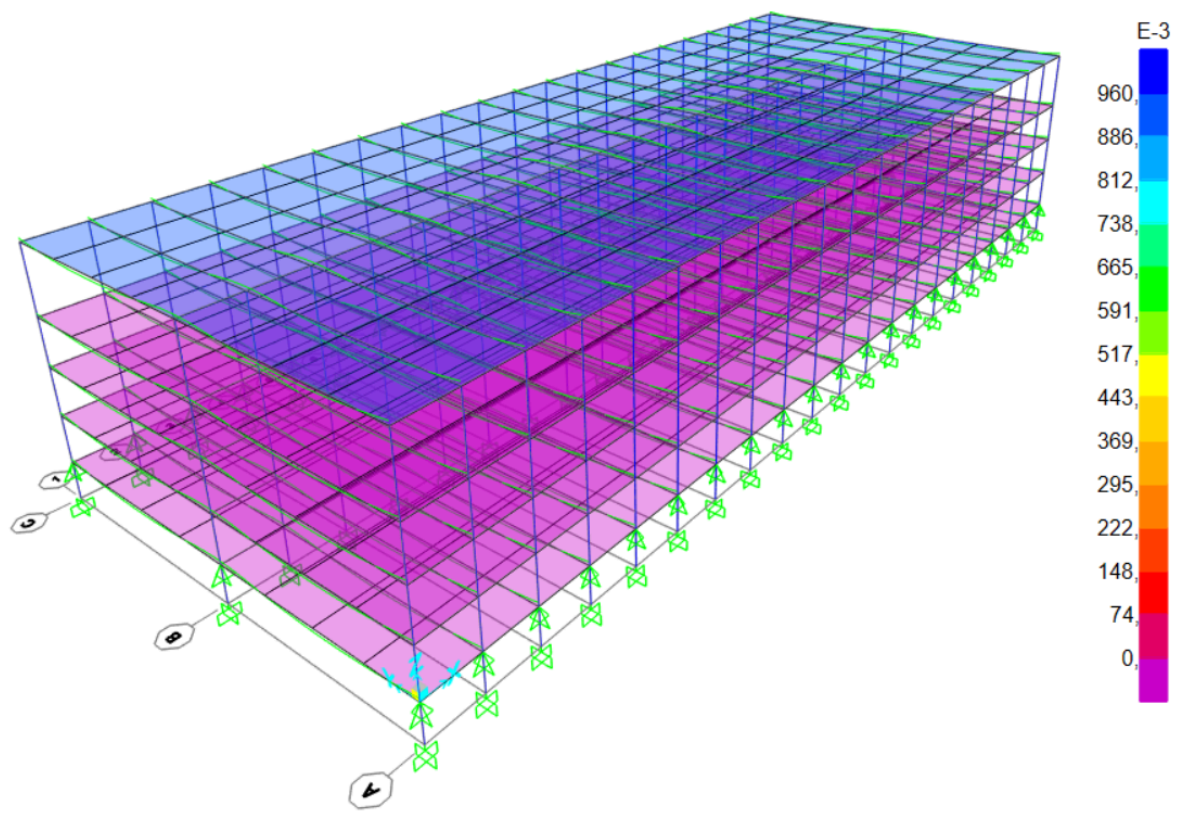


Figure 3.3.5.9. 3D Uniform Roof Live load distributed to Area (Slab)

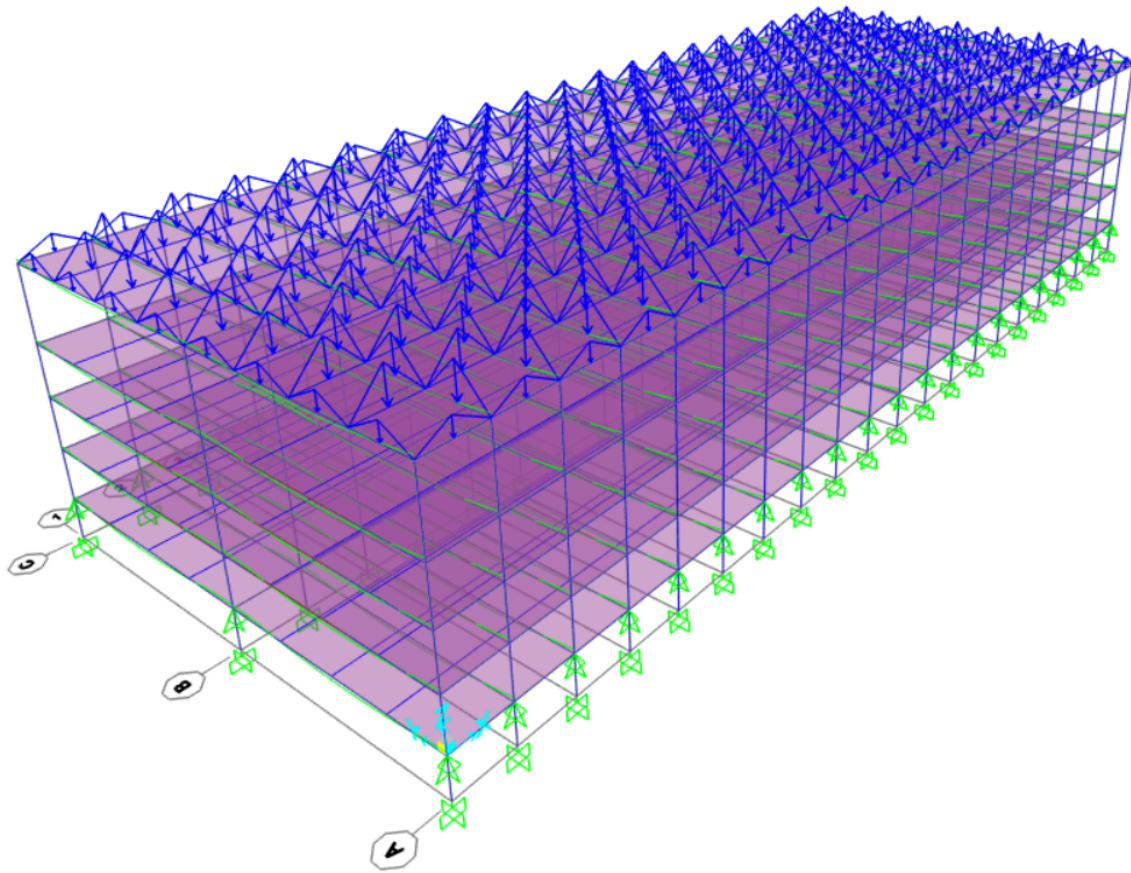


Figure 3.3.5.10. 3D Uniform Roof Live load distributed to Frames

3.4 Analysis and Design of Lateral Force Resisting System (LFRS)

3.4.1. Wind load with torsional effect

Similar to the calculation of GLRS, the calculation of the wind load was based on the recommendations in ASCE 7-22 (American Society of Civil Engineers, 2022). The implementation of the ASCE Hazard Tool map was carried out to calculate the basic wind speed in 130 West College Street, Los Angeles, California (ASCE Hazard Tool, 2024). Based on the data provided in the tool, the average wind speed in the place of selection is 42 m/s. The data obtained in the calculation of wind load was as follows, the data is provided in ASCE 7-22:

- Based on Table 26.6-1 Wind Directionality Factor, K_d , of Buildings with Main Wind Force Resisting System equal to 0.85.
- Section 26.7.3 has exposed the project to the B category.

- Topographical factor, $K_{zt} = 1$ because the construction site is flat.

In order to determine the constants of the Exposure Category B, the information of Table X.X should be utilized.

Table 3.4.1.1. Exposure Category Constants

SI Units										
Exposure	α	z_g (m)	$\hat{\alpha}$	$\hat{b}$	$\bar{\alpha}$	$\bar{b}$	c	l (m)	$\bar{e}$	z_{min} (m)*
B	7.5	1,000	1/7.5	0.84	1/4.5	0.47	0.30	97.54	1/3.0	9.14
C	9.8	750	1/9.8	1.00	1/6.4	0.66	0.20	152.40	1/5.0	4.57
D	11.5	590	1/11.5	1.09	1/8.0	0.78	0.15	198.12	1/8.0	2.13

* z_{min} = Minimum height used to ensure that the equivalent height $\bar{z}$ is the greater of $0.6h$ or z_{min} . For buildings or other structures with $h \leq z_{min}$, $\bar{z}$ shall be taken as z_{min} .

The Height Effect requires first that the velocity pressure exposure coefficient K_z be assessed:

$$K_z = 2.01 \left(\frac{4.572}{z_g} \right)^{2/\alpha}, \text{ for } z < 15ft$$

$$K_z = 2.01 \left(\frac{z}{z_g} \right)^{2/\alpha}, \text{ for } 15ft < z < z_g$$

The velocity pressure depends on the elevation, as seen in the equations above and thus Table X.X was created by using these equations at each height taken into account.

Table 3.4.1.2. Height Effect Coefficient Calculations

Exposure and Height Effect		
Story	z_i (m)	K_z
parapets	19.67	0.87
4	18.60	0.86
3	12.60	0.77
2	8.40	0.68
1	4.20	0.57

Then, Gust Effect has to be introduced to both longitudinal and transverse directions to receive the final wind loads.

In longitudinal direction: $B=100m$, $L=40m$, total number of frames is 21.

In transverse direction: B=40m, L=100m, total frame number is 3.

Longitudinal Direction, Case 1 (Y-axis)

The first is to categorize the structure in order to choose a method of calculation of gusts. The building has a total height of 18.6 m, which is above the 18 m mark and as such is not a low-rise structure. The gust effect should therefore be measured according to the right process that has been stipulated in the applicable code requirements. Firstly fundamental natural frequency of the building should be found according to the formula for moment-resisting frame buildings:

$$18.6 \text{ m} = 61.02 \text{ ft}$$

$$n_a = \frac{43.5}{h^{0.9}} = \frac{43.5}{61.02^{0.9}} = 1.07 > 1$$

Since fundamental natural frequency is more than 1 the building is classified as rigid and gust effect coefficient, G_f , is taken as 0.85.

In order to compute the relevant coefficients of external pressures, the proportion between length and width of the building should be considered first:

$$L/B = 40/100 = 0.4 < 1$$

The external pressure coefficients C_p were obtained with the help of this ratio, and are indicated in Table 3.4.1.3.

Table 3.4.1.3. External Pressure Coefficients

Wall Pressure Coefficients, C_p			
Surface	L/B	C_p	Use With
Windward Wall	All values	0.8	q_z
Leeward Wall	0-1	-0.5	q_h
	2	-0.3	
	≥ 4	-0.2	
Side Wall	All values	-0.7	q_h

Therefore, C_p for windward wall is 0.8 and for leeward wall is -0.5. Also, for the parapets, according to the ASCE 7-22 combined net pressure coefficient is +1.5 for windward parapets and -1 for leeward parapets.

Transverse Direction, Case 1 (X-axis)

This process is reiterated in the transverse direction with the only difference that the new L/B ratio has been updated. The gust factor was also determined as 0.85.

In the transverse direction the geometric ratio is:

$$B/L = 100/40 = 2.5$$

Using Table 3.4.1.3. gives us new external pressure coefficients: $C_{p, windward} = 0.8$ and

$$C_{p, leeward} = -0.275$$

With the calculation of these coefficients, the wind loads can now be determined. Four potential wind load cases have to be taken into consideration as highlighted in Figure 3.14 as specified in ASCE 7-22.

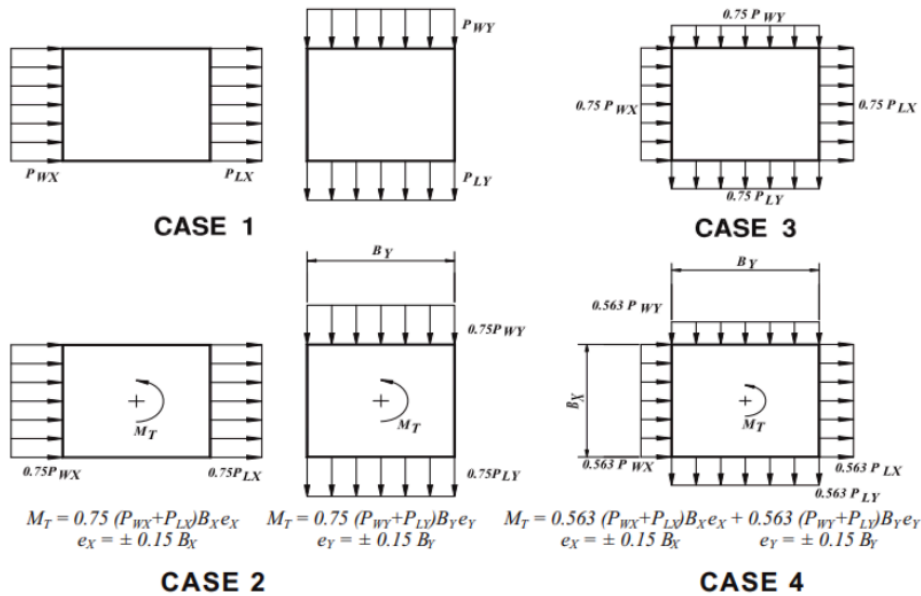


Figure 3.4.1.1. Wind Load Cases (American Society of Civil Engineers, 2022)

The expressions of velocity pressure and wind pressure have to be computed in order to obtain the wind forces on each floor:

$$q_z = 0.613 K_z K_{zt} K_d V^2$$

$$P = q_z GC_{p,windward} - q_h GC_{p,leeward}$$

The wind force of all the intermediate floors (i.e. all floors except the roof) is given by the following equation:

$$F_i = \frac{B}{\text{number of frames}} * (P_{i+1} * \frac{h_{i+1}}{2} + P_i * \frac{h_i}{2})$$

In case of the top floor, the wind load is calculated with the modified expression presented below and takes into consideration the height of the tributary that is smaller at the roof level:

$$F_{top} = \frac{B}{\text{number of frames}} * (P_{top} * \frac{h_{top}}{2} + P_{parapets} * h_{parapets})$$

As Figure 3.4.1.1.1 shows, the main difference between Cases 1 and 3 and Cases 2 and 4 is that the latter involve the torsional effects. The torsional part is calculated with the help of the following expressions:

$$e_x = 0.15 * B = 0.15 * 100 = 15m$$

$$e_y = 0.15 * L = 0.15 * 40 = 6m$$

Having all the necessary formulas and the appropriate structural features taken into account, one can now prepare the wind load tables of all the four cases.

Table 3.4.1.4. Wind Forces for Case 1, Longitudinal Direction (Y-axis)

Story	h (m)	Kz	q (Pa)	Gf	Pw (Pa)	Pl (Pa)	P (Pa)	Fi (kN)	F per frame
4	6.00	0.86	788.77	0.85	536.36	-340.63	876.99	445.33	21.21
3	4.20	0.77	705.71	0.85	479.88	-340.63	820.51	435.40	20.73
2	4.20	0.68	628.51	0.85	427.39	-340.63	768.01	333.59	15.89
1	4.20	0.57	528.24	0.85	359.21	-340.63	699.83	308.25	14.68

Table 3.4.1.5. Wind Forces for Case 1, Transverse Direction (X-axis)

Story	h (m)	Kz	q (Pa)	Gf	Pw (Pa)	Pl (Pa)	P (Pa)	Fi (kN)	F per frame
4	6.00	0.86	788.77	0.85	536.36	-187.35	723.71	159.74	53.25
3	4.20	0.77	705.71	0.85	479.88	-187.35	667.23	142.89	47.63
2	4.20	0.68	628.51	0.85	427.39	-187.35	614.73	107.68	35.89
1	4.20	0.57	528.24	0.85	359.21	-187.35	546.55	97.55	32.52

Table 3.4.1.6. Wind Forces for Case 2, Longitudinal Direction (Y-axis)

Story	Pw (Pa)	Pl (Pa)	Mt (kN*m)	0.75Fi (kN)	F direct (kN)
4	536.36	-335.23	226.28	334.00	15.90
3	479.88	-335.23	162.73	326.55	15.55
2	427.39	-335.23	103.68	250.19	11.91
1	359.21	-335.23	26.98	231.19	11.01

Table 3.4.1.6. Wind Forces for Case 2 Transverse Direction (X-axis)

Story	Pw (Pa)	Pl (Pa)	Mt (kN*m)	0.75Fi (kN)	F direct (kN)
4	536.36	-184.37	63.36	39.93	13.31
3	479.88	-184.37	53.19	35.72	11.91
2	427.39	-184.37	43.74	26.92	8.97
1	359.21	-184.37	31.47	24.39	8.13

In order to show clearly the torsional effects caused by load of wind, a distinct Table 3.4.1.7 was developed of Case 2 both longitudinally and in the transverse direction.

Table 3.4.1.7. Wind Force Calculations for third floor in both directions including the torsional effect (Case 2)

3rd floor	
Case 2 (Y-axis)	Case 2 (X-axis)

Frame	Ftortion (kN)	Fdirect (kN)	Ftotal (kN)	Frame	Ftortion (kN)	Fdirect (kN)	Ftotal (kN)
1	-0.42	15.55	15.13	A	-1.33	11.91	10.58
2	-0.38	15.55	15.17	B	0.00	11.91	11.91
3	-0.34	15.55	15.21	C	1.33	11.91	13.24
4	-0.30	15.55	15.25				
5	-0.25	15.55	15.30				
6	-0.21	15.55	15.34				
7	-0.17	15.55	15.38				
8	-0.13	15.55	15.42				
9	-0.08	15.55	15.47				
10	-0.04	15.55	15.51				
11	0.00	15.55	15.55				
12	0.04	15.55	15.59				
13	0.08	15.55	15.63				
14	0.13	15.55	15.68				
15	0.17	15.55	15.72				
16	0.21	15.55	15.76				
17	0.25	15.55	15.80				
18	0.30	15.55	15.85				
19	0.34	15.55	15.89				
20	0.38	15.55	15.93				
21	0.42	15.55	15.97				

The rest of the detailed floor-by-floor tables are shown in the Appendix. The calculation process was repeated in Case 3 and Case 4, and the results were obtained and put in Table 3.4.1.8.

Table 3.4.1.9. Wind Force Calculations for Case 3

Y-Axis Case 3									
Story	h (m)	Kz	q (Pa)	Gf	Pw	Pl (Pa)	P (Pa)	Fi (kN)	F per

					(Pa)				frame
4	6.0	0.86	788.77	0.85	536.36	-340.63	876.99	334.00	15.90
3	4.2	0.77	705.71	0.85	479.88	-340.63	820.51	326.55	15.55
2	4.2	0.68	628.51	0.85	427.39	-340.63	768.01	250.19	11.91
1	4.2	0.57	528.24	0.85	359.21	-340.63	699.83	231.19	11.01

X-Axis Case 3

Story	h (m)	Kz	q (Pa)	Gf	Pw (Pa)	Pl (Pa)	P (Pa)	Fi (kN)	F per frame
4	6.0	0.86	788.77	0.85	536.36	-187.35	723.71	119.80	39.93
3	4.2	0.77	705.71	0.85	479.88	-187.35	667.23	107.17	35.72
2	4.2	0.68	628.51	0.85	427.39	-187.35	614.73	80.76	26.92
1	4.2	0.57	528.24	0.85	359.21	-187.35	546.55	73.16	24.39

Table 3.4.1.10. Wind Force Calculations for Case 4

Story	Mt (kN*m)	0.563F1 (kN)	0.563F2 (kN)	F1 direct (kN)	F2 direct (kN)
4	328.40	250.72	29.98	11.94	9.99
3	217.42	245.13	26.82	11.67	8.94
2	162.09	187.81	20.21	8.94	6.74
1	110.66	173.54	18.31	8.26	6.10

As mentioned before, the complete list of intermediary tables is presented at the Appendix. To be clear, Table 3.4.1.11 below is a summary of the total wind forces including both direct and torsional forces with respect to the selected case in the two main directions.

Table 3.4.1.11. Wind Force Calculations for third floor in both directions including the torsional effect (Case 4)

3rd floor	
Case 4 (Y-axis)	Case 4 (X-axis)

Frame	Ftorsion (kN)	Fdirect (kN)	Ftotal (kN)	Frame	Ftorsion (kN)	Fdirect (kN)	Ftotal (kN)
1	-0.56	11.67	11.11	A	-5.44	8.94	3.50
2	-0.51	11.67	11.16	B	0.00	8.94	8.94
3	-0.45	11.67	11.22	C	5.44	8.94	14.37
4	-0.40	11.67	11.28				
5	-0.34	11.67	11.33				
6	-0.28	11.67	11.39				
7	-0.23	11.67	11.45				
8	-0.17	11.67	11.50				
9	-0.11	11.67	11.56				
10	-0.06	11.67	11.62				
11	0.00	11.67	11.67				
12	0.06	11.67	11.73				
13	0.11	11.67	11.79				
14	0.17	11.67	11.84				
15	0.23	11.67	11.90				
16	0.28	11.67	11.96				
17	0.34	11.67	12.01				
18	0.40	11.67	12.07				
19	0.45	11.67	12.12				
20	0.51	11.67	12.18				
21	0.56	11.67	12.24				

3.4.2. Seismic Load Calculation

The main process used to compute seismic design forces to be carried out in compliance with ASCE 7 (American Society of Civil Engineers, 2022) is the Equivalent Lateral Force (ELF) procedure. The initial process in the implementation of ELF approach is the establishment of the site soil category. According to the Geotechnical Investigation Report, the site of the project belongs to Site Class D (stiff soil).

Seismic Data

S_s	2.28
S_1	0.73
S_{MS}	2.29
S_{M1}	1.68
S_{DS}	1.53
S_{D1}	1.12
T_L	8
PGA_M	0.86
V_{S30}	260
Seismic Design Category	D

Figure 3.4.2.1. ASCE Hazard Tool Seismic Design Data

From the Hazard Tool $S_s=2.28$, $S_1=0.73$, $T_L=8s$. Next step is to identify soil condition correction coefficients from Figure 3.4.2.2 below:

F_a					
Site Class	$S_s \leq 0.25$	$S_s = 0.5$	$S_s = 0.75$	$S_s = 1.0$	$S_s \geq 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7				

F_v					
Site Class	$S_1 \leq 0.1$	$S_1 = 0.2$	$S_1 = 0.3$	$S_1 = 0.4$	$S_1 \geq 0.5$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	See Section 11.4.7				

Figure 3.4.2.2. Site Class Table

From the table, $F_a=1$ and $F_v=1.5$. So, S_{MS} and S_{M1} is computed as follows:

$$S_{MS} = F_a * S_s = 1 * 2.28 = 2.28 g$$

$$S_{M1} = F_v * S_1 = 1.5 * 0.73 = 1.095 g$$

Then, adjustments were done on design acceleration parameters:

$$S_{DS} = \frac{2}{3} * S_{MS} = \frac{2}{3} * 2.28 = 1.52$$

$$S_{D1} = \frac{2}{3} * S_{M1} = \frac{2}{3} * 1.095 = 0.73$$

The Seismic Design Category of the construction project has been obtained after the calculation of all the required seismic coefficients. The way Seismic Design Category can be determined has been clearly illustrated in Figure 3.4.2.3 below.

Table 11.6-2. Seismic Design Category Based on 1 s Period Response Acceleration Parameter.

Value of S_{D1}	Risk Category	
	I or II or III	IV
$S_{D1} < 0.067$	A	A
$0.067 \leq S_{D1} < 0.133$	B	C
$0.133 \leq S_{D1} < 0.20$	C	D
$0.20 \leq S_{D1}$	D	

Figure 3.4.2.3. Seismic Design Category Table (American Society of Civil Engineers, 2017)

According to the figures highlighted above, the Seismic Design Category (SDC) of the project is category D in which SD1 parameter of 1-second period response acceleration equals 0.73, and the structure belongs to Risk Category II.

The next step will be the selection of the selected Design Coefficients and Factors of the selected Seismic Force-Resisting System. Figure 3.4.2.4 gives the values to be used in Moment-Resisting Frame Systems, which is used in this building.

Table 12.2-1. Design Coefficients and Factors for Seismic Force-Resisting Systems.									
Seismic Force-Resisting System	ASCE 7 Section Where Detailing Requirements Are Specified	Response Modification Coefficient, R^a	Overstrength Factor, Ω_o^b	Deflection Amplification Factor, C_d^c	Structural System Limitations Including Structural Height, h_n , Limits (ft) ^d				
					Seismic Design Category				
					B	C	D ^e	E ^e	F ^f
C. MOMENT-RESISTING FRAME SYSTEMS									
1. Steel special moment frames	14.1 and 12.2.5.5	8	3	5½	NL	NL	NL	NL	NL
2. Steel special truss moment frames	14.1	7	3	5½	NL	NL	160	100	NP
3. Steel intermediate moment frames	12.2.5.7 and 14.1	4½	3	4	NL	NL	35 ^k	NP ^k	NP ^k
4. Steel ordinary moment frames	12.2.5.6 and 14.1	3½	3	3	NL	NL	NP ^j	NP ^j	NP ^j
5. Special reinforced concrete moment frames ^m	12.2.5.5 and 14.2	8	3	5½	NL	NL	NL	NL	NL
6. Intermediate reinforced concrete moment frames	14.2	5	3	4½	NL	NL	NP	NP	NP
7. Ordinary reinforced concrete moment frames	14.2	3	3	2½	NL	NP	NP	NP	NP
8. Steel and concrete composite special moment frames	12.2.5.5 and 14.3	8	3	5½	NL	NL	NL	NL	NL
9. Steel and concrete composite intermediate moment frames	14.3	5	3	4½	NL	NL	NP	NP	NP
10. Steel and concrete composite partially restrained moment frames	14.3	6	3	5½	160	160	100	NP	NP
11. Steel and concrete composite ordinary moment frames	14.3	3	3	2½	NL	NP	NP	NP	NP
12. Cold-formed steel—special bolted moment frame ⁿ	14.1	3½	3 ⁿ	3½	35	35	35	35	35

Figure 3.4.2.4. Design Coefficients and Factors for Seismic Force-Resisting System (American Society of Civil Engineers, 2017)

Figure 3.4.2.4 suggests that the structural system that will be chosen in this project is Special Reinforced Concrete Moment Frame, because the entire building will be made out of reinforced concrete with no elements of steel frames. Thus, the factors related to the seismic design are:
The coefficient of response modification, $R=8$.

Overstrength factor, $\Omega_o = 3$

Factor of deflection amplification, $C_d=5.5$.

In order to get the Importance Factor I_e , the values should be obtained by going to Figure 3.4.2.5 and adopting the appropriate values.

Risk Category from Table 1.5-1	Snow Importance Factor, I_s	Ice Importance Factor—Thickness, I_t	Ice Importance Factor—Wind, I_w	Seismic Importance Factor, I_e
I	0.80	0.80	1.00	1.00
II	1.00	1.00	1.00	1.00
III	1.10	1.25	1.00	1.25
IV	1.20	1.25	1.00	1.50

Figure 3.4.2.5. Importance Factor Table (American Society of Civil Engineers, 2022)

According to the data of the figure above the importance Factor, the $I_e=1.0$. The design spectrum periods are now:

$$T_0 = 0.2 \frac{S_{D1}}{S_{DS}} = 0.2 * \frac{0.73}{1.52} = 0.096 \text{ s}$$

$$T_s = \frac{S_{D1}}{S_{DS}} = \frac{0.73}{1.52} = 0.48 \text{ s}$$

The next approximate fundamental period should be calculated as for moment frames not exceeding 12 stories and having a minimum story height of 10 ft, T_a may be taken as 0.1N. So T_a will be:

$$T_a = 0.1N = 0.1 * 4 = 0.4 \text{ s}$$

For the fundamental period following formula was applied and corresponding C_u is 1.4:

$$T = C_u T_a = 1.4 * 0.4 = 0.56 \text{ s}$$

To determine T^* , a modal analysis in SAP2000 is required. First, a mass source is assigned. Only the dead load is considered, as snow loads are not applied to our building.

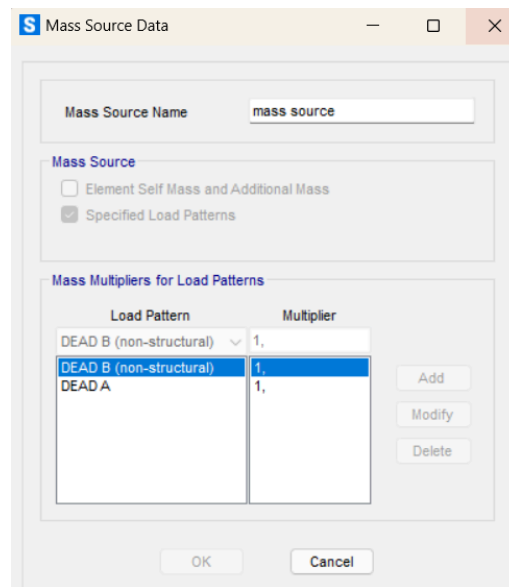


Figure 3.2.4.6. Assigning the Mass Source for Modal Analysis

Since our building is 4 story, and it has 3 axis, the maximum number of modes is 120

S Load Case Data - Modal [X]

Load Case Name
Modal analysis [Set Def Name] **Notes** [Modify/Show...]

Load Case Type
Modal [Design...]

Stiffness to Use
☒ Zero Initial Conditions - Unstressed State
☐ Stiffness at End of Nonlinear Case [v]
 Important Note: Loads from the Nonlinear Case are NOT included in the current case

Type of Modes
☒ Eigen Vectors
☐ Ritz Vectors

Number of Modes
 Maximum Number of Modes: 12
 Minimum Number of Modes: 1

Mass Source
 mass source

Loads Applied
☐ Show Advanced Load Parameters

Other Parameters
 Frequency Shift (Center): 0,
 Cutoff Frequency (Radius): 0,
 Convergence Tolerance: 1,000E-09
☒ Allow Automatic Frequency Shifting

[OK] [Cancel]

Figure 3.2.4.7. Defining Modal Analysis case

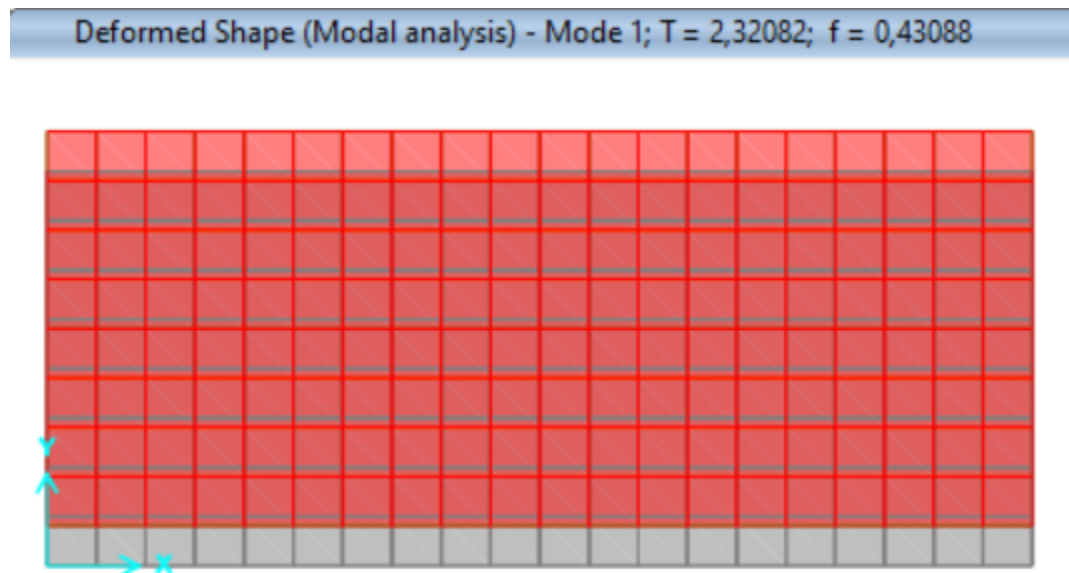


Figure 3.2.4.8. Mode 1 Results

Deformed Shape (Modal analysis) - Mode 2; T = 2,14685; f = 0,4658

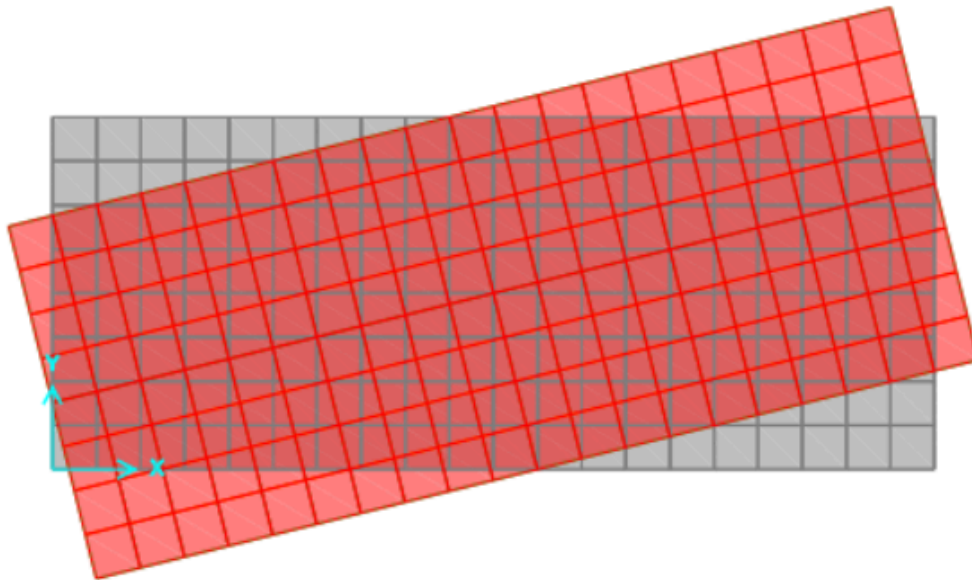


Figure 3.2.4.9. Mode 2 Results

Deformed Shape (Modal analysis) - Mode 3; T = 2,08114; f = 0,48051

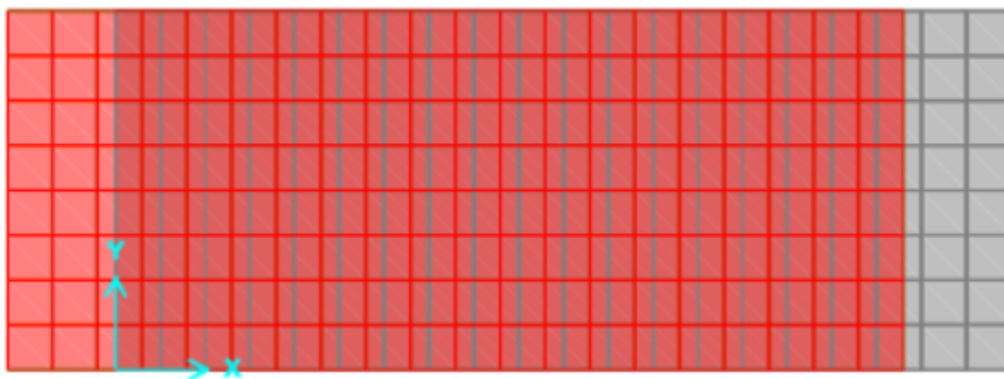


Figure 3.2.4.10. Mode 3 Results

From the SAP2000 the value of T^* is 2.32. Therefore, for the Strength design T value is used. The next stage is computing base shear by following equation:

$$V = C_s W$$

where C_s is seismic response coefficient and W is effective seismic weight. For the seismic response coefficient following steps are implemented:

$$C_s = \frac{S_{DS}}{R/I_e} = \frac{1.52}{8/1} = 0.19$$

C_s should not exceed:

$$C_s = \frac{S_{D1}}{T(R/I_e)} = \frac{0.73}{0.56*(8/1)} = 0.163$$

$$C_s = \frac{S_{D1}T_L}{T^2(R/I_e)} = \frac{0.73*8}{0.56^2(8/1)} = 2.33$$

So the minimum value of C_s among 3 calculations above is 0.163. As a lower boundary for C_s is computed by following equation:

$$C_{s,min} = 0.5 \frac{S_1}{R/I_e} = 0.5 * \frac{0.73}{8/1} = 0.0456 \text{ for } S_1 > 0.6g$$

Therefore, C_s value is 0.163 as it is bigger than the lower boundary.

Lastly, in calculating the base shear, the effective seismic weight of the structure will have to be calculated. To this effect, Table 3.4.2.1. was built including the self-weight items of the floor slabs, staircases, columns, beam and wall weights.

Table 3.4.2.1. Effective Seismic Weight

Floors	Floor finishing and slab	Column weight (kN)	Beam weight (kN)	Exterior kN/m2	Interior kN/m2	Stairs (kN)	Floor weight (kN)
1	5.42	2794.84	6621.83	0.79	0.68	128.40	37073.84
2	5.42	2001.04	6621.83	0.79	0.74	128.40	36524.70
3	5.42	1653.75	6621.83	0.79	0.70	128.40	36017.70
4	5.42	2362.50	6621.83	1.12	1.34	0.00	40511.08
Roof	3.83	0.00	0.00	0.08		0.00	15650.94
Effective Weight							165778.26

So, Base shera is:

$$V = C_s * W = 0.163 * 165778.26 = 27013 \text{ kN}$$

Distribution among the floors is obtained by following formulas:

$$F_x = C_{vx} V$$

$$C_{vx} = \frac{w_x h_x^k}{\sum w_i h_i^k}$$

k is defined as exponent related to the structural period and has such conditions:

$$k = 1, \text{ if } T \leq 0.5 \text{ s}$$

$$k = 2, \text{ if } T \geq 2.5 \text{ s}$$

using interpolation,

$$k = 2 - \frac{(2-1)(2.5-0.56)}{2.5-0.5} = 1.03$$

Now that all the parameters we need to calculate the lateral seismic forces are known, Table 3.4.2.2 has been prepared to show the calculated seismic force at each of the floor levels.

Table 3.4.2.2 Seismic force for each floor.

Floors	hi (m)	Wx (kN)	Wx h _x ^k	C _{vx}	F _x (kN)
4	18.6	57162.02	1160662.26	0.54	14739.83
3	12.6	36017.70	489663.35	0.23	6218.48
2	8.4	36524.70	327035.10	0.15	4153.18
1	4.2	37073.84	162560.26	0.08	2064.43
Total		166778.26	2139920.98		

3.4.2.1 Seismic Loads Including Torsional Effect

Torsional effects of seismic loading should also be taken into account and because the structural arrangement of the building allows both moment and torsional activities. In order to start this evaluation, one has to first locate the center of mass. Considering the dimensions of the building (length 100 m, width 40 m), the mass-center coordinates were calculated respectively.

$$x = 50\text{m and } y = 20\text{m}$$

(as building is symmetric in both directions)

The second step is to find the center of rigidity and this involves the determination of critical moments of inertia of the structural frames in each direction:

$$I_{b,cr} = 0.35 * \frac{1}{12} b h^3$$

$$I_{c,cr} = 0.7 * \frac{1}{12} b h^3$$

After acquiring these values, the general lateral stiffness of the building on either axis may be determined.

$$D = \frac{12E}{h^2} \left(\frac{I_{c,cr} I_{b,cr}}{L^2 I_{c,cr} + h^2 I_{b,cr}} \right)$$

It is possible to evaluate the story-level torsional forces, which are calculated with the expression relating the imbalance of stiffness to the rotation of inter-story:

$$C_f = h * D * \text{number of columns per frame} * \text{number of frames}$$

According to such calculations, the coordinates of the rigidity center are also $x'=50\text{m}$ and $y'=20\text{m}$. Having the mass center and rigidity center, the eccentricities along the X and the Y direction can be calculated:

$$e_x = 50 - 50 = 0 \text{ m}$$

$$e_{ax} = 0.05 * L_x = 0.05 * 100 = 5 \text{ m}$$

$$e_{xt} = e_x + e_{ax} = 0 + 5 = 5 \text{ m}$$

$$e_y = 20 - 20 = 0 \text{ m}$$

$$e_{ay} = 0.05 * L_y = 0.05 * 40 = 2 \text{ m}$$

$$e_{yt} = e_y + e_{ay} = 0 + 2 = 2 \text{ m}$$

The torsional moment, direct shear force, torsional shear force, and the resulting total seismic force are then calculated by using these eccentricities by the set of equations shown below:

$$T = e * F$$

$$F_{direct} = \frac{F_x}{\text{number of frames}}$$

$$F_{torsion} = \frac{T * C_f(x_i \text{ or } y_i)}{\sum C_f * (x_i^2 \text{ or } y_i^2)}$$

$$F_{total} = F_{direct} \pm F_{torsion}$$

Table 3.4.2.3 shows the horizontal forces, direct shear and torsion of the transverse and longitudinal directions.

Table 3.4.2.3 Direct forces on each frame including torsion

		Longitudinal (NS)		Traverse (NS)	
Floors	F _x (kN)	F _{direct} (kN)	T (kN- m)	F _{direct} (kN)	T (kN-m)
4	14739.83	4913.28	29479.66	701.90	73699.14
3	6218.48	2072.83	12436.96	296.12	31092.39
2	4153.18	1384.39	8306.36	197.77	20765.91
1	2064.43	688.14	4128.87	98.31	10322.17

The torsional effects are also introduced in Table 3.4.2.4, where the third floor is used as an example.

Table 3.4.2.4 Seismic load calculations for third floor including torsional effect

3rd floor Frame #	Y axis			X axis		
	F _{torsion} , kN	F _{direct} , kN	F _{total} , kN	F _{torsion} , kN	F _{direct} , kN	F _{total} , kN
Frame 1/A	-16.91	296.12	279.21	-283.38	2072.83	1789.44
Frame 2/B	-15.22	296.12	280.90	0.00	2072.83	2072.83
Frame 3/C	-13.53	296.12	282.59	283.38	2072.83	2356.21
Frame 4	-11.84	296.12	284.28			
Frame 5	-10.15	296.12	285.97			
Frame 6	-8.46	296.12	287.66			
Frame 7	-6.76	296.12	289.35			
Frame 8	-5.07	296.12	291.04			
Frame 9	-3.38	296.12	292.74			
Frame 10	-1.69	296.12	294.43			
Frame 11	0.00	296.12	296.12			
Frame 12	1.69	296.12	297.81			
Frame 13	3.38	296.12	299.50			
Frame 14	5.07	296.12	301.19			

Frame 15	6.76	296.12	302.88	
Frame 16	8.46	296.12	304.57	
Frame 17	10.15	296.12	306.26	
Frame 18	11.84	296.12	307.96	
Frame 19	13.53	296.12	309.65	
Frame 20	15.22	296.12	311.34	
Frame 21	16.91	296.12	313.03	

3.4.3 Assigning Lateral Forces to SAP 2000

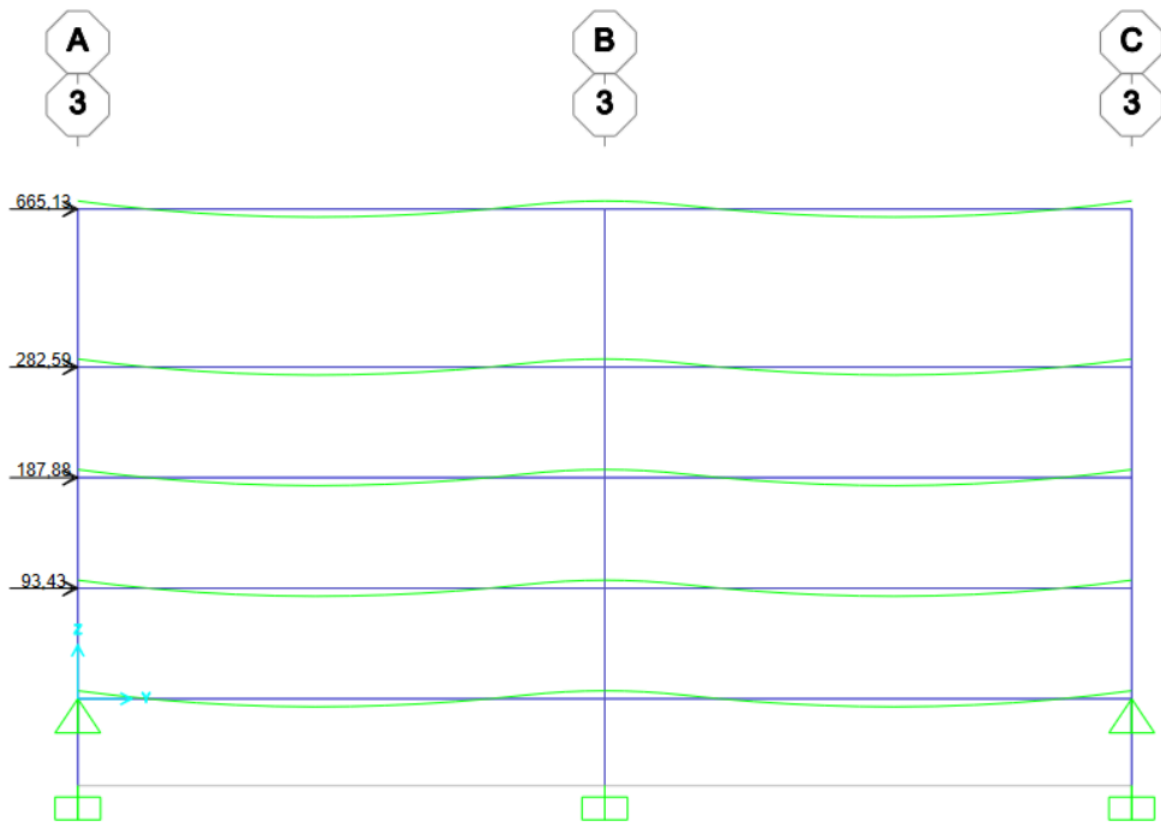


Figure 3.4.3.1. Seismic Load Assignment for Frame C

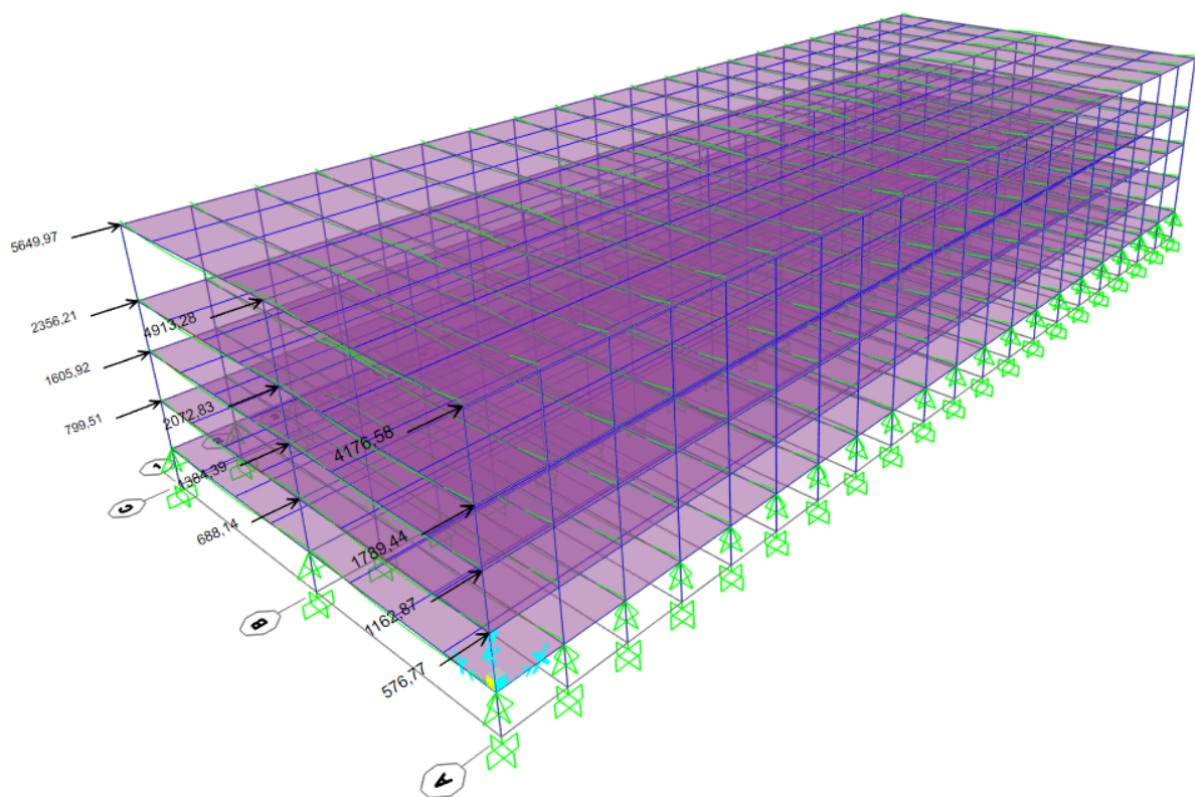
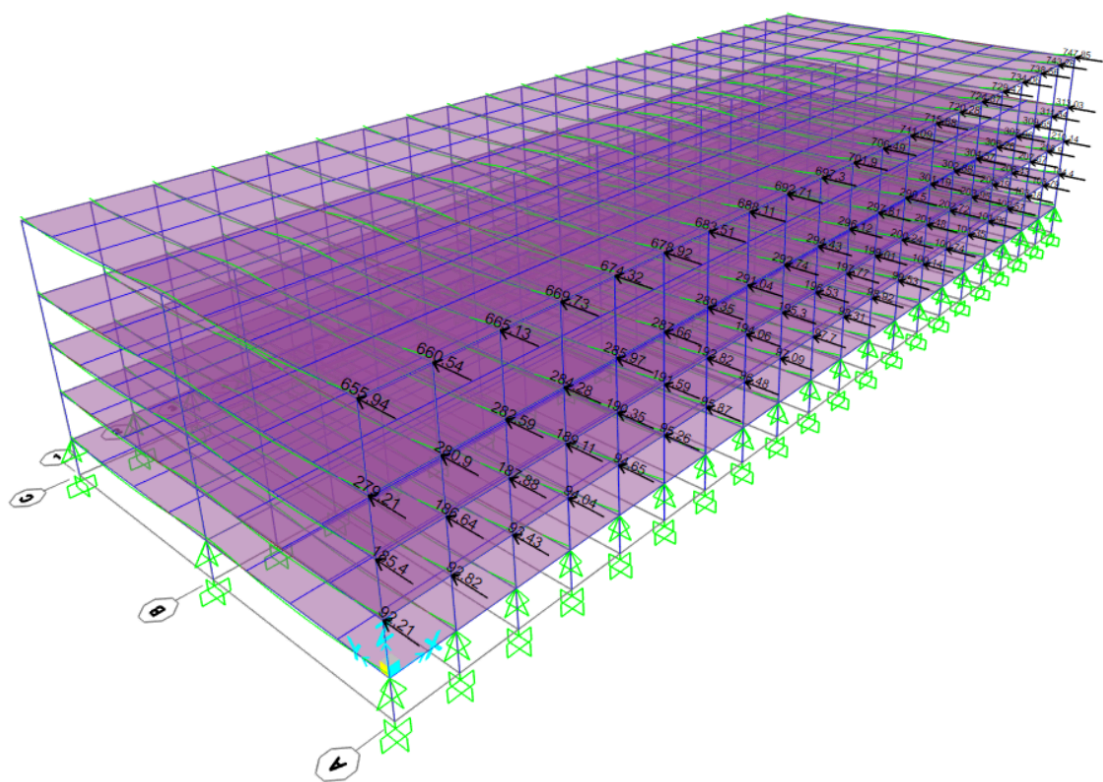


Figure 3.4.3.2. 3D Seismic Load Assignment

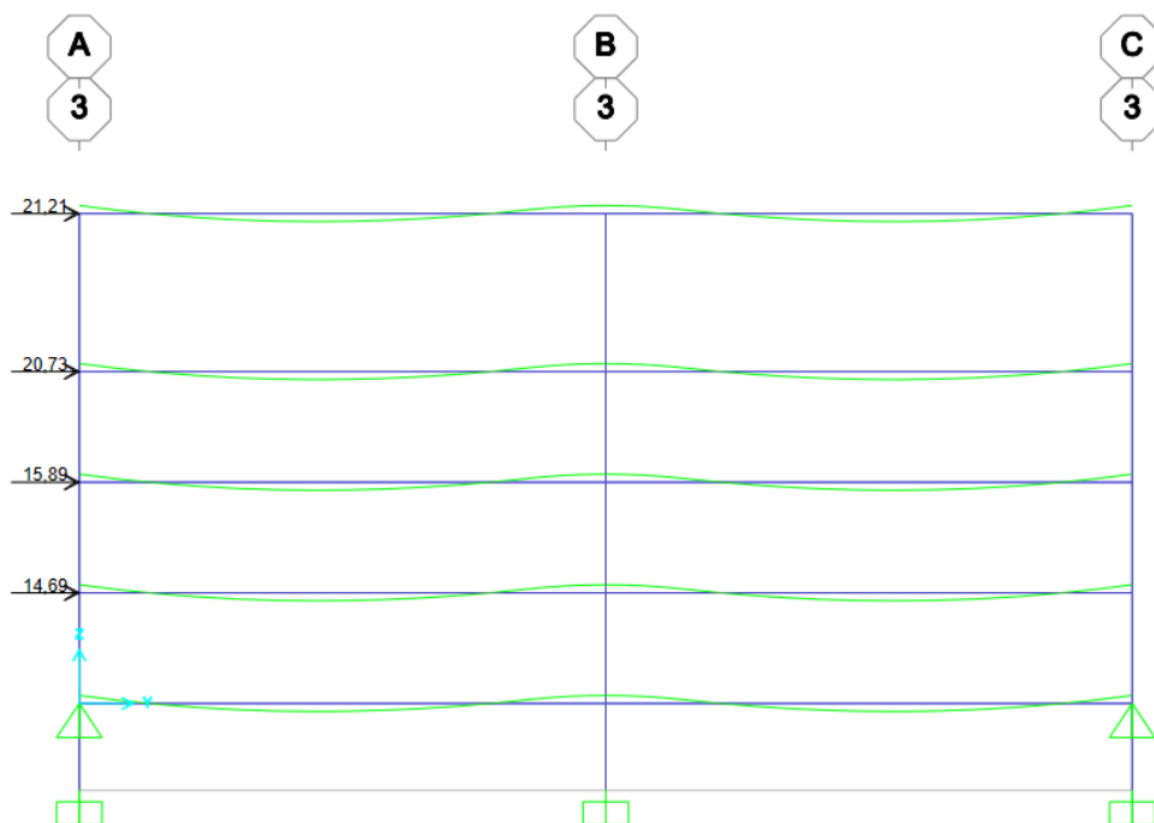


Figure 3.4.3.3. Case 1 Wind Load Assignment for Frame C

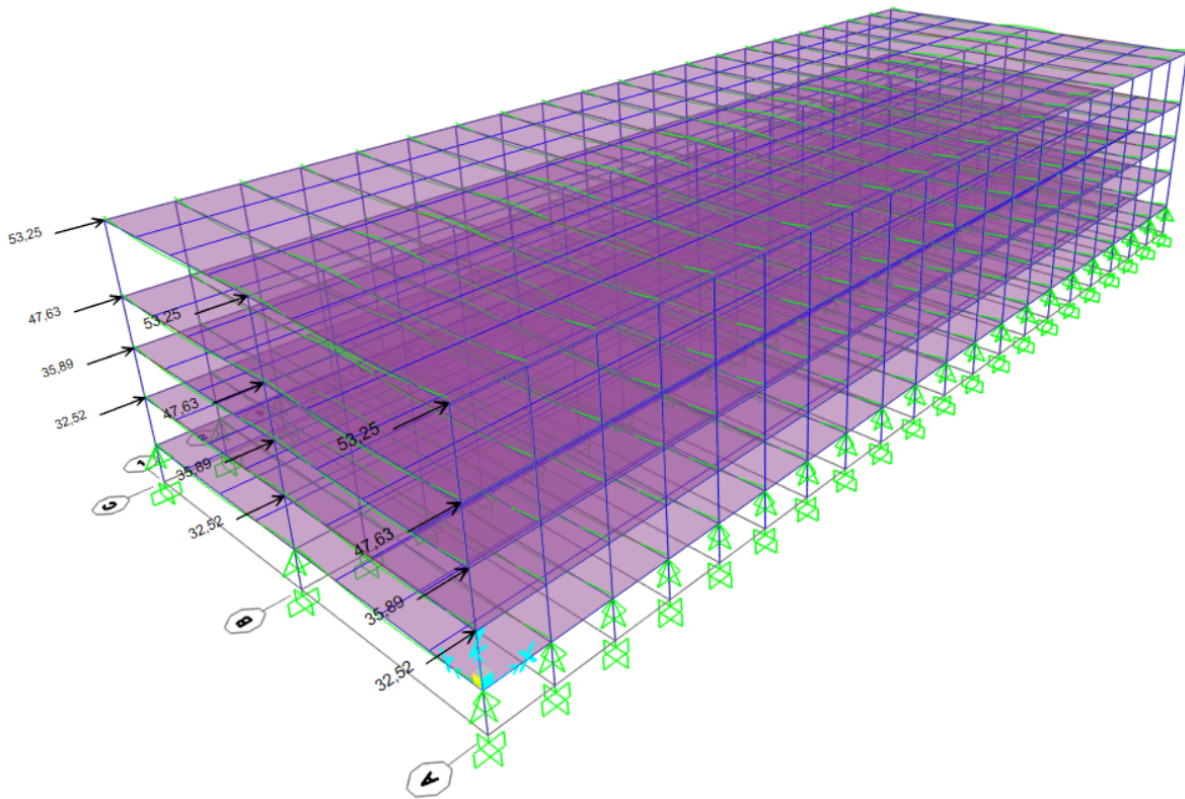
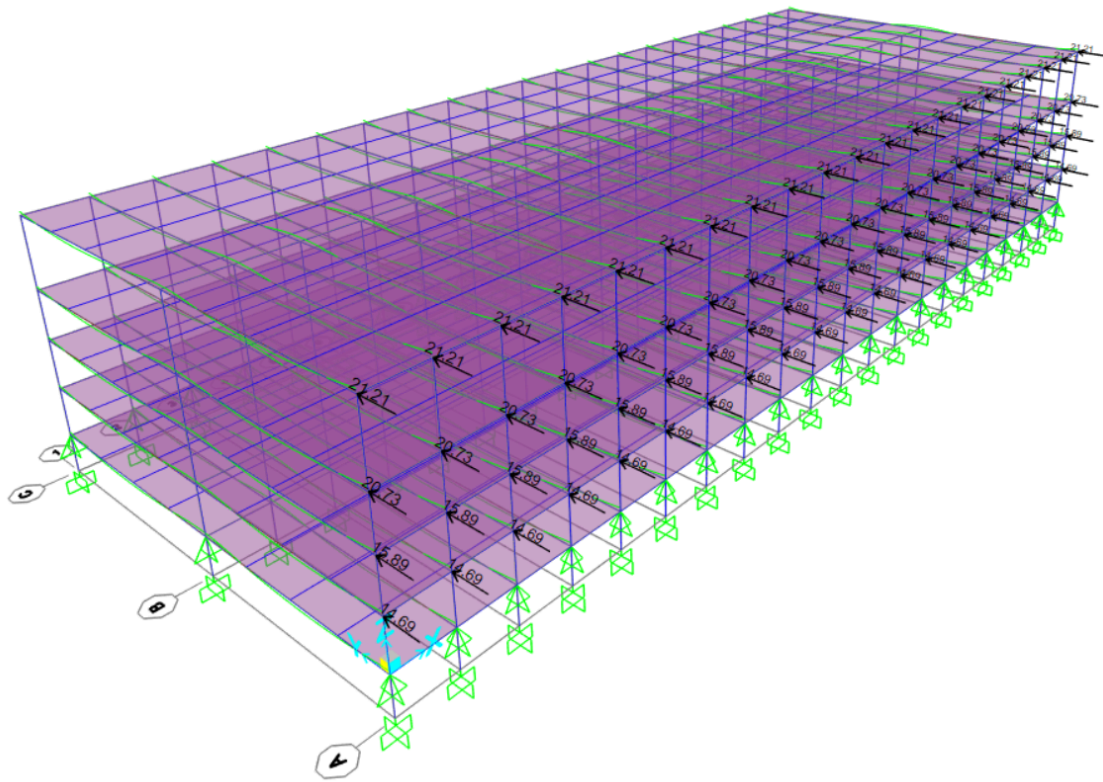


Figure 3.4.3.4. 3D Case 1 Wind Load Assignment

The rest of the Cases for Wind Forces are included in Appendix.

3.5. Lateral Drift Analysis and Check under Wind and Seismic Loads

3.5.1. Wind and Seismic Drifts Calculations

As part of the evaluation of the Lateral Force Resisting System, the effects of the wind and seismic load on the drift of the building was considered. In order to get meaningful and accurate results, lateral displacements were computed in both X and Y directions which are Frame A and Frame 1 respectively, taking into consideration the shape of the residential structure, which is a rectangle. The organizational plan, as well as the frame choice, is presented in Figure 3.5.1.1. Both shear and flexural drifts were established between the winds and the seismic applied loads.

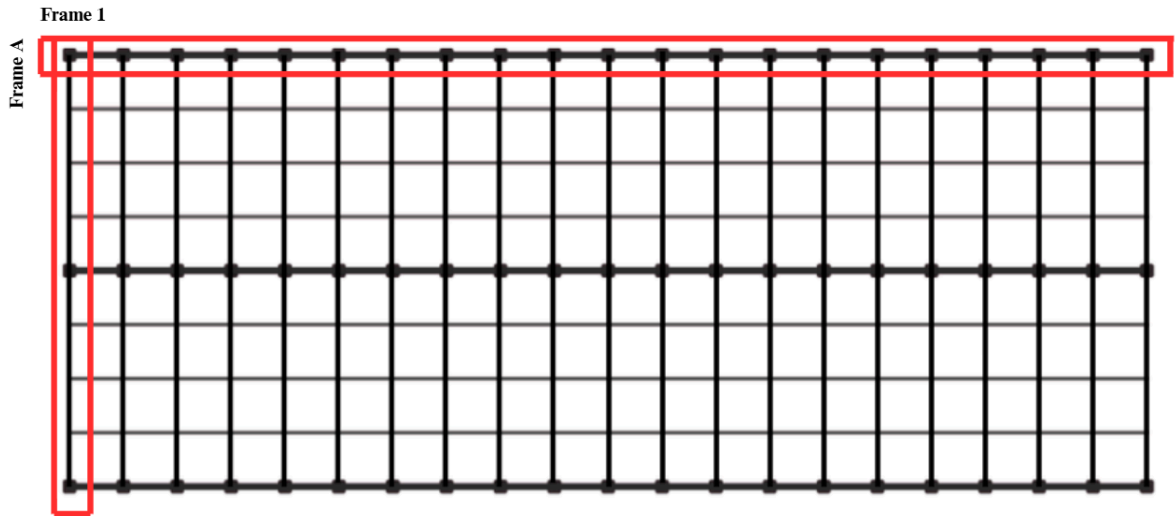


Figure 3.5.1.1. Structural layout with the chosen Frame A and Frame 1

The table below shows the calculated wind induced drifts on long and short sides of the building using the frame forces known at each level previously. The following formula was used to calculate the shear force:

$$V_i = F_i + V_{i+1}$$

Shear at base of each column in each level was determined by dividing the total shear force at the level by the number of frames (n = 20).

$$V_{i-col} = \frac{V_i}{n}$$

The critical moments of inertia of the elements were also considered. Applied formulas are presented below:

$$\delta_b^i = \frac{V_{i-avg} h_i^2 L}{12E I_{b,cr}} \text{ for floors } 1 - 3$$

$$\delta_b^i = \frac{V_{i-avg} h_i^2 L}{48E I_{b,cr}} \text{ for top floor}$$

$$\delta_c^i = \frac{V_{i-avg} h_i^3}{12E I_{c,cr}} \text{ for floors } 1 - 3$$

$$\delta_c^i = \frac{V_{i-avg} h_i^3}{24E I_{c,cr}} \text{ for top floor}$$

$$\delta_t^i = \delta_b^i + \delta_c^i$$

$$\delta_i^i = \delta_t^i + (\delta_t^{i-1}/2) \text{ for top floor}$$

$$\delta_i^i = \delta_t^i/2 + (\delta_t^{i-1}/2) \text{ for other floors}$$

δ_b^i is major beam inter-story drift; δ_c^i is column inter-story drift; δ_t^i is total inter-story drift.

Following Table 3.5.1.1 is for calculated results due to Wind Load in X-direction.

Table 3.5.1.1. Lateral Shear Drift due to Wind Load in X-direction

Story	hi (mm)	hi-avg (mm)	Fi (kN)	Vi (kN)	Vi-col (kN)	Vavg (kN)	Ic,cr (mm ⁴)	Ib,cr (mm ⁴)	Ic-avg (mm ⁴)	db (mm)	dc (mm)	dt (mm)	Inter-story (mm)	Absolute (mm)
4	6000	6000	21.21	21.21	1.06	1.06	3.65E+09	3.50E+09	3.65E+09	0.153	0.09	0.24	0.51	2.57

3	4200	5100	20.73	41.94	2.10	1.58	3.65E+09	3.50E+09	3.65E+09	0.446	0.09	0.54	0.67	2.06
2	4200	4200	15.89	57.83	2.89	2.49	7.56E+09	3.50E+09	5.60E+09	0.705	0.09	0.80	0.90	1.39
1	4200	4200	14.68	72.50	3.63	3.26	1.04E+10	3.50E+09	8.99E+09	0.920	0.08	1.00	0.50	0.50
0		2100				1.81					0.00	0.00		0.00

To work out the lateral flexural drift of the building, the first step entailed identifying the moment at each level. In the case of the top floor, this was achieved by multiplying the floor height by the base shear whereas the rest floors were accumulated to the contribution of the floor of consideration. Next, ϕ_i was determined by utilizing the moment and cross-sectional area of columns along with some constants. Necessary formulas are below:

$$M_i = h_i * V_i \text{ for top floor}$$

$$M_i = M_{i+1} + h_i * V_i \text{ for other floors}$$

$$\phi_i = \frac{2M_i}{E(mL)^2 * A_c}$$

$$\Delta\theta = \phi_i * h_i$$

$$\theta_i = \sum_1^n \Delta\theta$$

$$\delta_i = \theta_i * h_i$$

$$\Delta_i = \sum_1^n \delta_i$$

Table 3.5.1.2. Lateral Flexural Drift due to Wind Load in X-direction

Story	M (N-mm)	Column size (m)	A (mm ²)	f _i	Dq _i (rad)	q _i (rad)	Interstory (mm)	Absolute (mm)
4	1.27E+08	0.50	250000.00	2.14E-11	1.28E-07	5.22E-07	0.00	0.01

3	1.76E+08	0.50	250000.00	2.96E-11	1.24E-07	3.94E-07	0.00	0.00
2	2.43E+08	0.55	302500.00	3.38E-11	1.42E-07	2.69E-07	0.00	0.00
1	3.05E+08	0.65	422500.00	3.03E-11	1.27E-07	1.27E-07	0.00	0.00

In order to compute the lateral shear and flexural drift in the long direction of the building, the equations that were used to compute the shear drift of the short direction were used in the long direction. The forces per frame in the long direction were also determined previously per frame at each level of the floors. As the number of frames on the length side is two ($n = 2$), the formula of the base shear allocated to each column ($V_{i,col}$) will be different.

Table 3.5.1.3. Lateral Shear Drift due to Wind Load in Y-direction

Story	hi (mm)	hi-avg (mm)	Fi (kN)	Vi (kN)	Vi-col (kN)	Vavg (kN)	Ic,cr (mm ⁴)	Ib,cr (mm ⁴)	Ic-avg (mm ⁴)	db (mm)	dc (mm)	dt (mm)	Inter-story (mm)	Absolute (mm)
4	6,000	6000	53.25	53.25	26.62	26.62	3.65E+09	1.70E+09	3.65E+09	1.98	2.21	4.19	5.99	16.70
3	4,200	5100	47.63	100.88	50.44	38.53	3.65E+09	1.70E+09	3.65E+09	1.40	2.20	3.60	3.98	10.72
2	4,200	4200	35.89	136.77	68.39	59.41	7.56E+09	1.70E+09	5.60E+09	2.16	2.20	4.36	4.46	6.74
1	4,200	4200	32.52	169.29	84.64	76.51	1.04E+10	1.70E+09	8.99E+09	2.78	1.77	4.55	2.28	2.28
0	0					42.32					0.00	0.00		0.00

Table 3.5.1.4. Lateral Flexural Drift due to Wind Load in Y-direction

Story	M (N-mm)	Column size (m)	A (mm ²)	fi	Dqi (rad)	qi (rad)	Interstory (mm)	Absolute (mm)
4	3.19E+08	0.50	250000.00	8.60E-12	5.16E-08	2.01E-07	0.00120	0.00246
3	4.24E+08	0.50	250000.00	1.14E-11	4.79E-08	1.49E-07	0.00063	0.00125
2	5.74E+08	0.55	302500.00	1.28E-11	5.37E-08	1.01E-07	0.00043	0.00062
1	7.11E+08	0.65	422500.00	1.13E-11	4.76E-08	4.76E-08	0.00020	0.00020

The methodology and equations applied to compute the lateral shear drift with seismic loading is the same as those applied to compute wind load drift. The only distinction is the values of loads, which have been obtained through seismic forces. The rest of the parameters such as the constants are kept constant.

Table 3.5.1.5. Lateral Shear Drift due to Seismic Load in X-direction

Story	hi (mm)	hi-avg (mm)	Fi (kN)	Vi (kN)	Vi-col (kN)	Vavg (kN)	Ic,cr (mm ⁴)	Ib,cr (mm ⁴)	Ic-avg (mm ⁴)	db (mm)	dc (mm)	dt (mm)	Inter-story (mm)	Absolute (mm)
4	6000	6000	1375.72	1375.72	68.79	68.79	3.65E+09	3.50E+09	3.65E+09	10	5.71	15.63	19.75	43.49
3	4200	5100	580.39	1956.11	97.81	83.30	3.65E+09	3.50E+09	3.65E+09	6	2.37	8.25	8.92	23.74
2	4200	4200	387.63	2343.74	117.19	107.50	7.56E+09	3.50E+09	5.60E+09	8	1.99	9.58	9.80	14.82
1	4200	4200	192.68	2536.42	126.82	122.00	1.04E+10	3.50E+09	8.99E+09	9	1.41	10.03	5.01	5.01
0		2100				63.41					0.00	0.00		0.00

Table 3.5.1.6. Lateral Flexural Drift due to Seismic Load in X-direction

Story	M (N-mm)	Column size (m)	A (mm ²)	fi	Dqi (rad)	qi (rad)	Interstory (mm)	Absolute (mm)
4	8.25E+09	0.50	250000.00	1.39E-09	8.33E-06	2.43E-05	0.15	0.27
3	8.22E+09	0.50	250000.00	1.38E-09	5.80E-06	1.60E-05	0.07	0.13
2	9.84E+09	0.55	302500.00	1.37E-09	5.75E-06	1.02E-05	0.04	0.06
1	1.07E+10	0.65	422500.00	1.06E-09	4.45E-06	4.45E-06	0.02	0.02

Table 3.5.1.7. Lateral Shear Drift due to Seismic Load in Y-direction

Story	hi (mm)	hi-avg (mm)	Fi (kN)	Vi (kN)	Vi-col (kN)	Vavg (kN)	Ic,cr (mm ⁴)	Ib,cr (mm ⁴)	Ic-avg (mm ⁴)	db (mm)	dc (mm)	dt (mm)	Inter-story (mm)	Absolute (mm)
4	6,000	6000	196.53	196.53	98.27	98.27	3.65E+09	1.70E+09	3.65E+09	7.30	8.16	15.46	19.32	39.97

3	4,200	5100	82.91	279.44	139.72	118.99	3.65E+09	1.70E+09	3.65E+09	4.33	3.39	7.72	8.08	20.65
2	4,200	4200	55.38	334.82	167.41	153.57	7.56E+09	1.70E+09	5.60E+09	5.59	2.85	8.44	8.40	12.58
1	4,200	4200	27.53	362.35	181.17	174.29	1.04E+10	1.70E+09	8.99E+09	6.34	2.01	8.36	4.18	4.18
0	0					90.59					0.00	0.00		0.00

Table 3.5.1.8. Lateral Flexural Drift due to Seismic Load in Y-direction

Story	M (N-mm)	Column size (m)	A (mm ²)	f _i	Dq _i (rad)	q _i (rad)	Interstory (mm)	Absolute (mm)
4	1.18E+09	0.50	250000.00	3.17E-11	1.90E-07	5.56E-07	0.00	0.01
3	1.17E+09	0.50	250000.00	3.16E-11	1.33E-07	3.66E-07	0.00	0.00
2	1.41E+09	0.55	302500.00	3.13E-11	1.31E-07	2.33E-07	0.00	0.00
1	1.52E+09	0.65	422500.00	2.42E-11	1.02E-07	1.02E-07	0.00	0.00

3.5.2. Comparison of interstory drifts

With all the inter-story drifts established on the basis of hand computing, the findings were matched with the ones achieved with the 2D and 3D frame models developed in SAP2000. The inter-story displacements of Frame A and Frame 1 in the transverse and longitudinal directions are displayed in Tables 3.5.2.1 and 3.5.2.2 below.

Table 3.5.2.1. Comparison of interstory displacements on LFRS, Frame 1

Comparison of interstory displacements, Frame 1						
Levels	Interstory displacement from wind load			Interstory displacement from seismic load		
	2D	3D	Hand	2D	3D	Hand
4	6.52	7.02	5.99	89.83	92.55	19.32
3	5.11	6.33	3.98	33.05	35.97	8.08
2	5.05	6.23	4.46	30.04	34.62	8.40
1	3.29	3.63	2.28	18.05	21.41	4.18

Table 3.5.2.2. Comparison of interstory displacements on LFRS, Frame A

Comparison of interstory displacements, Frame A						
Levels	Interstory displacement from wind load			Interstory displacement from seismic load		
	2D	3D	Hand	2D	3D	Hand
4	0.4	0.6	0.51	47.6	51.14	16.29
3	0.18	0.34	0.67	15.63	23.38	6.77
2	0.26	0.42	0.90	18.63	25.34	7.96
1	0.4	0.37	0.50	26.36	20.09	4.23

After computing the wind and seismic drifts of Frame A and Frame 1 and coming up with comparison tables of both Frame A and Frame 1, graphical figures were generated as a better representation of the differences between hand-calculated and SAP2000 results. Figures 3.5.2.1 - 3.5.2.4 below indicate the inter-story drifts caused by the wind and seismic loads in the X and Y directions. It is necessary to add that these graphs reflect the structural behavior of the elastic range. Since the shopping mall is in a seismically active area, more factors are required to ensure that the actual impact of probable earthquakes is taken into consideration.

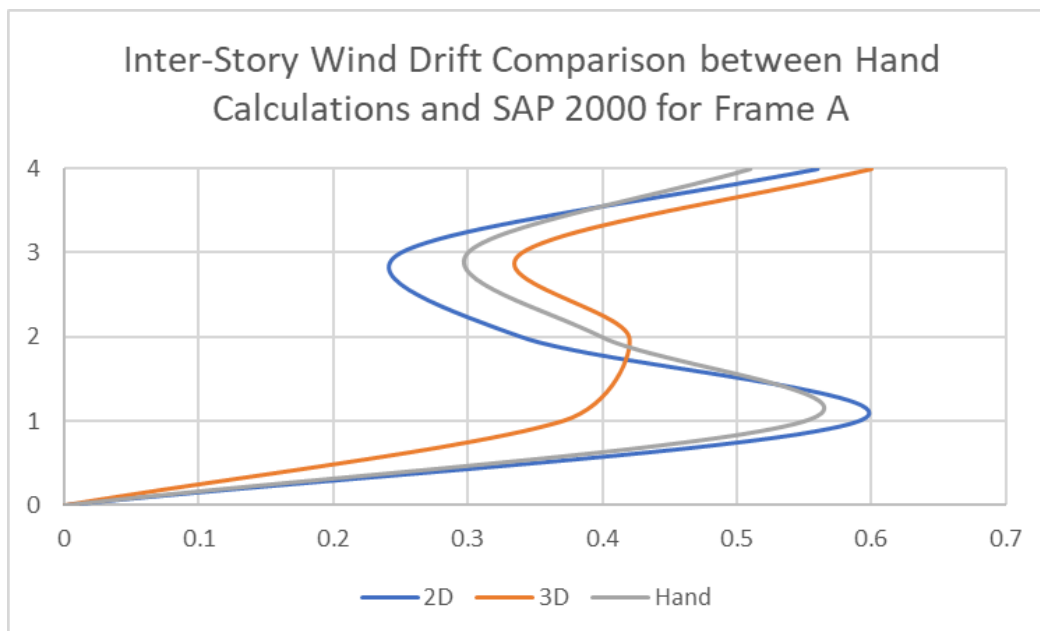


Figure 3.5.2.1 Inter-Story Wind Drift Comparison between Hand Calculations and SAP 2000 for Frame A, X - Direction

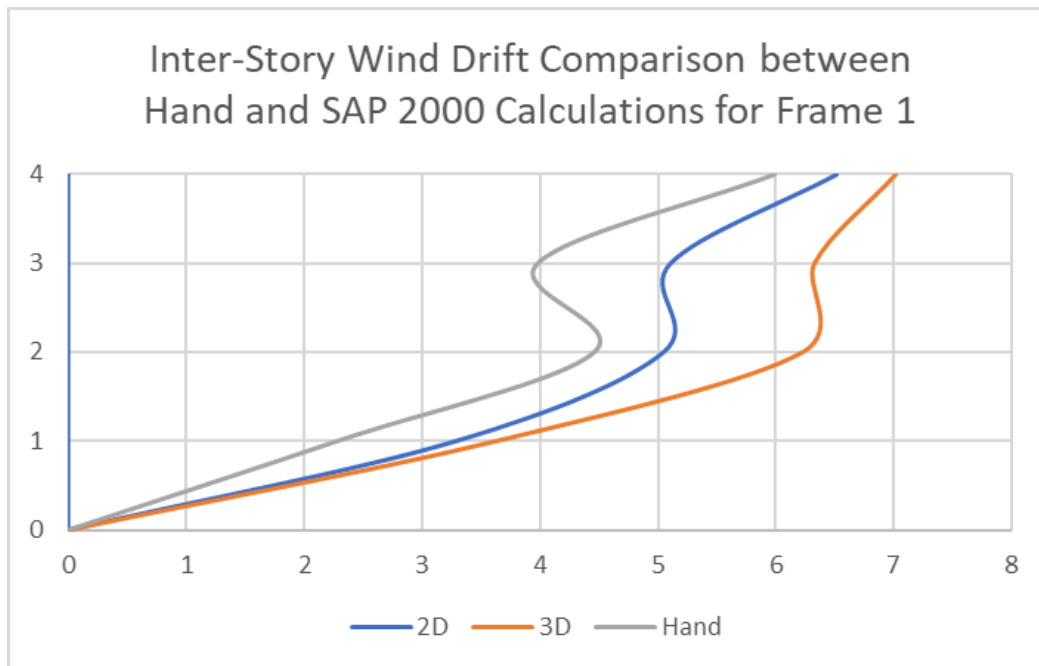


Figure 3.5.2.2 Inter-Story Wind Drift Comparison between Hand Calculations and SAP 2000 for Frame 1, Y - Direction

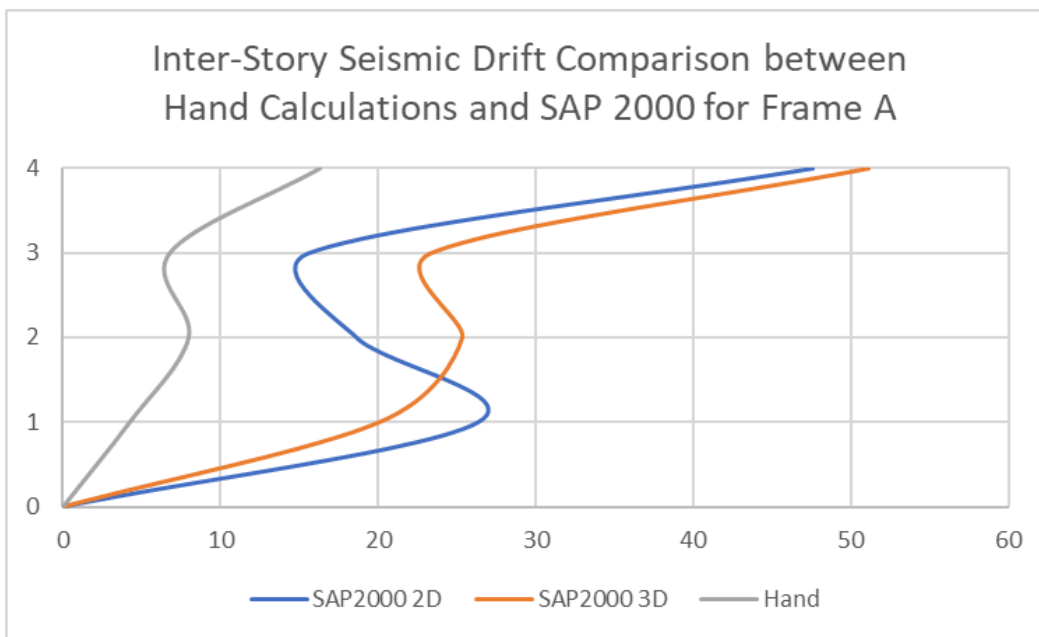


Figure 3.5.2.3. *Inter-Story Seismic Drift Comparison between Hand Calculations and SAP 2000 for Frame A, X - Direction*

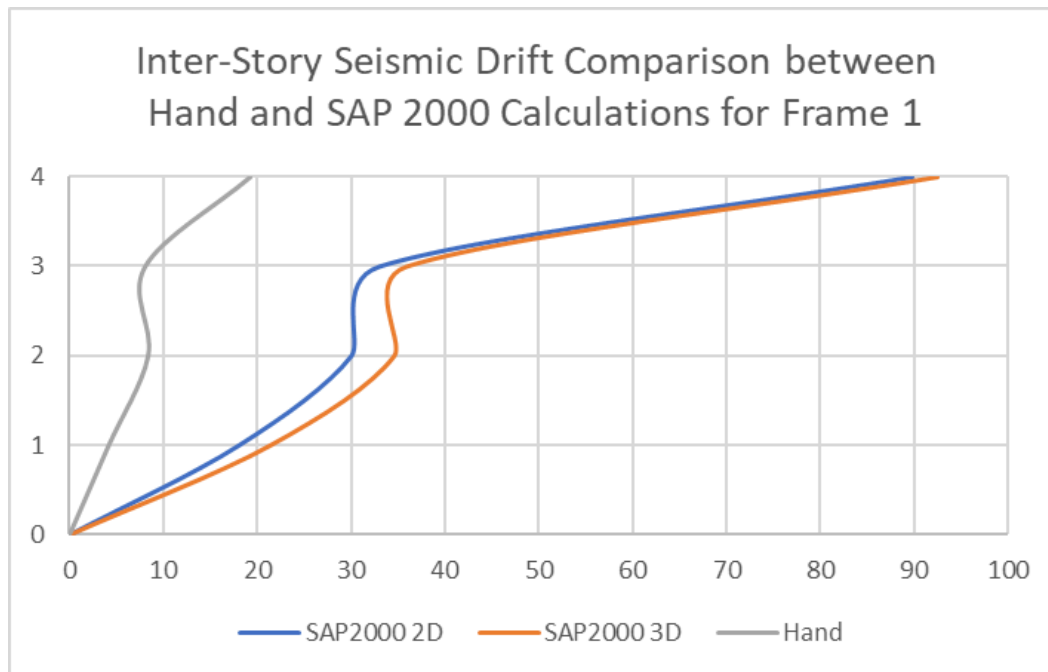


Figure 3.5.2.4. *Inter-Story Seismic Drift Comparison between Hand Calculations and SAP 2000 for Frame 1, Y - Direction*

As it has been mentioned above, the seismic effects have to be considered by introducing the factor of displacement amplification, $C_d = 5.5$. This aspect enables the inter-story displacements to give a more realistic approach to the structural behavior in the real earthquake conditions. The resultant drift values are compared with the acceptable seismic drift values. The formulae below show how the allowable drift is calculated:

$$\text{Allowable Limit} = 2\% \text{ of the story height}$$

$$\text{Forth Floor Allowable Limit} = 0.02 * 6000 = 120 \text{ mm}$$

$$1 - 3 \text{ Floor Allowable Limit} = 0.02 * 4200 = 84 \text{ mm}$$

The results of comparing the calculated values of inter-story drift and the permissible seismic drift limits are shown in figures 3.5.2.5 and 3.5.2.6 below. All the values are within the

acceptable range and it indicates the sufficiency and soundness of the series of design of the lateral force-resisting system.

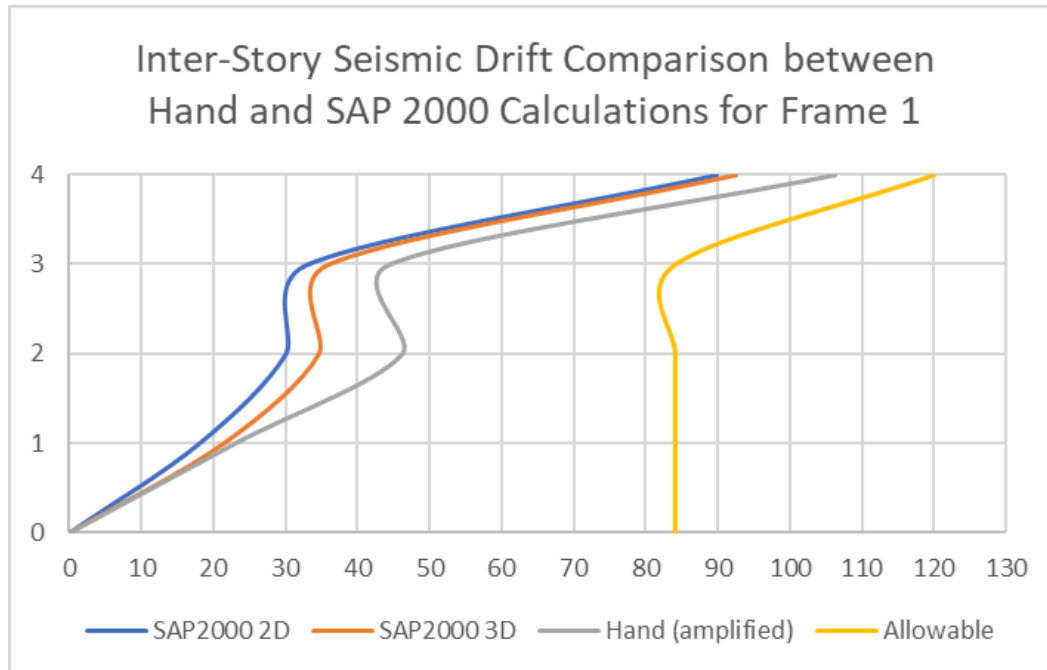


Figure 3.5.2.5. *Amplified Inter-Story Seismic Drift Comparison between Hand and SAP 2000 Calculations with the Allowable Limit for Frame 1, Y - Direction*

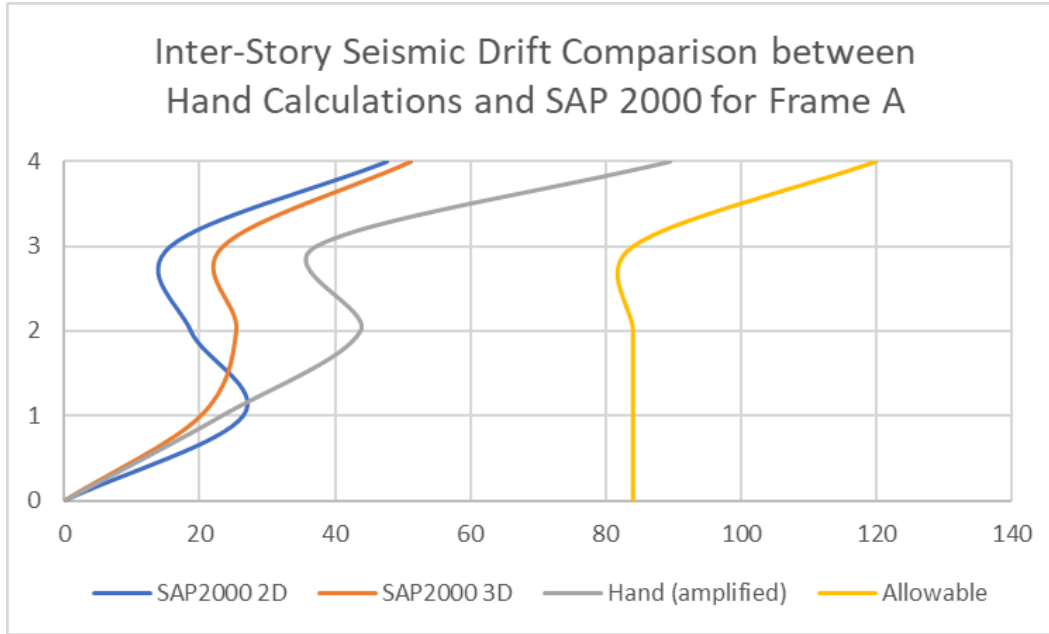


Figure 3.5.2.6. *Amplified Inter-Story Seismic Drift Comparison between Hand and SAP 2000 Calculations with the Allowable Limit for Frame A, X - Direction*

3.5.3. P-Δ Drift Calculation

The structural analysis will also measure the P-D effect to consider the extra moments created by the loads due to gravity on the laterally moved structural elements. The importance of this second-order effect is that the axial loads and lateral displacements interact and may increase the internal forces and affect the overall structure stability. The consideration of the P-D effect will give a more realistic evaluation of the behavior of the building under combined gravity and lateral loading especially in seismic cases. This effect will be calculated according to the set of formulas that will be identified below.

$$Q = \frac{\sum P_u \delta_{1st}}{V_{us} * L_c}$$

$$\beta = \frac{1}{1-Q}$$

All the calculations involving the P-D effect were done using the described methodology. The obtained values and the relevant parameters have been summarized in the following Tables 3.5.3.1-3.5.3.4.

Table 3.5.3.1. Amplified drift for Wind load, Frame A (P-D drift)

	Total						Amplified		Allowable	
Story	Interstory	Absolute	Vu	Pu	Q	Beta	Interstory	Absolute	Interstory	Absolute
4	5.99	16.71	63.6	31916.2	0.50	2	11.994	22.712	120	372
3	3.98	10.72	62.2	96145.9	1.47	1	3.982	10.718	84	252
2	4.46	6.74	47.7	155980.7	3.48	1	4.459	6.736	84	168
1	2.28	2.28	44.0	216795.0	2.67	1	2.277	2.277	84	84

Table 3.5.3.2. Amplified drift for Wind load, Frame 1 (P-D drift)

	Total						Amplified		Allowable	
Story	Interstory	Absolute	Vu	Pu	Q	Beta	Interstory	Absolute	Interstory	Absolute
4	0.51	2.58	63.6	31916.2	0.0428	1.04	0.535	5.604	120	372
3	0.67	2.06	62.2	96145.9	0.2459	1.33	0.886	5.070	84	252
2	0.90	1.40	47.7	155980.7	0.6994	3.33	2.985	4.184	84	168
1	0.50	0.50	44.0	216795.0	0.5842	2.40	1.198	1.198	84	84

Table 3.5.3.3. Amplified drift for Seismic load, Frame A (P-D drift)

	Total						Amplified		Allowable	
Story	Interstory	Absolute	Vu	Pu	Q	Beta	Interstory	Absolute	Interstory	Absolute
4	19.32	39.98	4127.2	31916.2	0.0249	1.03	108.991	258.496	120	372
3	8.08	20.66	1741.2	96145.9	0.1062	1.12	49.719	149.504	84	252
2	8.40	12.58	1162.9	155980.7	0.2682	1.37	63.113	99.785	84	168
1	4.18	4.18	578.0	216795.0	0.3732	1.60	36.672	36.672	84	84

Table 3.5.3.4. Amplified drift for Seismic load, Frame 1 (P-D drift)

Story	Total						Amplified		Allowable	
	Interstory	Absolute	Vu	Pu	Q	Beta	Interstory	Absolute	Interstory	Absolute
4	19.90	43.76	4127.2	31916.2	0.0256	1.03	112.325	297.629	120	372
3	8.99	23.87	1741.2	96145.9	0.1181	1.13	56.047	185.304	84	252
2	9.85	14.88	1162.9	155980.7	0.3145	1.46	79.007	129.258	84	168
1	5.03	5.03	578.0	216795.0	0.4493	1.82	50.250	50.250	84	84

3.5.4. Stability Check

One more vital aspect is to take into account the gravity effect on the lateral load resisting system, in this case, it is P-delta effect. In order to determine this effect as negligible, the stability coefficient has to be determined. The method of calculation follows the following formula:

$$\theta = \frac{P_x \Delta I_e}{V_x h_{sx} C_d}$$

Only when the stability coefficient is less than 0.1, then, the P-delta effect may be neglected.

Using the axial load and seismic forces which have been identified above as Pu and Vu respectively, the stability coefficient can be identified. Table 3.5.4.1 below illustrates the values that have been used and the stability coefficient that was determined to take into consideration the long and short sides of the residential building in Frame 1 and Frame A.

Table 3.5.4.1. Stability Coefficient Calculation for Frame 1

Level	Pu	Vu	Inter-story Drift	Story height	Theta
4	31916.2	4127.2	19.90	6000	0.005
3	96145.9	1741.2	8.99	4200	0.021
2	155980.7	1162.9	9.85	4200	0.057
1	216795.0	578.0	5.03	4200	0.082

Table 3.5.4.2. Stability Coefficient Calculation for Frame A

Level	Pu	Vu	Inter-story Drift	Story height	Theta
4	31916.2	4127.2	19.32	6000	0.005
3	96145.9	1741.2	8.08	4200	0.019
2	155980.7	1162.9	8.40	4200	0.049
1	216795.0	578.0	4.18	4200	0.068

3.5.5. Torsional Irregularity Check

Given that seismic loading is the most critical force laterally, it has the potential of causing torsional irregularities that can be structurally hazardous. Torsional irregularity check was thus conducted. This is by determining a ratio between the maximum inter-story movement and the mean inter-story movement and comparing it with the permissible limit. This ratio should not be over 1.2 according to the requirements of ACI 318-19 (ACI, 2019). Table 3.5.3.1 below is a summary of the torsional irregularity analysis of the shopping mall floors. As illustrated, all floors are not beyond the given limit meaning that the structure has not been predisposed to torsional irregularity.

Table 3.5.5.1. Torsional Irregularity Check

Ratio		
Level	Transverse	Longitudinal
4	1.1495	1.0652
3	1.1474	1.0634
2	1.1481	1.0629
1	1.1489	1.0629

3.6. Internal Force Analysis

3.6.1. Dead Load Internal Force Calculation

To illustrate the internal forces of the building frames, the internal force analysis dead load, and wind load were selected. All the Dead Load internal force results of the SAP 2000 software were obtained and are presented below, both in 3D and in 2D frame.

+

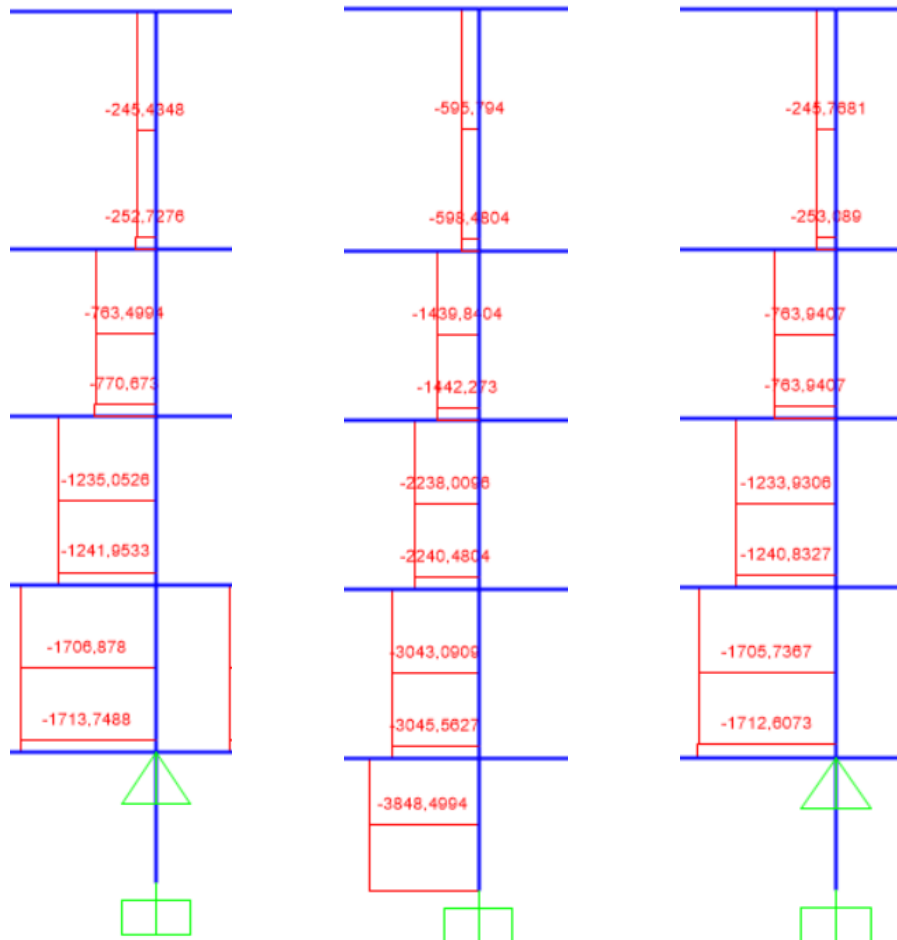


Figure 3.6.1.1 Axial Force from Dead Load in SAP2000 3D frame model

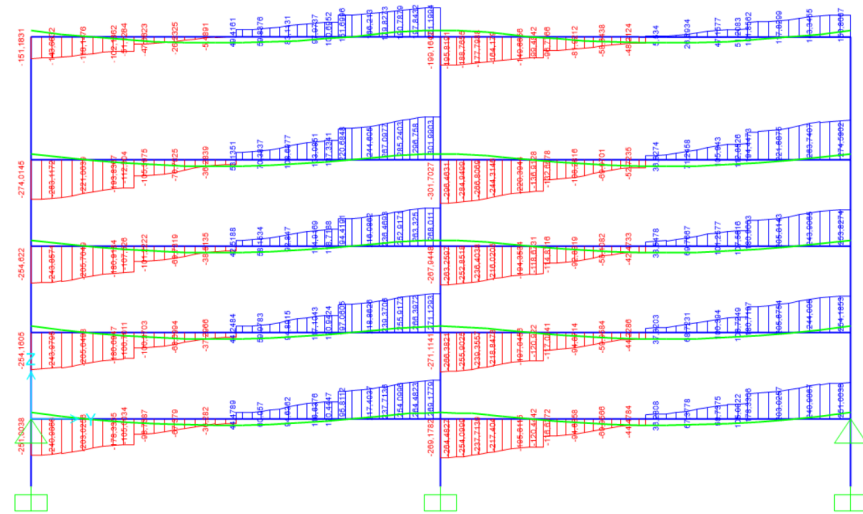


Figure 3.6.1.2 Shear Force from Dead Load in SAP2000 3D frame model

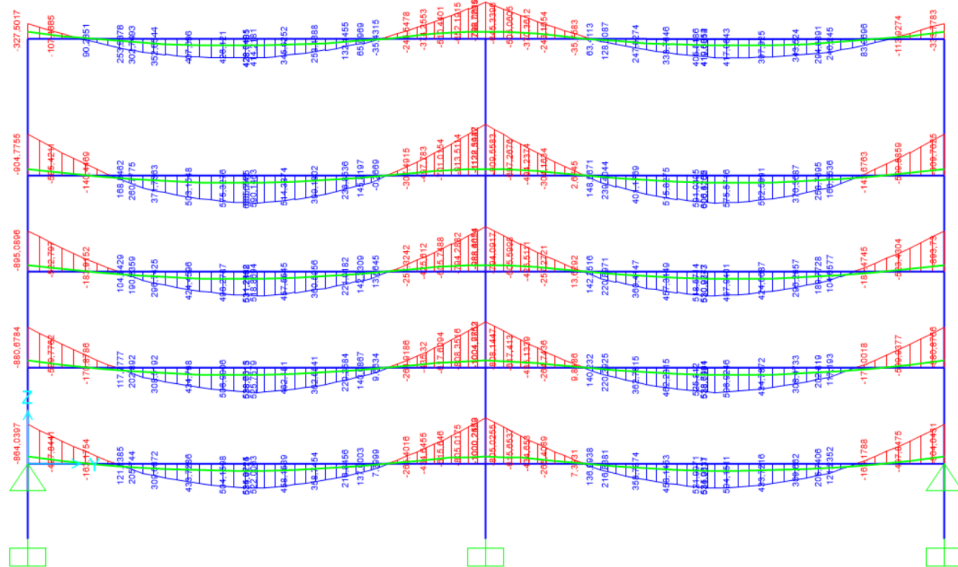


Figure 3.6.1.3 Moment Diagram of Dead Load in SAP2000 3D frame model

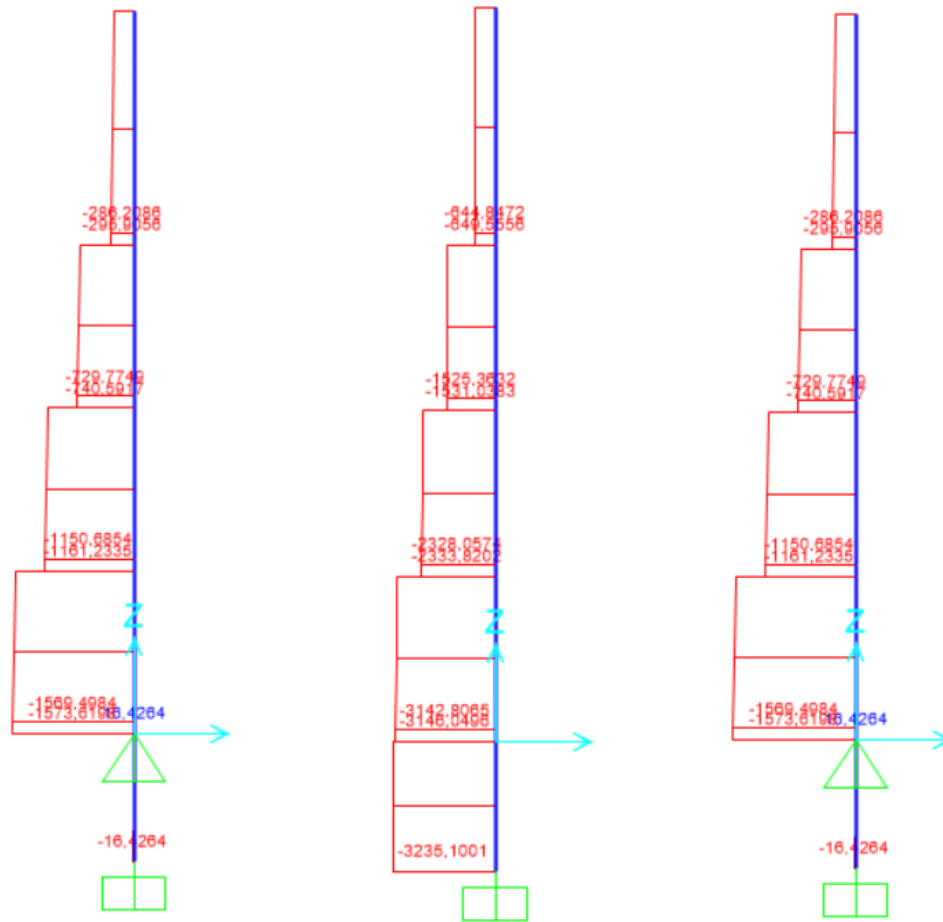


Figure 3.6.1.4 Axial Force from Dead Load in SAP2000 2D frame model

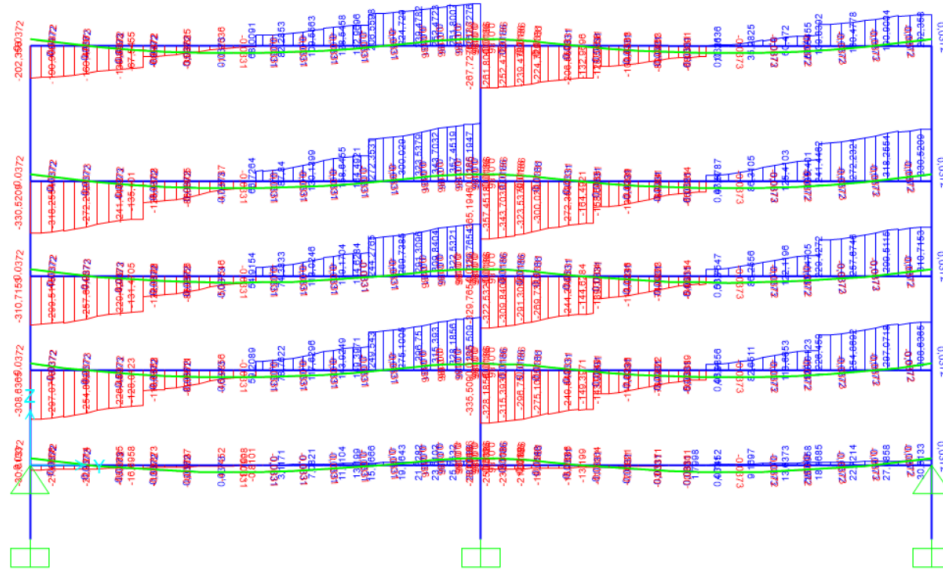


Figure 3.6.1.5 Shear Force from Dead Load in SAP2000 2D frame model

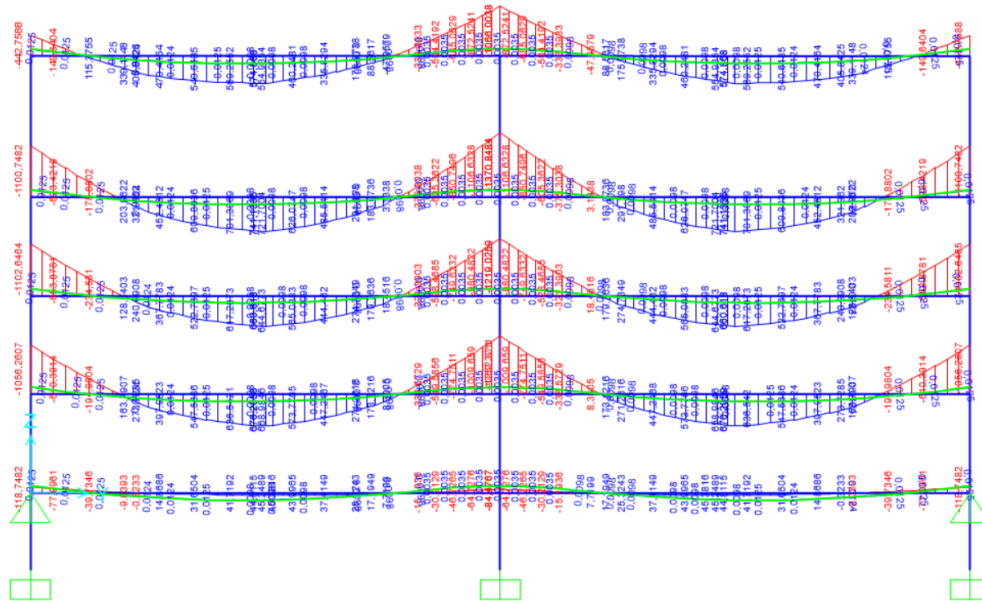


Figure 3.6.1.6 Moment Diagram of Dead Load in SAP2000 2D frame model

Now the SAP results require hand calculations. In addition, they will have to be instructed to perform comparative analysis and develop internal force comparison graphs.

The schematic diagram shown below represents the method adopted to compute the internal forces under the Gravity Loads.

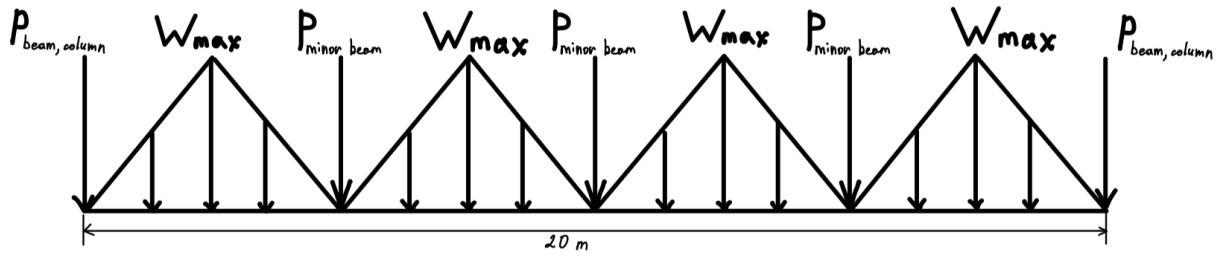


Figure 3.6.1.7 Applied Dead Load Illustration

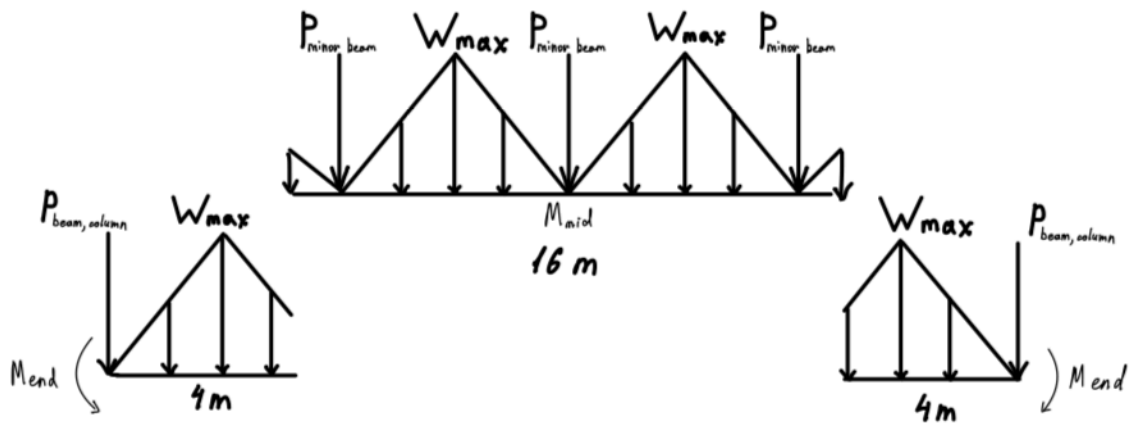


Figure 3.6.1.8 Analyzed Vertical Load Frame

Final results for hand calculation are presented in the figures below.

Axial Force from Dead Load by Hand Calculation, kN

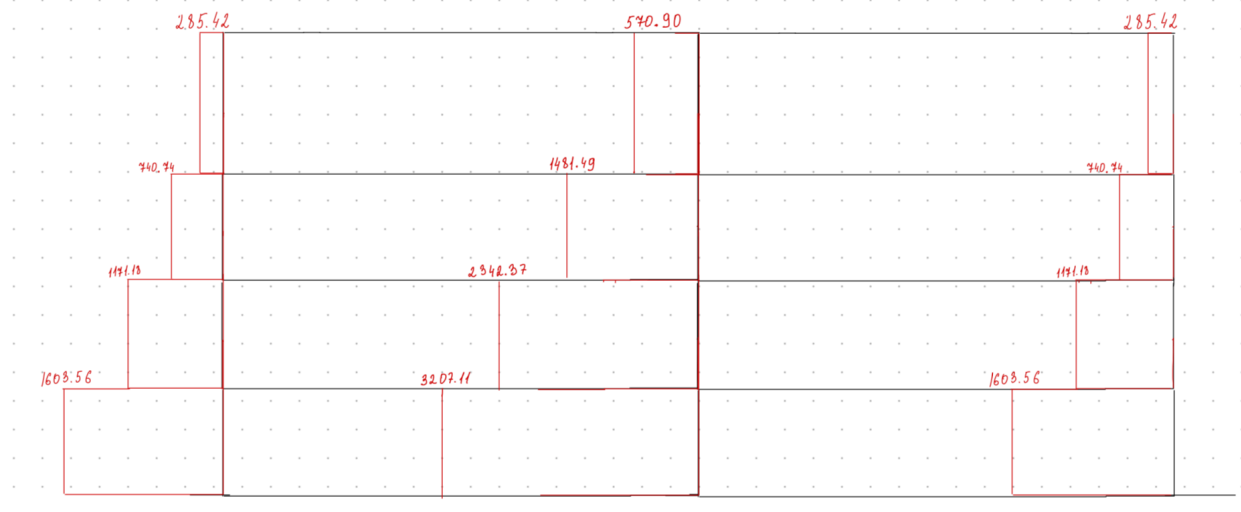


Figure 3.6.1.9 Hand Calculation Results of Axial Force from Dead Load

Shear Force from Dead Load by Hand Calculation, kN

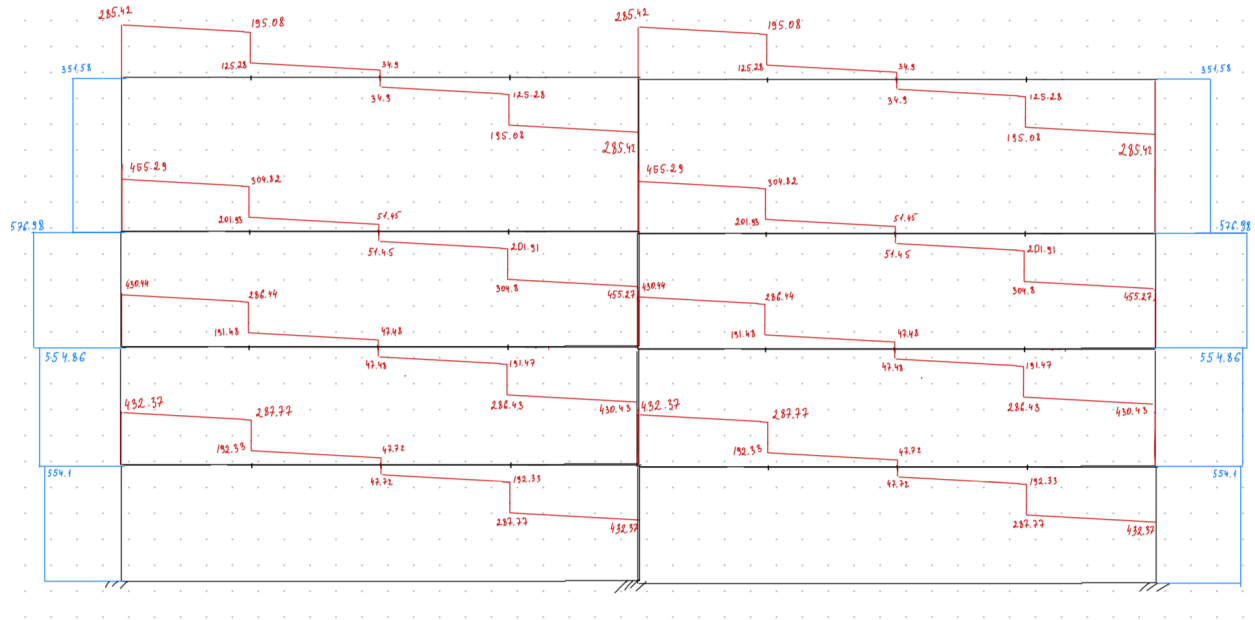


Figure 3.6.1.10 Hand Calculation Results of Shear Force from Dead Load

Moment Diagram of Dead Load by Hand Calculation, kN-m

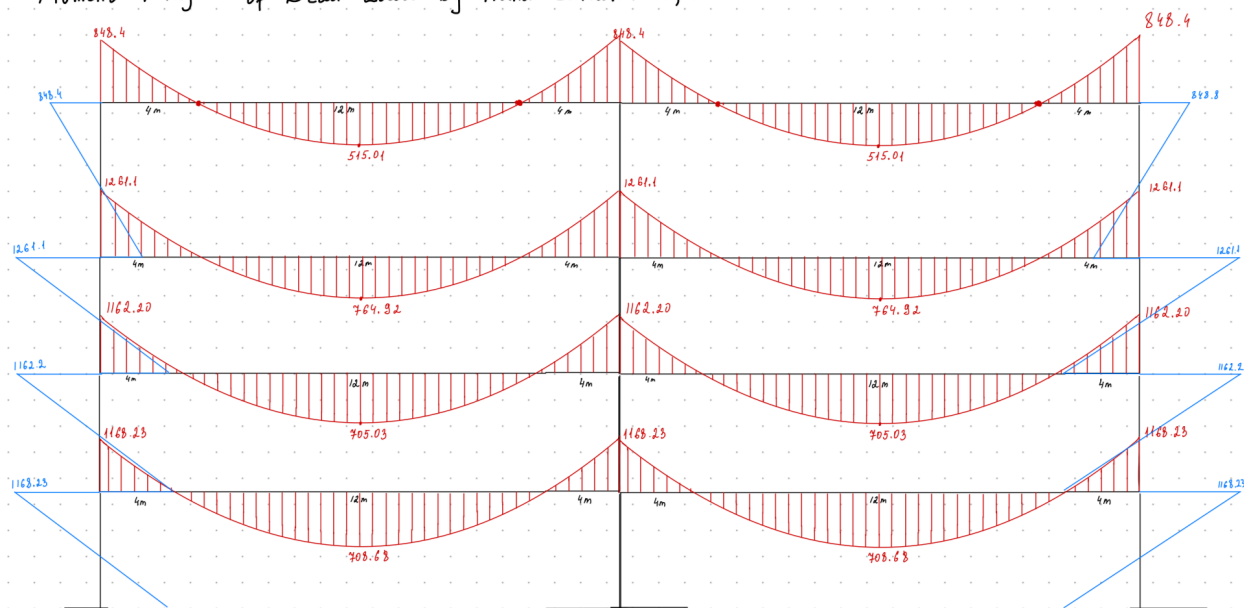


Figure 3.6.1.10 Hand Calculation Results of Moments from Dead Load

Due to hand calculations and simulation of SAP 2000, the comparative tables were prepared to show the variance between the two approaches. Table 3.6.1.1 and Figure 3.6.1.9 of the axial load, shear force and moment created under dead load can be obtained below.

Table 3.6.1.1. Comparison of Axial Forces under Dead Load

Levels	Axial force in 3D, kN	Axial force in 2D, kN	Axial force in Hand, kN
4	252.72	295.97	285.42
3	770.67	740.59	740.74
2	1241.95	1161.23	1171.18
1	1713.75	1573.61	1603.56

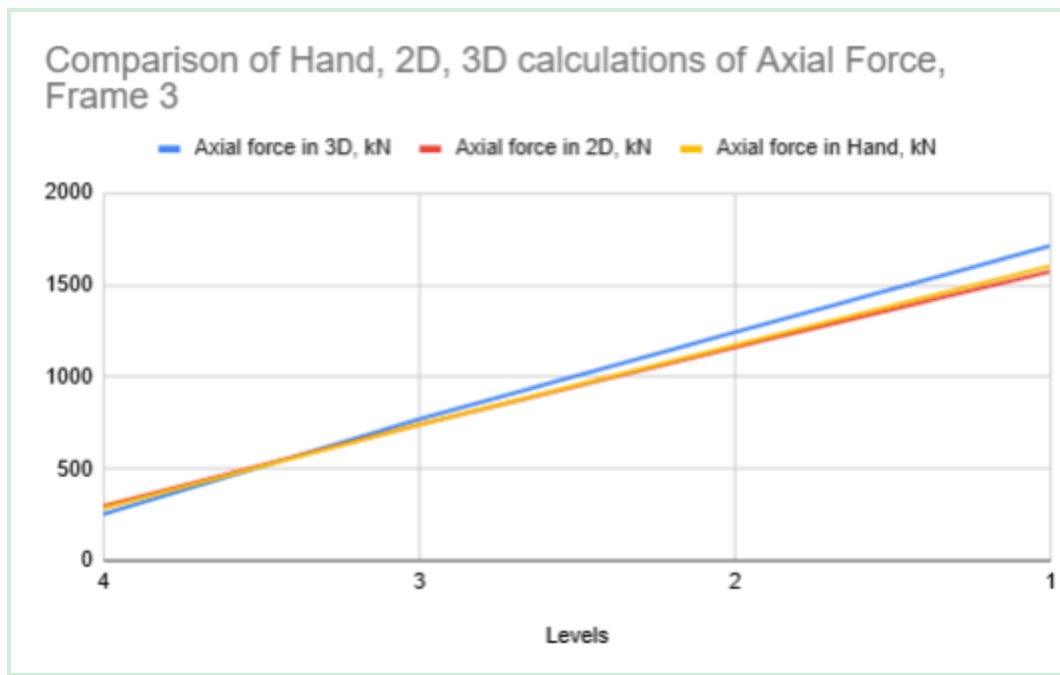


Figure 3.6.1.11. Dead Load Axial Force Comparison between Hand Calculations and SAP2000

Table 3.6.1.2. Comparison of Shear Forces under Dead Load

Levels	Shear force in 3D, kN	Shear force in 2D, kN	Shear force in hand, kN
4	282.02	278.76	285.42
3	426.07	414.36	455.29

2	402.66	395.53	430.44
1	412.93	393.32	432.37

Comparison of Hand, 2D, 3D calculations of Shear Force, Frame 3

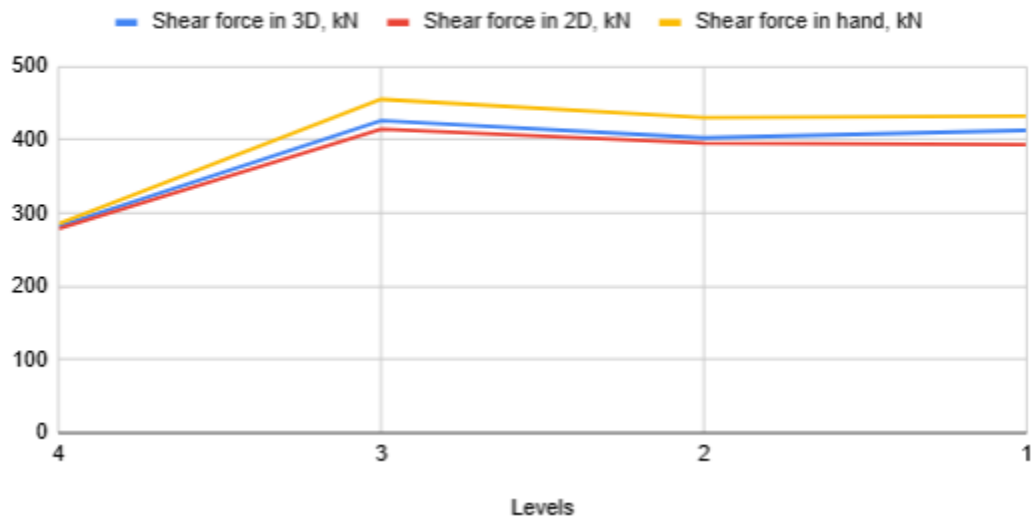


Figure 3.6.1.12. Dead Load Shear Force Comparison between Hand Calculations and SAP 2000

Table 3.6.1.3. Comparison of Moments under Dead Load

Levels	Moment in 3D, kN-m	Moment in 2D, kN-m	Moment in hand, kN-m
4	747.89	719.15	848.4
3	1454.90	1254.37	1261.1
2	1405.32	1182.70	1162.2
1	1416.18	1173.44	1168.23

Comparison of Hand, 2D, 3D calculations of Moment, Frame 3

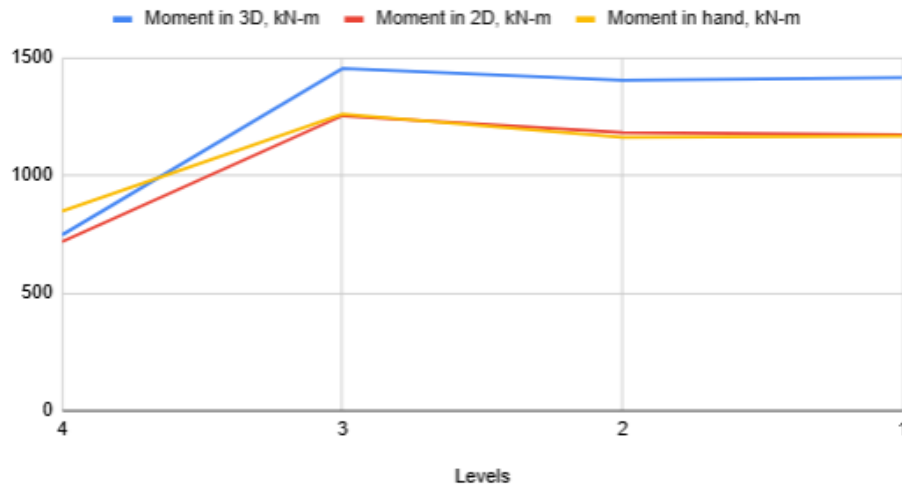
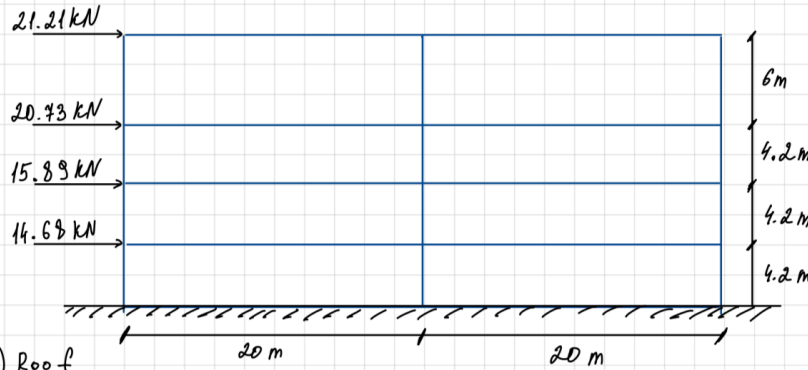


Figure 3.6.1.13. Dead Load Moment Comparison between Hand Calculations and SAP 2000

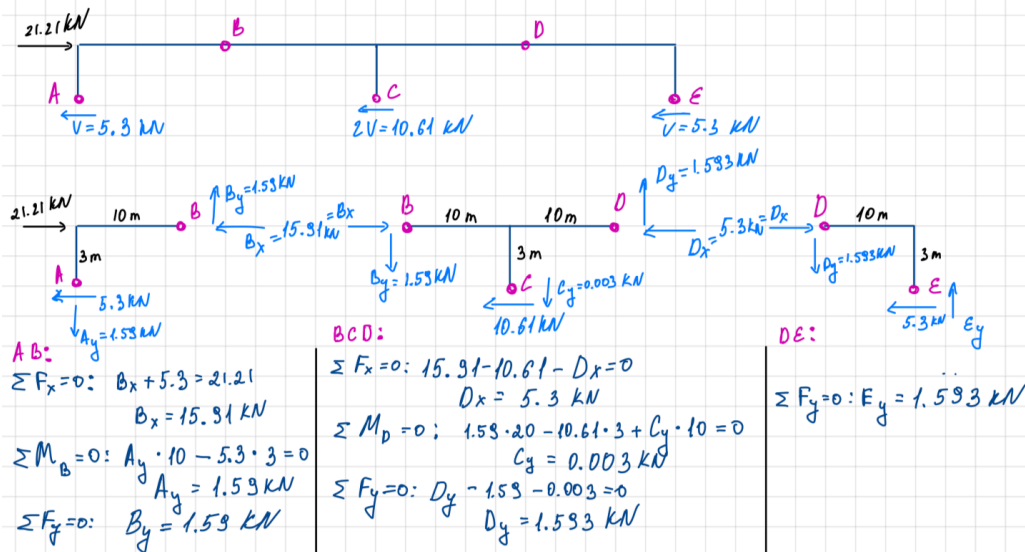
3.6.2. Wind Load Internal Force Calculation

Moving on to the calculations of internal forces at the wind load, the Portal Frame method was used to make the hand calculations. The computations of all the Portal Frame are as shown below. The manual calculations are clear evidence of the process of calculating the internal forces (axial load, shear force and moment) of all 4 floors.

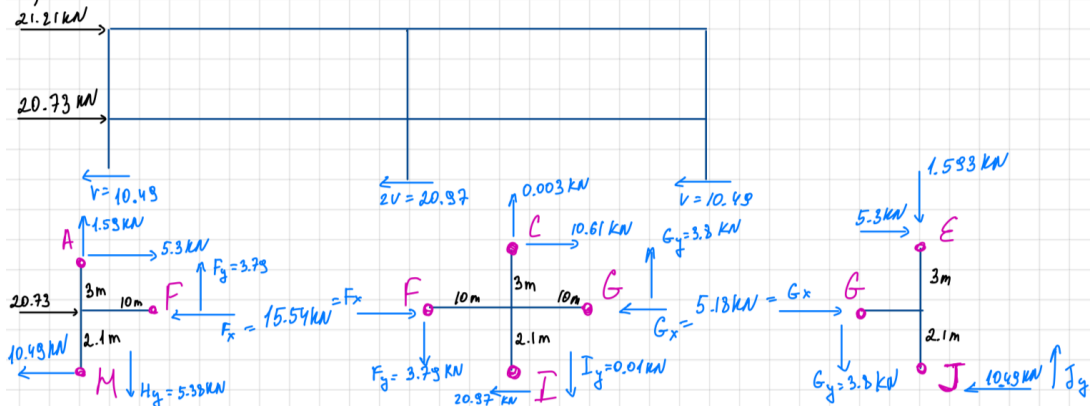
Internal forces due to Wind load



1) Roof



2) Floor 4.



$$\sum F_x = 0: 20.73 + 5.3 - 10.49 - F_x = 0$$

$$F_x = 15.54 \text{ kN}$$

$$\sum M_F = 0: -1.53 \cdot 10 - 5.3 \cdot 3 - 10.49 \cdot 2.1 + H_y \cdot 10 = 0$$

$$H_y = 5.38 \text{ kN}$$

$$\sum F_y = 0: 1.53 - 5.38 + F_y = 0$$

$$F_y = 3.79 \text{ kN}$$

$$\sum F_x = 0: 15.54 + 10.61 - 20.97 - G_x = 0$$

$$G_x = 5.18 \text{ kN}$$

$$\sum M_G = 0: -0.003 \cdot 10 - 10.61 \cdot 3 +$$

$$+ 3.79 \cdot 20 - 20.97 \cdot 2.1 + I_y \cdot 10 = 0$$

$$I_y = 0.01 \text{ kN}$$

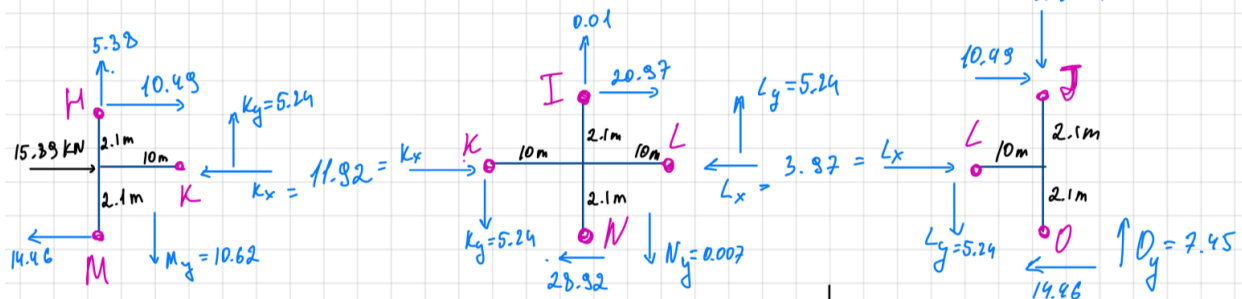
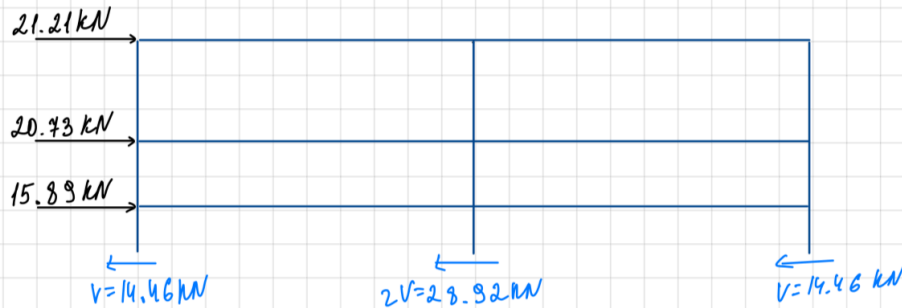
$$\sum F_y = 0: G_y + 0.003 - 3.79 - 0.01 = 0$$

$$G_y = 3.80 \text{ kN}$$

$$\sum F_y = 0: J_y - 3.3 - 1.593 = 0$$

$$J_y = 2.207 \text{ kN}$$

3) Floor 3



$$\sum F_x = 0: 15.89 + 10.49 - 14.46 - K_x = 0$$

$$K_x = 11.92 \text{ kN}$$

$$\sum M_F = 0: -5.38 \cdot 10 - 10.49 \cdot 2.1 - 14.46 \cdot 2.1 + M_y \cdot 10 = 0$$

$$M_y = 10.62 \text{ kN}$$

$$\sum F_y = 0: 5.38 - 10.62 + K_y = 0$$

$$K_y = 5.24 \text{ kN}$$

$$\sum F_x = 0: 11.92 + 20.97 - 28.92 - L_x = 0$$

$$L_x = 3.97 \text{ kN}$$

$$\sum M_L = 0: -0.01 \cdot 10 - 20.97 \cdot 2.1 +$$

$$+ 5.24 \cdot 20 - 28.92 \cdot 2.1 + N_y \cdot 10 = 0$$

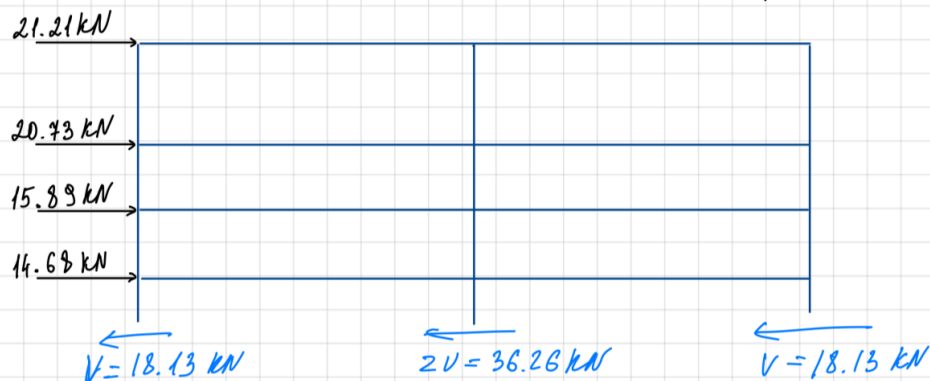
$$N_y = 0.007 \text{ kN}$$

$$\sum F_y = 0: L_y + 0.01 - 5.24 - 0.007 = 0$$

$$L_y = 5.24 \text{ kN}$$

$$\sum F_y = 0: O_y - 2.207 - 5.24 = 0$$

$$O_y = 7.45 \text{ kN}$$



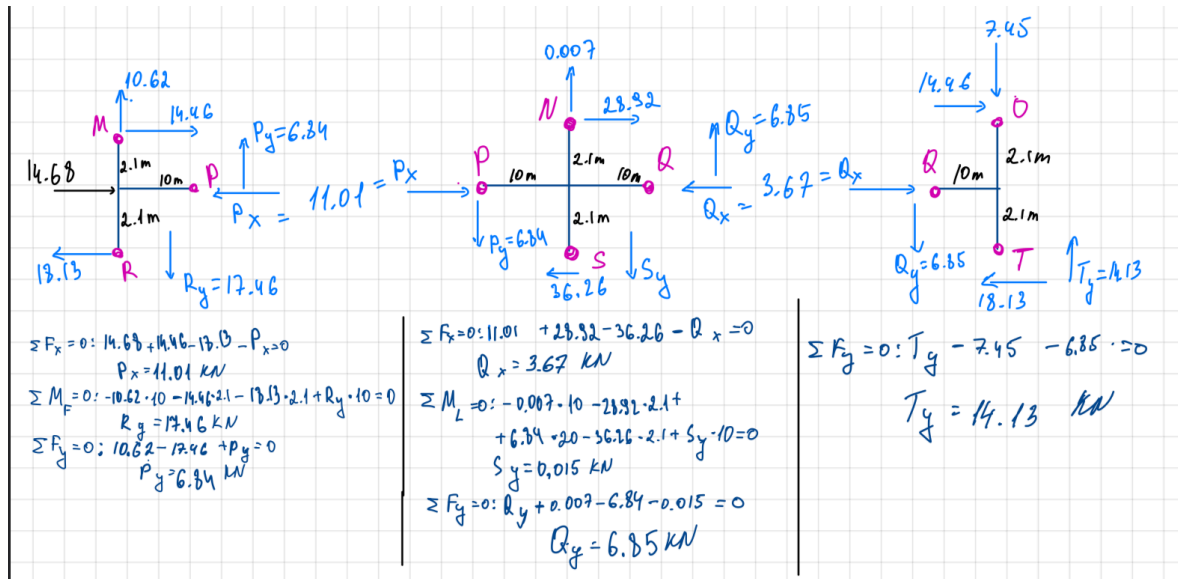


Figure 3.6.2.1. Portal Method Calculations for Wind Load Internal Force

Final results for hand calculation are presented in the figures below.

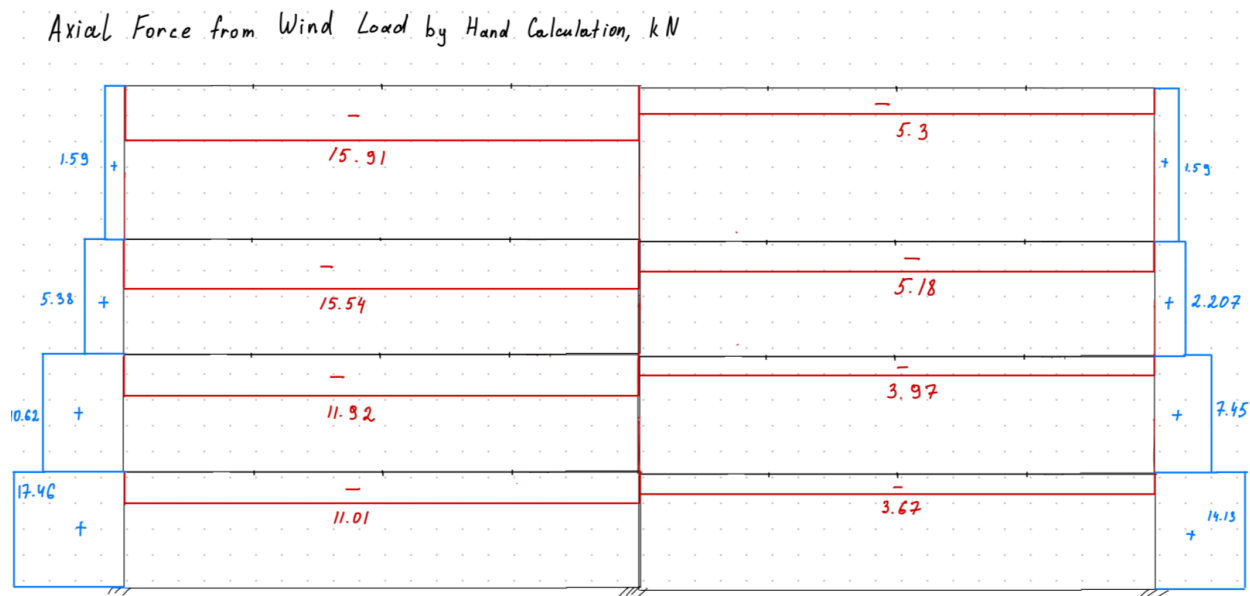


Figure 3.6.2.2 Hand Calculation Results of Axial Force from Wind Load

Shear Force from Wind Load by Hand Calculation, kN

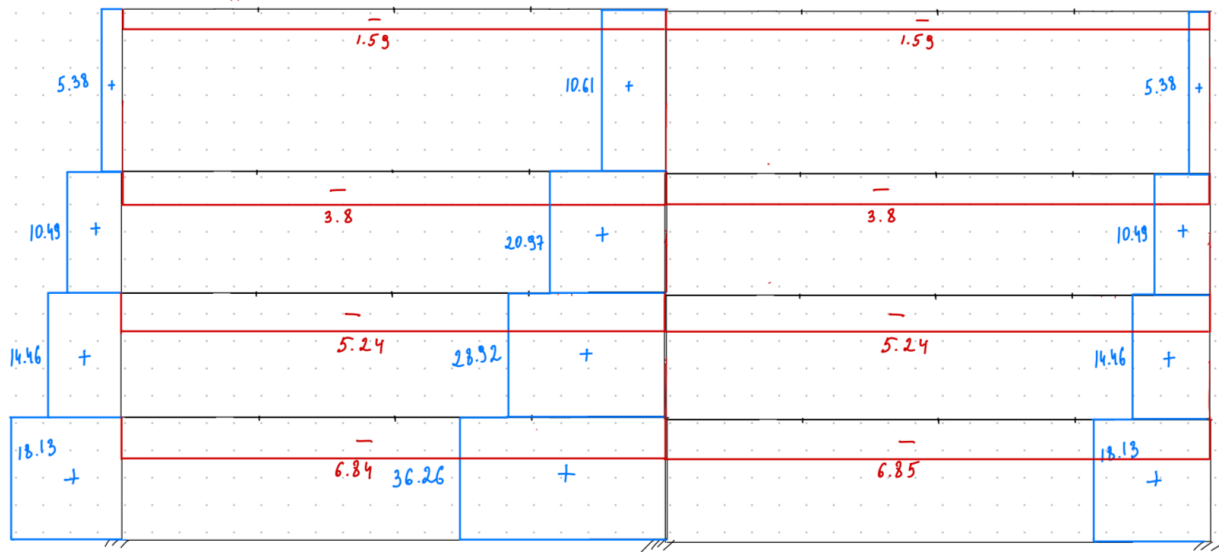


Figure 3.6.2.3 Hand Calculation Results of Shear Force from Wind Load

Moment Diagram of Wind Load by Hand Calculation, kN-m

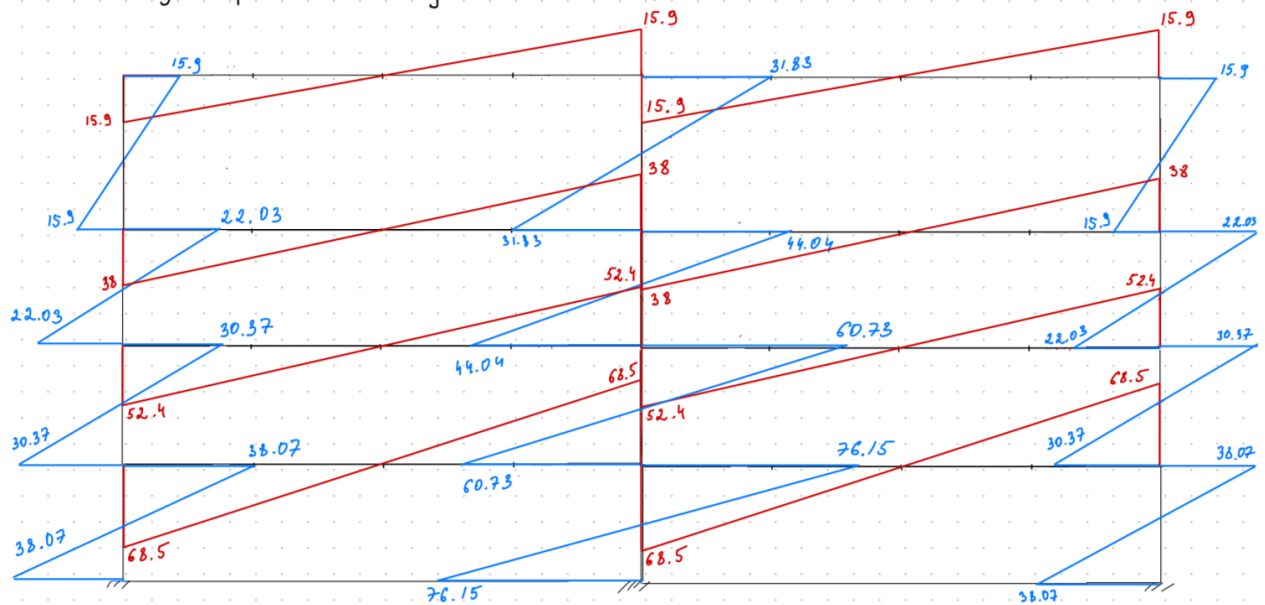


Figure 3.6.2.4 Hand Calculation Results of Moments from Wind Load

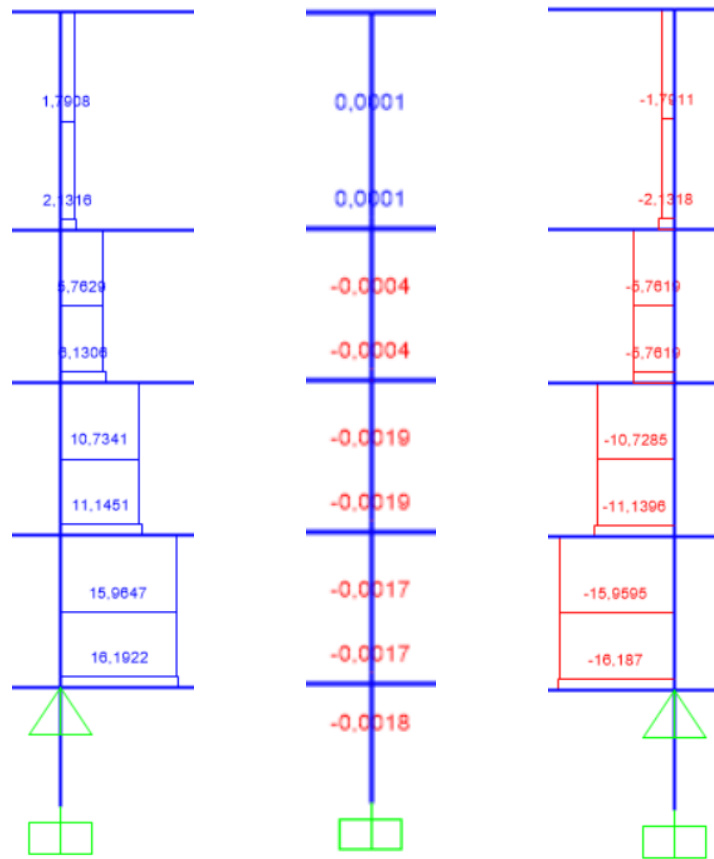


Figure 3.6.2.5 Axial Force Diagram of Wind Load in SAP2000 3D frame model for Columns

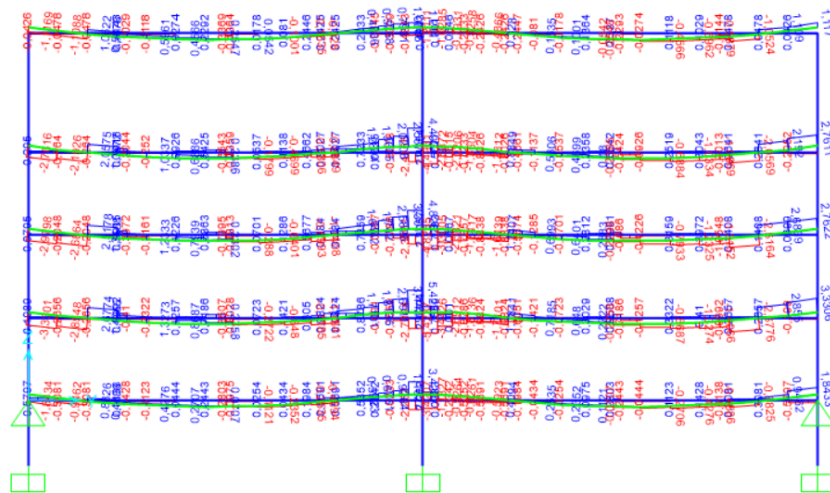


Figure 3.6.2.6 Axial Force Diagram of Wind Load in SAP2000 3D frame model for Beams

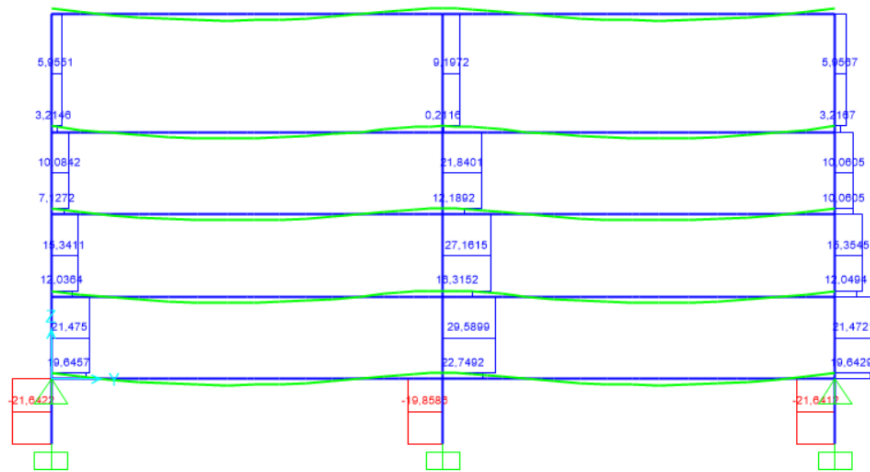


Figure 3.6.2.7 Shear Force Diagram of Wind Load in SAP2000 3D frame model for Columns

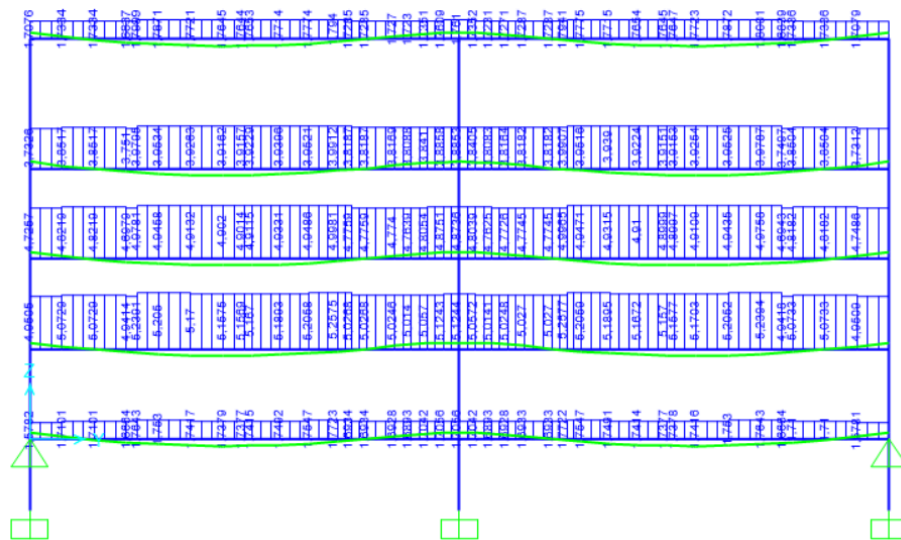


Figure 3.6.2.8 Shear Force Diagram of Wind Load in SAP2000 3D frame model for Beams

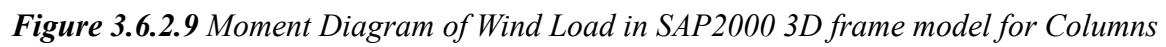


Figure 3.6.2.10 Moment Diagram of Wind Load in SAP2000 3D frame model for Beams

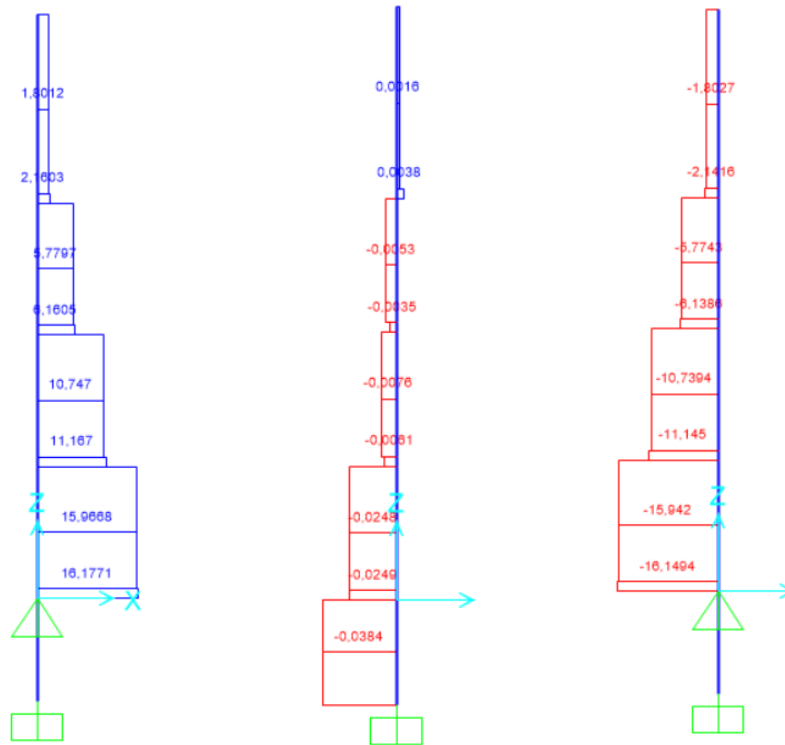


Figure 3.6.2.11 Axial Force Diagram of Wind Load in SAP2000 2D frame model for Columns

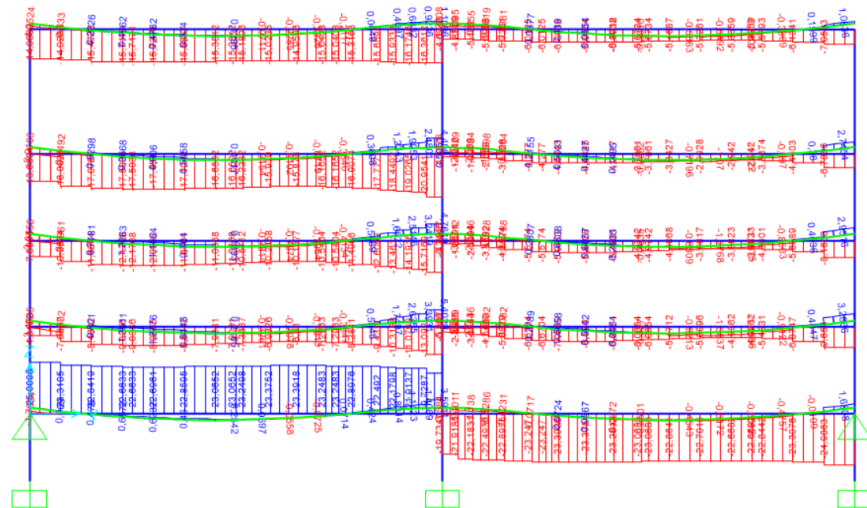


Figure 3.6.2.12 Axial Force Diagram of Wind Load in SAP2000 2D frame model for Beams

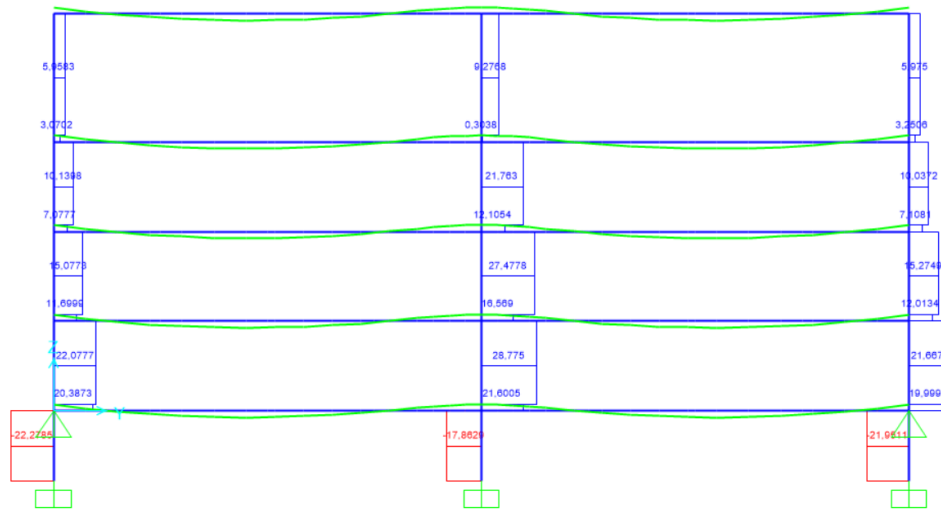


Figure 3.6.2.13 Shear Force Diagram of Wind Load in SAP2000 2D frame model for Columns

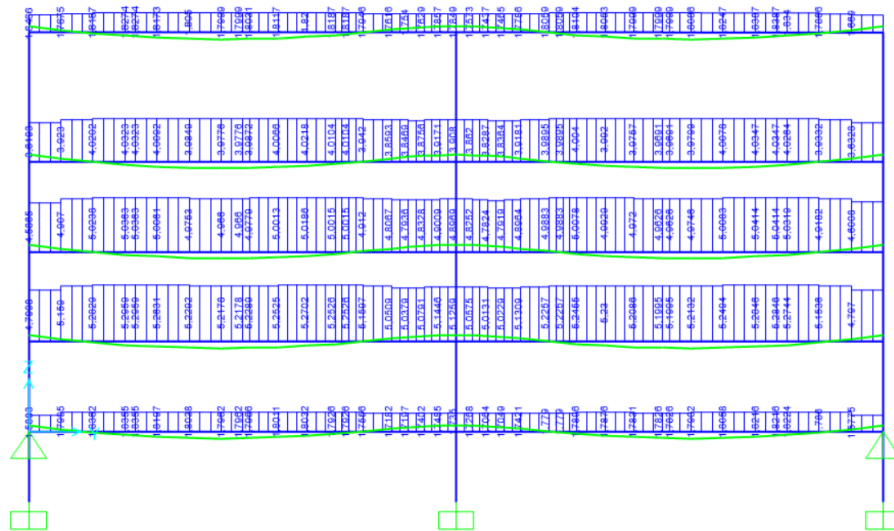


Figure 3.6.2.14 Shear Force Diagram of Wind Load in SAP2000 2D frame model for Beams

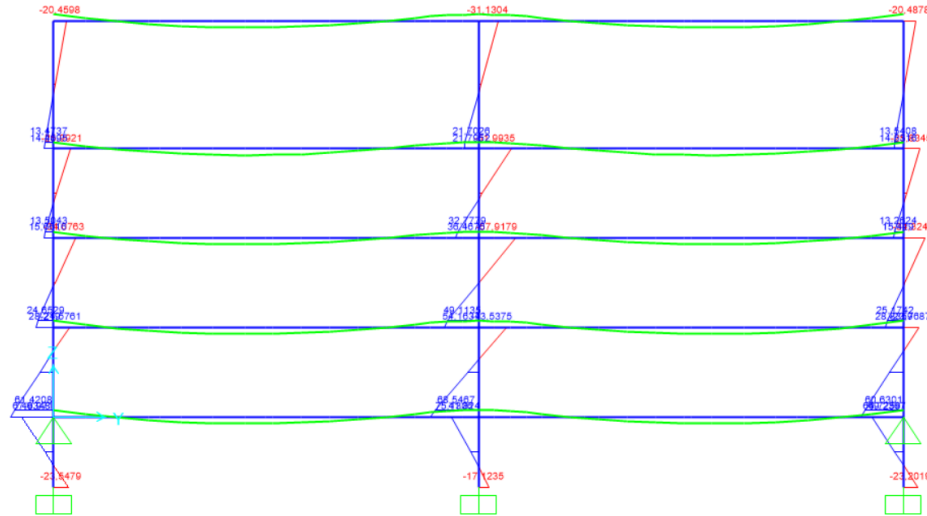


Figure 3.6.2.15 Moment Diagram of Wind Load in SAP2000 2D frame model for Columns

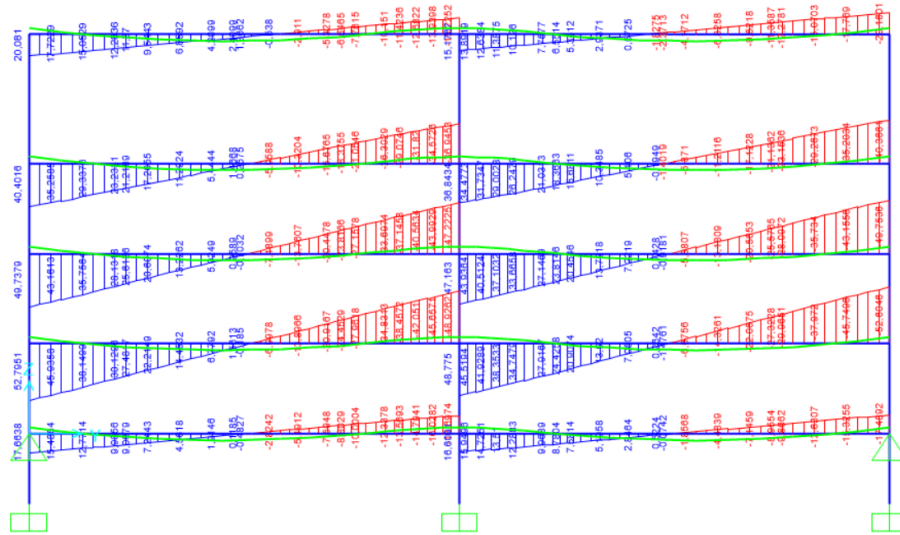


Figure 3.6.2.16 Moment Diagram of Wind Load in SAP2000 2D frame model for Beams

The hand calculations should be compared, like always to SAP 2000 simulation results. Thus, similarly to the last section, the following Tables 3.6.2.1 and Figures 3.6.2.2 -3.6.2.5 were developed to illustrate the difference between the hand-calculations and SAP. It is relevant to note that SAP 2000 simulations are never less precise because they apply more accurate techniques of arriving at the internal forces, whereas the hand calculations apply less accurate techniques of determining the variables.

The very concept of portal frame method presupposes the following: 1) internal frames are sheared twice more than external frames as the portals are adding to each other; 2) the moment at the center of a column and a beam are set to zero. The values for the first floor were significantly different in the Sap200. This can be explained by the fact that support was assigned as a pin.

Table 3.6.2.1. Comparison of internal forces under wind load (2D, 3D, and hand calculations)

	Hand			2D			3D		
Floor	Shear force, kN	Axial force, kN	Moment, kN-m	Shear force, kN	Axial force, kN	Moment, kN-m	Shear force, kN	Axial force, kN	Moment, kN-m
4	5.3	1.59	15.9	5.33	2.16	20.46	5.29	2.13	20.38
3	10.49	5.38	22.03	10.67	5.82	25.99	10.2	5.72	25.92
2	14.46	10.62	30.37	15.64	9.75	34.07	16.02	9.59	34.35
1	18.13	17.46	38.07	23.67	13.17	24.57	22.29	12.97	23.77

Axial force in 3D, kN, Axial force in 2D, kN and Axial force in Hand, kN

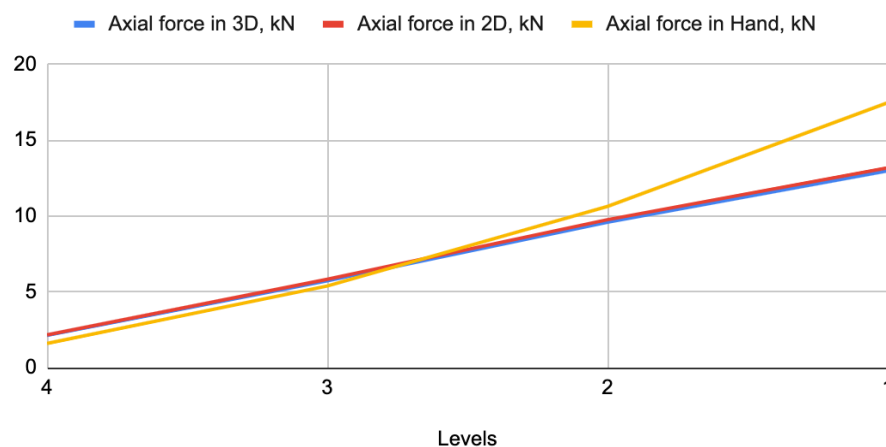


Figure 3.6.2.2. *Wind Load Axial Force Comparison between Hand Calculations and SAP 2000*

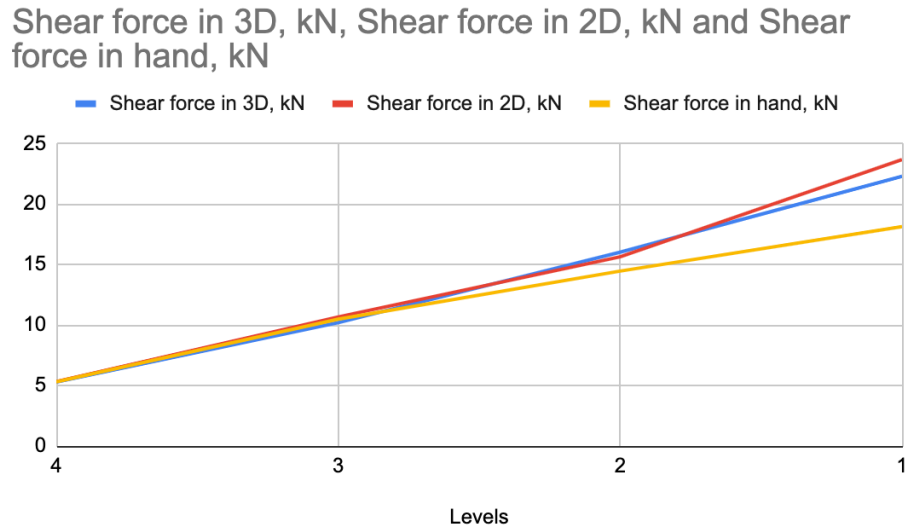


Figure 3.6.2.3. *Wind Load Shear Force Comparison between Hand Calculations and SAP 2000*

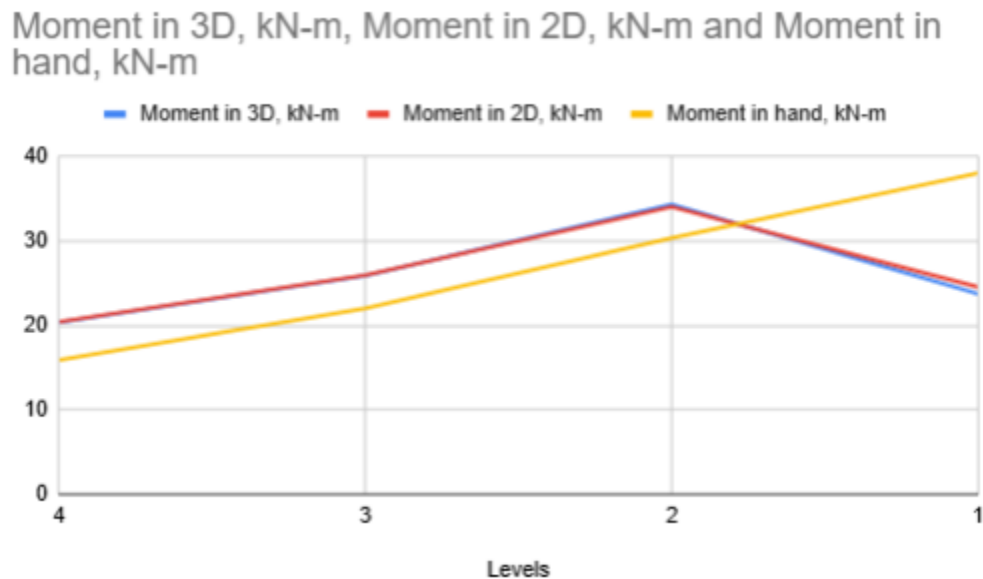


Figure 3.6.2.4. *Wind Load Moment Comparison between Hand Calculations and SAP 2000*

3.6.3. Load Combinations

Under the next section Structural Design, the next load combinations represented in Figure 3.6.3.1 will serve to consider any potential case or scenario that can occur to the building. As one

can observe in the figure, all the combinations of loads will be in accordance with the ASCE-7 code.

ASCE-7

1. $1.4D$
2. $1.2D + 1.6L + 0.5(L_r \text{ or } S \text{ or } R)$
3. $1.2D + 1.6(L_r \text{ or } S \text{ or } R) + (L \text{ or } 0.5W)$
4. $1.2D + 1.0W + L + 0.5(L_r \text{ or } S \text{ or } R)$
5. $1.2D + 1.0E + L + 0.2S$
6. $0.9D + 1.0W$
7. $0.9D + 1.0E$

Figure 3.6.3.1. Load Combinations

3.7. Structural Member Design

The designing of structural members was highly utilised with the use of SAP 2000. The software shows percentages of steel rebar of certain loading cases. All the values obtained in SAP are checked by hand-calculations that are presented below. SAP 2000 is in compliance with the standards of ACI 318-19.

Figure X below represents the Structural Design Verification process to determine whether the structural members can pass the applied loading combinations. It can be observed that as the frames are able to surpass all the structural load combinations, and thus, the model is applicable in the design of the structural members.

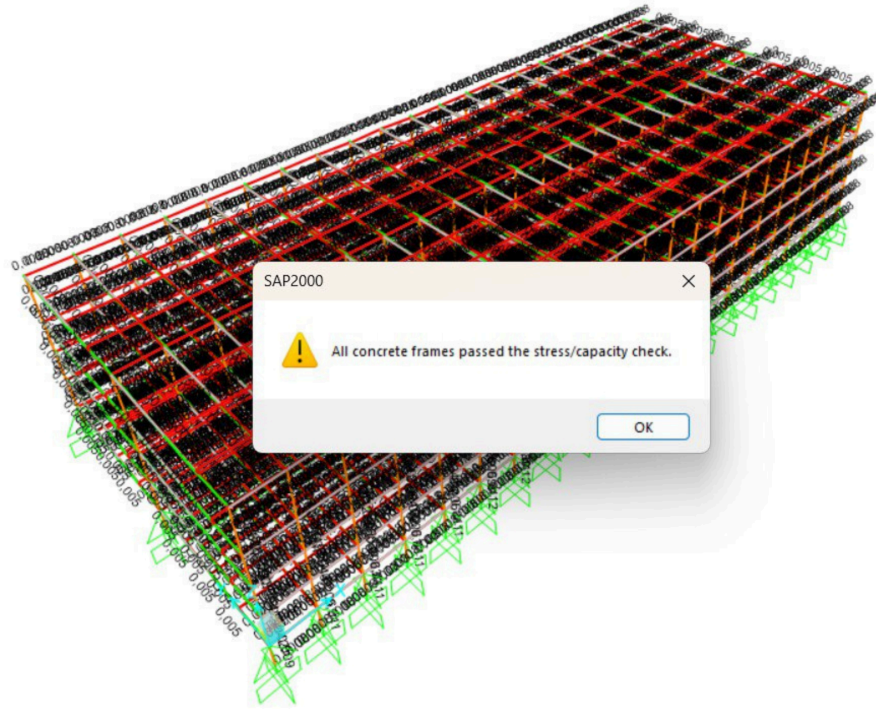


Figure 3.7.0.1. Capacity Check Verification

3.7.1. Major and Minor Beam Design

3.7.1.1. Longitudinal Reinforcement Design

Figure 3.7.1.1.1 provided below is taken in SAP 2000 to confirm the value of the required rebar. To illustrate the case, we will demonstrate the whole design process when the negative moment is the key of the major beam. The negative moment of the major beam in this particular case is -131 kN/m. The top reinforcement will be designed with negative moment and the bottom reinforcement with positive moment in the following figure since both of these SAP 2000 summaries are in the END section of the major beam where the compressive forces are larger than the tensile ones.

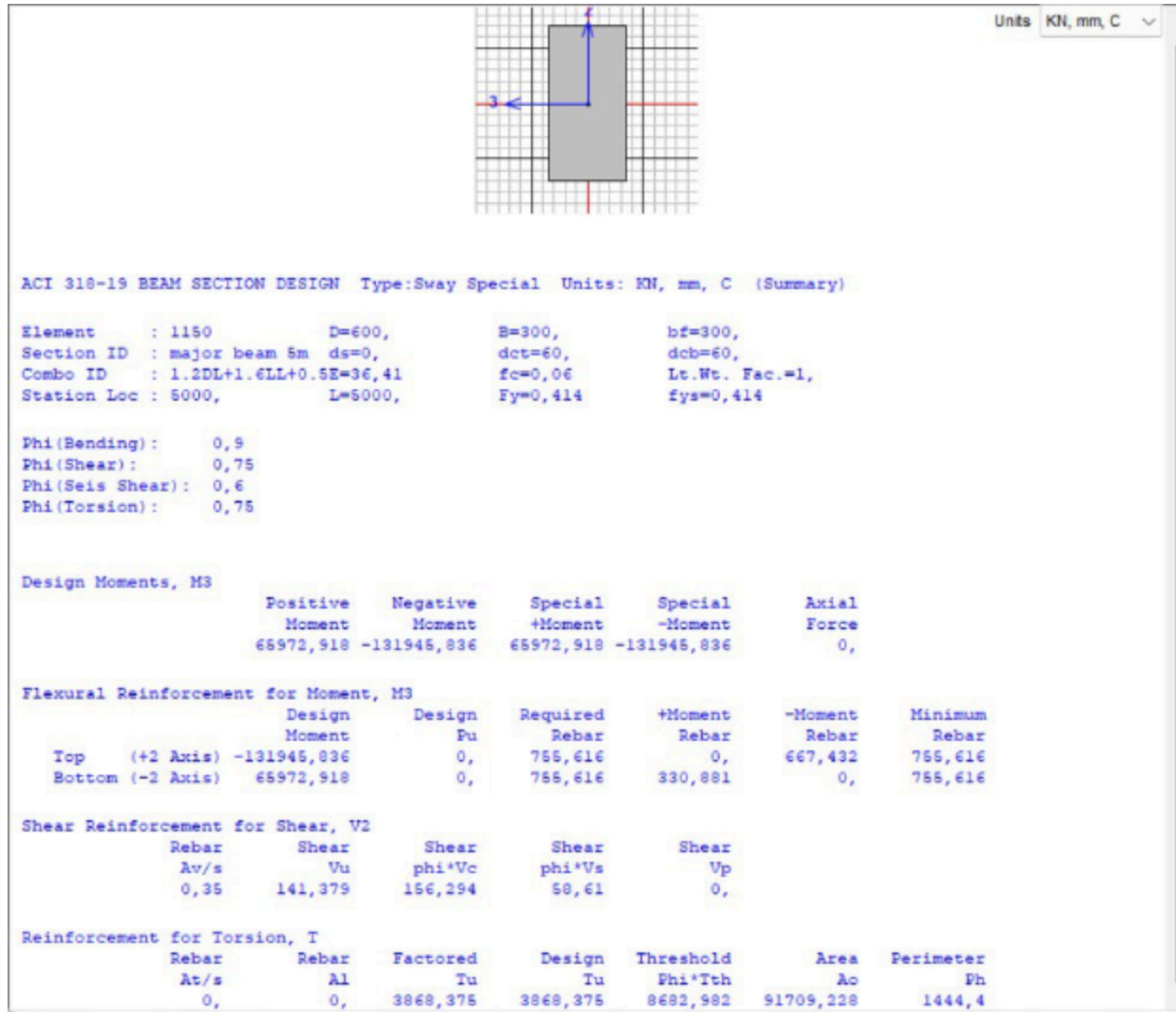


Figure 3.7.1.1.1 Major Beam Reinforcement Design for Negative Moment

5m Major beam

Longitudinal Reinforcement design for Negative Moment:

$$M_u = 131.9 \text{ kN} - \text{m}$$

$$b = 300 \text{ mm}, h = 600 \text{ mm}$$

$$d = 600 - 2.5 \text{ in} \cdot 25.4 \text{ mm/in} = 536.5 \text{ mm} \sim 540 \text{ mm}$$

$$f'_c = 40 \text{ MPa}, f_y = 413 \text{ MPa}$$

$$R_n = \frac{M_u}{\phi b d^2} = \frac{131}{0.9 \cdot 0.3 \cdot 0.54^2} = 1663.87 \text{ kPa}$$

$$\rho = 0.85 \frac{f'_c}{f_y} \left(1 - \sqrt{1 - \frac{2R_n}{0.85 f'_c}} \right) = 0.0041$$

$$A_s = \rho b d = 663.67 \text{ mm}^2$$

$$a = \frac{A_s * f_y}{0.85 * f'_c * b} = 17.91 \text{ mm}$$

$$\beta_1 = 0.085 - 0.05 \frac{f'_c - 4000}{1000} = 0.615$$

$$c = \frac{a}{\beta_1} = 17.91 \text{ mm}$$

$$\epsilon_t = \frac{0.003(d-c)}{c} = 0.052 > 0.005 \rightarrow \text{tension - controlled and ductile}$$

$$A_{s, SAP2000} = 755.6 \text{ mm}^2$$

SAP2000 calculated 755.6 mm², which is larger than the calculated value. To discover the error, it would be reasonable to find the error first:

$$\text{Error} = \frac{755.6 - 663.67}{755.6} * 100\% = 12.17\%$$

The mistake is not so serious and it can be justified by several various factors. The area of the minimal steel can also be verified, and as may be seen by lower equations, this is met:

$$A_{s,min} = \max\left(\frac{3\sqrt{f'_c}}{f_y} bd, \frac{200bd}{f_y}\right) = 628.47 \text{ mm}^2$$

$$A_s = 755.6 \text{ mm}^2 > 628.47 \text{ mm}^2 \rightarrow \text{sufficient reinforcement}$$

Thus, the size of the bar can be selected with the help of the $A_s = 755.6 \text{ mm}^2$ (1.17 in²). So at the highest reinforcements 4#5 bars will be adopted ($A_s = 1.24 \text{ in}^2$).

$$b_{min} = 10.88 \text{ in} = 0.276 \text{ m} < 0.3 \text{ m}$$

The stirrup is selected as #4 with a center-to-center of 180 mm. In Technical Drawings, all the specifics of the key beam reinforcements, dimension, spacing, clear cover, edge distance can be identified.

The next moment is the positive moment that is taken to design the bottom reinforcement of the end section of the major beam. It should be noted that the reinforcement plan was developed on each individual section of the major beam on each of the floors. In this case we just give the calculation process that is required to demonstrate the SAP 2000 steel percentage. The detailed design outcomes are all on Appendix.

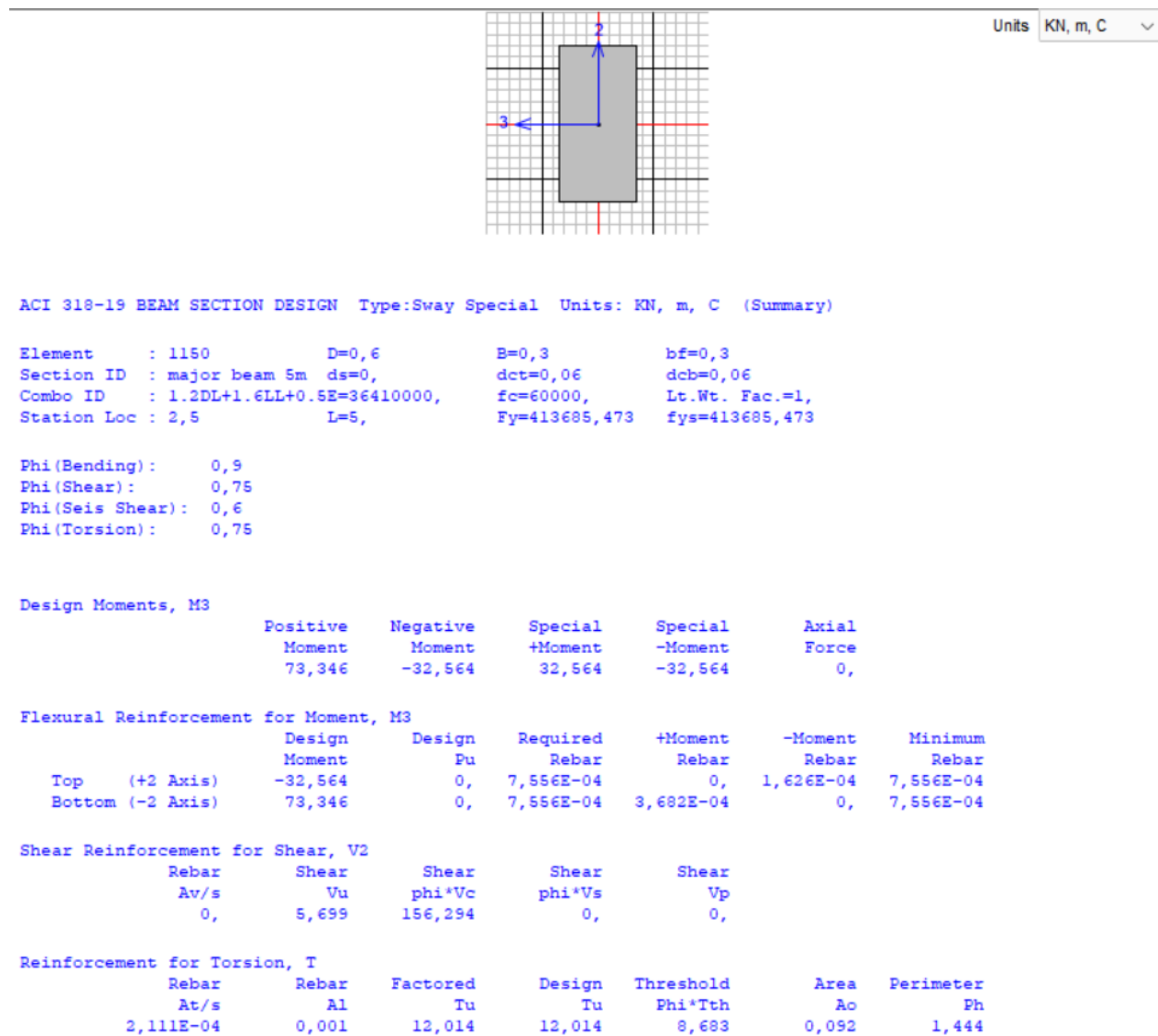


Figure 3.7.1.1.2 Major Beam Reinforcement Design for Positive Moment

$$M_u = 73.346 \text{ kN} - m$$

$$b = 300 \text{ mm}, h = 600 \text{ mm}$$

$$d = 600 - 2.5 \text{ in} \cdot 25.4 \text{ mm/in} = 536.5 \text{ mm} \sim 540 \text{ mm}$$

$$f'_c = 40 \text{ MPa}, f_y = 413 \text{ MPa}$$

$$R_n = \frac{M_u}{\phi b d^2} = \frac{73.346}{0.9 \cdot 0.3 \cdot 0.54^2} = 928.95 \text{ kPa}$$

$$\rho = 0.85 \frac{f'_c}{f_y} \left(1 - \sqrt{1 - \frac{2R_n}{0.85f'_c}} \right) = 0.00227$$

$$A_s = \rho b d = 367.76 \text{ mm}^2$$

$$a = \frac{A_s * f_y}{0.85 * f'_c * b} = 9.93 \text{ mm}$$

$$c = \frac{a}{\beta_1} = 16.14 \text{ mm}$$

$$\epsilon_t = \frac{0.003(d-c)}{c} = 0.097 > 0.005 \rightarrow \text{tension - controlled and ductile}$$

$$A_{s, SAP2000} = 489.54 \text{ mm}^2$$

$$A_s = 489.54 \text{ mm}^2 > 628.47 \text{ mm}^2 \rightarrow \text{insufficient reinforcement}$$

Since reinforcement is insufficient the minimum A_s will be used. Therefore, $A_s = 628.47 \text{ mm}^2$ and 2#7 bars were selected.

Table 3.7.1.1.1 Detailed Major Beam Reinforcement Design for Floor 4 in the Exterior Frame

5m Major Beam for Floor 4 for Exterior frame			
Variables	SAP2000		
	End	Mid	End
SAP 2000 Tension Area, mm2	794.2	774.2	850.6
Area, in2	1.23	1.200	1.318
SAP 2000 Compression Area, mm2	310.45	220.8	340.25
Area, in2	0.481	0.342	0.527
Tension Bars	4#5	2#7	3#7
Compression Bars	2#5	2#4	2#5
Shear Reinforcement	#3 at 180 mm	#3 at 250 mm	#3 at 180 mm
Reinforcement Ratio	0.00659	0.00570	0.00864

The same procedure was done for the minor beams and are represented below.

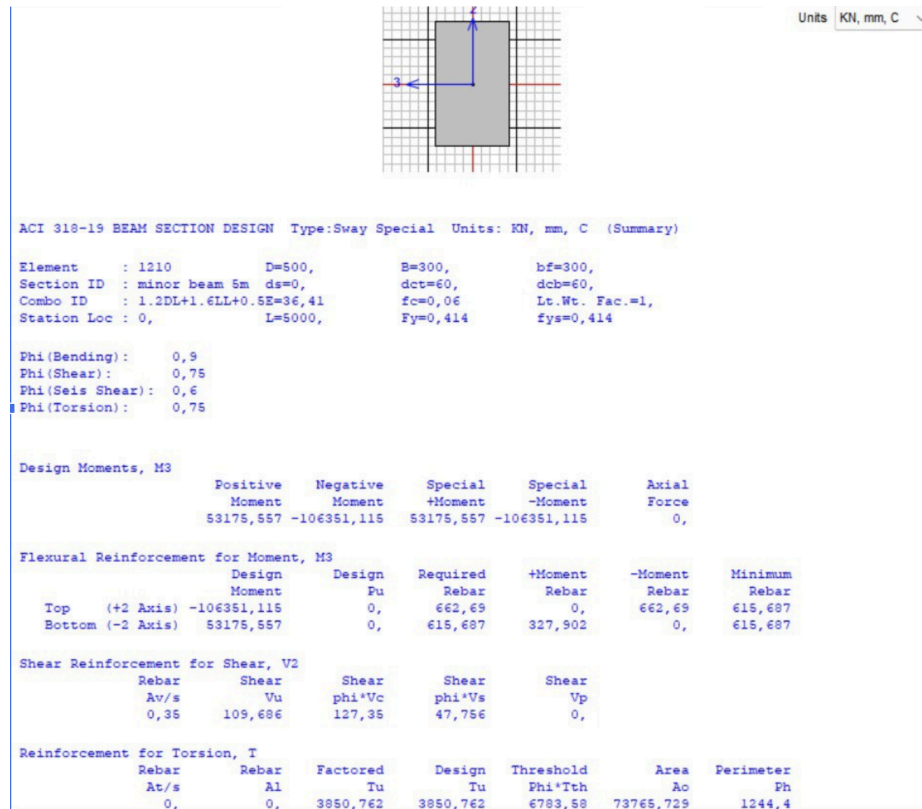


Figure 3.7.1.1.4 Minor Beam Reinforcement Design for Negative Moment

5m Minor beam

Longitudinal Reinforcement design:

$$M_u = 106 \text{ kN} - m$$

$$b = 300 \text{ mm}, h = 500 \text{ mm}, d = 440 \text{ mm}$$

$$f'_c = 40 \text{ MPa}, f_y = 413 \text{ MPa}$$

$$R_n = \frac{M_u}{\phi b d^2} = \frac{106}{0.9 \cdot 0.3 \cdot 0.54^2} = 2034.57 \text{ kPa}$$

$$\rho = 0.85 \frac{f'_c}{f_y} \left(1 - \sqrt{1 - \frac{2R_n}{0.85 f'_c}}\right) = 0.005$$

$$A_s = \rho b d = 663.78 \text{ mm}^2$$

$$a = \frac{A_s f_y}{0.85 f'_c b} = 17.92 \text{ mm}$$

$$c = \frac{a}{\beta_1} = 29.14 \text{ mm}$$

$$\epsilon_t = \frac{0.003(d-c)}{c} = 0.042 > 0.005 \rightarrow \text{tension - controlled and ductile}$$

$$A_{s,min} = \max\left(\frac{3\sqrt{f'_c}}{f_y}bd, \frac{200bd}{f_y}\right) = 512.09 \text{ mm}^2$$

$$A_{s,SAP2000} = 662.6 \text{ mm}^2$$

$$Error = \frac{663.78-662.6}{663.78} * 100\% = 1.7\%$$

Selected Bar: 2#7, b_min= 200.152 mm

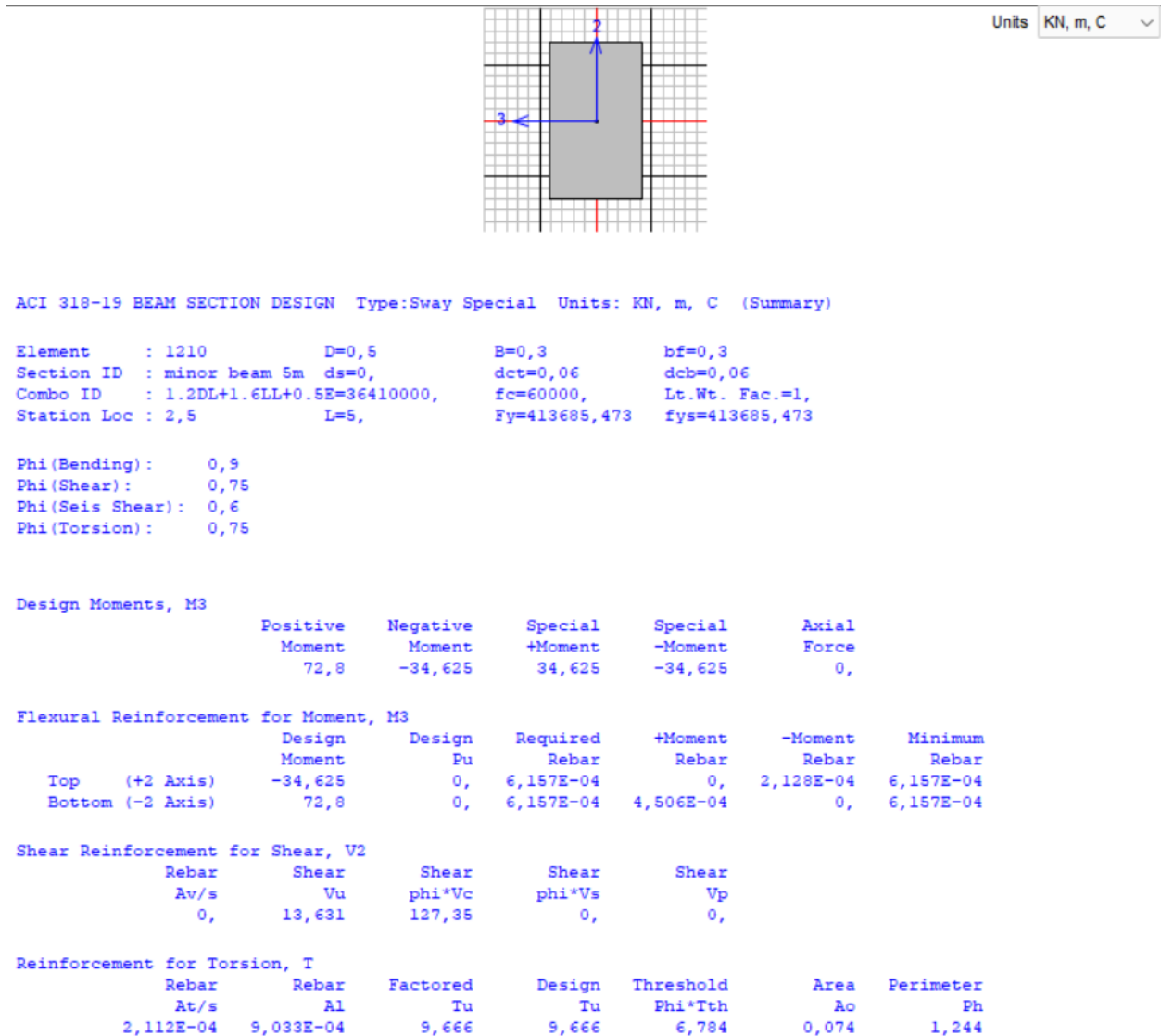


Figure 3.7.1.1.5 Minor Beam Reinforcement Design for Positive Moment

$$Mu = 72.8 \text{ kN} - \text{m}$$

$$b = 300 \text{ mm}, h = 500 \text{ mm}, d = 440 \text{ mm}$$

$$f'_c = 40 \text{ MPa}, f_y = 413 \text{ MPa}$$

$$R_n = \frac{M_u}{\phi b d^2} = \frac{72.8}{0.9 * 0.3 * 0.54^2} = 1328.25 \text{ kPa}$$

$$\rho = 0.85 \frac{f'_c}{f_y} \left(1 - \sqrt{1 - \frac{2R_n}{0.85 f'_c}}\right) = 0.00326$$

$$A_s = \rho b d = 439.98 \text{ mm}^2$$

$$a = \frac{A_s * f_y}{0.85 * f'_c * b} = 11.88 \text{ mm}$$

$$c = \frac{a}{\beta_1} = 19.31 \text{ mm}$$

$$\epsilon_t = \frac{0.003(d-c)}{c} = 0.066 > 0.005 \rightarrow \text{tension - controlled and ductile}$$

$$A_{s,min} = \max\left(\frac{3\sqrt{f'_c}}{f_y} b d, \frac{200 b d}{f_y}\right) = 512.09 \text{ mm}^2$$

$$A_{s,SAP2000} = 662.6 \text{ mm}^2$$

Selected Bar: 2#7, b_min= 200.152 mm

Table 3.7.1.1.2 Detailed Minor Beam Reinforcement Design for Floor 4

5m Minor Beam for Floor 4 for Exterior frame			
Variables	SAP2000		
	End	Mid	End
SAP 2000 Tension Area, mm2	662.6	662.6	678.2
Area, in2	1.027	1.027	1.051
SAP 2000 Compression Area, mm2	615.7	615.7	615.7
Area, in2	0.954	0.954	0.954
Tension Bars	2#7	2#7	2#7
Compression Bars	2#7	2#7	2#7
Shear Reinforcement	#3 at 200 mm	#3 at 250 mm	#3 at 200 mm
Reinforcement Ratio	0.01032	0.01032	0.01032

All other structural detailings for beams are shown in the Appendix section and it was done by the same procedure as described before.

3.7.1.2. Shear Reinforcement Analysis

Shear Reinforcement Design for Major Beam:

$$V_u = 141.379 \text{ kN} > \phi * 2\lambda\sqrt{f'_c}bd = 59.52 \text{ kN}$$

Shear reinforcement is required

$$V_s = \frac{V_u - \phi V_c}{\phi} = 109.141 \text{ kN} < V_{c1} \text{ and } V_{c2}$$

Torsional Reinforcement Design:

$$T_u = 3.87 \text{ kN} - m$$

$$T_u < T_n = \phi\lambda\sqrt{f'_c}\left(\frac{A_{cp}^2}{P_{cp}}\right) = 4.31 \text{ kN} - m$$

no torsional reinforcement is needed

$$s1 = \frac{A_v f_{yt} d}{V_s} = 182.14 \text{ mm} \simeq 180 \text{ mm}$$

$$s2 = \frac{d}{2} \leq 24 \text{ in. for } V_s \leq V_{c1}$$

$$s3 = \frac{A_v f_{yt}}{50b_w} \geq \frac{A_v f_{yt}}{0.75\sqrt{f'_c}b_w}$$

$$S_{max} = \min(S_2; S_3) = 270 \text{ mm}$$

The shear reinforcement to be used is #3 at spacing 180 mm center-to-center.

Shear Reinforcement Design for Minor Beam:

$$V_u = 109.686 \text{ kN} > \phi * 2\lambda\sqrt{f'_c}bd = 48.5 \text{ kN}$$

Shear reinforcement is required

$$V_s = \frac{V_u - \phi V_c}{\phi} = 84.58 \text{ kN} < V_{c1} \text{ and } V_{c2}$$

Torsional Reinforcement Design:

$$T_u = 3.85 \text{ kN} - m$$

$$T_n = \phi\lambda\sqrt{f'_c}\left(\frac{A_{cp}^2}{P_{cp}}\right) = 4.17 \text{ kN} - m$$

no torsional reinforcement is needed

$$s1 = \frac{A_v f_{yt} d}{V_s} = 207.14 \text{ mm} \approx 200 \text{ mm}$$

$$s2 = \frac{d}{2} \leq 24 \text{ in. for } V_s \leq V_{c1}$$

$$s3 = \frac{A_v f_{yt}}{50 b_w} \geq \frac{A_v f_{yt}}{0.75 \sqrt{f'_c} b_w}$$

$$S_{max} = \min(S_2; S_3) = 304 \text{ mm}$$

The shear reinforcement to be used is #3 at spacing 200 mm center-to-center.

The longitudinal rebars were calculated with respect to the flexural moment envelope to determine the cutting length. The theoretical cutoff point in each case of bar size was found at the point where the moment capacity of the bars meets the applied moment diagram. To allow proper development and anchorage above the point where the bars are no longer needed by analysis, however, the cutoff length was continued by a distance of 3db (three times the bar diameter) beyond the point of intersection. This change is in line with codes (e.g. ACI 318 or similar) and provides sufficient embedment length to the terminated bars.

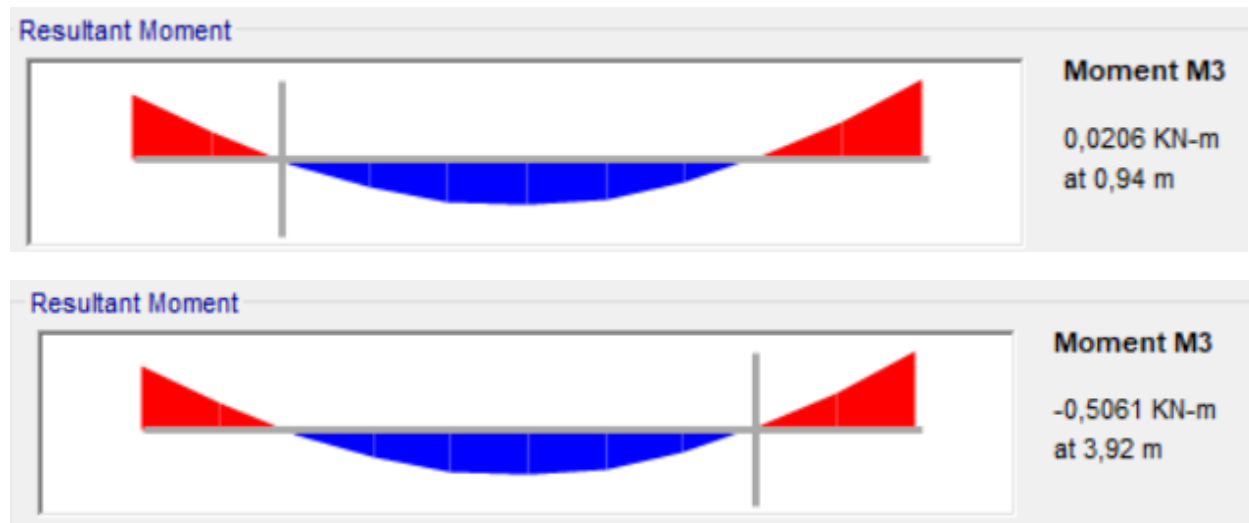


Figure 3.7.1.1.6. Determination of theoretical cutoff points from moment envelop

$$\text{cut - off distance for right tension bar} = 940 + 3 * 19.05 = 997.15 \text{ mm}$$

$$\text{cut - off distance for right compression bar} = 940 + 3 * 15.875 = 987.625 \text{ mm}$$

$$\text{cut - off distance for right tension bar} = 1080 + 3 * 19.05 = 1137.15 \text{ mm}$$

$$\text{cut - off distance for right compression bar} = 940 + 3 * 15.875 = 1127.625 \text{ mm}$$

3.7.2. Prestressed Beam Design

3.7.2.1. Geometry Definition and Section Properties

The geometry and structural system is the first step of the ten steps designed to create a beam frame with a post-tensioned frame, which identifies the physical parameters that must be used in all the following calculations and checks. The members of the structure are first categorized and in this case a beam framing to columns of equal span, 20 m long are taken into consideration with two spans. Instead of considering the whole structure simultaneously, a representative portion is cut out- a beam stem and a flange width that is attached to it and it is then analyzed in isolation by application of the principles of plane frame analysis. An important point in the definition of the geometry is to find out the effective width of the flange that is acting with the beam stem and which varies depending on the type of force being opposed: in case of axial effects, the full tributary width is taken since the pre-compression caused by post-tensioning diffuses throughout the entire slab; in case of bending effects, a smaller width is taken in accordance with ACI 318. According to this methodology the beam geometry was defined with the following parameters:

$$h = 1000\text{mm}; h_f = 130\text{mm}; b_w = 500\text{mm}; b_{eff} = 2580\text{mm}$$

$$\text{Cross - sectional area: } A = A_{flange} + A_{web} = 335,400 + 500,000 = 835,400 \text{ mm}^2$$

$$\text{Moment of inertia: } I_{total} = 1.062 \times 10^{11} \text{ mm}^4$$

$$\text{Centroid location from top fiber: } y = 403.16 \text{ mm}$$

$$\text{Section modulus (top): } S_{top} = \frac{I}{Y_t} = 2.635 \times 10^8 \text{ mm}^3$$

$$\text{Section modulus (bottom): } S_{bot} = \frac{I}{Y_b} = 1.461 \times 10^8 \text{ mm}^3$$

$$\text{Radius of gyration: } r = \sqrt{\frac{I}{A}} = 356.58 \text{ mm}$$

3.7.2.2. Material Properties

The material properties of concrete, prestressing steel, and non-prestressed reinforcement are established during this step as the material values determine the response of the structure to loads

and levels of stresses that it can safely maintain. In the case of the concrete, compressive strength under service conditions and the initial strength at transfer was determined taking into consideration that the concrete had not attained full matured strength at the time prestressing was done. Unit weight and modulus of elasticity was defined and a coefficient of creep was supposed to provide the long term deformations and loss of prestress. In the case of the prestressing steel, unbonded twelve wire strands were chosen. Final tensile strength and modulus of elasticity and post-losses stress were calculated and friction coefficients and anchor set were provided according to the code requirements. In the case of the non-prestressed reinforcement, the yield strength was given. The summary of these material properties is presented in the following table.

Table 3.7.2.2.1 Material Properties for Concrete, Prestressing Steel, and Non-Prestressed Reinforcement

Material	Property	Symbol	Value	Unit
Concrete	Compressive strength	f_c'	40	MPa
	Strength at transfer	f_{ci}'	28	MPa
	Unit weight	γ	25	kN/m ³
	Modulus of elasticity	E_c	29,725	MPa
	Creep coefficient	ϕ	2	—
	Material factor	γ_c	1	—
Prestressing Steel	Number of strands	n	12	—
	Strand diameter	d	15.2	mm
	Strand area	A_{ps}	140	mm ²
	Ultimate strength	f_{pu}	1860	MPa
	Modulus of elasticity	E_{ps}	200,000	MPa
	Stressing force	—	80	%
	Stress after losses	f_{pe}	1302	MPa
	Angular friction coefficient	μ	0.07	—
	Wobble friction coefficient	K	0.003	/m
	Anchor set	Δa	6	mm

Non-Prestressed Reinforcement	Yield strength	f_y	414	MPa
-------------------------------	----------------	-------	-----	-----

3.7.2.3. Load Definition

The forces on the structure are needed to compute the demand values; the moments and reactions that the beam should be capable of opposing. The dead loads, which included the permanent weight of the structure and fixed installations were divided into two, namely; self-weight, calculated considering cross-sectional geometry of the beam with the corresponding slab multiplied by unit weight of concrete and superimposed dead load, which included other permanent items such as mechanical installations. Live loads which are temporary or moving weights that the structure must have were identified as per the building code stipulations and the reduction of live loads applied according to the tributary area of the beam. The dead and the live loads were calculated and their values are available in the sections above. These load definitions define the minimum requirements to be used in further serviceability and strength checks.

3.7.2.4. Design Parameters and Allowable Stresses

This step defines the design parameters, and the permissible stresses that the beam has to work within, which sets the boundaries that the beam has to operate within, to ensure functionality and safety throughout its service life. The tensile stress to be used at the extreme fibers was decided according to ACI 318 where member categories were either uncracked, transition, or cracked depending on the calculated tensile stress as compared to f'_c . In this project, the tensile stress was set at 4.74 MPa, within the transition region that suggests partial cracking to occur and this will require the consideration of such cracking when calculating deflection.

$$\sigma \leq 0.62\sqrt{f'_c} = 3.92 \text{ MPa} - \text{Uncracked}$$

$$\sigma \sim 0.75\sqrt{f'_c} = 4.74 \text{ MPa} - \text{Target (Transition)}$$

$$1\sqrt{f'_c} < \sigma = 6.32 \text{ MPa} - \text{Cracked}$$

Sustained compressive loads were allowed to be applied with separate allowable compressive stresses imposed to provide control in creep deflection, whereas total compressive loads were allowed to be applied with separate allowable compressive stresses imposed to allow infrequent peak loading operations. The initial condition during jacking was also limited to make sure that the extreme tension and compression due to the fibers and compression against the concrete

would not be exceeded to a predetermined code limit as the initial strength of the concrete is known. Such design parameters are the ruling principles of the future analysis; in case stresses are surpassed, there is a need to reform the geometry or material assumptions. The permissible stresses can be summarized as shown in the table below.

Table 3.7.2.4.1 Allowable Stresses

Condition	Location and Type	Allowable Stress (MPa)
Service	Bottom Fiber, Tension	6.32
Service	Top Fiber, Tension	4.74
Service	Compression (Total)	-24
Service	Compression (Sustained)	-18
Initial	Tension	1.32
Initial	Compression	16.8

3.7.2.5. Calculation of Demand Values

In this step, the calculation of actions which can also be said to be determining the demand values is done using the previously defined geometry and loads. The model of structural analysis was created based on SAP2000 in which the ends of bottom supports were put on rollers to adjust the shortening of the member horizontally that takes place when the post-tensioning is utilized. With the help of this model, moments and reactions were developed in each span at significant points that included the face of the supports and midspan. The tabulated values of the calculated demand are as shown in the following table.

Table 3.7.2.5.1 Actions Due to Dead and Live Loads

	SPAN 1			SPAN 2		
Moments	Left	Mid	Right	Left	Mid	Right
MD (kN-m)	-1508.54	801.94	-1658.54	-1658.54	801.94	-1508.54
ML (kN-m)	-779.65	417.89	-819.65	-819.65	417.89	-779.65

MD + 0.3ML	-1742.44	927.31	-1904.44	-1904.44	927.31	-1742.44
MD+ML	-2288.19	1219.83	-2478.19	-2478.19	1219.83	-2288.19

3.7.2.6. Post Tensioning

The prestressing strain F_p is calculated to be equal to an integer number of prestressing strands.

The force per strand is obtained as:

$$F \text{ per strand} = f_{se} \times A = 1302 \times 140 \times 10^{-6} = 182.28 \text{ kN/strand}$$

The amount of precompression needed is 1150 kN, calculated as:

$$1 \text{ N/mm}^2 \times 11.5 \times 10^5 \text{ mm}^2 / 1000 = 1150 \text{ kN}$$

To determine the number of strands, we divide the amount of precompression needed by the force on each strand:

$$N \text{ of strands} = 1150 / 182.28 = 6.3 \Rightarrow 7 \text{ strands selected}$$

This gives a total of 7 strands to form an initial design giving a force $F_p = 7 \times 182.28 = 1275.96$ kN.

The tendon profiles are in a simplistic parabolic form. The minimum point of all the tendons in the three spans is fixed to produce a homogeneous force upwards. In spans 1 and 2, the tendon curve takes the shape that is similar to that in the figure. The real profile as built will be less jagged, more like a reversed parabola, rather than a steep parabolic curve.

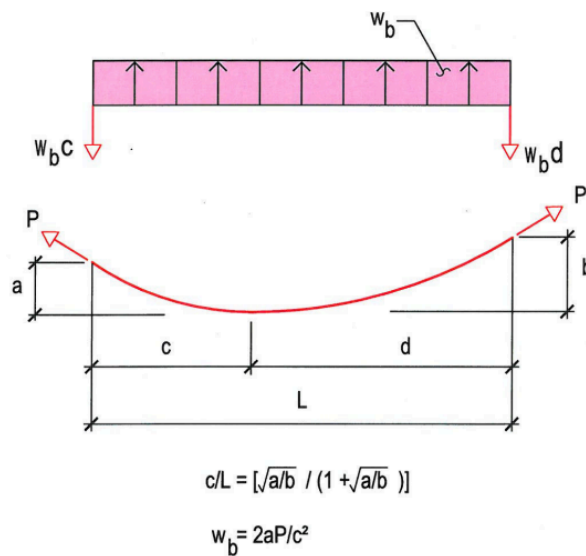


Figure 3.7.2.6.1. Geometry and actions of a parabolic tendon

The tendon geometry and forces are as follows:

- Distance between the point of the highest tendon left and the lowest: $a=Yb-70=860\text{mm}$ ($e=930\text{mm}$).
- Distance $b=930-70=860\text{mm} = a$.
- $L=20\text{m}$ is the total span length.
- The distance c calculated is: $c = \left(\frac{\sqrt{1}}{1+\sqrt{1}}\right) \times 20000 = 10000\text{ mm}$

Upward force due to post-tensioning: $w_b = 1275.96 \times 2 \times 0.860/10^2 = 21.95\text{ kN/m}$

% balancing = $21.95/100.95 = 22\% < 80\%$ (target)

Adj. N of strands = $\frac{80\%}{22\%} \times 7 = 26.4 \Rightarrow \text{choose 27 strands}$

$$F_p = 27 \times 182.28 = 4921.56\text{ kN}$$

$$w_b = 4921.56 \times 2 \times 0.860/10^2 = 84.65\text{ kN/m (balances 84\% of DL)}$$

Since the loading is uniform on the two spans and the supports are fixed, the distance b is equalized to a resulting in a simple parabolic tendon profile. Also the tendon profile obtained through the software indicates that the lowest point of the tendon exists according to the geometry $a=860\text{mm}$ and $b=860\text{mm}$. This is similar to the manually designed profile with all tendons being symmetric and similar.

The tendon profile is calculated by the program and this makes sure that the tendon point is positioned in the right position to produce the necessary upward force. The final tendon design is a composite product of the manual design and the software-generated geometry, which produces an effective balance of the dead load (DL) with a smooth and continuous curve shape of parabolism.

3.7.2.7. Serviceability Check

SAP2000 was used to get the moments required to evaluate stress and deflection. The following load checks (using ACI and IBC standards) are based on the following assumptions:

Total Load Condition: $1\text{DL} + 1\text{LL} + 1\text{PT}$.

Sustainable Load State: $1\text{DL} + 0.3\text{LL} + 1\text{PT}$.

$$\text{Effective moment factor } I_e = (1 - 0.3(4.92 - 0.5\sqrt{f'_c})/0.5\sqrt{f'_c})I_g$$

Stress:

$$\sigma_{total-load} = (M_{DL} + M_{LL} + M_{PT})/S + F_p/A$$

$$\sigma_{sust-load} = (M_{DL} + 0.3M_{LL} + M_{PT})/S + F_p/A$$

Table 3.7.2.7.1. Stress check at critical points

Load Combination	ACI 318	Point A	Point B	Point C	Point D	Point E	Point F
DL+0.3LL+PT	ft, MPa	-4.68	-6.71	-4.08	-4.08	-6.83	-4.46
	fb, MPa	-8.08	-4.41	-9.15	-9.15	-4.21	-8.48
	Ft, MPa	4.74	-18.00	4.74	4.74	-18.00	4.74
	Fb, MPa	-18.00	4.74	-18.00	-18.00	4.74	-18.00
		OK	OK	OK	OK	OK	OK
DL+LL+PT	ft, MPa	-2.61	-7.82	-1.90	-1.90	-7.94	-2.39
	fb, MPa	-11.81	-2.41	-13.08	-13.08	-2.20	-12.21
	Ft, MPa	4.74	-18.00	4.74	4.74	-24.00	4.74
	Fb, MPa	-24.00	6.32	-24.00	-24.00	6.32	-24.00
		OK	OK	OK	OK	OK	OK

As tensile stresses lie within the range of transition conditions, a stress check has passed, and additional checks can be performed by calculating deflections in SAP2000. Long term deflections (prestress, dead load and fraction of live load):

Table 3.7.2.7.2. Deflection Check Results for Prestressed Beam

Deflection	DL, mm	PT, mm	DL+PT, mm	LL, mm	L/360	Long Term	L/240
Check	54.66	-41.39	16.27	36.19	55.56	81.38	83.33

The design passes the deflection checks.

3.7.2.8. Design for Flexure, Shear, and Torsion

Because prestress force is already presented as an explicit load case and is part of all of the load combinations, the design moments that the program reports are the residual (net) moments that

remain after considering the opposing effects of the prestressing - secondary moments that tendons cause. Thus, the reinforcement area, which is necessary in SAP2000, can be shown as passive reinforcement only since the contribution of the tendon has already been taken into account.

Table 3.7.2.8.1. Final Structural Detailing for Prestressed Beam for Floor 1-4

Location	End	Mid	End
Top selected bars	5#11	3#9	6#11
Bottom selected bars	4#8	6#10	5#10
Shear reinforcement	#4 @ 200 mm	#4 @ 250 mm	#4 @ 200 mm

3.7.3. Column Design

3.7.3.1. Slenderness Ratio

From Sap2000,

$$P_u = 1219.4 \text{ kN}$$

$$I_{column} = 0.0066 \text{ mm}^4; I_{beam, 5m} = 0.450 \text{ mm}^4; I_{beam, 20m} = 0.0583 \text{ mm}^4$$

$$L_{column} = 6 \text{ m}, L_{beam, 5m} = 5 \text{ m}; L_{beam, 20m} = 20 \text{ m}$$

Exterior column : (500 mm × 500 mm):

$$\psi_A = \frac{\sum EI/l \text{ of column}}{\sum EI/l \text{ of beam}} = 0.0060$$

$$\psi_B = \frac{\sum EI/l \text{ of column}}{\sum EI/l \text{ of beam}} = 0.0120$$

$$K = 1.08 \rightarrow \frac{Kl_u}{r} = 33.52 > 22 \rightarrow \text{slender}$$

Corner column : (500 mm × 500 mm):

$$\psi_A = \frac{\sum EI/l \text{ of column}}{\sum EI/l \text{ of beam}} = 0.007$$

$$\psi_B = \frac{\sum EI/l \text{ of column}}{\sum EI/l \text{ of beam}} = 0.013$$

$$K = 1.08 \rightarrow \frac{Kl_u}{r} = 39.6 > 22 \rightarrow \text{slender}$$

Interior column : (500 mm × 500 mm):

$$\psi_A = \frac{\sum EI/l \text{ of column}}{\sum EI/l \text{ of beam}} = 0.012$$

$$\psi_B = \frac{\sum EI/l \text{ of column}}{\sum EI/l \text{ of beam}} = 0.025$$

$$K = 1.08 \rightarrow \frac{Kl_u}{r} = 20.4 < 22 \rightarrow \text{short}$$

To proceed with Slender Column Design, sway magnification is necessary:

$$\text{Firstly, the column buckling is calculated: } P_c = \frac{\pi^2 EI}{(kl_u)^2} = 69666.9 \text{ kN}$$

2) After that the sway magnification coefficient is determined:

$$\delta_s = \frac{1}{1 - \frac{\sum P_u}{0.75 \sum P_c}} = 1.024 < 2.5$$

3) To identify whether the further magnification is required, the following equation must be computed:

$$\frac{l_u}{r} = 31.034 < \frac{35}{\sqrt{P_u / (f_c' A_g)}} = 116.27 \rightarrow \text{No further magnification is required}$$

4) Thus, these are the amplified moments that will be used in further column design:

$$M1 = \delta_s M1 = 35.95 \text{ kN} - m$$

$$M2 = \delta_s M2 = 107.42 \text{ kN} - m$$

3.7.3.2. Longitudinal Rebars

Column Design

$$P_n = \frac{P_u}{0.65} = 1876 \text{ kN}$$

$$M_n = \frac{M_u}{0.65} = 55.31 \text{ kN}$$

$$e = \frac{M_u}{P_u} = 0.029 \text{ m}$$

$$K_n = \frac{P_n}{f'_c A_g} = 0.929$$

$$R_n = \frac{P_n e}{f'_c A_g h} = 0.047 \rightarrow \text{from interaction diagram } \rho = 0.014$$

$$A_s = 0.0022 \text{ m}^2 = 3.88 \text{ in}^2 \rightarrow \text{selected bars: 8\#8}$$

3.7.3.3. Shear Reinforcement

Shear Strength Check

$$V_{u1} = 37.87 \text{ kN}; V_{u2} = 11.74 \text{ kN}; V_c = 376.984 \text{ kN}$$

$$\phi V_c = 282.738 \text{ kN}$$

$$\frac{\phi V_c}{2} = 141.369 \text{ kN}$$

$$Vu < 0.5\phi V_c \rightarrow \text{Only minimum reinforcement is required} \rightarrow \#3 \text{ ties @200mm}$$

3.7.3.4. Biaxial Bending

$$M_{ux} = 35.95 \text{ kN} - m \rightarrow M_{nx} = 55.31 \text{ kN} - m$$

$$M_{uy} = 107.42 \text{ kN} - m \rightarrow M_{ny} = 165.26 \text{ kN} - m$$

$$\gamma_x = \gamma_y = 0.759; e_x = 0.029 \text{ m}; e_y = 0.088 \text{ m}$$

$$P_{nx} = 26912 \text{ kN}; P_{ny} = 14491 \text{ kN}; P_o = 11440 \text{ kN}; P_n = 53333 \text{ kN}$$

$$P_n > 0.1P_{n_o} \rightarrow \text{The method is feasible}$$

$$\phi P_n > P_u \rightarrow 34666.4 \text{ kN} > 1219.4 \text{ kN} \rightarrow \text{Safe against biaxial bending}$$

3.7.4. Two-way slab

In the case of the two-way slab design the same procedure as the one used in the major beam was adopted, the two-way slab was sub-divided into strips, which were further sub-divided into sections. The dimensions of the slab are 5000mm by 5000mm and slab thickness is 130mm.

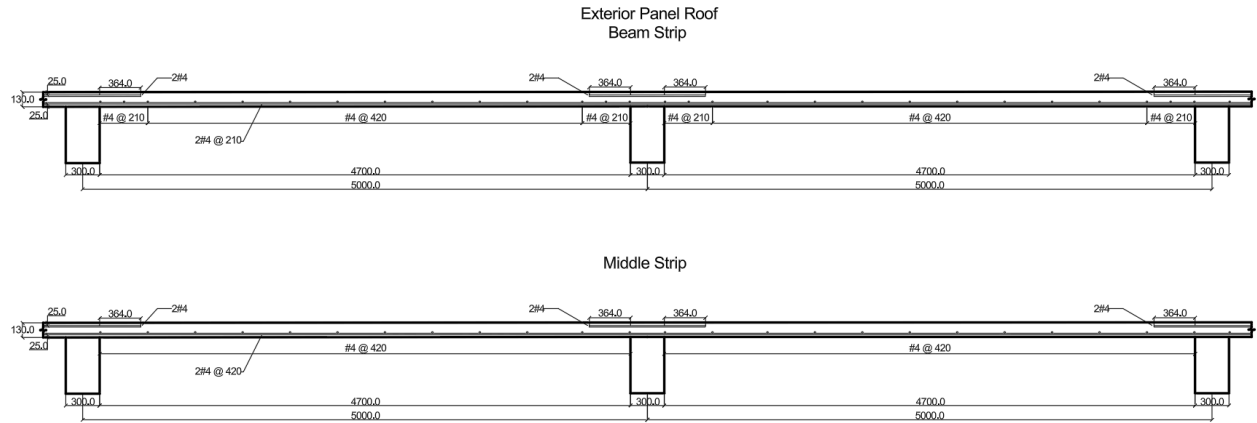


Figure 3.7.4.1.1. Two-Way Slab Design

3.7.4.1. Flexural Analysis

$$\bar{y} = \frac{201500 \cdot 65 + 150000 \cdot 380}{201500 + 150000} = 199.42 \text{ mm}$$

$$I_b = \frac{300 \cdot 500^3}{12} + 201500(199.42 - 65)^2 + \frac{1550 \cdot 130^3}{12} + 150000(380 - 65)^2 = 2.33 \cdot 10^9 \text{ mm}^4$$

$$I_s = \frac{5000}{2} \cdot \frac{130^3}{12} = 4.57 \cdot 10^8 \text{ mm}^4$$

$$\alpha_{fm} = \alpha = \frac{EI_b}{EI_s} = 5.96$$

Minimum height can be determined through the following:

$$h_{min} = \frac{l_n(0.8 + \frac{f_y}{1400})}{36 + 9\beta} = 4700 \cdot (0.8 + 60000/200000)/(36 + 9 \cdot 1) = 114.8 \text{ mm}$$

Factored load is calculated here:

$$q_u = 1.2q_d + 1.6q_l = 1.2 \cdot 6.731 + 1.6 \cdot 4.79 = 15.74 \text{ kN/m}^2$$

Total static moment:

$$M_o = \frac{q_u l_2 l_0^2}{8} = \frac{15.74 \cdot 5 \cdot 4.35^2}{8} = 186.16 \text{ kN} \cdot \text{m}$$

$$M(-) = 0.65 \cdot M_o = 0.65 \cdot 186.16 = 121.01 \text{ kN} \cdot \text{m}$$

$$M(+) = 0.35 \cdot M_o = 0.35 \cdot 186.16 = 65.16 \text{ kN} \cdot \text{m}$$

$$\alpha_{f1} = \alpha_s = 5.96 > 1$$

$$\frac{l_2}{l_1} = 1$$

$$\bar{y} = \frac{201500 \cdot 65 + 0 \cdot 380}{201500 + 0} = 65 \text{ mm}$$

$$I_b = 2.13 \cdot 10^9 \text{ mm}^4$$

$$I_s = 3.64 \cdot 10^8 \text{ mm}^4$$

$$\alpha_{fm} = \alpha = \frac{EI_b}{EI_s} = 5.87$$

As α_{fm} is greater than 2.0, minimum height can be calculated as follows:

$$h_{min} = \frac{l_n(0.8 + \frac{f_y}{1400})}{36 + 9\beta} = 4700 \cdot (0.8 + 60000/200000)/(36 + 9 \cdot 1) = 114.8 \text{ mm}$$

As the thickness is satisfied, factored loads can be calculated:

$$q_u = 1.2q_d + 1.6q_l = 1.2 \cdot 6.731 + 1.6 \cdot 4.79 = 15.74 \text{ kN/m}^2$$

Table 3.7.4.1. Two-Way Slab Design Procedure for Exterior panel

	Beam Strip			Middle Strip		
	Negative	Positive	Negative	Negative	Positive	Negative
%	96.35	75.00	75.00	3.65	25.00	25.00
new Mu (kN-m)	51.02	21.38	39.71	1.93	7.13	13.24
Mu slab (kN-m)	7.65	3.21	5.96	0.29	1.07	1.99
b	0.63	1.25	1.25	1.25	1.25	1.25
Rn (kPa)	1535.30	321.77	597.56	29.10	107.26	199.19
Ro	0.0037	0.0008	0.0014	0.0001	0.0003	0.0005
As (m ²)	0.00022	0.00009	0.00017	0.00001	0.00003	0.00006
As (in ²)	0.34124	0.14040	0.26182	0.01264	0.04665	0.08675
Asmin (in ²)	0.227	0.227	0.227	0.227	0.227	0.227
As final (in ²)	0.341	0.227	0.262	0.227	0.227	0.227
Bar Selected	2#4	2#4	2#4	2#4	2#4	2#4
Spacing (m)	0.21	0.42	0.42	0.42	0.42	0.42

Table 3.7.4.2. Two-Way Slab Design Procedure for Interior panel

	Beam Strip			Middle Strip		
	Negative	Positive	Negative	Negative	Positive	Negative
%	75.00	75.00	75.00	25.00	25.00	25.00
new Mu (kN-m)	39.71	21.38	39.71	13.24	7.13	13.24
Mu slab (kN-m)	5.96	3.21	5.96	1.99	1.07	1.99
b	1.25	1.25	1.25	1.25	1.25	1.25
Rn (kPa)	597.56	321.77	597.56	199.19	107.26	199.19
Ro	0.0014	0.0008	0.0014	0.0005	0.00026	0.00048
As (m ²)	0.00017	0.00009	0.00017	0.00006	0.00003	0.00006
As (in ²)	0.26182	0.14040	0.26182	0.08675	0.04665	0.08675
Asmin (in ²)	0.227	0.227	0.227	0.227	0.227	0.227
As final (in ²)	0.227	0.227	0.227	0.227	0.227	0.227
Bar Selected	2#4	2#4	2#4	2#4	2#4	2#4
Spacing (m)	0.42	0.42	0.42	0.42	0.42	0.42

3.7.4.2. Shear analysis

$$d_{average} = 5.12 - 3/4 - 4/8 = 3.87in = 98.3mm$$

$$V_u = q_u(l_1l_2 - (b + d_{avg})(h + d_{avg})) =$$

$$= 15.74(25 - (0.75 + 98.3/1000)(0.13 + 98.3/1000)) = 390.45kN$$

$$\lambda = 1.0$$

$$\phi V_{c1} = \phi(0.17\lambda\sqrt{f'_c} + \frac{N_u}{6A_g})b_wd = 274.38kN$$

$$\phi V_{c2} = \phi(0.66\lambda\rho^{(1/3)}\sqrt{f'_c} + \frac{N_u}{6A_g})b_wd = 23.99kN$$

3.7.5. Joint Selection

The following procedure below is in accordance with ACI 318-19. Joint design can be presented as follows:

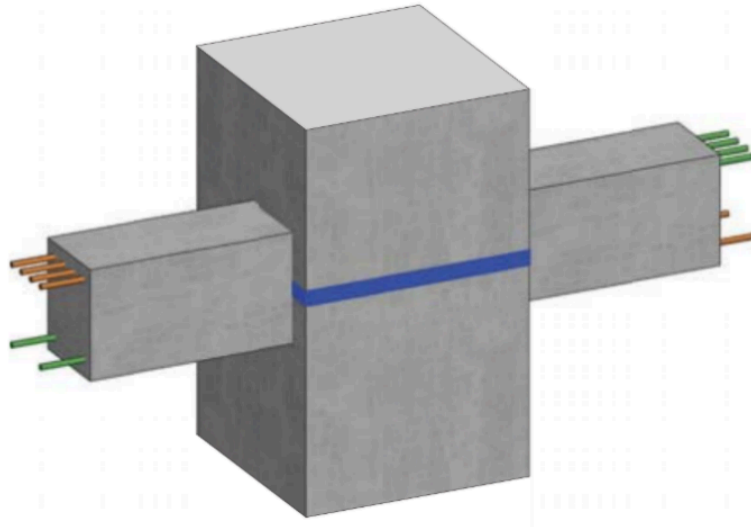


Figure 3.7.5.1 Joint design between the column and beam (ACI, 2019)

An example was given of the interior beam and column to demonstrate the following process. The shear of these two structural elements was calculated using the following equation:

$$V_{u,joint} = T_{pr1} + C_{pr2} - V_{col}$$

The reinforcement details to calculate the joint shear were identified using SAP2000 to represent interior beam-column joint in the figure below:

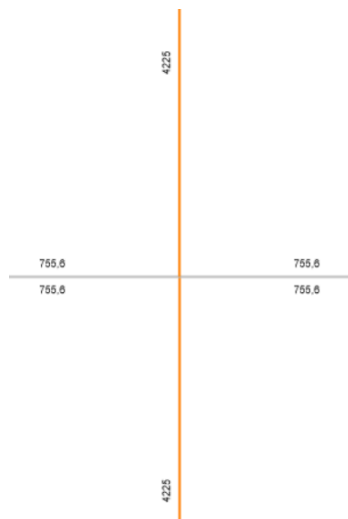


Figure 3.7.5.2. Reinforcement details at the beam-column joint.

The parameters in the equation above have been calculated as follows:

$$T_{pr1} = \alpha A_s f_y = 1.25 * 755.6 * 0.517 = 488.3kN$$

$$C_{pr1} = \alpha A_s f_y = 1.25 * 755.6 * 0.517 = 488.3kN$$

SAP 2000's results are the following:

$$V_{col} = 124kN$$

The shear demand on joint:

$$V_{u, joint} = 488.3 + 488.3 - 124 = 852.6kN$$

In order to find the shear strength of the joint, the effective width, b_j , was first calculated:

$$b_j = \frac{b_b + b_{col}}{2} = \frac{300 + 650}{2} = 475mm < b_b + h_{col} = 300 + 650 = 950mm$$

$$b_j = 475mm < b_b + 2x = 300 + 650 = 950mm$$

The shear capacity is calculated using the following formula:

$$V_n = \gamma \sqrt{f'_c} A_j$$

Using this formula we find 1.7 as γ of confined interior joints and $A_j = b_j * h_{col}$. Therefore:

$$V_n = \gamma \sqrt{f'_c} A_j = 1.7 \sqrt{40} * 475 * 650 = 3319.6kN$$

The nominal shear strength should meet $\phi V_n \geq V_u$. The strength reduction factor ϕ is considered to be 0.85 in order to represent the strain hardening of the reinforcement.

$$0.85 * 3319.6 = 2821.66 kN \geq V_u = 852.6kN$$

As a result, it was confirmed that the joint is safe under shear forces. The same method was used to check the joint's shear capacity in other directions, and it was also found to be safe. In addition, the spacing requirements were calculated based on ACI 15.4.2, using the following formula:

$$A_v = \max\left(\frac{0.062 \sqrt{f'_c} b_c s}{f_{yt}}, \frac{0.35 b_c s}{f_{yt}}\right)$$

$$s = \min\left(\frac{A_v f_{yt}}{0.062 \sqrt{f'_c} b_c}, \frac{A_v f_{yt}}{0.35 b_c}\right)$$

The minimum spacing with 2#4 ties:

$$s = \min\left(\frac{2*129*517.1}{0.062\sqrt{40*500}}, \frac{2*129*517.1}{0.35*500}\right) = \min(680.45, 762.4) = 680.45$$

Therefore, two #4 ties spaced at 250 mm will be used to satisfy the code requirements — a spacing tighter than the minimum requirement due to seismic zone considerations.

3.7.6. Bar Selection

The summary of the chosen reinforcement bars is presented in Table 3.62 below. As illustrated, no single major beam is presented in the table, with every beam in every floor having a different reinforcement design. The description of each particular beam is presented in the Appendix.

Table 3.7.6.1. Final selected bars.

Roof	Exterior Panel					
	Beam Strip			Middle Strip		
	(-ve)	(+ve)	(-ve)	(-ve)	(+ve)	(-ve)
As final (in ²)	0.341	0.227	0.262	0.227	0.227	0.227
Bar Selected	2#4	2#4	2#4	2#4	2#4	2#4
Spacing (m)	0.21	0.42	0.42	0.42	0.42	0.42
Roof	Interior Panel					
	Beam Strip			Middle Strip		
	(-ve)	(+ve)	(-ve)	(-ve)	(+ve)	(-ve)
As final (in ²)	0.227	0.227	0.227	0.227	0.227	0.227
Bar Selected	2#4	2#4	2#4	2#4	2#4	2#4
Spacing (m)	0.42	0.42	0.42	0.42	0.42	0.42
Floor 1-4	Exterior Panel					
	Beam Strip			Middle Strip		
	(-ve)	(+ve)	(-ve)	(-ve)	(+ve)	(-ve)
As final (in ²)	0.765	0.307	0.576	0.227	0.227	0.227
Bar Selected	3#5	3#3	3#4	3#3	3#3	3#3

Spacing (m)	0.21	0.42	0.42	0.42	0.42	0.42
Floor 1-4	Interior Panel					
	Beam Strip			Middle Strip		
	(-ve)	(+ve)	(-ve)	(-ve)	(+ve)	(-ve)
As final (in ²)	0.227	0.227	0.227	0.227	0.227	0.227
Bar Selected	3#3	3#3	3#3	3#3	3#3	3#3
Spacing (m)	0.42	0.42	0.42	0.42	0.42	0.42
Column types			Reinforcem ent	Reinforcem ent Ratio	Ties	
Column Floor 4 (500 x 500 mm)			8#8	0.0163	#4 @ 200 mm at hinge zone #4 @ 250 mm at mid height	
Column Floor 3 (650 x 650 mm)			8#9	0.0122	#4 @ 200 mm at hinge zone #4 @ 250 mm at mid height	
Column Floor 2 (650 x 650 mm)			8#11	0.0191	#4 @ 200 mm at hinge zone #4 @ 250 mm at mid height	
Column Floor 1 (650 x 650 mm)			10#10	0.0194	#4 @ 200 mm at hinge zone #4 @ 250 mm at mid height	
Column Basement (650 x 650 mm)			12#11	0.0286	#4 @ 200 mm at hinge zone #4 @ 250 mm at mid height	

3.7.7. Development Length

Calculations of the length of development were done with ACI 318-19. The reinforcement bars that are selected are the number 6 and 5 whose diameter is 0.75 inch and 0.625 inches respectively. The tension reinforcement was calculated using the following equation:

$$\frac{l_d}{d_b} = \begin{cases} \frac{f_y \psi_t \psi_e \psi_g}{25 \lambda \sqrt{f'_c}} & \text{(no. 6 and smaller bar)} \\ \frac{f_y \psi_t \psi_e \psi_g}{20 \lambda \sqrt{f'_c}} & \text{(no. 7 and larger bars)} \\ \frac{3 f_y \psi_t \psi_e \psi_g}{50 \lambda \sqrt{f'_c}} & \text{(no. 6 and smaller bars)} \\ \frac{3 f_y \psi_t \psi_e \psi_g}{40 \lambda \sqrt{f'_c}} & \text{(no. 7 and larger bars)} \end{cases}$$

Figure 3.7.7.1. Utilized formulas for development Length Calculations

$$\frac{l_d}{d_b} = \frac{f_y \psi_t \psi_e \psi_g}{25 \lambda \sqrt{f'_c}} \rightarrow l_d = 688.6 > 500 \text{ mm (column width)}$$

As shown in the equation above, a hooked bar is needed in order to accommodate the entire length of the longitudinal reinforcement, which cannot fit into the largest cross-section of the column. The dimensions of the hooked rebars should be determined using the following equation:

$$l_{dh} = \left(\frac{f_y \psi_t \psi_e \psi_g}{55 \lambda \sqrt{f'_c}} \right) d_b^{1.5} \text{ (modification factor)}$$

$$l_{dh} = 351.38 \text{ mm} > 8d_b = 152.4 \text{ mm}$$

Thus the length l_{dh} of the bars in tension developed was 478.26 mm and the 90 degree hook parameters were obtained as follows:

$$r = D/2 = 3 \text{ in or } 76.2 \text{ mm}$$

$$\text{Hook distance} = 12d_b = 9 \text{ in or } 228.6 \text{ mm}$$

Further, stirrups will be installed in the development length at the given distance $\leq 3d_b = 57.15 \text{ mm}$. In the case of the compression bars the development length was

determined by the formula:

$$l_{dc} = \left(\frac{f_y \psi_r}{50 \lambda \sqrt{f'_c}} \right) d_b \geq 0.0003 f_y \psi_r d_b$$

$$l_{dc} = 285.75 \text{ mm} < 500 \text{ mm}$$

Consequently, we can conclude that the development length requirements of all the ACI 318-19 requirements are met in full.

3.7.8. Lap splices

Each member was calculated in the lap splices. The lap splice value of tension-controlled beams is given by:

$$l_{st} = l_{dh} = 351.38 \text{ mm}$$

However, at the joints due to seismicity it is not allowed to place lap splices there. Therefore, lap splices will be placed only in members parts that are in compression, which is calculated according to the following equation:

$$l_{sc} = 0.0005f_y d_b = 30 \text{ in} = 762 \text{ mm}.$$

To calculate the lap splice value for columns, the following Table 3.63 is created to demonstrate how the lap splices differ on each floor.

Table 3.7.8.1. Column Lap Splices

Component	Lap splice, mm
Column Floor 4 (500 x 500 mm)	762.00
Column Floor 3 (650 x 650 mm)	859.54
Column Floor 2 (650 x 650 mm)	1074.42
Column Floor 1 (650 x 650 mm)	967.74
Column Basement (650 x 650 mm)	1074.42

3.8. Special seismic provisions for reinforcement detailing

3.8.1. Beams

Since the building is located in a high seismicity area, special seismic detailing was introduced. The clear span as well as the width-depth ratio is according to the requirements stated in ACI 318-19 Section 18.6. Also, two or more continuous longitudinal bars were provided at the top and bottom of the beams, as per Section 18.6.3.1. The positive moment capacity at the face of the

joint was to be greater than half the negative moment capacity. Hoops were installed to secure sufficient confinement, the length of which is twice the length of the member depth at the support face with a minimal spacing of either $d/4$, $6d_b$, or 150 mm. The initial hoop was set 50 mm apart from the support face.

3.8.2. Columns

The column's seismic detailing was carried out according to the requirements of Section 18.7 of ACI 318-19. The reinforcement zone of the columns was to be 1 to 6 percent of the cross-sectional area. On each side of the joint, a transverse reinforcement was placed over a distance, l_o , wherein l_o was considered as a sixth of the clear span of the column, making the $l_o=400\text{mm}$.

3.8.3. Joints

The beam's reinforcement was continued in the far face of the columns at the joints. The depth h of the columns was to be larger (more than 20 times) the diameter of the bars ($20d_b$) in cases where the longitudinal bars extended into the columns. The lengths of the reinforcement bars were developed properly to meet the necessary requirements:

$$l_{dh} \geq \begin{cases} \frac{f_y d_b}{65 \lambda \sqrt{f'_c}} \\ 8d_b \\ 6 \text{ in.} \end{cases}$$

3.9. Structural serviceability design

3.9.1. Minimum Deflection

The ACI 318-14 provides minimum deflection requirements of structural members including beams and slabs. When these components fall short of the minimum thickness requirements, then their deflections should be calculated. In this case, the minimum thickness requirements mean that the deflections are probably tolerable.

$$h_b = L/21 \text{ for beams}$$

$$h_s \geq L_n/36 \text{ to } L_n/33 \text{ for slabs}$$

In the equation, L is the span of the main beam. In this case, the following statement is applicable:

$$h_b = \frac{5000}{21} = 238.1 \text{ mm} < 500 \text{ mm and } 600 \text{ mm (depth of minor and major beam)}$$

$$h_b = \frac{20000}{21} = 952.38 \text{ mm} < 1000 \text{ mm (depth of large span beam)}$$

$$h_s = \frac{4700}{36} = 129.5 \text{ mm} < 130 \text{ mm (depth of the slab)}$$

The thickness of the beams and slabs is actually higher than the minimum required thickness hence the requirements are satisfied.

3.9.2. Crack width

Controlling cracks is essential in the design of reinforced concrete to ensure the building does not collapse during service. Thus, the crack widths should be ensured to not exceed the allowable values specified in the code. The peak crack width under flexure is going to be determined by the formula of McCormac and Brown (2016):

$$w = 0.076 \beta_h f_s \sqrt[3]{d_c A}$$

Where w is the estimated width of the crack in inches, β_h is the relationship between the distance the extreme tension fiber is located and the ratio of the distance between the reinforcement centroid (which can be assumed as 1.2) and the neutral axis (fs); d_c is the concrete cover and A is the effective tension area.

$$d_c = 2.5 \text{ in}$$

$$\# \text{ of tension bars} = 3$$

$$A = \frac{5 * 11.81}{3} = 19.7 \text{ in}^2$$

$$w = 0.076 \beta_h f_s \sqrt[3]{d_c A} = 0.076 * 1.2 * 0.6 * 60 * \sqrt[3]{2.5 * 19.7} = 0.012 \text{ in} \leq 0.016 \text{ in}$$

In this structure, the allowable width of the crack is 0.016 inches and the calculated value of the crack width is less than this figure. Thus, the crack width can be considered

satisfactory. Moreover, the longitudinal reinforcing bars spacing can be calculated to the greatest extent possible with the following.

$$s(in) = (15(\frac{40}{f_s}) - 2.5C_c)$$

$$s = (15\frac{40}{60^{2/3}} - 2.5 * 2.5) = 9.83 \text{ in or } 250 \text{ mm}$$

In this way, the shear reinforcement should not be spaced more than 250 mm to comply with the serviceability requirements.

4. Geotechnical Design

4.1 Site description

The project location is 130 West College Street, Los Angeles, California. The specific location of the project is indicated in Figure 4.1.1. This site lies in the Chinatown district, Northern section of Downtown LA. Los Angeles is located at the intersection of the San Andreas systems and the Transverse Ranges thrust faults, making it one of the highest seismic regions in California (Bilodeau et al., 2007).

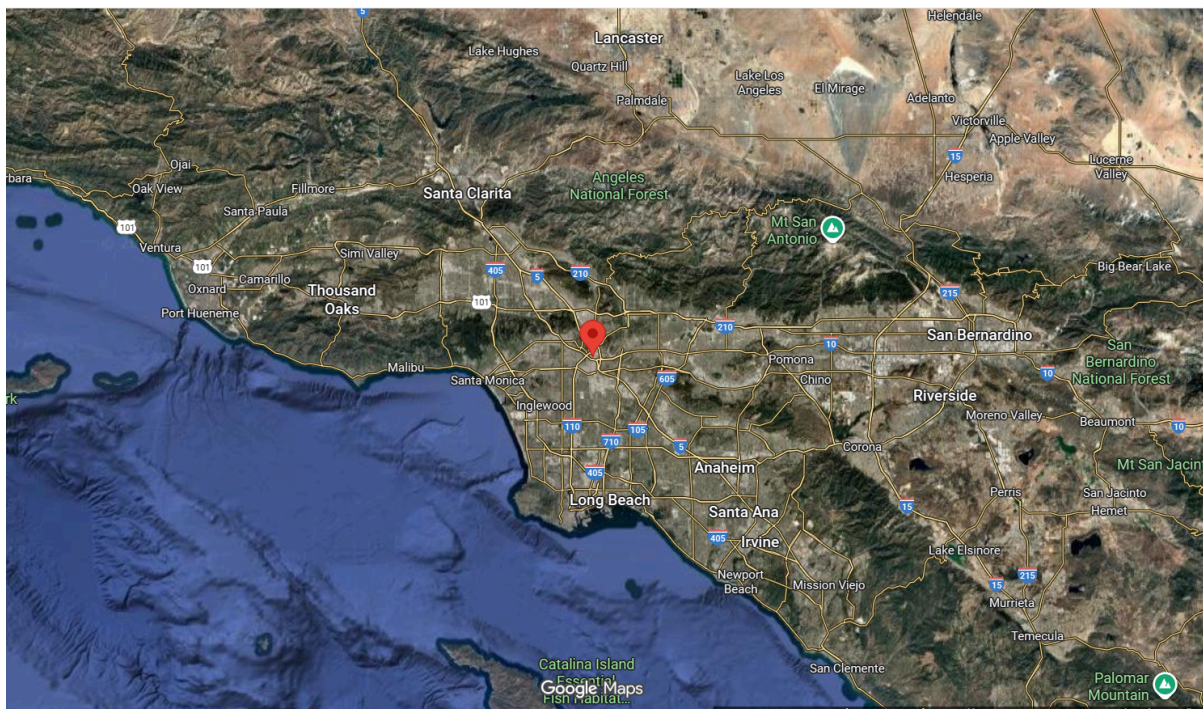


Figure 4.1.1 Location map

According to the satellite image of the site on Figure 4.1.2, currently this location is occupied by a parking lot. For the project realization, the existing parking lot will be removed and a 4-story shopping mall with a 1 basement floor will be constructed.



Figure

Figure 4.1.2 *Satellite photo of existing lot*

Due to the lack of some geotechnical data on the project location, geotechnical reports from neighbouring locations were reviewed. The report of the 765 W College Street area was used. It was assumed that the soil properties are similar in these two locations, since the distance between them is about 848m according to Figure 4.1.3.



Figure 4.1.3 Distance to the auxiliary investigation site

4.2 Geotechnical field investigation

The primary geotechnical investigation report was completed by Langan engineering company between February and October 2023, while the secondary supporting investigation was carried out by GEOBASE INC in January 2017. In the primary report, four borings were drilled across the construction site with depth ranging from 15.7 m (51.5 ft) to 22.3 m (66.5 ft) below ground surface.

The deepest exploration was borehole LB-1, which is 22.3 m deep. For this reason LB-1 was selected for the soil profile presentation. SPT tests were performed for the soil classification purposes. Four CPT tests were carried out, including cone resistance, friction ratio and pore pressure data. All soil layers were identified by the United Soil Classification System (USCS). Other important soil tests included Atterbeg limits, direct shear test, grain size distribution test, liquefaction analysis, unit weight and moisture content. Groundwater was encountered at depths between 7.5 m (24.5 ft) and 8.2 m (27 ft)

The main geotechnical report by Langan engineering did not provide any information on the shear wave measurements. Since this data is crucial for identifying seismic site classification we introduced the geotechnical report by GEOBASE INC conducted on 765 W College Street. This

site location is relatively close to our location and has a comparable soil profile. Based on this we assumed the shear wave velocity from 765 W College Street as a substitute for our site.

4.3 Soil profile

There are 4 boreholes located in our investigation site with depths approximately from 15.7 m to 20.27m. The subsurface conditions of the construction site were mainly extracted from the deepest boring, LB-1, which is 22.3 m (66.5 ft) deep. This boring provided the deepest and most completed investigation. However, for the accuracy of our data we interpolated and compared data from all borings and identified common layers.

The ground surface is paved with 0.3 m thick asphalt. Below the asphalt there is a thin 0.9 m thick layer of clayey sand. Between depths of approximately 1.2 and 10 m, the soils are generally fine to coarse sand with SPT blow counts ranging from 14 to 78 blows per foot. Deeper layers from about 10 to 14 m consist of stiff silt and SPT blow counts fall between 7 to 10 blows/ft. Between 14 and 15 m there is a thin layer of stiff sandy clay. The bottom layers from 15 to 20 m are mostly packed with a stiff lean silt, where SPT blow counts vary from 19 to 54 blows/ft.

Groundwater was encountered at 7.5 m (54.5 ft) depth of the borehole. No bedrock was encountered at the LB-1, which means the soil profile mostly consists of young alluvial soils typical for the Los Angeles region. The soil profile is illustrated on Table 4.3.1.

Table 4.3.1 Soil profile

Layer	Height , m	Depth, m	Soil Type	USCS	N spt	Unit Weight, kN/m ³
1	1.219	0-1.2	Dark brown, clayey SAND , trace fine gravel, moist	SC	0	17.7

2	1.372	1.2-2.6	Medium dense, brown, fine to medium coarse silty SAND , trace fine gravel, moist	SM	18	18.5
3	4.419	2.6-7.0	Very dense, yellow-brown, fine to coarse SAND , some silt, trace fine gravel, moist.	SM	22	19.2
4	3.353	7.0-10.3	Dense, gray, fine to coarse SAND , some silt, wet.	SM	23	19.5
5	3.353	10.3-13.6	Stiff, dark gray, SILT , some fine to medium sand, trace clay, wet.	ML	9	18.9
6	1.524	13.6-15.1	Stiff, dark gray, fine to medium sandy CLAY , trace silt, wet.	CL	8	18.4
7	2.438	15.1-17.5	Stiff, dark gray, lean SILT , some fine sand, trace clay, low plasticity, wet.	ML	32	20.1
8	2.591	17.5-20.1	Stiff, dark gray, lean SILT , some fine sand, trace clay, low plasticity, wet.	ML	44	20.4

As it is seen in Table 4.3.2, the soil profile is presented with an SPT investigation for each layer. Up to the middle part of the soil profile mostly consists of sand layers with classifications of SC in the upper part, and SM until the middle. ML layers were investigated in the below part including clay layer (CL) in the depth of around 14 m below the ground surface. Table 4.3.2 represents soil properties for each layer for further investigations and research.

Table 4.3.2 Soil properties

Layer	Height, m	Depth, m	Soil Type	USCS	N spt	Unit Weight, kN/m ³	Cohesion, kPa	Friction angle, °	Elastic modulus, MPa	Poisson's ratio
-------	-----------	----------	-----------	------	-------	--------------------------------	---------------	-------------------	----------------------	-----------------

1	1.219	1.2	SAND	SC	0	17.7	0	20.00	0	-
2	1.372	2.6	SAND	SM	18	18.5	0	36.65	0	0.3
3	4.419	7.0	SAND	SM	22	19.2	0	38.41	0	0.3
4	3.353	10.3	SAND	SM	23	19.5	0	38.82	0	0.3
5	3.353	13.6	SILT	ML	9	18.9	9.6	31.77	16	0.3
6	1.524	15.1	CLAY	CL	8	18.4	17.6	31.10	38	0.3
7	2.438	17.5	SILT	ML	32	20.1	9.6	42.20	10	0.4
8	2.591	20.1	SILT	ML	44	20.4	9.6	46.03	15	0.4

4.4 Seismicity analysis

a. Liquefaction analysis

Table 4.4.1 Liquefaction values

Layer	Height, m	Depth, m	USCS	Correc tion factor	(N1)60	Vertical Stress, kN/m ²	rd	CSR	CRR	FoS	LPI
1	1.219	1.2	SC	0.92	0	0	1	0.546	4	2	0
2	1.372	2.6	SM	0.89	16.02	25.378	0.99	0.587	1.9	1.4	0.01
3	4.419	7.0	SM	0.82	18.04	110.23	0.98	0.607	0.6	0.7	0.12
4	3.353	10.3	SM	0.7	16.1	175.60	0.97	0.611	4	2	0
5	3.353	13.6	ML	0.66	5.94	238.97	0.95	0.642	1.5	1.1	0.01
6	1.524	15.1	CL	0.69	5.52	267.01	0.93	0.674	4	2	0
7	2.438	17.5	ML	0.68	21.76	316.02	0.9	0.698	4	2	0

8	2.591	20.1	ML	0.64	28.16	368.87	0.87	0.704	4	2	0
---	-------	------	----	------	-------	--------	------	-------	---	---	---

For the liquefaction analysis the following procedures represent the data obtained in Table 4.1.1:

$$CSR_{7.5} = 0.65 \left(\frac{\alpha_{max}}{g} \right) \left(\frac{\sigma_v}{\sigma'_v} \right) \cdot r_d \cdot \frac{1}{MSF} \cdot \frac{1}{K_\sigma} \quad (4.4.1)$$

Where,

$$MSF = 1.19$$

$$K_\sigma \simeq 0.9$$

$$g = 9.81 \text{ m/s}^2$$

α_{max} - peak horizontal acceleration on ground surface,

r_d - stress reduction coefficient,

σ_v - initial total overburden stress,

σ'_v - initial effective overburden stress.

$$CRR_{7.5} = \exp \left(\frac{q_{c1Ncs}}{113} + \left(\frac{q_{c1Ncs}}{1000} \right)^2 - \left(\frac{q_{c1Ncs}}{140} \right)^3 + \left(\frac{q_{c1Ncs}}{137} \right)^4 - 2.80 \right) \quad (4.4.2)$$

Where,

$$q_{c1Ncs} = q_{c1N} + \Delta q_{c1N}$$

$$\Delta q_{c1N} = \left(11.9 + \frac{q_{c1N}}{14.6} \right) \cdot \exp \left(1.63 - \frac{9.7}{FC+2} - \left(\frac{15.7}{FC+2} \right)^2 \right)$$

$$q_{c1N} = C_{Ne} \cdot \frac{q_c}{P_a}$$

$$C_{Ne} = \left(\frac{P_a}{\sigma_v} \right)^m \leq 1.7$$

$$m = 1.338 - 0.249(q_{c1Ncs})^{0.264} \text{ with } 0.264 \leq m \leq 0.782$$

$$FS_{liquefaction} = \frac{CRR_{7.5}}{CSR_{7.5}} \quad (4.4.3)$$

By obtaining the values of CSR and CRR following the above procedures and formulas, the results shown in Table 4.4.1 the Factor of Safety for liquefaction could be calculated in (4.4.3). The study represents their own criteria of finding FS for liquefaction, the table values were investigated by interpolation of the values according to the formula 4.4.3. Interpolation was proceeded for analyzing the vulnerable layers in the soil profile. LPI values are obtained from:

$$LPI = \int_0^{20} (10 - 0.5_z) \cdot F_L \cdot d_z \quad (4.4.4)$$

Where,

$F_L = 1$ - when FS less than 1,

$F_L = 0$ - when FS greater than 1,

z - depth of measurements in meters.

LPI evaluation:

$LPI = 0$ - Liquefaction risk is very low,

$LPI = 0 \sim 5$ - Liquefaction risk is low,

$LPI = 5 \sim 15$ - Liquefaction risk is high,

$LPI > 15$ - Liquefaction risk is very high.

b. Shear wave velocity

Shear wave velocity graph

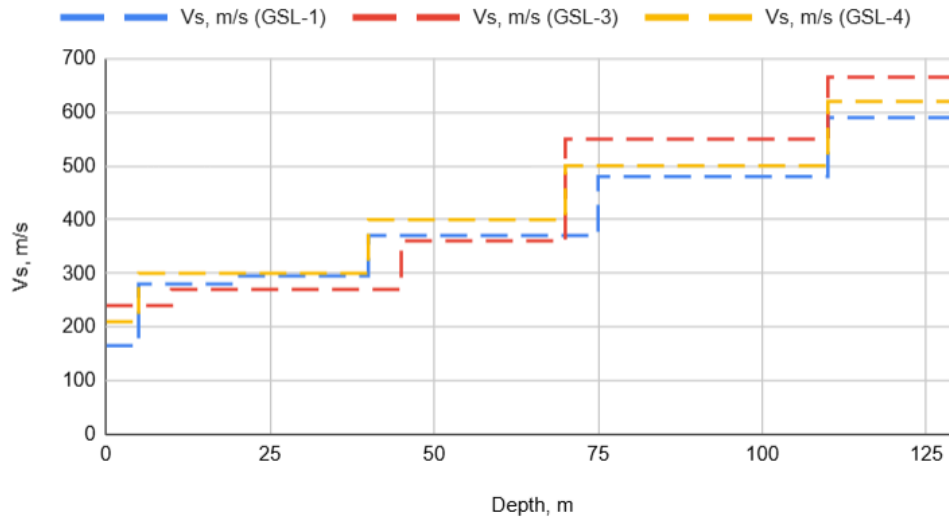


Figure 4.4.2 Shear wave velocity graph

The investigation from 765 W College Street, conducted 3 tests on shear wave velocity of the soil. The site classification was identified using the average shear wave velocities of the first 30m (V_{s30}). According to these tests the average V_{s30} was equal to approximately 280 m/s. This value falls into Site Class D (Stiff Soil) (ASCE/SEI 7-16, 2017).

4.5 Shallow foundation design

The first stage in the geotechnical part of this project was to create and test a shallow foundation in order to be able to get a reference point of how the conditions beneath the surface react to the near surface load transfer. This type of foundation is based on the structural loads being distributed over a wide area near the surface of the ground hence this is a simple point of beginning the analysis of bearing capacity and settlement behavior under the loads expected to be placed on the buildings. Through the analysis of the performance of a shallow system first, we can easily measure the constraints of the current soil profile.

The design procedure was centered at determining the pertinent soil parameters, estimation of immediate and long term settlements, and also whether the induced bearing pressures can be compatible with the mechanical properties of the grounds. Such analyses are the basis upon which alternative systems are compared in the report later. Results of the shallow foundation analysis are calculated, assimilated, and the values presented in the subsequent sections.

a. Bearing capacity: Shallow foundation, Footing Foundation

Table 4.5.1 Soil properties for bearing capacity, Terzaghi's method

Cohesion c'	0 (for sand)
Unit weight (kN/m ³)	19
Saturated unit weight (kN/m ³)	22.08
Depth of foundation (m)	1
N_c	42.16
N_q	29.44
N_γ	41.06
r_y (reduction factor)	0.93
$B \cdot N_\gamma$	151.88

To calculate the bearing capacity the following procedures have been proceeded:

Terzaghi's general equation:

$$q_u = c'N_c + qN_q + \frac{1}{2}\gamma BN_\gamma \quad (4.5.1)$$

Where,

N_c , N_q , N_γ - bearing capacity factor,

c' - cohesion,

q - effective stress at the bottom of the foundation,

γ - unit weight of soil,

B - width of the foundation.

$$N_c = \tan \phi' (K_c + 1)$$

$$N_q = K_q \tan \phi'$$

$$N_\gamma = \frac{1}{2} \tan \phi' (K_\gamma \tan \phi' - 1)$$

As footing foundation type for shallow foundation is required, the formula below represents:

For square foundation:

$$q_u = 1.3c'N_c + qN_q + 0.4\gamma BN_\gamma \quad (4.5.2)$$

Table 4.5.2 Soil properties for bearing capacity, Meyerhof method

Cohesion c'	0 (for sand)
Unit weight (kN/m ³)	19
Saturated unit weight (kN/m ³)	22.08
Depth of foundation (m)	1
N_c	45.96
N_q	32.12
N_γ	28.28
r_γ (reduction factor)	0.93
$B*N_\gamma$	151.88

K	14.65
s_c	3.35
s_q	2.17
s_y	2.17
d_c	1.19
d_q	1.10
d_y	1.10

Meyerhof's equation:

$$q_u = c'N_c F_{cs} F_{cd} F_{ci} + qN_q F_{qs} F_{qd} F_{qi} + \frac{1}{2}\gamma B N_\gamma F_{\gamma s} F_{\gamma d} F_{\gamma i} \quad (4.5.3)$$

Where,

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

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

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

All factors and formulas were taken from the book “Principles of FOUNDATION ENGINEERING” (Das, B. M., & Sivakugan, N. (2019). *Principles of foundation engineering* (9th ed., SI ed.). Cengage Learning.)

Hansen's equation:

$$q_u = c'N_c s_c d_c i_c g_c b_c + \bar{q} N_q s_q d_q i_q g_q b_q + \frac{1}{2}\gamma B' N_\gamma s_\gamma d_\gamma i_\gamma g_\gamma \quad (4.5.4)$$

Where,

s_c, s_q, s_γ - shape factors

d_c, d_q, d_γ - depth factors

i_c, i_q, i_γ - inclination factors

g_c, g_q, g_γ -ground surface factors

b_c, b_q - base inclination factors

$$K_p = \tan^2(45 + \frac{\phi}{2}) \quad (4.5.5)$$

$$N_c = (N_q - 1)\cot\phi \quad (4.5.6)$$

$$N_q = e^{\pi \tan\phi} \tan^2(45 + \frac{\phi}{2}) \quad (4.5.7)$$

$$N_\gamma = (N_q - 1)\cot(1.4\phi) \quad (4.5.8)$$

Base inclination, ground surface slope, load inclination factors taken as 1, because the foundation is assumed to be under ideal loading and ground conditions.

Table 4.5.3 Footing foundation results

Method	for footing	3 x 4	4 x 5	5 x 6
Terzaghi	$q_{ultimate}(kN/m^2)$	1 454.32	1 713.65	1 964.42
	$Q_{allowable}(kN)$	5 741.26	11 297.64	19 454.16
Meyerhof	$q_{ultimate}(kN/m^2)$	3 267.74	3 817.47	4 351.31
	$Q_{allowable}(kN)$	12 994.98	25 323.16	43 323.02

Hansen	$q_{ultimate} (kN/m^2)$	1 406.07	1 559.57	1 701.63
	$Q_{allowable} (kN)$	5 548.29	10 270.46	16 826.32

b. Bearing capacity: Shallow foundation, Mat Foundation

Table 4.5.4 Mat foundation results

B = 40 m, L = 100 m		
Method	$q_{ultimate} (kN/m^2)$	$Q_{allowable} (kN)$
Terzaghi	8 981.66	11 950 210.45
Meyerhof	12 586.72	16 756 955.51
Hansen	6 836.96	9 090 620.123

Results of bearing capacity and calculating loads

Dead Load (DL): Estimated at **8.5 kPa** for typical floors and **7.0 kPa** for the roof, accounting for concrete slabs, finishes, partitions, and MEP systems.

Live Load (LL): Estimated at **4.8 kPa** (100 psf) for retail floors, **1.9 kPa** (40 psf) for the parking garage, and **1.0 kPa** for the roof, based on ASCE 7 guidelines.

The load takedown for a representative interior column resulted in a total service load of approximately **10,780 kN**. For the entire 40m x 100m building with an estimated 36 columns, the total building load is approximately **360,000 kN**. These values are considered reasonable for a structure of this scale.

$$q_{net,ult} = q_{ult} - \gamma D_f \quad (4.5.9)$$

$$q_{net,all} = \frac{q_{net,ult}}{FS} \quad (4.5.10)$$

$$Q_{all} = q_{all} \times B \times L \quad (4.5.11)$$

The analysis shows that a footing of at least 4m x 5m is required to safely support the column load. However, for a 40m x 100m building with 36 columns, using footings of this size is impractical. The footings would be spaced too closely, leading to overlap of stress zones and potential overlap of the footings themselves. This would create significant constructability issues and lead to excessive and unpredictable differential settlement. Therefore, isolated footings were rejected as a viable solution.

All three methods show that the allowable bearing capacity of the mat foundation is significantly greater than the total applied load of ~ 360,000 kN. From a bearing capacity standpoint, the mat foundation is a safe and viable solution.

4.6 Shallow Foundation settlement

While the mat foundation passed the bearing capacity check, it must also satisfy settlement criteria. The calculated static settlement, comprising elastic and consolidation components, is summarized in Table 4.6.1

Table 4.6.1 Settlement of mat foundation

Settlement	m	mm
Elastic Settlement	0.07348	73.48
Consolidation Settlement	0.00863	8.63
Total	0.08211	82.11

Formula for Elastic settlement:

$$S_e = \frac{q_0 B}{E_s} (1 - \mu_s^2) I_s I_f \quad (4.6.1)$$

$$I_s = F_1 + \left(\frac{1-2\mu_s}{1-\mu_s} \right) F_2 \quad (4.6.2)$$

Formula for Consolidation settlement:

$$S_{c(p)} = \frac{C_c H_c}{1+e_0} \log\left(\frac{\sigma'_0 + \Delta\sigma_{avg}}{\sigma'_0}\right) \quad (4.6.3)$$

Even though the shallow foundation analysis offers a good reference point in the behaviour of the soils at structural loading, the findings make it evident that this system cannot be applied to the proposed building. Both the elastic and consolidation calculated settlements are above the generally accepted serviceability limits and result in deformation levels that would jeopardize structural performance as well as long term reliability. Besides the size of the overall settlement, the heterogeneity of the soil profile creates issues surrounding the aspect of varying movements that could further affect the building. This has shown that the shallow foundation is incapable of giving the amount of control that is needed in a project of this magnitude and thus other foundation solutions should be looked into in future design phases.

4.7 Deep Foundation Design of a Single Pile

After considering the shallow foundation system, the second step of the geotechnical examination will deal with one pile foundation as an option. A pile, in contrast to a shallow one, loads of structural weight onto more capable and deeper strata of soil or rock, avoiding the weak strata or highly compressible strata which increase the excessive settlement. Through the investigation of a single pile behaviour under axial loading, it is possible to determine the load carrying mechanisms, evaluate end bearing, and shaft resistance and ascertain the possibility of the deeper subsurface conditions to offer a stable and reliable system of support to the building. The following calculations describe the approach to determining the estimation of capacity, settlement, and the performance of one pile in the geotechnical setting of a site.

In order to design a deep foundation, bearing capacity of a single pile should be estimated. The ultimate bearing capacity (Q_u) of a pile should be calculated by:

$$Q_u = Q_p + Q_s$$

Where,

Q_p - point resistance of a pile tip

Q_s - frictional resistance of a pile shaft

It is also essential to ensure that our pile foundation design will withstand the applied structure load. Thus, the following formula of allowable bearing capacity will be introduced:

$$Q_{all} = \frac{Q_u}{FS}$$

Where,

Q_{all} - allowable pile capacity

FS=3

The allowable bearing capacity of the pile must satisfy the following requirement:

$$Q_{all} \geq Q_{design}$$

Where,

Q_{design} -column load applied by the building

For further calculations the diameter of the piles are taken as 0.4, 0.5 and 0.6 m which is typical for bored precast concrete piles. The ultimate bearing capacity was calculated for piles with lengths between 7 and 16 meters. The basement excavation of 3.5 m from upper layers is considered in the further bearing capacity estimations.

4.7.1 Point Resistance (Qp)

To find the point resistance, from the observation of the above parts regarding Shallow Foundation, it has been decided that Shallow Foundation could not fit for the safety measures of the basement for the Shopping Mall. Pile (or Deep) Foundation is assumed to fit the design.

Since pile tips for lengths between 7 and 16 meters all end within clay layers, the point resistance only for clay layers should be calculated. To calculate point resistance: Meyerhof's and Vesics methods were applied following procedures:

Meyerhof method:

$$Q_p = 9c_u A_p$$

Where,

c_u - undrained cohesion of the soil below the tip of the pile, kN/m^2

A_p - cross section area of a pile, m^2

$$A_p = \left(\frac{D}{2}\right)^2 \pi = \left(\frac{0.6}{2}\right)^2 \pi = 0.2826 m^2$$

Vesic's method:

$$Q_p = A_p c_u N_c^*$$

Where,

N_c^* -bearing capacity factor of a pile

According to the Vesic theory of expansion of cavity,

$$N_c^* = \frac{4}{3} (\ln I_{rr} + 1) + \frac{\pi}{2} + 1$$

I_{rr} can be approximated as

$$I_{rr} = I_r = 347 \left(\frac{c_u}{p_a}\right) - 33 \leq 300$$

Where,

p_a -atmospheric pressure = 100 kN/m^2

The Table 4.7.1.1 illustrates Undrained Shear Strength value for clay layers to investigate the shaft resistance in further calculations.

Table 4.7.1.1 Undrained Shear Strength values for each layer

Layer	Height, m	Depth, m	Undrained Shear Strength, kPa
1	1.219140506	1.2	0
2	1.371533069	2.6	0
3	4.419384334	7.0	0
4	3.352636391	10.3	0
5	3.352636391	13.6	40.5
6	1.523925632	15.1	36
7	2.438281012	17.5	144
8	2.590673575	20.1	198

The results for the pile tip bearing capacity of a single pile with diameters of 0.4, 0.5 and 0.6 m and length varying from 7 to 16 meters are represented in the following table:

Table 4.7.1.2 Point bearing capacity in clay.

Pile diameter, m	0.4		
Pile length, m	Meyerhof Q_p , kN	Vesic Q_p , kN	Average Q_p , kN
7	45.8	51.61	48.705
8	45.8	51.61	48.705
9	45.8	51.61	48.705

10	45.8	51.61	48.705
11	40.72	44.93	42.825
12	162.86	208.25	185.555
13	162.86	208.25	185.555
14	162.86	208.25	185.555
15	223.93	286.34	255.135
16	223.93	286.34	255.135
Pile diameter, m	0.5		
Pile length, m	Meyerhof Q_p, kN	Vesic Q_p, kN	Average Q_p, kN
7	71.57	80.64	76.105
8	71.57	80.64	76.105
9	71.57	80.64	76.105
10	71.57	80.64	76.105
11	63.62	70.2	66.91
12	254.47	325.4	289.935
13	254.47	325.4	289.935
14	254.47	325.4	289.935
15	349.89	447.41	398.65
16	349.89	447.41	398.65
Pile diameter, m	0.6		

Pile length, m	Meyerhof Q_p , kN	Vesic Q_p , kN	Average Q_p , kN
7	103.01	116.06	109.54
8	103.01	116.06	109.54
9	103.01	116.06	109.54
10	103.01	116.06	109.54
11	91.56	101.04	96.3
12	366.25	468.33	417.29
13	366.25	468.33	417.29
14	366.25	468.33	417.29
15	503.59	643.95	573.77
16	503.59	643.95	573.77

The results of these two methods indicated similar results with less than 20% difference.

Averaging these two results provide an accurate estimate of the final point bearing capacity value without relying on a single method.

4.7.2 Shaft Resistance (Q_s)

a. Shaft Resistance in Sand

The skin friction by the upper granular layers from the excavation level at 3.5m down till the clay layers at 10.3m was calculated using an effective stress method and 3 empirical methods based on the SPT test results: Meyerhof's Method, Briaud et al., and Coyle and Castello. The effective stress method is considered the most accurate for frictional resistance in sand as it is based on the fundamental soil mechanics principles, this method was applied only to the 6.8m sand segment since sand layers are located only till 10.3 m below the surface and the shortest pile of 7 m considering the 3.5m excavation already reaches the clay layer located at 10.5 m

below the ground surface. The empirical methods rely on general correlations, therefore, they were not used in final design.

Effective stress method:

$$f = K\sigma_0' \tan \delta'$$

$$Q_s = \sum p \Delta L f = \sum p \Delta L K \sigma_0' \tan \delta'$$

Where,

p – perimeter of the pile, m

ΔL – width of the layer, m

K – earth pressure coefficient =1.5 for precast concrete piles

σ_0' – effective vertical stress, kN/m^2

δ' – soil-pile friction angle= $0.8 \phi'$

The results of frictional resistance in sand for different pile lengths using effective stress method are provided in the Table 4.7.2.1:

Table 4.7.2.1 Shaft resistance values using effective stress method

D, m	0.4				
Pile length, m	Depth, m	Perimeter of the pile, m	σ_0' at depth, kN/m^2	f at depth, kN/m^2	Qs, kN
7	10.5	1.257	132.58	87.09	769.18
8	11.5	1.257	132.58	87.09	851.27
9	12.5	1.257	132.58	87.09	933.36

10	13.5	1.257	132.58	87.09	1015.44
11	14.5	1.257	132.58	85.96	825.68
12	15.5	1.257	132.58	117.82	797.18
13	16.5	1.257	132.58	117.82	908.23
14	17.5	1.257	132.58	117.82	1019.27
15	18.5	1.257	132.58	127.47	872.91
16	19.5	1.257	132.58	127.47	993.05
D, m	0.5				
Pile length, m	Depth, m	Perimeter of the pile, m	σ_0' at depth, kN/m^2	f at depth, kN/m^2	Qs, kN
7	10.5	1.571	132.58	87.09	957.37
8	11.5	1.571	132.58	87.09	1039.46
9	12.5	1.571	132.58	87.09	1121.55
10	13.5	1.571	132.58	87.09	1203.63
11	14.5	1.571	132.58	85.96	1013.87
12	15.5	1.571	132.58	117.82	985.37
13	16.5	1.571	132.58	117.82	1096.42
14	17.5	1.571	132.58	117.82	1207.46
15	18.5	1.571	132.58	127.47	1061.1
16	19.5	1.571	132.58	127.47	1181.24
Pile	0.6				

diameter, m					
Pile length, m	Depth, m	Perimeter of the pile, m	σ_0' at depth, kN/m^2	f at depth, kN/m^2	Qs, kN
7	10.5	1.885	132.58	87.09	1145.59
8	11.5	1.885	132.58	87.09	1227.68
9	12.5	1.885	132.58	87.09	1309.77
10	13.5	1.885	132.58	87.09	1391.85
11	14.5	1.885	132.58	85.96	1202.09
12	15.5	1.885	132.58	117.82	1173.59
13	16.5	1.885	132.58	117.82	1284.64
14	17.5	1.885	132.58	117.82	1395.68
15	18.5	1.885	132.58	127.47	1249.32
16	19.5	1.885	132.58	127.47	1369.46

Meyerhof method:

$$Q_s = f_{av} pL = (0.02 p_a N_{60}) pL$$

Where,

N_{60} – average value of SPT resistance

Coyle and Castello method:

$$Q_s = f_{av} pL = (K \sigma_0' \tan \delta') pL$$

The Earth pressure coefficient, K, was found using a variation of K with the L/D diagram redrawn after Coyle and Castello.

Briaud et al. method:

$$Q_s = f_{av} pL = K p_a (N_{60})^{0.29} pL$$

The following table represents calculated frictional resistance of a pile using all 3 empirical methods:

Table 4.7.2.2 Frictional resistance in sand using Meyerhof's, Briaud et al., and Coyle and Castello methods.

D, m	0.4					
L, m	P, m	N₆₀	K	Qs, kN (Meyerhof)	Qs, kN (Coyle and Castello)	Qs, kN (Briaud)
7	1.257	11.22	0.85	197.4	245.5	397.24
8	1.257	11.22	0.79	225.59	228.17	453.99
9	1.257	11.22	0.71	253.79	205.06	510.74
10	1.257	11.22	0.65	281.99	187.74	567.49
11	1.257	10.27	0.6	283.92	258.05	608.42
12	1.257	11.91	1.5	359.24	1237.15	692.9
13	1.257	11.91	1.5	389.18	1237.15	750.64
14	1.257	11.91	1.5	419.11	1237.15	808.38
15	1.257	13.94	1.5	525.62	1784.2	906.58
16	1.257	13.94	1.5	560.66	1784.2	967.02
D, m	0.5					
L, m	P, m	N₆₀	K	Qs, kN (Meyerhof)	Qs, kN (Coyle and Castello)	Qs, kN (Briaud)

					Castello)	
7	1.571	11.22	0.85	246.74	306.87	496.55
8	1.571	11.22	0.79	281.99	285.21	567.49
9	1.571	11.22	0.71	317.24	256.33	638.42
10	1.571	11.22	0.65	352.49	234.67	709.36
11	1.571	10.27	0.6	354.91	322.56	760.53
12	1.571	11.91	1.5	449.05	1546.44	866.12
13	1.571	11.91	1.5	486.47	1546.44	938.3
14	1.571	11.91	1.5	523.89	1546.44	1010.47
15	1.571	13.94	1.5	657.02	2230.25	1133.23
16	1.571	13.94	1.5	700.83	2230.25	1208.78
D, m	0.6					
L, m	P, m	N₆₀	K	Qs, kN (Meyerhof)	Qs, kN (Coyle and Castello)	Qs, kN (Briaud)
7	1.885	11.22	0.85	296.1	368.26	595.88
8	1.885	11.22	0.79	338.4	342.26	681
9	1.885	11.22	0.71	380.7	307.6	766.13
10	1.885	11.22	0.65	423	281.61	851.25
11	1.885	10.27	0.6	425.7	387.08	912.66
12	1.885	11.91	1.5	538.87	1855.77	1039.37
13	1.885	11.91	1.5	583.78	1855.77	1125.98

14	1.885	11.91	1.5	628.69	1855.77	1212.6
15	1.885	13.94	1.5	788.45	2676.36	1359.91
16	1.885	13.94	1.5	841.01	2676.36\	1450.57

b. Shaft Resistance in Clay

Estimating various lengths of the piles to compare, the lengths were taken between 7 m and 16 m. 16 m as maximum, as the Soil Profile is observed until 19.5 m, with consideration of excavation for the basement. To find the shaft resistance in clay - alpha (α), betta (β), and lambda (λ) methods were used by following procedures:

Alpha (α) method:

$$f = \alpha C_u$$

$$Q_s = \sum f p \Delta L = \sum \alpha C_u p \Delta L \quad (4.7 \dots)$$

Where,

f – unit skin resistance,

p – perimeter of the pile, m

ΔL – width of the layer, m

α – empirical adhesion factor derived from Terzhagi's table,

C_u – undrained shear strength, kN/m²

Alpha (α) is taken by interpolation based on the Terzhagi's table following $\frac{c_u}{p_a}$ values. In this regard, the obtained shaft resistance for each pile length and 3 diameters (0.4 m, 0.5 m, 0.6 m) for the provided soil layers are illustrated in Table 4.7.2.2 by Alpha (α) method:

Table 4.7.2.2 Alpha (α) Method Qs values

Pile Length, m	Qs value (D=0.4), kN	Qs value (D= 0.5), kN	Qs value (D=0.6), kN
7	9.86	12.32	14.78
8	59.13	73.91	88.68
9	108.41	135.50	162.58
10	157.69	197.09	236.48
11	201.06	251.28	301.5
12	278.03	347.48	416.94
13	408.36	510.37	612.37
14	543.67	679.48	815.29
15	630.78	788.35	945.92
16	717.89	897.22	1076.55

Lambda (λ) method:

$$f_{av} = \lambda(\overline{\sigma_0'} + 2c_u)$$

$$Q_s = pLf_{av} = pL\lambda(\overline{\sigma_0'} + 2c_u)$$

Where,

c_u - mean undrained shear strength, kPa

$\overline{\sigma_0'}$ - mean effective vertical stress for the entire embedment length, kN/m^2 .

The mean effective stress is obtained by the following formula:

$$\overline{\sigma_0'} = \frac{A_1 + A_2 + A_3 + \dots}{L}$$

Where,

$A_1, A_2, A_3, \dots$ - areas of effective vertical stress diagrams

$$A_3 = \frac{(25.37 + 110.23) \cdot 4.41}{2} = 299.63$$

The following table shows areas of effective vertical stress diagrams:

Table 4.7.2.3 Effective vertical stress areas

Depth, m	Effective stress, kN/m^2	Areas of effective stress diagrams, m^2
1.2	0	0
2.6	25.37	17.4
7.0	110.23	299.63
10.3	148.13	433.09
13.6	178.61	547.73
15.1	191.7	282.16
17.5	216.79	498.01
20.1	244.23	597.17

Mean effective stress and mean undrained shear strength was calculated for each pile length, lambda values were obtained via interpolation of variation of λ with pile embedment length. The skin friction in clay layers using lambda method for each pile length was obtained in the following table:

Table 4.7.2.4 Shaft resistance using λ method

D, m	0.4						
L, m	Depth, m	P, m	λ	c_u , kPa	$\overline{\sigma_o'}$, kN/m ²	f_{av}	Q_s , kN
7	10.5	1.257	0.299	1.16	198.29	59.98	527.6
8	11.5	1.257	0.281	6.07	175.43	52.71	529.91
9	12.5	1.257	0.263	9.9	157.66	46.67	527.85
10	13.5	1.257	0.245	12.96	157.98	45.05	566.17
11	14.5	1.257	0.236	15.1	169.21	47.06	650.48
12	15.5	1.257	0.227	20.44	211.07	57.19	862.42
13	16.5	1.257	0.218	29.94	195.81	55.74	910.61
14	17.5	1.257	0.209	38.09	182.73	54.11	952
15	18.5	1.257	0.2	58.35	227.99	68.94	1299.44
16	19.5	1.257	0.195	67.08	214.59	68.01	1367.34
D, m	0.5						
L, m	Depth, m	P, m	λ	c_u , kPa	$\overline{\sigma_o'}$, kN/m ²	f_{av}	Q_s , kN
7	10.5	1.571	0.299	1.16	198.29	59.98	659.51
8	11.5	1.571	0.281	6.07	175.43	52.71	662.39
9	12.5	1.571	0.263	9.9	157.66	46.67	659.82
10	13.5	1.571	0.245	12.96	157.98	45.05	707.72
11	14.5	1.571	0.236	15.1	169.21	47.06	813.1
12	15.5	1.571	0.227	20.44	211.07	57.19	1078.02

13	16.5	1.571	0.218	29.94	195.81	55.74	1138.26
14	17.5	1.571	0.209	38.09	182.73	54.11	1190
15	18.5	1.571	0.2	58.35	227.99	68.94	1624.3
16	19.5	1.571	0.195	67.08	214.59	68.01	1709.17
D, m	0.6						
L, m	Depth, m	P, m	λ	c_u , kPa	$\overline{\sigma'_o}$, kN/m ²	f_{av}	Q_s , kN
7	10.5	1.885	0.299	1.16	198.29	59.98	791.43
8	11.5	1.885	0.281	6.07	175.43	52.71	794.89
9	12.5	1.885	0.263	9.9	157.66	46.67	791.79
10	13.5	1.885	0.245	12.96	157.98	45.05	849.28
11	14.5	1.885	0.236	15.1	169.21	47.06	975.74
12	15.5	1.885	0.227	20.44	211.07	57.19	1293.65
13	16.5	1.885	0.218	29.94	195.81	55.74	1365.95
14	17.5	1.885	0.209	38.09	182.73	54.11	1428.04
15	18.5	1.885	0.2	58.35	227.99	68.94	1949.20
16	19.5	1.885	0.195	67.08	214.59	68.01	2051.06

Betta (β) method:

$$f_{av} = \beta \sigma'_0 = (K \tan \phi'_R) \sigma'_0 = (1 - \sin \phi'_R) \tan \phi'_R \sigma'_0$$

$$Q_s = \sum f_{av} pL = \sum \beta \sigma'_0 pL$$

Where,

ϕ'_R – drained friction angle of remolded clay

K – earth pressure coefficient

σ_0' – vertical effective stress, kN/m^2

Table 4.7.2.5 Shaft resistance using β method

D, m	0.4						
L, m	Depth, m	P, m	β	K	σ_0', kN/m^2	f_{av}	Q_s, kN
7	10.5	1.257	0.296	0.442	198.29	58.6	526.05
8	11.5	1.257	0.296	0.442	175.43	51.845	528.79
9	12.5	1.257	0.296	0.442	157.66	46.593	532.18
10	13.5	1.257	0.293	0.459	157.98	46.321	650.18
11	14.5	1.257	0.293	0.459	169.21	49.614	696.4
12	15.5	1.257	0.295	0.443	211.07	62.186	947.94
13	16.5	1.257	0.295	0.443	195.81	63.231	1014.38
14	17.5	1.257	0.295	0.427	182.73	63.275	1089.45
15	18.5	1.257	0.295	0.427	227.99	63.314	1156.72
16	19.5	1.257	0.295	0.427	214.59	63.386	1290.25
D, m	0.5						
L, m	Depth, m	P, m	β	K	σ_0', kN/m^2	f_{av}	Q_s, kN
7	10.5	1.571	0.296	0.442	198.29	58.6	657.45
8	11.5	1.571	0.296	0.442	175.43	51.845	660.89
9	12.5	1.571	0.296	0.442	157.66	46.593	665.12
10	13.5	1.571	0.293	0.459	157.98	46.321	812.6
11	14.5	1.571	0.293	0.459	169.21	49.614	870.36

12	15.5	1.571	0.295	0.443	211.07	62.186	1184.73
13	16.5	1.571	0.295	0.443	195.81	63.231	11294.5
14	17.5	1.571	0.295	0.427	182.73	63.275	1445.67
15	18.5	1.571	0.295	0.427	227.99	63.314	1537.82
16	19.5	1.571	0.295	0.427	214.59	63.386	1612.55
D, m	0.6						
L, m	Depth, m	P, m	β	K	$\sigma'_0, kN/m^2$	f_{av}	Q_s, kN
7	10.5	1.885	0.296	0.442	198.29	58.6	788.86
8	11.5	1.885	0.296	0.442	175.43	51.845	792.98
9	12.5	1.885	0.296	0.442	157.66	46.593	798.05
10	13.5	1.885	0.293	0.459	157.98	46.321	975.01
11	14.5	1.885	0.293	0.459	169.21	49.614	1044.32
12	15.5	1.885	0.295	0.443	211.07	62.186	1421.53
13	16.5	1.885	0.295	0.443	195.81	63.231	1395.72
14	17.5	1.885	0.295	0.427	182.73	63.275	1582.94
15	18.5	1.885	0.295	0.427	227.99	63.314	1782.45
16	19.5	1.885	0.295	0.427	214.59	63.386	1934.86

After conducting all of the calculations, the alpha method gave the lowest value for the shaft resistance and therefore was selected for the final design. This method demonstrates the worst-case scenario and provides a better safety against the pile bearing capacity overestimation.

By finalizing the results of the Point Resistance and Shaft Resistances on Clay and Sand layers, the results have been generalized in the according table below:

Table 4.7.3. Ultimate Bearing Capacity of a single pile with L=16m

D, m	0.6	0.5	0.4
Qs (sand), kN	1369.46	1181.24	993.05
Qs (clay), kN	1076.55	897.11	897.11
Qp, kN	573.77	398.65	255.14
Qu, kN	3019.78	2477.01	2145.29
Qall, kN	1006.59	825.67	715.1
Q design, kN	6580.29	6580.29	6580.29

The Factor of Safety (FoS) is taken as 3, as from the previous section for Shallow Foundation.

The calculated bearing capacities of single piles with diameter ranging from 0.4 m to 0.6m are all significantly lower than the design column load, which is equal to 6580 kN for interior columns.

4.7.3 Negative skin friction

Negative skin friction (NSF) is an additional drag force on the foundation created when the soil around the pile settles more than the pile. It decreases the pile's available bearing capacity while the downward load exerted on the pile increases. Since our soil profile contains compressive clay and silt layers beneath a 7.5 m GWT it is susceptible to NSF. After excavation 3.5 m for the basement, the soil resulting profile will have granular soil layers overlying the consolidating clay and silt layers. To make sure the pile design is safe for the total load, the following calculations have been made.

$$Q_n = \frac{pK'\gamma_f'H_f^2 \tan\delta'}{2}$$

$$f_n = K'\sigma_0' \tan\delta'$$

Where,

f_n – negative skin stress, kN/m^2

K' – earth pressure coefficient = $1 - \sin\phi'$

σ_0' – vertical effective stress, kN/m^2

γ_f' – effective unit weight of fill, kN/m^3

δ' – soil-pile friction angle = $(0.5 - 0.7)\phi'$

p – pile perimeter = $\pi D = \pi * 0.6 = 1.885\text{ m}$

H_f – height of the clay fill, m

The effective unit weight below the GWT (7.5) is calculated by

$$\gamma' = \gamma_{sat} - \gamma_{water}$$

Vertical effective stress formula:

$$\sigma_0' = \Sigma(H \times \gamma')$$

Where,

H – thickness of the soil layer, m

$$\sigma_3' = (\gamma_3 * H) + \sigma_2' = (19.2 * 4.4) + 25.37 = 110.23$$

$$\begin{aligned}\sigma_4' &= (\gamma_4 * H - 9.81 * (10.3\text{m} - 7.5\text{m})) + \sigma_3' = \\ &= (19.5 * 3.35 - 9.81 * 2.8) + 110.23 = 175.6\end{aligned}$$

The following table represents the vertical effective stresses for each soil layer including the unit weights considering the GWT at 7.5 m depth.

Table 4.7.1 Lateral Active Pressure

Layer number	Depth, m	Effective unit weight, kN/m^3	Vertical effective stress, kN/m^2
1	1.2	17.7	0
2	2.6	18.5	25.37
3	7.0	19.2	110.23
4	10.3	19.5	175.6
5	13.6	18.9	238.97
6	15.1	18.4	267.01

7	17.5	20.1	316.02
8	20.1	20.4	368.87

Following the previous NSF formulas, the final NSF and downward drag force can be calculated. Since the 3.5 m depth of the soil will be excavated from the site, we will be left with 6 layers.

Table 4.7.2 Negative Skin Friction and Downward Drag Force

Layer #	Depth, m	Thickness, m	$\phi', ^\circ$	$\delta', ^\circ$	K'	$f_n, kN/m^2$	Q_n, kN
3 SAND	7	3.5	36.67	18.33	0.69	25.04	50.35
4 SAND	10.3	3.3	35.75	17.87	0.69	33.11	44.73
5 SILT	13.6	3.3	29.56	14.78	0.74	35.11	38.13
6 CLAY	15.1	1.5	29.22	14.61	0.75	37.37	7.61
7 SILT	17.5	2.4	38.31	19.15	0.67	50.59	25.46
8 SILT	20.1	2.6	40.82	20.41	0.65	59.19	31.49

4.8 Deep foundation settlement of a single pile

The total settlement is the summation of elastic settlement (S_e) and consolidation settlement (S_c) of a pile.

$$S_t = S_e + S_c$$

Consolidation settlement is included in calculations because it represents the long-term compression of saturated fine-graded soils like clay and silt layers under the pile tip due to drainage of pore water.

4.8.1 Elastic settlement of a single pile

Elastic settlement is the immediate deformation of soil that happens after load is applied. The total elastic settlement formula according to Veisic's method:

$$S_e = S_{e1} + S_{e2} + S_{e3}$$

Where,

S_{e1} – elastic shortening of the pile

S_{e2} – settlement due to working load on the tip of the pile (Q_{wp})

S_{e3} – settlement due to working load along the shaft of the pile (Q_{ws})

$$S_{e1} = \frac{(Q_{wp} + \xi Q_{ws})L}{A_p E_p}$$

Where

Q_{wp} – working load on the pile tip, kN

Q_{ws} – working load along the shaft of the pile, kN

ξ – a constant value taken as 0.5

L – length of the pile, m

A_p – cross-sectional area of the pile, m^2

E_p – modulus of elasticity of the pile, kN/m^2

We can estimate the modulus of elasticity of the pile (E_p) from compressive strength (f'_c) of the concrete, assuming it's equal to 20 MPa.

$$E_p = 4700\sqrt{f'_c} = 4700\sqrt{20} = 21019.04 \text{ MPa} \approx 21019039 \text{ kN/m}^2$$

Working loads on the pile tip and shaft were calculated based on the following formulas:

$$Q_{wp} = \frac{Q_p}{FS}$$

$$Q_{ws} = \frac{Q_s}{FS} = Q_{all} - Q_{wp}$$

Table 4.8.1.1 Elastic shortening of a single 16m pile with different diameters

L, m	16				
D, m	A, m	$E_p, \text{ kN/m}^2$	$Q_{wp}, \text{ kN}$	$Q_{ws}, \text{ kN}$	$S_{e1}, \text{ m}$
0.4	0.126	21019039	191.26	815.34	0.00363
0.5	0.196				0.00232
0.6	0.283				0.00161

$$S_{e2} = \frac{q_{wp} + D}{E_s} (1 - \mu_s^2) I_{wp}$$

Where

q_{wp} – point load per unit area at the pile point, kN/m^2

E_s – modulus of elasticity of soil, kN/m^2

μ_s – Poisson's ratio of soil

I_{wp} – influence factor = 0.85

The point load per unit area at the pile tip of D=0.4m is equal to:

$$q_{wp} = \frac{Q_{wp}}{A_p} = \frac{191.26}{0.1256} = 1521.97$$

Table 4.8.1.2 Settlement due to working load on the tip of the 16m pile

L, m	16				
D, m	q_{wp}, kN	$E_s, kN/m^2$	μ	I_{wp}	S_{e2}, m
0.4	1521.97	15000	0.4	0.85	0.02897
0.5	974.06				0.02318
0.6	676.43				0.01932

$$S_{e3} = \left(\frac{Q_{ws}}{pL}\right) \frac{D}{E_s} (1 - \mu_s^2) I_{ws}$$

Where,

p – perimeter of a pile, m

L – embedded length of a pile, m

I_{ws} – influence factor

The calculation of influence factor for each pile diameter was completed by the following formula:

$$I_{ws} = 2 + 0.35\sqrt{\frac{L}{D}}$$

Table 4.8.1.3 Settlement due to working load along the shaft of the 16m pile

L, m	16	
-------------	-----------	--

D, m	p, m	Q_{ws}	μ	$E_s, kN/m^2$	I_{ws}	S_{e3}, m
0.4	1.257	815.34	0.4	15000	4.214	0.00383
0.5	1.571				3.98	0.00362
0.6	1.885				3.807	0.00346

Table 4.8.1.4 Total elastic settlement of the 16m pile

L, m	16				
D, m	S_{e1}, m	S_{e2}, m	S_{e3}, m	S_e, m	S_e, mm
0.4	0.00363	0.02897	0.00383	0.03646	36.46
0.5	0.00232	0.02318	0.00362	0.02912	29.12
0.6	0.00161	0.01932	0.00346	0.02439	24.39

4.8.2 Consolidation settlement of a single pile

The consolidation for normally consolidated soils formula:

$$S_{c(p)} = \frac{C_c H_c}{1+e_0} \log \frac{\sigma'_0 + \Delta\sigma'_{av}}{\sigma'_0}$$

Where,

C_c – compression index

H_c – thickness of the clay layer, m

e_0 – void ratio

σ_0' – average effective pressure on the clay layer before the consolidation, kN/m^2

$\Delta\sigma_{av}'$ – average increase in effective pressure after the consolidation, kN/m^2

All input values for the consolidation settlement were obtained from the consolidation test from the geotechnical site investigation report.

The following table represents the initial consolidation settlement of a 16 m long single pile:

Table 4.8.2.1 Consolidation settlement of the 16m pile

L, m	Qu, kN	C_c	H_c , m	e_0	σ_o' , kN/m^2	$\Delta\sigma_{av}'$, kN/m^2	S_c , mm
16	6580.29	0.06	0.53	0.94	130	451.38	10.6

Table 4.8.2.2 Total settlement of 16m pile

L, m	16		
D, m	S_e	S_c	S_t
0.4	36.46	10.6	47.06
0.5	29.12	10.6	39.72
0.6	24.39	10.6	34.99

The total settlement values exceed the acceptable settlement limit of 25 mm, making the single piles unsuitable for the requirements. Due to both insufficient bearing capacity and excessive settlement, single piles were not considered as a final foundation design. Therefore, a group pile foundation was designed to provide enough bearing capacity against structural loads and do maintain the acceptable settlement.

4.9 Group Pile Design

Having determined the behavior of an individual pile, the analysis goes on to the design that will be employed in practice: a group pile foundation. Although one pile gives some understanding of the basic load transfer processes, a real-scale structural support needs the consideration of the interaction between multiple piles when placed in a shared cap. Under a group system, the spacing between the piles, overlapping of stress zones, group efficiency, and possible capacity losses are some of the factors that are critical to a system. Through the evaluation of the cumulative sense of the pile group in regard to the ultimate capacity as well as settlement we can evidently know whether the deeper soil formations can successfully carry the loads of the building when engaged in united pile activity. The calculations and design considerations used on the chosen pile group layout are presented in the following section.

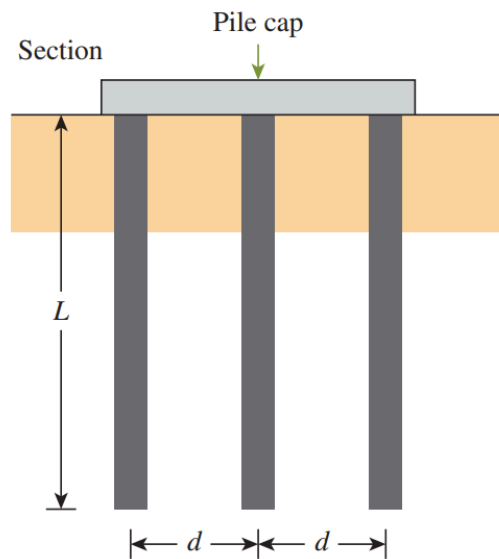


Figure 4.9.1. Group Pile Foundation (Das & Sivakugan, 2019)

4.9.1 Hand calculation of Group Pile Design

The efficiency with which a pile group resists applied loads can be defined as:

$$\eta = \frac{2(n_1 + n_2 - 2)d + 4D}{pn_1n_2}$$

Where,

η - group efficiency

n_1, n_2 - number of piles in a group

p - perimeter of pile

Load bearing capacity of group piles is determined as:

when $\eta < 1$:

$$Q_{g(u)} = \eta \sum Q_u$$

when $\eta \geq 1$:

$$Q_{g(u)} = \sum Q_u$$

Allowable bearing capacity of group pile:

$$Q_{g(all)} = \frac{Q_{g(u)}}{FS}$$

$Q_{g(u)}$ - ultimate bearing capacity

Q_u - ultimate bearing capacity of single pile

$Q_{g(all)}$ - allowable bearing capacity

The group of piles is constructed under a pile cap characterized by dimensions L_g and B_g :

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

$$B_g = (n_2 - 1)d + 2(D/2)$$

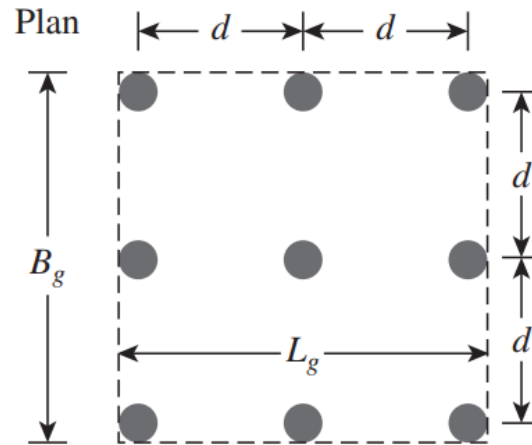


Figure 4.9.1.1. Length and Width of Group Piles and Cap (Das & Sivakugan, 2019)

There are 8 types of base column carrying different loads that need an installation of group piles. Loads on each column are shown in Table 4.9.1.1:

Table 4.9.1.1. Load Distribution for Different Column Types

Column Type	Total Load, kN
Interior Column	6580.291
Exterior Column (5 span)	4018.801
Exterior Column (20 span)	5695.253
Exterior Column (5 span, Curtain Wall)	3615.524
Corner Column Typical	3287.324
Corner Column (Curtain Wall)	3085.767

The systems of group piles are developed based on a column load to ensure overall economical structural efficiency. The piles which were tested were circular deep piles with diameter of 0.4 m and the results were recorded in Table 4.9.1.2 It is observed that the behavior of a pile group depends on the number of piles present in the arrangement and the process of computing the group bearing capacity should be changed.

Table 4.9.1.2. Load Distribution for Different Column Types

Column Type	L, m	D=0.4 m			D=0.5 m			D=0.6 m			Q_{des}, kN
		n_1	n_2	$Q_{g(all)}, kN$	n_1	n_2	$Q_{g(all)}, kN$	n_1	n_2	$Q_{g(all)}, kN$	
I-1	16	4	3	7096	3	3	6829	3	3	8976	6580
E-5	16	3	3	5843	3	2	4595	2	2	4026	4019
E-20	16	3	3	5843	3	3	6829	3	2	6040	5695
ECW-5	16	3	2	3932	3	2	4595	2	2	4026	3615
C-1	16	3	2	3932	3	2	4595	2	2	4026	3287
CCW-1	16	3	2	3932	3	2	4595	2	2	4026	3086

The dimensions of Pile Cap for each columns were calculated:

Table 4.9.1.3. Dimensions of Pile Caps

Column	L_g, m	B_g, m
I-1	4.0	2.8
E-5	2.8	2.8
E-20	2.8	2.8
ECW-5	2.8	1.6
C-1	2.8	1.6
CCW-1	2.8	1.6

The cost of using circular piles, whose diameter equals $D = 0.4$ m, would be economical as compared to using $D = 0.5$ m and $D = 0.6$ m since a smaller diameter of the piles would yield lesser length of the pile and fewer piles per group. This choice of suitable pile group arrangements to each column loading is hence a good strategy to ensure that the total cost of the foundation is minimized. Figure 4.9.1.1 provides the initial design and size of the pile groups.

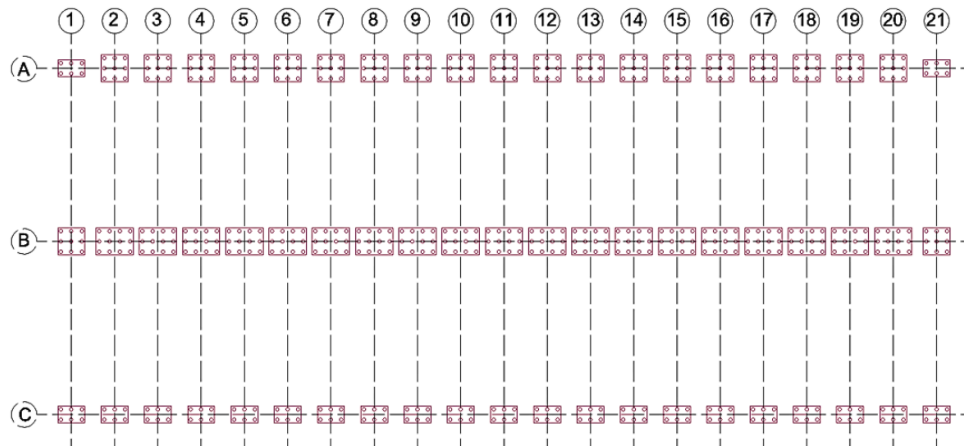
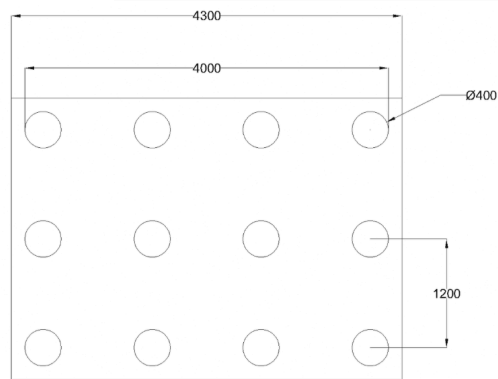
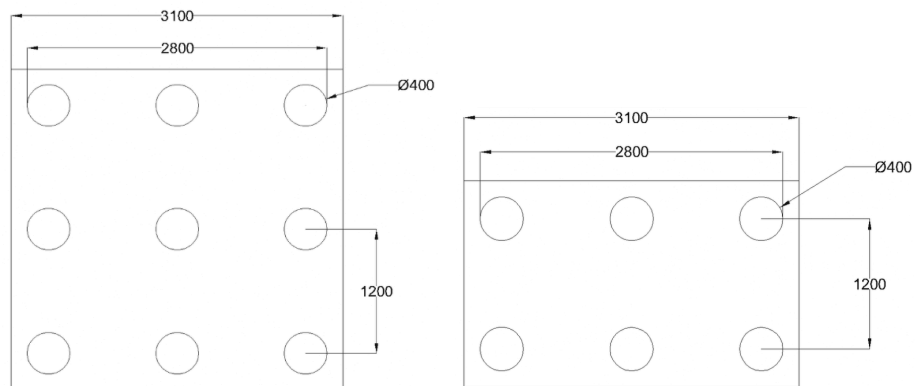


Figure 4.9.1.2. Group Piles Layout

All group piles dimensions and design:



(a)



(b)

(c)

Figure 4.9.1.3. (a) 4x3, (b) 3x3, (c) 3x2 Group Piles**4.9.2. Software Calculation and Comparison of Group Piles with Hand Calculations****Table 4.9.2.1** Comparison of Group Piles on Software values and Hand Calculation values

	Interior Piles		Exterior Piles		Corner Piles	
	Hand Calculation	Software Calculation	Hand Calculation	Software Calculation	Hand Calculation	Software Calculation
Qu (single), kN	3019.78	6580	3019.78	4207.82	3019.78	3615.52
Qu (group), kN	14192	16065.3	11686	12338.73	7864	8612.15
FS	2	2.33	2	2.09	2	2.3
ng	0.99	0.71	1.17	0.73	1.27	76
Qall (group), kN	7096		5843		3932	
Vd, kN	6580	6886.59	5695	5916.03	3615.52	3750.47
Se, mm	14.85	36.6	13.75	44.7	10.47	32.9

Analysis of bearing capacity of pile group in cohesionless soils

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

Pile skin bearing capacity $R_s = 189,62 \text{ kN}$ Pile base bearing capacity $R_b = 1696,46 \text{ kN}$ Vertical bearing capacity of single pile $R_c = 1886,08 \text{ kN}$ Efficiency of pile group $\eta_g = 0,71$ Vertical bearing capacity of pile group $R_g = 16065,30 \text{ kN}$ Maximum vertical force $V_d = 6886,59 \text{ kN}$

Safety factor = 2,33 > 2,00

Vertical bearing capacity of pile group is SATISFACTORY

Analysis of settlement of pile group in cohesionless soils

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

Group settlement factor $g_f = 2,78$
 Load at the onset of mobilization of skin friction $R_{yu} = 3245,60 \text{ kN}$
 The settlement for the force R_{yu} $s_y = 7,2 \text{ mm}$
 Total resistance $R_c = 8540,94 \text{ kN}$
 Maximum settlement $s_{lim} = 50,0 \text{ mm}$
 The settlement for maximum service load $V = 6886,59 \text{ kN}$ is $36,6 \text{ mm}$.

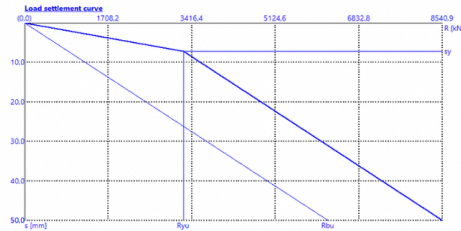


Figure 4.9.2.1 4x3 Piles Group Bearing Capacity Check

Analysis of bearing capacity of pile group in cohesionless soils

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

Pile skin bearing capacity $R_s = 189,62 \text{ kN}$
 Pile base bearing capacity $R_b = 1696,46 \text{ kN}$
 Vertical bearing capacity of single pile $R_c = 1886,08 \text{ kN}$
 Efficiency of pile group $\eta_g = 0,73$
 Vertical bearing capacity of pile group $R_g = 12338,73 \text{ kN}$
 Maximum vertical force $V_d = 5916,03 \text{ kN}$
 Safety factor = $2,09 > 2,00$

Vertical bearing capacity of pile group is SATISFACTORY

Analysis of settlement of pile group in cohesionless soils

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

Group settlement factor $g_f = 2,78$
 Load at the onset of mobilization of skin friction $R_{yu} = 2434,20 \text{ kN}$
 The settlement for the force R_{yu} $s_y = 7,2 \text{ mm}$
 Total resistance $R_c = 6405,71 \text{ kN}$
 Maximum settlement $s_{lim} = 50,0 \text{ mm}$
 The settlement for maximum service load $V = 5916,03 \text{ kN}$ is $44,7 \text{ mm}$.

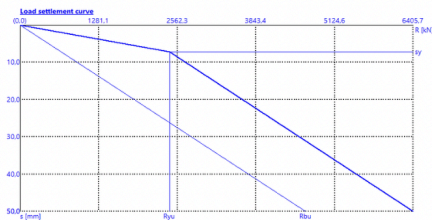


Figure 4.9.2.2 3x3 Pile Group Bearing Capacity

Analysis of bearing capacity of pile group in cohesionless soils

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

Pile skin bearing capacity	$R_s = 189,62 \text{ kN}$
Pile base bearing capacity	$R_b = 1696,46 \text{ kN}$
Vertical bearing capacity of single pile	$R_c = 1886,08 \text{ kN}$
Efficiency of pile group	$\eta_g = 0,76$
Vertical bearing capacity of pile group	$R_g = 8612,15 \text{ kN}$
Maximum vertical force	$V_d = 3750,47 \text{ kN}$
Safety factor	$= 2,30 > 2,00$

Vertical bearing capacity of pile group is SATISFACTORY

Analysis of settlement of pile group in cohesionless soils

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

Group settlement factor	$g_f = 2,18$
Load at the onset of mobilization of skin friction	$R_{yu} = 1622,80 \text{ kN}$
The settlement for the force R_{yu}	$s_y = 5,6 \text{ mm}$
Total resistance	$R_c = 5085,37 \text{ kN}$
Maximum settlement	$s_{lim} = 50,0 \text{ mm}$
The settlement for maximum service load $V = 3750,47 \text{ kN}$ is 32,9 mm.	

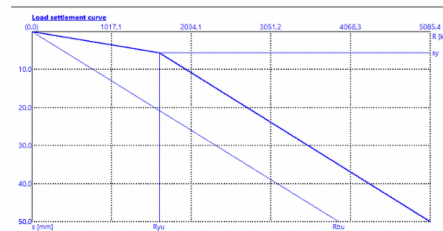


Figure 4.9.2.3 3x2 Pile Group Bearing Capacity

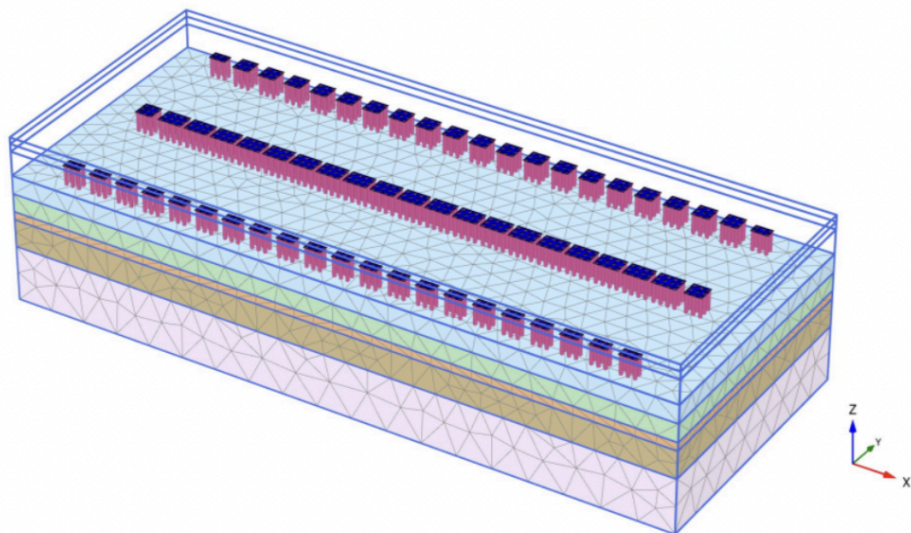


Figure 4.9.2.4 Group Pile Location in Plaxis

The bearing capacity check was done in Geo5 to compare a single pile with pile groups (4×3, 3×3 and 3×2). From the results, all pile group arrangements were satisfactory because the ultimate capacity was higher than the applied load, so the factor of safety was within the acceptable range. However, the single pile result was unsatisfactory since its capacity was not enough to resist the load safely. This shows that pile groups perform better because the load is shared between several piles, reducing stress on each pile. Overall, group of piles gives more stability and safer foundation compared to single pile.

4.9.3 Plaxis Assessment of Seismicity

The main purpose of this analysis is to assess the local site effects. This means assessing how the stratigraphic layer of the Nazarbayev University site changes the seismic waves (amplitude, frequency content and duration) from the bedrock to the surface.

The Acceleration Response Spectrum indicates the maximum response of a single-degree-of-freedom (SDOF) system. The peak at short periods ($T < 0.5$ s) means high Pseudo-Spectral Acceleration (PSA). This is a key design factor for low-rise structures with stiff structural systems whose natural periods may resonate with this peak.

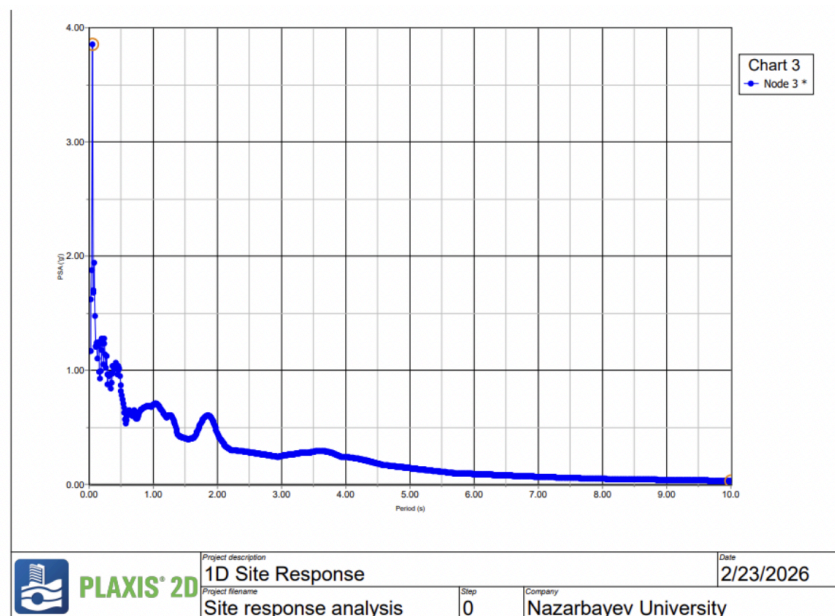


Figure 4.9.3.1 Response Spectra Plot

The Fourier transform breaks down the time-history into the constituent frequencies. The peaks in this range (0.5-2.0 Hz) show the dominant frequencies at the site. This gives an insight into the frequency-specific amplification of the soil deposits.

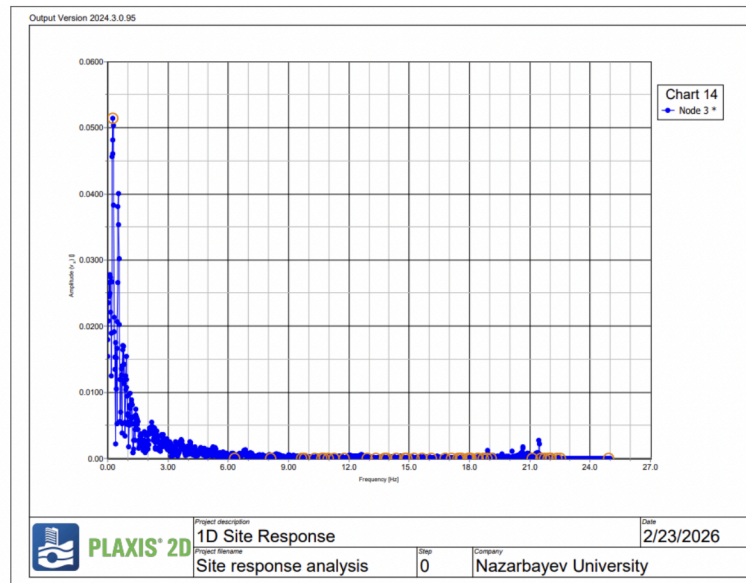


Figure 4.9.3.2 Fourier Spectra Plot

Arias Intensity is a measure of the energy of the accelerogram. The Significant Duration of the earthquake is represented by the change in slope between 5 and 15 seconds. This is an important factor for determining the potential for liquefaction and for permanent deformations.

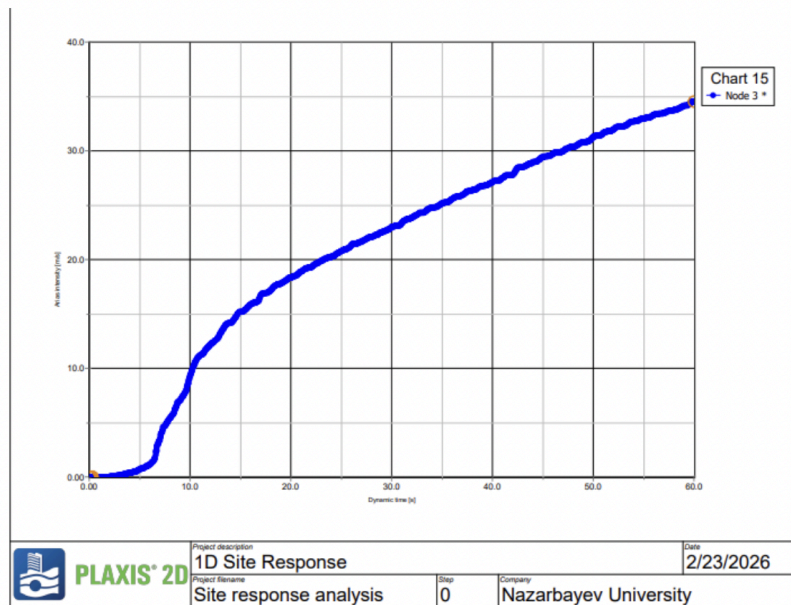


Figure 4.9.3.3 Arias Intensity Plot

The accelerogram offers the surface Peak Ground Acceleration (PGA). Here, the PGA is about 0.7g. The graph illustrates the time-history of the acceleration, which is crucial for non-linear dynamic analysis.

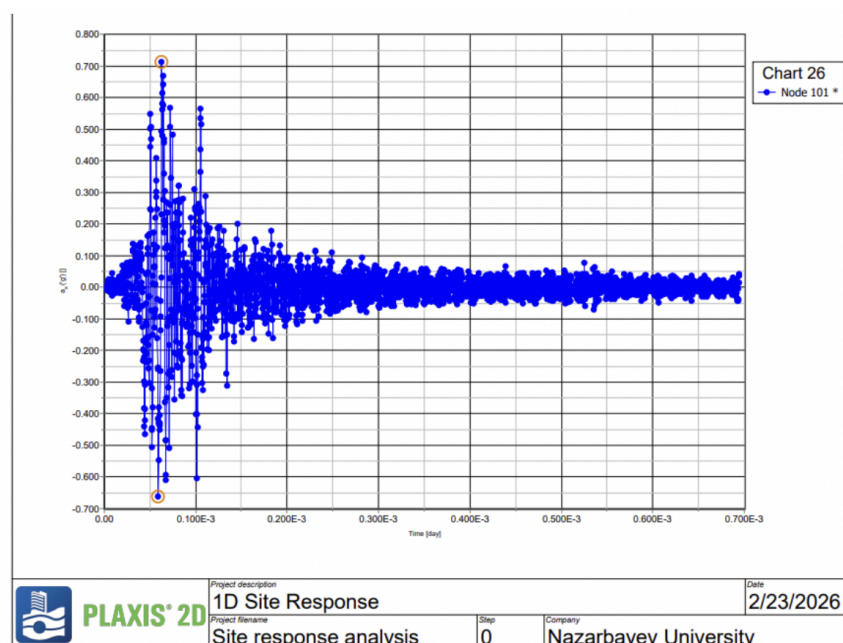


Figure 4.9.3.4 Signal Plot

Using the Site Response Analysis for PLAXIS 3D

In our project, the outcomes of this 1D Site Response Analysis were used as the key input for the further 3D structural analysis:

- **Seismic Load Input:** The acceleration time-history (Signal Plot) from the 2D analysis at the ground surface was used and applied as a prescribed dynamic displacement/multiplier in PLAXIS 3D.
- **Dynamic Multipliers:** The spectral response and the peak ground accelerations (PGAs) were used to scale the seismic loads for the 3D model to ensure that the complex soil-structure interaction (SSI) was simulated using an actual site-specific earthquake.
- **Validation:** By analysing first, we could ensure that the boundary conditions and soil damping values in our 3D model were properly adjusted to account for the amplified site effects.
- Moreover, we used the outcomes of the 1D analysis in our PLAXIS 3D model. We then used the accelerograms at the surface to specify the Seismic Load (Dynamic Multipliers). This enabled us to conduct a more realistic Soil-Structure Interaction analysis, since the foundation and the superstructure in the 3D model will be subjected to the exact motions amplified by our stratigraphy.

4.10 Group Pile Settlement

4.10.1 Hand Calculation of Group Pile's Settlement

Single pile behavior was used to determine the elastic and consolidation components used to determine the settlement of a pile group after taking into account the effects of group. The process of calculating is outlined in the following formulas and the parameters. Every equation is supported by explanation.

Elastic Settlement of the Group Piles:

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

Where,

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

B_g - width of group pile section

s_e - elastic settlement of each pile at comparable working load

D - width or diameter of each pile in the group

The elastic settlement of each pile at comparable working load was calculated by this formula:

$$s_e = s_{e(1)} + s_{e(2)} + s_{e(3)}$$

Where,

$s_{e(1)}$ - elastic shortening of the pile

$s_{e(2)}$ - settlement of a pile caused by the load carried at the pile point

$s_{e(3)}$ - settlement of a pile caused by the load carried by the pile shaft

All methods and formulas for elastic settlement of single pile were described and written in section 4.8.1

Consolidation Settlement of the Group Piles:

Consolidation settlement is estimated for each compressible soil layer beneath the group.

The increase in effective stress caused at the middle of each soil layer by the load Q_g :

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

Where,

$\Delta\sigma'_i$ - increase in effective stress at the middle of layer i

L_g, B_g - length and width

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

The consolidation settlement of each layer caused by the increased stress was calculated. The formula is:

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

Where,

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

$\Delta e_{(i)}$ - change of void ratio caused by the increase in stress in layer i

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

H_i - thickness of layer i

For normally consolidated clays, the following formula was used:

$$\Delta s_{e(i)} = \left[\frac{C_{c(i)} H_i}{1 + e_{0(i)}} \right] \log \left[\frac{\sigma'_{e(i)} + \Delta \sigma'_{(i)}}{\sigma'_{e(i)}} \right]$$

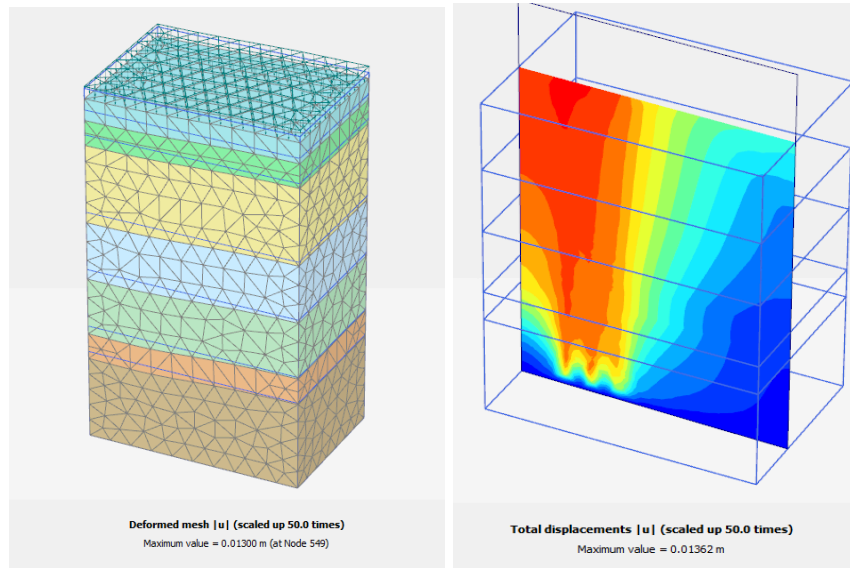
Table 4.10.1.1. Settlement of Group Piles

Single Pile				
	Elastic shortening of the pile, mm	Settlement at the pile point, mm	Settlement along the pile shaft, mm	Total Elastic Settlement, mm
Pile	1.088	3.223	2.882	7.193
Group Pile				
Column	$n_1 \times n_2$	Elastic	Consolidation	Total Settlement,

		Settlement, mm	Settlement, mm	mm
I-1	4 × 3	9.258	5.593	14.851
E-5	3 × 3	9.258	4.496	13.754
E-20	3 × 3	9.258	6.299	15.557
ECW-5	3 × 2	4.179	6.286	10.465
C-1	3 × 3	4.179	6.498	10.677
CCW-1	3 × 2	4.179	5.854	10.033

Due to the settlement analysis and comparison of alternative foundations systems, the group pile foundation was found as the most viable and certain solution to the project. The elastic as well as the consolidation aspect of settlement is kept in control with the acceptable range of group configurations evaluated and the deeper load transfer effectively addresses the observed undesirable deformation of shallow foundation alternatives. The design attains the necessary structural performance at a reasonable cost by design with pile groups choice depending on the load on the column itself. In totality, the group pile foundation offers a consistent and stable system of support that complies with the geotechnical living of the location, as well as the building functionality requirements.

4.10.2 Software Calculation of Group Pile's Settlement with Plaxis



(a)

(b)

Figure 4.10.2.1 Results of Plaxis 3D analysis (a) general view (b) contour plot of vertical settlement

The table of results of the PLAXIS 3D analysis of the pile foundation system can be found in figure 4.13.1. Figure 4.13.1(a) indicates deformed finite element mesh of the soil profile when it is under loading, and the magnification of deformation is 50 times in order to clarify the graph. The diagram shown in Figure 4.13.1(b) shows the contour plot of total vertical displacement; this shows how the mass of soil was distributed in getting the settlement. The highest settlement, which is attained as a result of the analysis is about 0.013 m and this reflects good performance of the foundation system within the loading conditions applied.

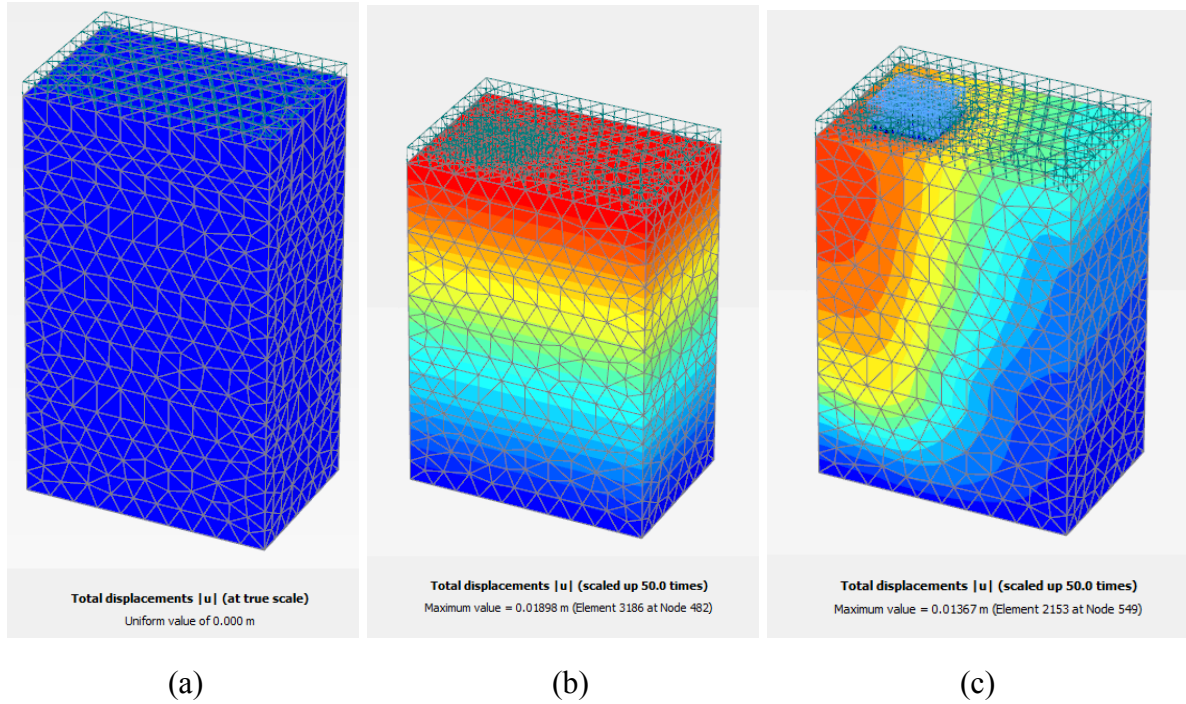


Figure 4.10.2.2 Total displacement in Plaxis 3D (a) before excavation and applying load (b) after excavation without applying load (c) after excavation with load

The total displacement analysis of the PLAXIS 3D analysis was plotted in figure 4.13.2 at various stages of construction. The first situation is Figure 4.13.2(a) which is the pre excavation and loading situation where no movement is noted. The displacement distribution at excavation without the application of structural load is presented in figure 4.13.2(b). It can be seen in figure 4.13.2(c) that there is greater deformation adjacent to the foundation area due to the displacement following excavation and load application. The data illustrates how excavation and loading impact the deformation and settlement of the soil.

Embedded beam - Pile

General

Mechanical

Property	Unit	Value
Material set		
Identification		Pile
Material type		Elastic
Colour		RGB 199, 82, 143
Comments		
Unit weights		
γ	kN/m ³	24.00
Rayleigh damping		
Input method		SDOF equivalent
Rayleigh α		0.000
Rayleigh β		0.000
ξ_1	%	0.000
ξ_2	%	0.000
f_1	Hz	0.1000
f_2	Hz	1.000

Embedded beam - Pile

General

Mechanical

Property	Unit	Value
Properties		
Cross section type		Predefined
Predefined cross section type		Solid circular beam
Diameter	m	0.4000
A	m ²	0.1257
I_2	m ⁴	1.257E-3
I_3	m ⁴	1.257E-3
Stiffness		
E	kN/m ²	30.00E6
Axial skin resistance		
Axial skin resistance		Linear
$T_{skin, start, max}$	kN/m	680.6
$T_{skin, end, max}$	kN/m	680.6
Base resistance		
F_{max}	kN	33.48E3

Figure 4.10.2.3 Properties of Pile (Embedded beam)

Figure 4.13.3.3 indicates the material and mechanical properties that the pile will have in the PLAXIS 3D system as an embedded beam. The pile was characterized as a solid circular section of diameter 0.4 m and elastic behavior.

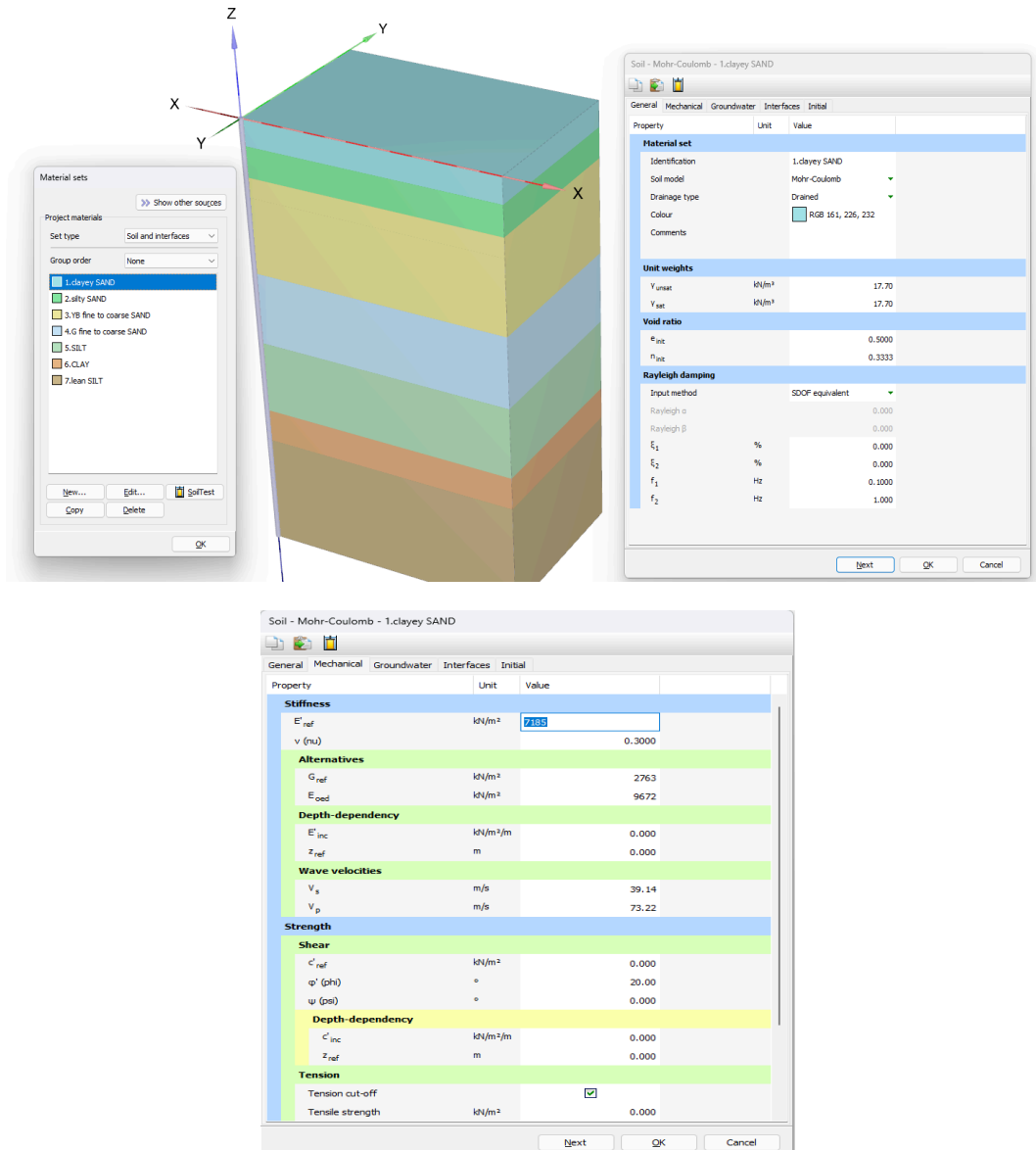


Figure 4.10.2.4 Properties of soil layer as example

The distribution of material properties into a layer of soil in PLAXIS 3D using the Mohr Coulomb model is explained in Figure 4.13.4 and demonstrates the determined parameters of stiffness, strength, and unit weight of the chosen sand layer.

4.10.3 Software Calculation of Foundation Settlement with Plaxis

This section of the report offers a detailed technical assessment of the settlement response for the entire foundation of the proposed building, building on the initial group-pile design calculations. While preliminary calculations were based on individual groups, in this analysis the complete

interaction of the superstructure, the composite pile foundation, and the multi-layered soil structure is taken into account. The analysis, performed in the PLAXIS 3D finite element software, assesses the stress-strain behavior over a meticulous 13-step construction sequence, from the initial greenfield state to the final static and seismic load conditions, in which soil deformations are tracked. The end result of this study suggests a maximum total settlement of 0.01895 m, which is a reflection of the load redistribution within large foundation structures.

Pile, Raft and Retaining Wall Elements

The piles are modeled as embedded beams. The raft and retaining walls are modelled as plates. Plates in PLAXIS 3D are 6-node surface elements with both bending and translational degrees of freedom. The raft is given a certain thickness and stiffness to spread the loads from the superstructure onto the pile groups. Modelling the raft as a rigid/semi-rigid plate allows the evaluation of differential settlements over the building footprint, which is essential to understand the risk of structural tilting or cracking of the brick infill.

The lateral support during the excavation is provided by sheet piles and reinforced concrete wall. The contact between the retaining walls and the soil is modeled using interface elements that permit the occurrence of gapping and slip at the interface.

Construction Procedure

The modeling of the construction phases starts with excavation:

- Phase 1 (Excavation): Mass excavation down to the basement level is conducted. Soil excavation will produce a stress relief, resulting in the heave of the pit base.
- Phase 2 (Retaining Wall): The permanent retaining walls are installed to maintain the stability of the excavation boundaries.
- Phase 3 to 5 (Pile A, B, and C): The piles are installed in three stages. This enables the model to simulate the step-wise changes in the stiffness and stress transfer within the soil profile as the deep foundation elements are engaged.

The installation of the foundation piles is followed by the completion of the foundation structure:

- Phase 6 (Pile Cap): The pile caps are activated, which groups the piles together and prepares for the slab activation.
- Phase 7 (Slab): The raft foundation (slab) is turned on. Here, the plate elements that represent the basement floor are activated, and the raft is allowed to interact with the soil.

- Phase 8 (Loads): The dead loads and live loads are applied. This is usually the phase where the largest settlement is observed, due to the transfer of weight.
- Phase 9 (Seismic Loads): A dynamic analysis is carried out to assess the foundation's behaviour under seismic loads. This involves the activation of dynamic boundary conditions (Free-field and Compliant base) to simulate the dynamic wave propagation.

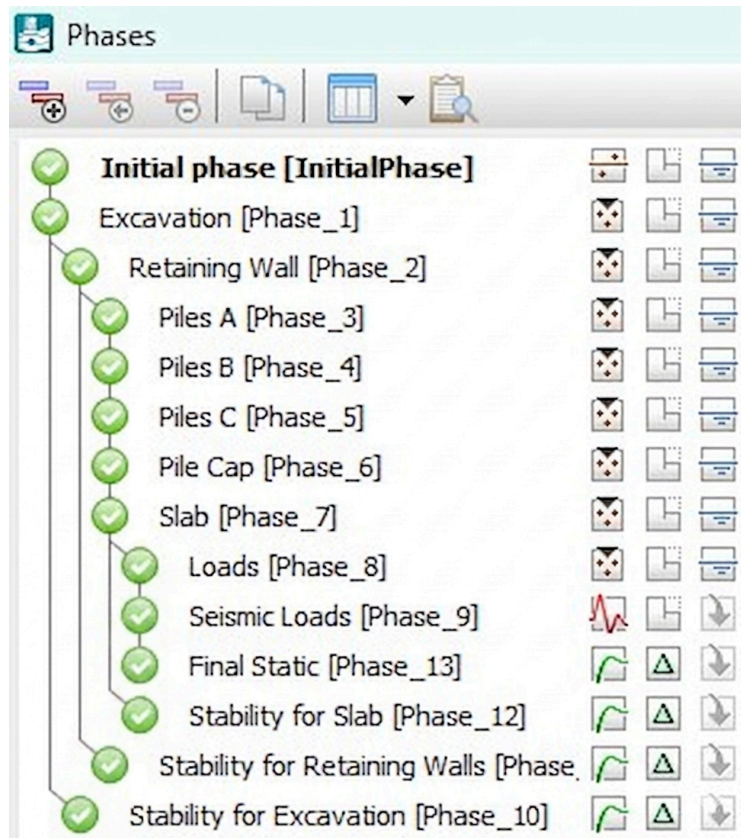


Figure 4.10.3.1 Construction Procedure

Analysis of Foundation Settlement Results

The highest value of settlement (0.01895 m) occurs at the center of the raft, where Pile Group A, B and C effects are the strongest. This is an increase on the 0.013 m found in the single group pile case study and shows the effect of the bigger loaded area and the effect of interaction with other groups.

The color map shows the settlement changing from red (max) at the center to blue (min) towards the boundaries. The settlement pattern indicates a semi-rigid foundation system. The settlement difference between the raft center and corners is about 6.5 mm, which is below the allowable level to avoid damage to the superstructure.

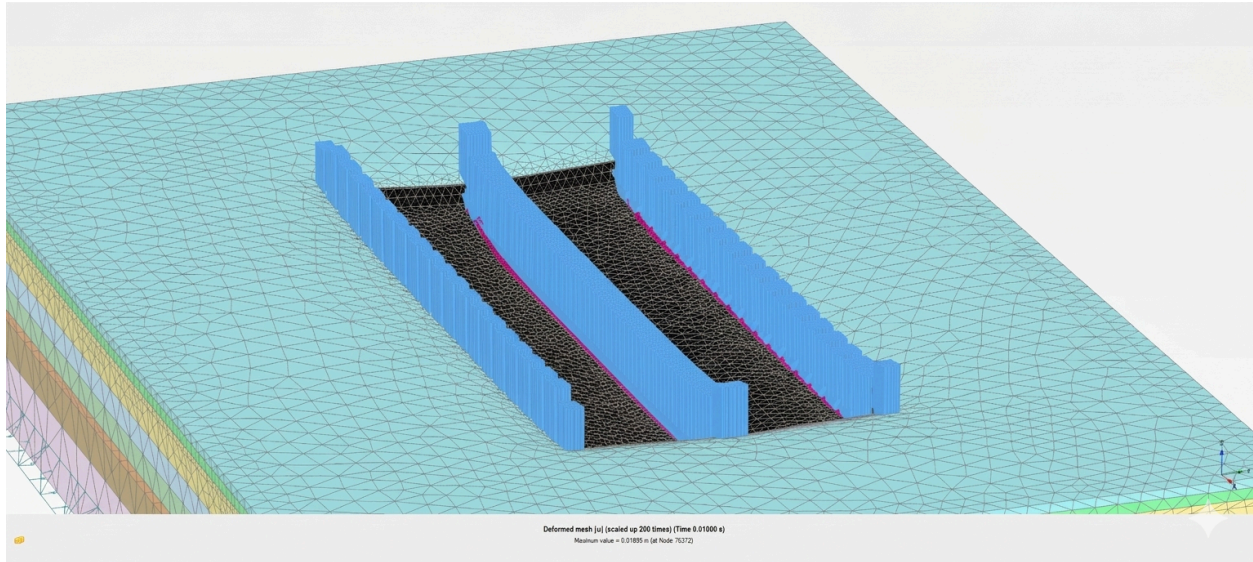


Figure 4.10.3.2 Deformation Analysis in Plaxis 3D

Dynamic Settlement and Pore Pressure

During the seismic event, there is a potential for temporary excess pore water pressure increase in the silty sand layers. While the condition is set as "Drained" for the major phases, the dynamic analysis assesses the possibility of "shakedown" or cyclic softening settlement. The amplified total displacement to 0.01895 m accounts for the permanent settlement due to this earthquake. The free-field elements on the sides and compliant base at the bottom of the model ensure that the energy from the earthquake is not confined within the model, which results in a realistic estimate of dynamic settlement.

Safety Factors and Global Stability

The stability phases (Phases 10-12) use the strength reduction method to determine the critical failure mechanism. The global factor of safety (FOS) for the final state was estimated to be more than 1.5, implying a high degree of safety against global ground failure. The results show that the retaining walls and the slab are safe even under final static loading phase conditions.

Comparative Analysis with Analytical Methods

The theoretical prediction (18.95 mm) was compared with the analytical methods used in the preliminary design. Analytical methods, like Vesic, assume that the soil is a uniform half-space and may overestimate settlement due to the absence of the raft effect and the non-linear behaviour of the soil.

Table 4.10.3.1 Hand and Plaxis 3D Settlement Comparison

Method	Settlement	Variation from Plaxis3D
Hand	22.40 mm	+18.2%
Plaxis 3D	18.95 mm	

The Plaxis 3D analysis is more economical and more accurate because it takes into account the 3D geometry of the excavation and the installation of the piles in stages. The 2% range of values reported by classmates for smaller group sizes (3x3) suggests that hand calculations are good for preliminary checks but not precise enough when designing a "whole building" raft foundation where the group-group interactions are the critical issue.

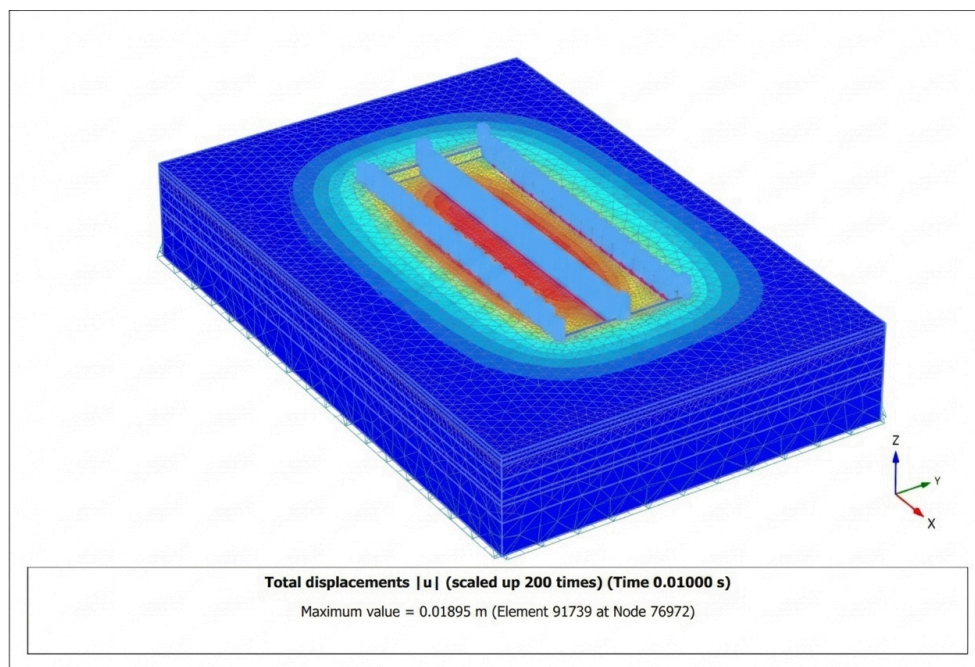


Figure 4.10.3.3 Deformation Analysis in Plaxis 3D

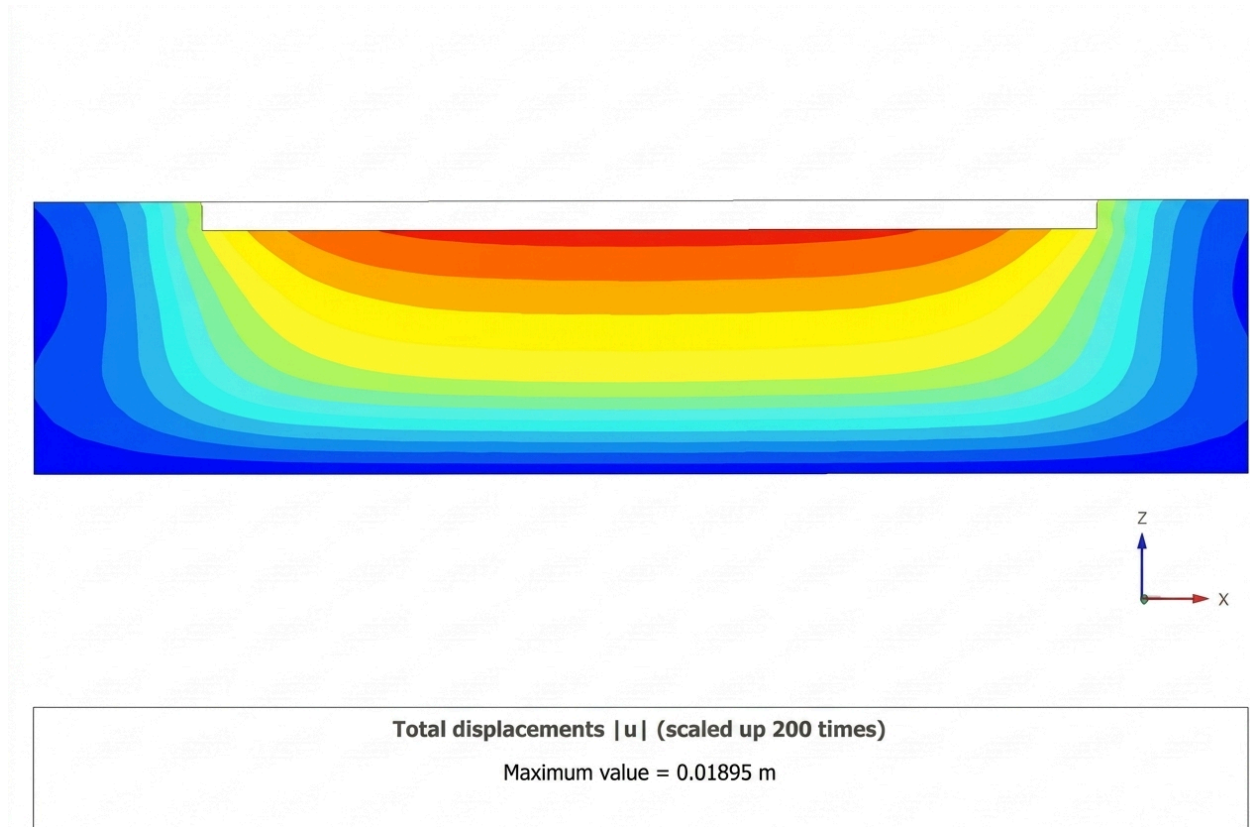


Figure 4.10.3.4 Cross Section of Total Displacement in Plaxis 3D

The Plaxis 3D model, with a detailed 13-phase construction sequence, verifies the foundation design. The predicted maximum settlement of 0.01895 m is below serviceability limits, proving sufficient load transfer to the silty sand soil. Safety calculations for all phases confirm a satisfactory factor of safety, leading to the conclusion that the pile-raft foundation is adequate and optimised for the site conditions.

4.11 Foundation Design Under Lateral Loading

After designing the foundation using the axial loadings it was crucial to ensure that the selected foundation design is able to withstand the lateral loads from the earthquake and wind. For this analysis the maximum lateral loads were taken from the SAP2000 software for 6 different column types, as shown in the table.

Table 4.11.1 Lateral loads

Column type	#of piles	$Q_{g(seismic)}$, kN	$Q_{g(seismic)}$ per pile, kN	$Q_{g(wind)}$, kN	$Q_{g(wind)}$ per pile, kN
I-1, 19	4x3=12	541.56	45.13	43.69	3.64
E-5, 19	3x3=9	751.82	83.54	447.76	49.75
E-20, 2	3x3=9	673.87	74.87	93.94	10.44
ECW-5, 19	3x2=6	751.82	125.3	447.76	74.63
C-1, 2	3x2=6	624.7	104.12	290.26	48.38
CCW-1, 2	3x2=6	697.61	116.27	293.08	48.85

4.11.1 Hand calculation of lateral bearing capacity

The lateral bearing capacity was calculated using the method of Broms and was done for the entire pile length excluding the basement excavation depth. Since the diameter of a single pile in a group pile for each column type is equal to 0.4m, the hand calculation was carried out only once for this pile diameter. Firstly, the length category (long/rigid) should be determined. The following tables represent the decision on the pile category.

Table 4.11.1.2 Modulus of subgrade reaction

Layer #	Depth m	N	Density	$n_h (avg)$
1	1.2	0	loose	2000
2	2.6	18	medium	6250
3	7.0	22	medium	6250
4	10.3	23	medium	4000
5	13.6	9	loose	1400
6	15.1	8	loose	1400
7	17.5	32	dense	10500

8	20.1	44	dense	10500
---	------	----	-------	-------

The value for $n_h(avg)$ was determined from Table 12.16 from Das & Sivakugan (2019) by comparing the density of each soil layer based on the SPT N values.

Table 4.11.1.3 Determination of pile type

Layer #	Depth, m	$n_h(avg)$	$E_p, kN/m^2$	I_p, m^4	T	$\frac{L}{T}$	Long/rigid
1	1.2	2000	0.4	0.001256	2.64	6.1	Long
2	2.6	6250	0.4	0.001256	0.85	18.94	Long
3	7.0	6250	0.4	0.001256	0.85	18.94	Long
4	10.3	4000	0.4	0.001256	1.32	12.12	Long
5	13.6	1400	0.4	0.001256	3.11	5.14	Long
6	15.1	1400	0.4	0.001256	3.11	5.14	Long
7	17.5	10500	0.4	0.001256	0.5	31.82	Long
8	20.1	10500	0.4	0.001256	0.5	31.82	Long

The moment of inertia for a circular pile with a diameter of 0.4m was calculated using this formula:

$$I_p = \frac{\pi d^4}{64} = \frac{\pi \times 0.4^4}{64} = 0.001256 m^4$$

The formula of characteristic length of the soil-pile system:

$$T = \sqrt[5]{\frac{E_p I_p}{n_h}}$$

Since L/T is greater than 5 for various depths, the pile is considered to be a long pile, thus the analysis of Brom's for long piles can be used when calculating the lateral resistance of the pile and pile deflection.

For the calculation of pile ultimate lateral resistance Rankine's passive earth pressure constant should be calculated.

$$K_p = \tan^2(45 + \frac{\phi'}{2})$$

Table 4.11.1.4 Rankine's passive earth pressure constant for each layer

Layer #	Friction angle, °	K
1	20.00	2.04
2	35.71	3.8
3	36.67	3.96
4	35.75	3.81
5	29.56	2.95
6	29.22	2.91
7	38.31	4.26
8	40.82	4.78

The yield moment for the pile was calculated using the formula:

$$M_y = SF_y = \frac{I_p}{\frac{D}{2}} F_y = \frac{0.001256}{\frac{0.4}{2}} 248000 = 1557.44 \text{ kNm}$$

Where,

S – section modulus of pile section

F_y – yield stress of the pile taken as 248 MN/m^2

$$\frac{M_y}{D^4 \gamma K_p} = \frac{1557.44}{0.4^4 * 17.7 * 2.04} = 1684.88$$

This relationship was used to identify the ultimate lateral resistance of a pile using a Figure 12.42a from Das & Sivakugan (2019). For the calculated $\frac{M_y}{D^4 \gamma K_p}$ the corresponding value of

$\frac{Q_{u(g)}}{D^3 \gamma K_p}$ was found from the figure, approximately equal to 120 for the $\frac{M_y}{D^4 \gamma K_p} = 1684.88$. Thus,

the ultimate lateral resistance formula is:

$$Q_{u(g)} = 120 \times (D^3 \gamma K_p)$$

$$Q_{u(all)} = \frac{Q_{u(g)}}{FS}$$

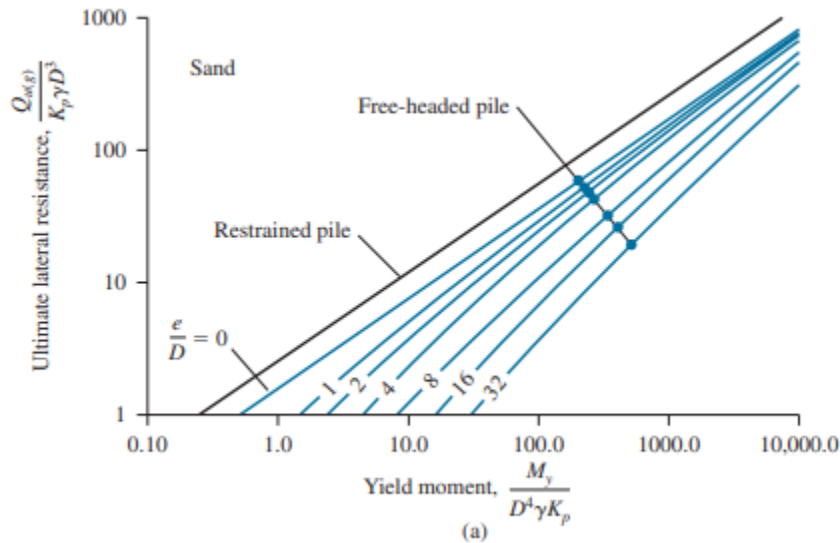


Figure 4.11.1.1 Cross Section of Total Displacement in Plaxis 3D

Table 4.11.1.5 Lateral resistance values

Layer #	M_y , kNm	Unit Weight, kN/m^2	$\frac{M_y}{D^4 \gamma K_p}$	$\frac{Q_{u(g)}}{D^3 \gamma K_p}$	$Q_{u(g)}$, kN	$Q_{u(all)}$, kN
1	1557.44	17.7	1684.88	120	277.26	92.42

2	1557.44	18.5	864.59	85	382.79	127.59
3	1557.44	19.2	799.16	80	389.77	129.92
4	1557.44	19.5	818.88	79	375.63	125.21
5	1557.44	18.9	1091.94	108	385.1	128.37
6	1557.44	18.4	1137.19	112	383.47	127.82
7	1557.44	20.1	710.3	72	394.68	131.56
8	1557.44	20.4	624.41	66	411.55	137.18

Table 4.11.1.6 Comparing lateral forces with pile lateral resistance

Column types	$Q_{g(seismic)}$ per pile, kN	$Q_{g(wind)}$ per pile, kN	Qu all, kN
I-1, 19	45.13	3.64	137.18
E-5, 19	83.54	49.75	137.18
E-20, 2	74.87	10.44	137.18
ECW-5, 19	125.3	74.63	137.18
C-1, 2	104.12	48.38	137.18
CCW-1, 2	116.27	48.85	137.18

According to these results, the allowable bearing capacity of a 0.4 m diameter pile is greater than the applied seismic and wind loads. Thus each pile group has enough capacity to resist the lateral loads.

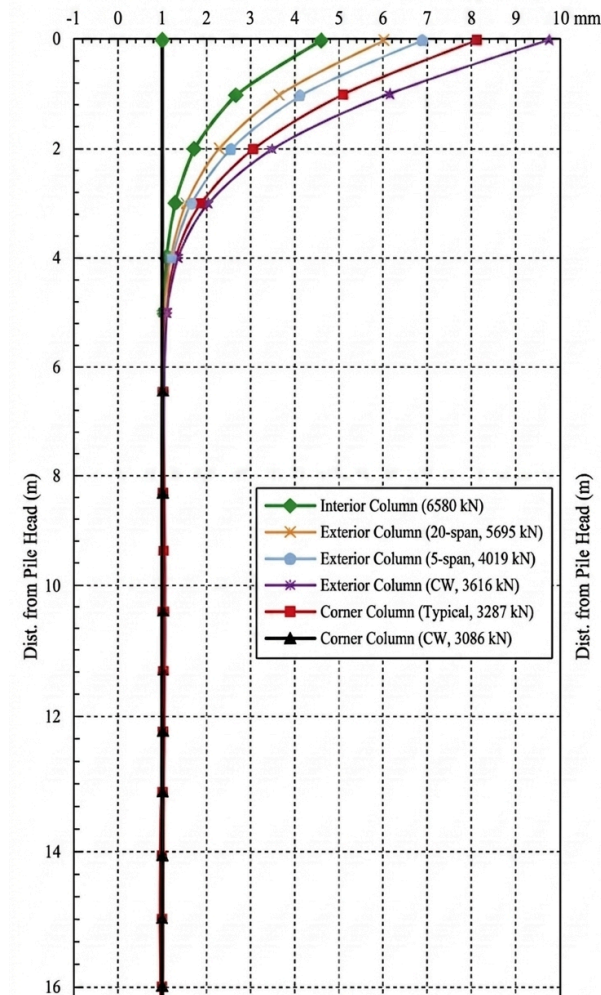


Figure 4.11.1.2 Pile Deflection

4.12 Lateral Active and Passive Pressures

The construction of a one-story basement requires excavation to a depth of approximately 3.5 m. Therefore, a proper design of retaining walls is required to resist lateral earth pressure by soils. Consequently, sand layers will be liable to the lateral earth pressure. Rankine Theory was used for determining active and passive earth pressure coefficients based on the friction angle (ϕ).

4300

The coefficients are stated as:

$$K_a = \tan^2\left(45 - \frac{\phi'}{2}\right)$$

$$K_p = \tan^2(45 + \frac{\phi'}{2})$$

Where,

K_a -active earth pressure coefficient

K_p -passive earth pressure coefficient

Rankine pressure formulas are given by:

$$\sigma_a' = \sigma_0' K_a - 2c'\sqrt{K_a}$$

$$\sigma_p' = \sigma_0' K_p - 2c'\sqrt{K_p}$$

$$\sigma_0' = \gamma H$$

Where,

σ_a' - Rankine active earth pressure

σ_p' - Rankine passive earth pressure

σ_0' - Effective stress at $z = H$

To calculate the force per unit length of the wall following formulas were used:

$$P_a = \frac{1}{2}\gamma H^2 K_a - 2c'H\sqrt{K_a}$$

$$P_p = \frac{1}{2}\gamma H^2 K_p - 2c'H\sqrt{K_p}$$

Where,

P_a - active force per unit length

P_p - passive force per unit length

Since the sand layers are non cohesive soils, we assume they have $c'=0$. Table 4.7.1 represents the calculated active and passive earth pressures.

Table 4.12.1 Lateral Active Pressure

Depth	σ_0' , kN/m ²	K_a	σ_a' , kN/m ²	P_a , kN
0	0	0.39	0	5.24
1.23+	21.771	0.39	8.52	
1.23-	21.771	0.23	4.97	11.41
1.99+	36.815	0.23	8.40	
1.99-	36.815	0.25	9.09	24.48
2.6+	48.1	0.25	11.87	
2.6-	48.1	0.21	10.14	40.46
3.67+	70.464	0.21	14.86	

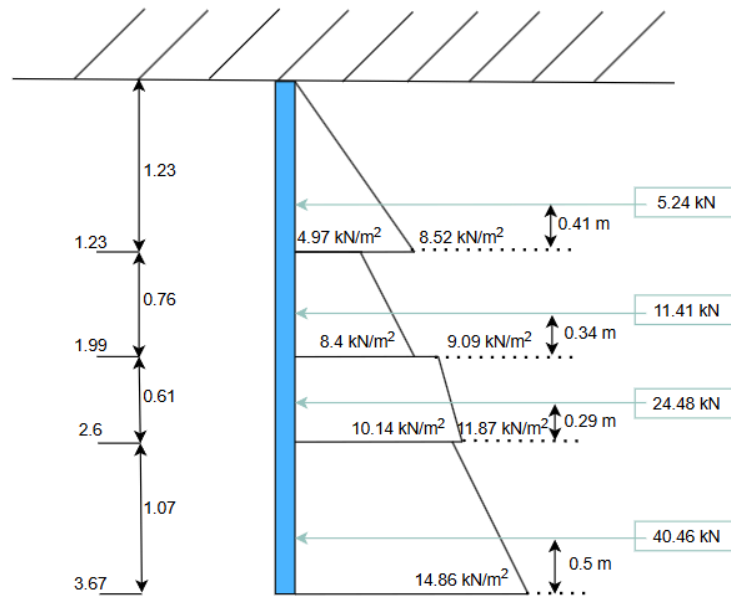


Figure 4.12.1 Lateral Active Pressure

Table 4.12.2 Lateral Passive Pressure

Depth	σ_0' , kN/m ²	K_p	σ_p' , kN/m ²	P_p , kN
2.6-	48.1	4.74	228.06	1146.227
3.67+	70.464	4.74	334.09	
3.67-	70.464	4.74	290.56	1718.158
5.5+	104.5	4.12	430.90	
5.5-	104.5	4.12	493.86	3407.566
7.02+	123.552	4.12	583.89	

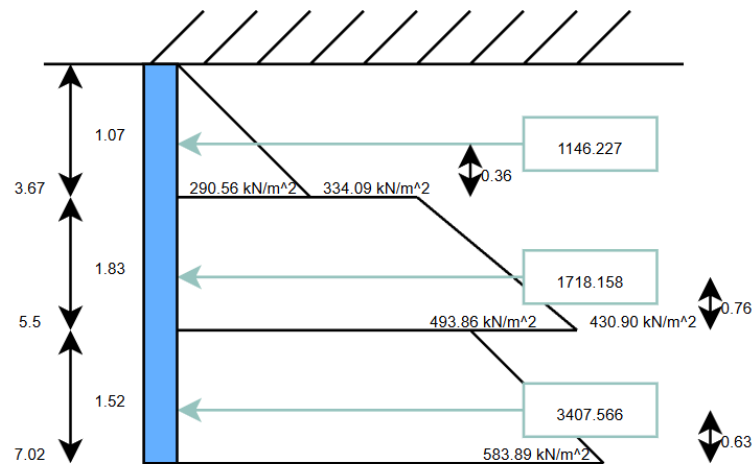


Figure 4.12.2 Lateral Passive Pressure

Based on the comments, the fixing works have been done for the depth of the active and passive pressures. The passive pressure's depth starts from 2.6 m until the groundwater table which is around 7 m. The works have been carried out with deliberate calculations of K_p values, Passive Pressures and Forces. The diagram is shown in Figures above 4.12.1 and 4.12.2.

4.13 Retaining Wall Design

4.13.1 Hand Calculation of Retaining Wall

Design parameters chosen for Retaining Wall with total height of the wall (H_{total}) is 3.5 m (3.5 m excavation - Underground parking as basement). Base of the Width is assumed $\sim 0.7H = 2.5$ m. Cantilever Wall is chosen for the wall design because of optimal, efficient, and economical reasons. According to the procedures for the lateral active and passive pressures and Table 4.7.1 the Retaining Wall design could be proceeded as follows:

a. Overturning Failure Check:

To proceed with an overturning failure check, the resisting forces of the wall should be calculated first. The formula is below:

$$FS_{Overturning} = \frac{\Sigma M_R}{\Sigma M_O} \quad (4.12.1)$$

Where,

ΣM_R – sum of Resisting Moments, kN*m/m

ΣM_O – sum of Overturning Moments, kN*m/m

For the Cantilever Wall, parts are divided to three. The materials for Stem and Base Slab are assumed to be Concrete. The third part is the soil heel. The Table 4.12.1 provides the detailed view of how ΣM_R is found. Concrete unit weight is taken as 24 kN/m³, and retained backfill soil is 18.75 kN/m³. FS typically more than 2 illustrates safe structure.

Table 4.13.1.1 Calculation of ΣM_R

Part	Area, m ²	Weight/unit length of wall, kN/m	Arm from Toe, m	Moment from Toe Mr, kN*m/m
Stem (Concrete)	1.24	29.76	1	29.76
Base Slab (Concrete)	1.6	38.4	2	76.8
Soil on Heel	4.56	162.75	1.3	211.575
		$\Sigma P_u = 230.91$		$\Sigma M_R = 318.135$

Table 4.13.1.2 Calculation of $\sum M_o \sum P_{a,i}$

Depth,	Pai, kN/m	Depth zi, m	Arm from Toe yi, m	Mo, kN*m/m
0-1.23	5.24	0.41	3.09	16.19
1.23-1.99	11.41	1.61	1.89	21.56
1.99-2.6	24.48	2.295	1.205	29.5
2.6-3.5	34.02	3.05	0.45	15.31
	$\sum P_{a,i} = 75.15$			$\sum M_o = 82.56$

$$FS_{Overturning} = \frac{318.135}{82.56} = 3.85 > 2.0 - \text{OK}$$

b. Sliding Failure Check

The resisting force should be calculated for this procedure as follows:

$$FS_{Sliding} = \frac{\sum F_{R'}}{\sum F_d} \quad (4.12.2)$$

Where,

$\sum F_{R'}$ – sum of the horizontal resisting forces, kN/m

$\sum F_d$ – sum of the horizontal driving forces, kN/m

$$F_R = (\sum V)\tan(\delta') + Bc'_a + P_p \quad (4.12.3)$$

Where,

δ' – friction angle between the soil and base slab, °

c'_a – adhesion between the soil and base slab, kPa

P_p – lateral passive pressure, kPa

$$F_d = P_a \cos(\alpha) = \sum P_a \quad (4.12.4)$$

Where,

P_a – total horizontal resultant forces per area or horizontal active pressure, kPa

α – angle of backface, °

$$F_R = 230.91 * 0.311 + 2.5 * 6.67 + 27.75 = 116.24 \text{ kN/m}$$

$$FS_{Sliding} = \frac{116.24}{75.15} = 1.55 > 1.5 - \text{OK}$$

c. Bearing Capacity Failure Check

Eccentricity:

$$e = \frac{B}{2} - \frac{\sum M_R - \sum M_O}{\sum V} \quad (4.12.5)$$

$$e = \frac{2.5}{2} - \frac{529.71-82.56}{230.91} = -0.686 \text{ m}$$

Actual Maximum Bearing Pressure:

$$q_{toe} = \frac{\Sigma V}{B} \left(1 - \frac{6e}{B}\right) \quad (4.12.6)$$

$$q_{heel} = \frac{\Sigma V}{B} \left(1 + \frac{6e}{B}\right) \quad (4.12.7)$$

$$q_{toe} = \frac{230.91}{3.5} \left(1 + \frac{6*(-0.686)}{3.5}\right) = 11.61 \text{ kPa}$$

$$q_{heel} = \frac{230.91}{3.5} \left(1 - \frac{6*(-0.686)}{3.5}\right) = 143.56 \text{ kPa}$$

In order to find the bearing capacity failure check, the ultimate bearing capacity of the soil should be proceeded as follows:

$$q_u = c'N_c + qN_q + 0.5\gamma B'N_\gamma \quad (4.12.8)$$

$$B' = B - 2e$$

$$B' = 2.5 - 2 * (-0.686) = 3.872 \text{ m}$$

$$q_u = 10 * 22.25 + 9.5 * 11.85 + 0.5 * 19 * 3.872 * 12.54 = 796.35 \text{ kN/m}^2$$

$$FS_{bearing} = \frac{q_u}{q_{max}} \quad (4.12.9)$$

$$FS_{bearing} = \frac{796.35}{143.56} = 5.55 > 3 - \text{OK}$$

4.13.2 Software Calculation of Retaining Wall

All three measures as from the calculations are illustrating safe structure for Cantilever Design. The safety factors for three cases have proceeded and the design of the Cantilever Wall for the basement is illustrated in Figures 4.13.1 and Figures 4.13.2.

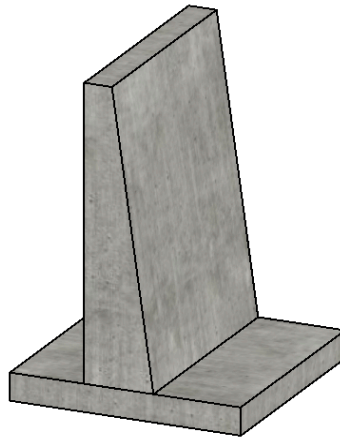


Figure 4.13.2.1 Cantilever Retaining Wall Design in 3D

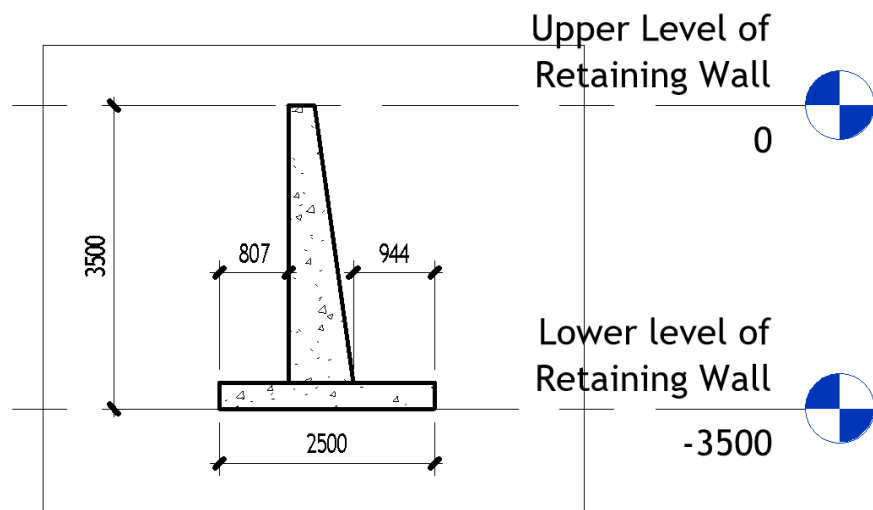


Figure 4.13.2.2 Cantilever Retaining Wall Dimensions

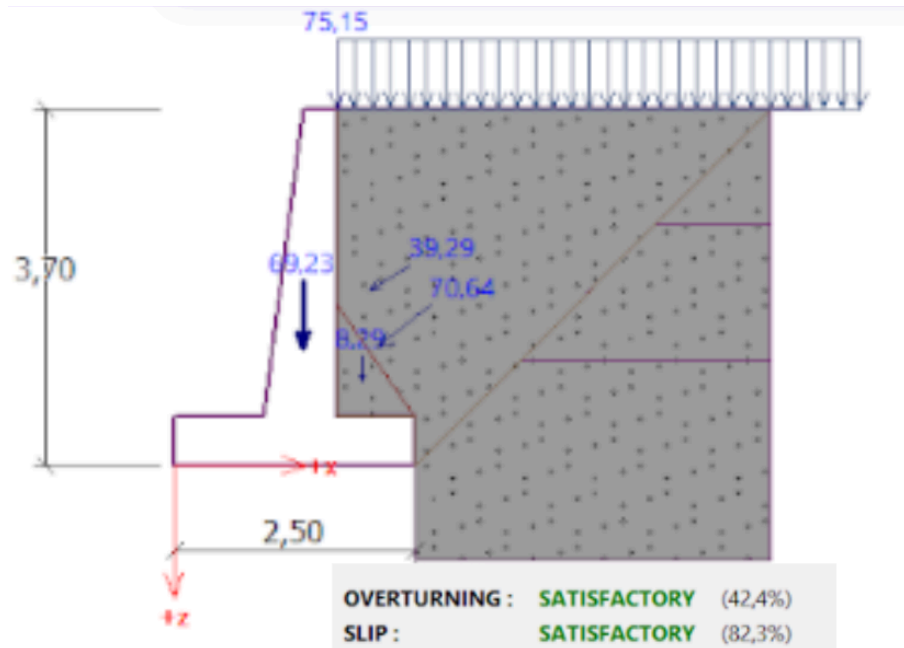


Figure 4.13.2.3 Retaining Wall Overturning Failure Check

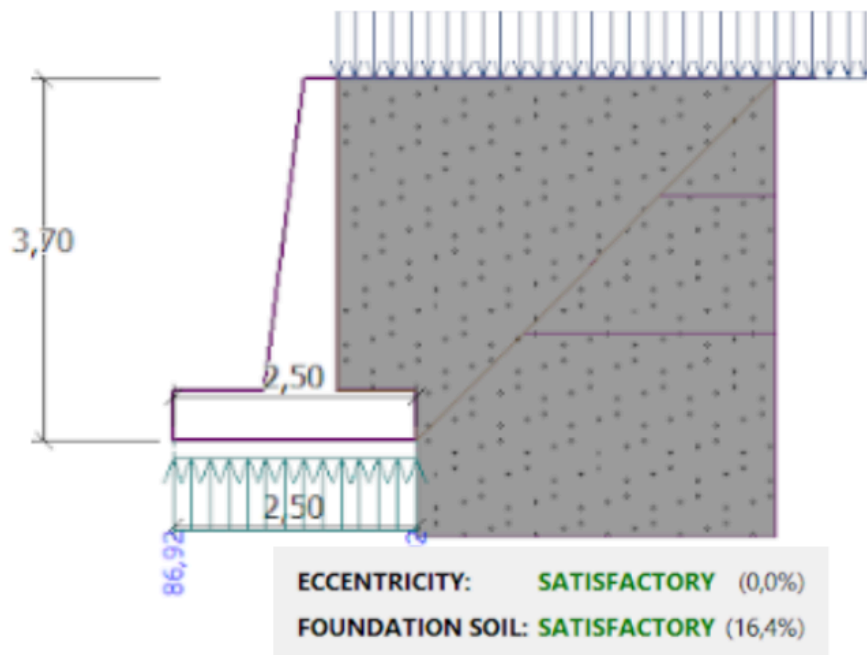


Figure 4.13.2.4 Retaining Wall Sliding and Bearing Capacity checks

Overall, the Geo5 results shown in hand and software calculations satisfied the minimum requirements. The Factors of Safety stated in the software for Overturning Failure check is 3.0, for Sliding Failure Check is 1.5, and Bearing Capacity Failure Check is 3.0. The results are provided in the Figures above 4.13.3 and 4.13.4.

4.14 Construction Procedure

Site Investigation and Preparatory Works: Removal of all surface vegetation, organic topsoils to prevent any contamination of structural fill. Establishment of survey control and engineering working platform.

Earth Retention: Guide Walls to ensure verticality and alignment, casting RC templates. Drilling reinforced structural piles.

Excavation and Dewatering: Activate Deep Wall systems to diminish the water and perform staged excavation to reach formation level.

Pile Cap Construction: Lay a thin "blinding" layer of concrete for a clean working surface. Connect the pile steel to the Foundation Mat. Pour the final layer of concrete and use vibrators to create a solid foundation for the mall.

Precast concrete piles were chosen because:

- Able to resist heavy axial loads/tension forces/bending moment
- Higher reliability with proper curing (fewer defects, better durability)
- High resistance to corrosion/chemical contamination/biological effects
- Cost-effective and time efficient in maintaining

4.15 Group pile cap reinforcement

A minimum cover to reinforcement is determined to be 75 mm, according to the standards (*IS 2911-1-3, 2010*). A pile diameter is equal to 400 mm. For reinforcement design a bar with 20 mm diameter is assumed. The calculation of effective depth in x and y directions is done using formulas bellow:

$$d_x = h - cover - \frac{d_{bar}}{2} = 1000 - 150 - \frac{24}{2} = 840$$

$$d_y = h - cover - d_{bar} - \frac{d_{bar}}{2} = 1000 - 150 - 24 - \frac{24}{2} = 820$$

$$K_x = \frac{M}{f_{cu} * B * d_x} = \frac{1245.63 * 10^6}{40 * 2800 * 840} = 0.016$$

$$K_y = \frac{M}{f_{cu} * L * d_y} = \frac{1245.63 * 10^6}{40 * 4000 * 820} = 0.012$$

$$z_{x/y} = d_{x/y} (0.5 + \sqrt{(0.25 - \frac{K_{x/y}}{0.9})})$$

Where,

H- pile cap width, mm

M- bending moment, kNm

$$f_{cu} = 40 \text{ NM/mm}^2$$

$$f_y \text{ - steel yield strength, } 460 \text{ N/mm}^2$$

To calculate the area used for reinforcement, the equation below was used:

$$A_{s,x} = \frac{M}{0.87 * f_y * z_x} = \frac{1245.63 * 10^6}{0.87 * 460 * 901.3} = 3772.7$$

The number of bars in each direction is calculated by:

$$\frac{A_{s,x}}{A_{bar}} = \frac{3772.7}{314} = 12$$

Next, to calculate the spacing of the bars in y and x directions the following equations were used:

$$s_x = \frac{B - 2c - d_{bar}}{n_x - 1} = \frac{2800 - 2(150) - 20}{11 - 1} = 248 \text{ mm}$$

$$s_y = \frac{L - 2c - d_{bar}}{n_y - 1} = s_y = \frac{4000 - 2(150) - 20}{12 - 1} = 335 \text{ mm}$$

By repeating these steps the following table of pile cap reinforcement for each pile cap type can be summarized:

Table 4.15.1 Pile Cap Reinforcement Design for Each Column Type

Group of piles	L, mm	B, mm	d_x , mm	d_y , mm	$A_{s,x}$, mm ²	$A_{s,y}$, mm ²	A_{bar} , mm ²	X-direction reinforcement	Y-direction reinforcement
I-1	4000	2800	840	820	3772.66	3845.89	314	12T20 @ 248	13T20 @ 335
E-5	2800	2800	840	820	5141.32	5273.39	314	16T20 @ 178	17T20 @ 174
E-20	2800	2800	840	820	4596.10	4713.49	314	15T20 @ 190	15T20 @ 200
ECW-5	2800	1600	840	820	5243.84	5273.39	314	17T20 @ 91	18T20 @ 165
C-1	2800	1600	840	820	4322.77	4362.01	314	14T20 @ 116	14T20 @ 210
CCW-1	2800	1600	840	820	4846.73	4881.27	314	16T20 @ 98	16T20 @ 184

After performing hand calculation of pile reinforcement, Geo5 software was used to identify the reinforcement design for the single pile. The software results verify that the chosen reinforcement satisfy the shear, bending and compression requirements. The following screenshot shows the final chosen reinforcement.

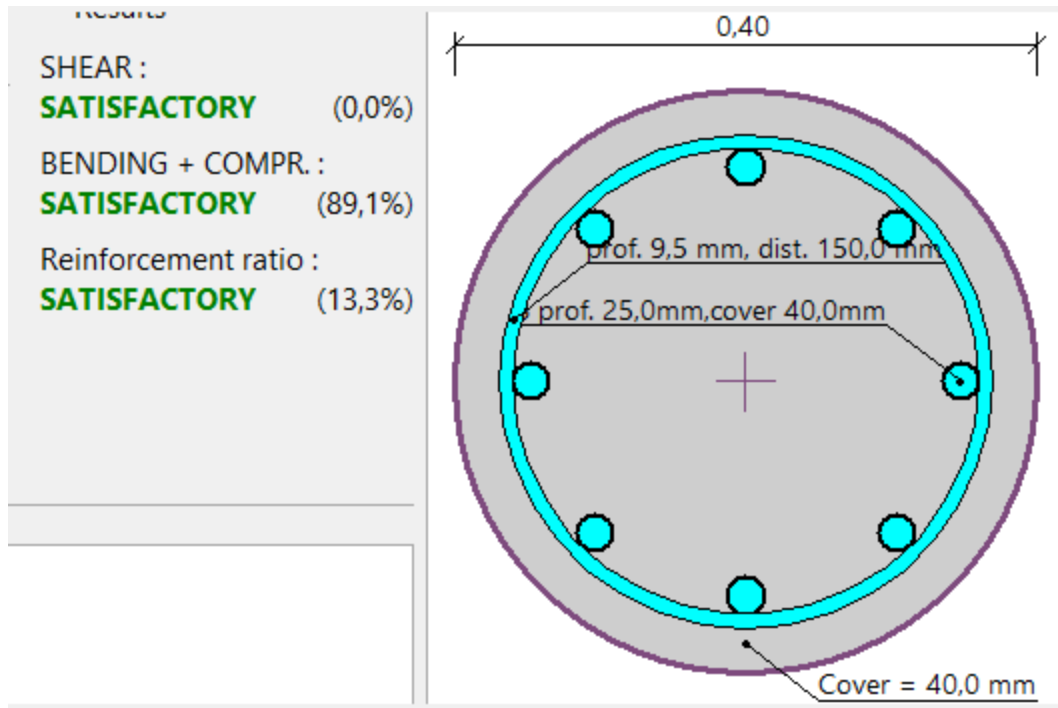


Figure 4.15.1 Single pile reinforcement

In order to visualise the pile reinforcement design as well as the column-pile connection Autocad software was used. The following figure illustrates the group of piles reinforcement for the interior column type:

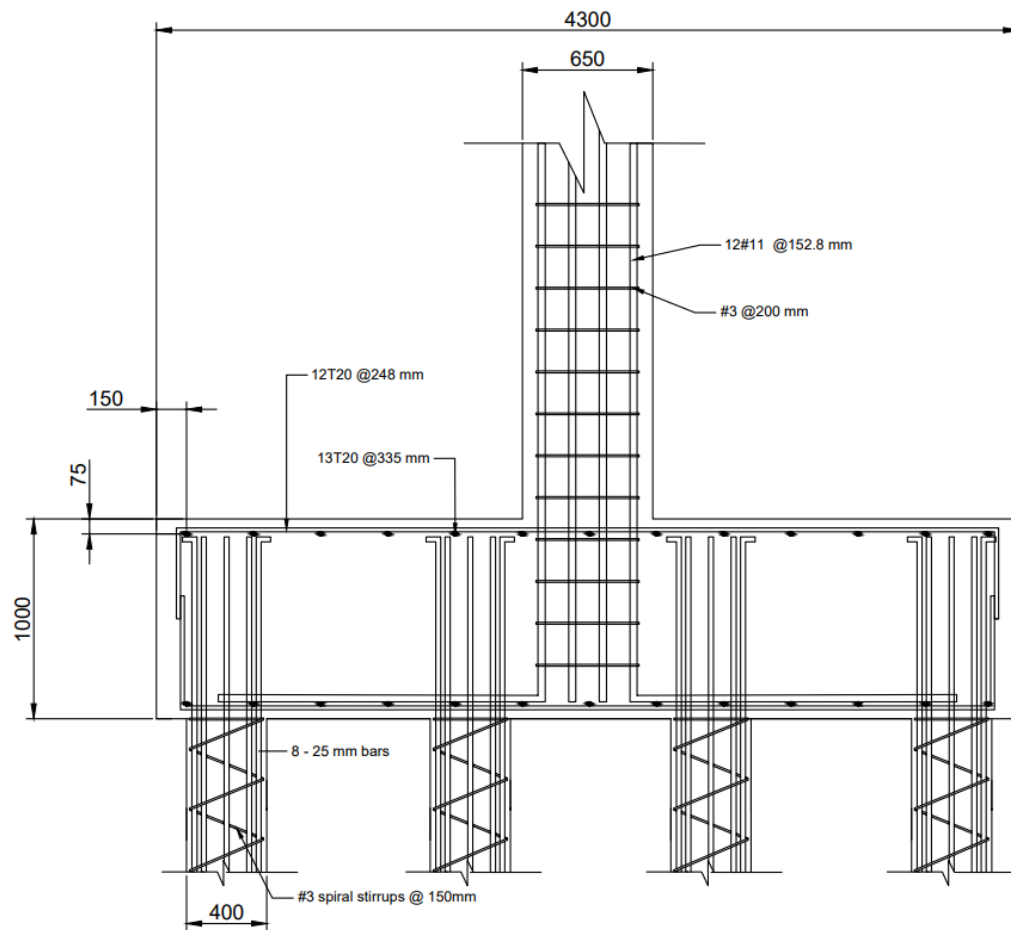


Figure 4.15.2 I-I Column Reinforcement of Foundation

5 Construction Management

5.1 Project Charter

Table 5.1.1 Project Charter

Information			
Title	Design of 4-story shopping mall “KHAN TENGRI MALL” in Los Angeles, USA		
Description	The shopping mall is going to consist of 100*40 meter size 4 floors, and the perimeter is 280 m, which is about 919 LF. The area of the mall is 16,000 m ² (172 223 ft ² , 4 floors together). Vertical connections are provided by 3 elevators, one technical and two lifts for users. In addition, the first floor constitutes a huge retail store, technical and engineering rooms, and boutiques. Hardware stores and retail pavilions are situated on the third floor. There is a food court and miscellaneous stores on the third floor, and cinemas are located on the fourth floor.		
Location	The building is a 4 storey shopping mall located by 130 W College St, Los Angeles, California.		
Facilities	4 story, basement parking		
Timeline	945 days		
Project start date	September 2025	End date	April 2028
Budget	\$43 153 112		
Stakeholders	Owners		
Problem statement	Nowadays, the selected area is an open bus parking lot, which appears to be an inefficient use of valuable and dense urban land. The		

	parking lot can be relocated to city outskirts to develop another social amenity of higher value. The surroundings of Chinatown is aging as well, requiring some new retail facilities and contemporary leisure infrastructure. A 4-storey shopping mall “KHAN TENGRI MALL” with parking could optimize the situation, giving an opportunity to serve both locals and tourists. This solution is viable to boost the local economy with the increase of foot traffic and can revitalize the urban environment.
Objectives	The main goal of this project is to build a 4 storey shopping mall with parking in the basement level, the project must comply with all regulations of design codes in Los Angeles, CA. Other aims are: • To ensure effective arrangement of zones on each floor (stores, retail stores, cinema, food courts) • To provide convenient transport facilities inside shopping mall (2 public elevators, 1 technical elevator, and escalators on each level) • To ensure accordance with all regulations in terms of fire safety and evacuation systems • To minimize consumption of electricity and other electricity generating sources, utilizing specific technologies. • To generate urban architectural design for the building to match with city landscape
Deliverables	4 floor shopping mall Underground parking Food courts Cinema halls Boutiques Stores

Potential risks	Utility connection delays - potential risks of having some issues with the connection of the electricity, ventilation systems, and water Increase of material prices - rise of material costs can affect construction process, plans, and budget Social Factors - locals can complain about construction works Technical issues - potential problems with implementation of engineering systems (HVAC, fire alarm systems)
Key project assumptions	Timely financing - uninterrupted financial support of investors is expected throughout all construction stages Skilled workforce availability - it is predicted that number of engineers, installers and builders are going to be sufficient Stable economic conditions - the demand for the shopping is anticipated to remain high Reliable logistics - materials, tools, and equipments are going to be delivered without interruptions Commissioning - lifts, escalators, HVAC, and fire alarm systems work properly Support of local authorities - the government sustains the project without any unnecessary obstacles
Scope	4000 S.M./ 43 056 S.F.
Project members	Adina Serikkyzy Alibi Yeskazy Nurislam Nuraddinov Zhanbolat Sadratdinov Baidibek Mussayev Rassul Ibrayev

	Adelina Nurgaliyeva
--	---------------------

5.2 Feasibility Study

This feasibility paper suggests a four-story enclosed shopping mall of 130 W College St, Chinatown, Los Angeles whereby an underutilized bus parking lot would be transformed to a modern and multi-anchor shopping and retail facility. The building comprises a total area of =146,387 SF (=43,055/floor) with an estimated efficiency of 85 percent. Having necessities and experiential applications, such as supermarket and electronics anchors, inline retail, a food court, and four small cinemas, the program stabilizes the weekday and weekend footfalls. Vertical circulation includes 4 escalators, 2 passenger lifts and 1 freight lift. The core systems consist of 4 RTUs (50-70 ton) central chiller/ boiler and approximately 40,000 SF of ductwork, 500 LED fixtures and 50 exit signs, and plumbing which is designed to serve 40 WCs, 20 urinals, 40 lavatories and 4 janitor sinks. The concept aims at district revitalization, job creation, tax-base growth and modernization of the infrastructure as well as matching Chinatown mixed residential tourism catchment.

- **Project Overview.** Four story mall level: L1 supermarket + inline + BOH/MEP; L2 large electronics anchor + inline + staff rooms; L3 food court + inline; L4 four small cinema + inline. **Vertical circulation:** 4 escalators, 2 lifts of passengers, 1 freight lift. **Key systems:** 4 RTUs (50 70 ton) + central chiller/boiler; approximately 100000 SF ductwork; approximately 600 LED, approximately 50 exit signs plumbing fixtures as marked.
- **Need & Objectives.** **Need:** rebuild an old lack of productivity bus lot into a new high-tech multi-anchor center. **Goals:** (i) re-invention of the district and safer streetscape activation; (ii) construction and long-term employment; (iii) increased tax base and tourist attraction; (iv) improved infrastructure and life-safety/ADA compliance.
- **Site & Access Summary.** Central location in Chinatown with exposure and mixed uses of trip; close to downtown/transit. Primary street connection with shoppers, restricted loading of **BOH** associated with the freight core. Utility tie-ins (power, water, sewer, telecom) can be done, size of fire flow/transformer to be established during design development.

- **Space Program/ Planning Requirement.** Balanced anchors (grocery + electronics) and inline retail, food court and cinemas to balance out the demand over the week. **Probing ratios:** GLA = -85% of GSF, transparent spans, pliable demising, enduring store finishes. Life safety: multi-stair spiral, lit exits, voice evac/alarms, complete sprinklers, smoke control in concurrence with escalator volumes, ADA all over.
- **Technical Approach (Architecture/MEP/VT) Structure/Envelope:** 4 storey retail plates (approximately 43k SF), gypsum, acoustic ceilings, calibrated glazing, daylight. **HVAC:** included 4 RTUs + central chiller/boiler, approximately 40k SF ductwork; tenant bay zoned; grease/MAU meal court. **Electrical/Lighting:** two switchboards, approximately 500 luminaire LED luminaires, approximately 50 exit signs; exterior roadway and bollard lighting; tenant POS/IT capacity. **Plumbing:** sanitary / domestic to 40 WCs / 20 urinals / 40 lavs / 4 janitor sinks; 400-gal water heater; grease control to food service. **Vertical Transportation:** 4 retail spine escalators, 2 passenger and 1 logistic elevator.

5.3 Work Breakdown Structure

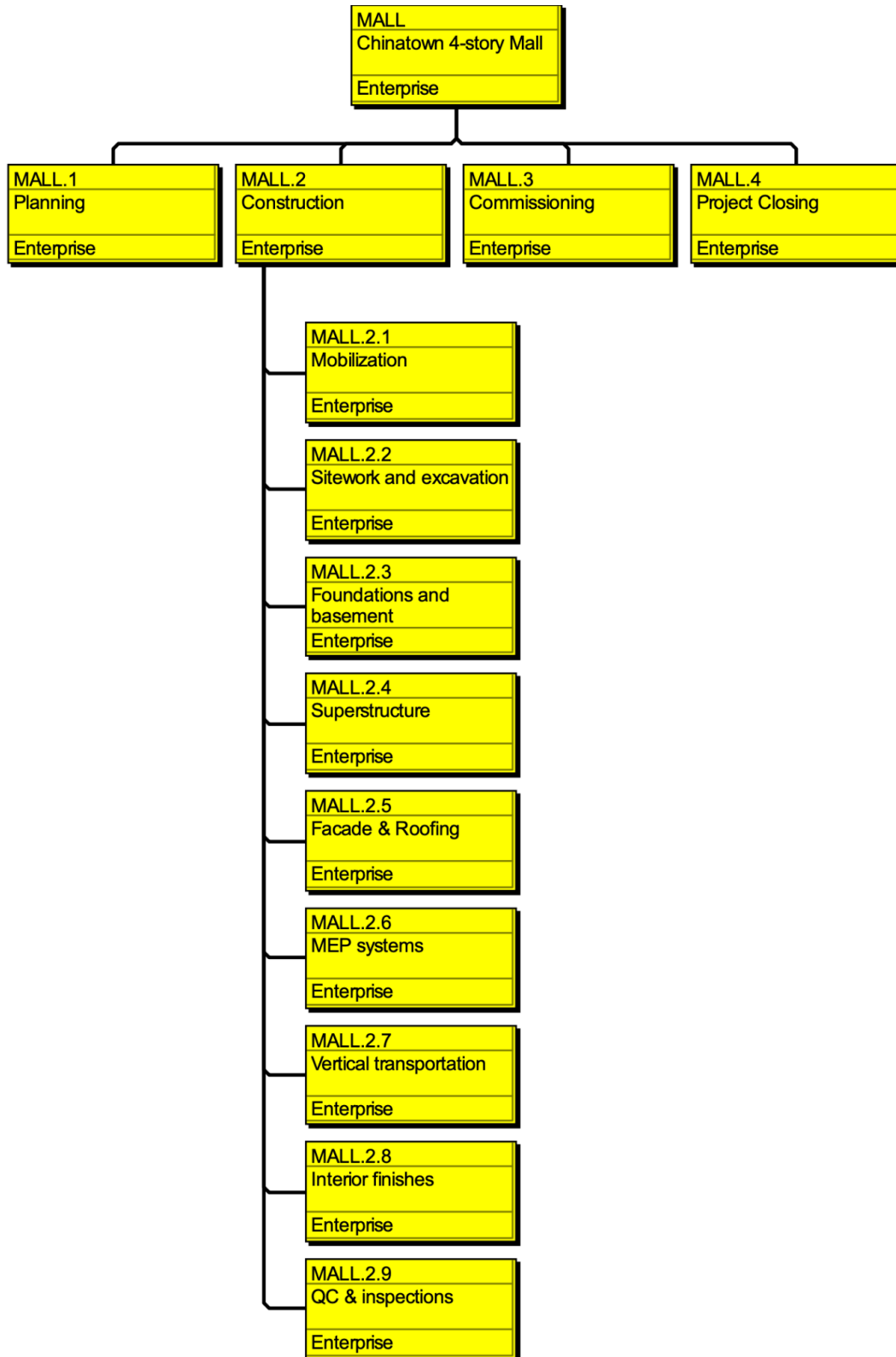


Figure 5.3.1. Work breakdown structure from Primavera P6.

The WBS divides the 4-story mall into understandable work packages with clearly-defined deliverables. Pre-Design and permitting services include building site assessment, preliminary building design, building code/ADA/ fire life safety review, and permit applications. Procurement & Contracts prepares bid packages and orders long lead (elevators, escalators, switchgear, RTUs) Site & Substructure including site planning, excavation/shoring, foundations, one level basement car park (slab, walls, waterproofing and ventilation). Superstructure & Envelope constructs 4 floors of structural frames and cores, completes the facade/roof to provide the weather-tightness to the building. MEP & Vertical Transportation installs HVAC (RTUs + Chiller/Boiler, Duct), Electrical, Plumbing, Fire Protection, and Vertical Transportation. Interiors & Fit-Out will add partitions, ceilings, flooring, golf-course build-out, food-court, sanitary rooms, etc. Testing, Commissioning & Handover - The MEP/VP Commissioning, testing, implementation of punch-list and O&M manuals, training and closeout.

5.4 Scheduling



Figure 5.4.1. Gantt Chart (Microsoft Project).

The Gantt Chart is used to identify work sequences of: Pre-Design, Permitting, Site/Substructure, Superstructure, Envelope (weather-tight), MEP rough-in, Interiors, Commissioning/Handover. Sheets of interiors are stacked one atop the other and prepare them only when the floor is completed, saving time on the parallel movement of the MEP, frame and facade/roof from floor to floor. Key milestones are: permit granting, foundation and parking level completion, structure topped out, weather-tight envelope, permanent power, vibration testing, commissioning, occupancy certificate. The key pathway bypasses permissions through excavations/foundation and leading through frame, weather tight envelope, main MEP read through to commissioning/inspections. Buffer is added for long-lead items (VT, switchgear, RTUs) and "wet" works (excavation advice, waterproofing) so as to protect the final completion date.

5.5 Cost/Benefit Analysis

5.5.1 Costs Estimation

Costs were estimated by RSMeansOnline Demo version. This estimation gives a preliminary overview on future spendings. The costs are going to be corrected later with hand calculations as well.

\$7,361,691.09 Building Cost	LOS ANGELES, CA Location	4 Stories (Ea.)	Yes Basement
\$170.92 Cost per S.F.	43,072 Floor Area	14.00 Story Height	\$135,406.05 Additive Cost

Figure 5.5.1. Cost estimation by RSmeans (2018)

Methodology

Calculations were based on RSMeans Online database values, but the data was published for earlier years (2018). To take inflation into account and align costs with the cost environment of Los Angeles in 2025, a multi step escalation method was applied. Historical Cost Index for 2018 and 2022 was 90.2 and 100 respectively.

Adjustment is:

$$Factor_{2018 \rightarrow 2022} = \frac{100}{90.2} = 1.1086$$

Analyzing current data on RSMeans 2025 HCI updates, ULI inflation in construction reports the annual increase in inflation is about 4% from 2022 to 2025. This was obtained directly from published RSMeans Historical Cost Index (HCI) values. An index of 261.6 is given for January 2022 and an index of about 294 for January 2025, on a basis of January 1, 1993 = 100. The 2025 value can be found in the official RSMeans cost index updates, which reports the "actual January 2025 Historical Index" as 293.9, and is independently reported as 294.0 in a state-level escalation memo which cites the Heavy Construction Costs with RSMeans Data series (2025 RSMeans Data Cost Book Updates, n.d.).

Over these three years the total cost index change is:

$$\frac{294}{261.6} = 1.1239$$

This is 12% of annual growth over 3 years. Following this, compound annual rate is:

$$CAGR = \sqrt[3]{\frac{294}{261.6}} - 1 = 0.0397 \approx 4.0\% \text{ per year}$$

In result:

$$Factor_{2022 \rightarrow 2025} = 1.04^3 = 1.125$$

Combining these factors, the final value for the factor is:

$$Factor_{2022 \rightarrow 2025} = 1.125 * 1.1086 = 1.247$$

Table 5.5.1.1 Cost Estimation report by RSMeansOnline.

Description	Quantity	% of Total	Cost per S.F. (\$)	Cost (2018) (\$)	Cost per S.F. (2025) (\$)	Cost (2025) (\$)

Standard Foundations		9.52%	13.70	589 993.45	17.08	735721.8
Strip footing, concrete, reinforced, load 11.1 KLF, soil bearing capacity 6 KSF, 12" deep x 24" wide	919		3.97	171 159.43	4.95	213435.8
Spread footings, 3000 PSI concrete, load 100K, soil bearing capacity 6 KSF, 4'-6" square x 15" deep	256.28		2.95	127 253.47	3.68	158685.1
Slab on Grade			1.63	70 061.35	2.03	87366.5
Slab on grade, 4" thick, non industrial, reinforced	10768		1.63	70 061.35	2.03	87366.5
Basement Excavation			1.23	52 948.41	1.53	66026.67
Excavate and fill, 10,000 SF, 8' deep, sand, gravel, or common earth, on site storage	10768		1.23	52 948.41	1.53	66026.67
Basement Walls			6.87	295 824.26	8.57	368 892.9
Foundation wall, CIP, 12' wall height, pumped, .444 CY/LF, 21.59 PLF, 12" thick	919		6.87	295 824.26	8.57	368 892.9
Floor Construction	3075.4	48.24%	25.23	1 086 621.03	31.46	135 5016
Cast-in-place concrete column, 12" square, tied, 200K load, 12' story height, 142 lbs/LF, 4000PSI	891.43		6.96	299 787.72	8.68	373 835.3
Cast-in-place concrete column, 12" square, tied, minimum reinforcing, 150K load, 10'-14' story height, 135 lbs/LF, 4000PSI	1020.09		1.71	73 612.86	2.13	91 795.24

Concrete I beam, precast, 18" x 36", 790 PLF, 25' span, 6.44 KLF superimposed load	10768		7.81	336 333.87	9.74	419 408.3
Flat slab, concrete, with drop panels, 6" slab, 25.5" panel, 12" column, 15'x15' bay, 75 PSF superimposed load, 153 PSF total load	10768		4.27	183 830.76	5.32	229 237
Precast concrete double T beam, 2" topping, 24" deep x 8' wide, 50' span, 75 PSF superimposed load, 165 PSF total load	10768		4.48	193 055.81	5.59	240 740.6
Exterior Walls	43744.4		34.56	1 488 551.94	43.1	1 856 224
Stucco, 3 coat, self furring metal lath 3.4 Lb/SY, on regular CMU, 12" x 8" x 16"	43744.4		34.56	1 488 551.94	43.1	1 856 224
Exterior Windows				222 891.20		277 945.3
Aluminum flush tube frame, for 1/4"glass,1-3/4"x4", 5'x6' opening, no intermediate horizontals	514.64		5.17	16 673.72	6.45	20 792.13
Glazing panel, insulating, 1/2" thick, 2 lites 1/8" float glass, clear	7204.96		4.79	206 217.48	5.97	257 153.2
Exterior Doors			1.85	79 781.73	2.31	99 487.82
Door, aluminum & glass, without transom, bronze finish, hardware, 3'-0" x 7'-0" opening	16.15		1.48	63 881.16	1.85	79 659.81

Door, steel 18 gauge, hollow metal, 1 door with frame, no label, 3'-0" x 7'-0" opening	5.38		0.37	15 900.57	0.46	19 828.01
Roof Coverings			2.56	110 167.77	3.19	137 379.2
Roofing, single ply membrane, EPDM, 60 mils, loosely laid, stone ballast	10768		2.56	110 167.77	3.19	137 379.2
Insulation, rigid, roof deck, extruded polystyrene, 40 PSI compressive strength, 4" thick, R20	10768		0.47	20 178.69	0.59	25 162.83
Roof edges, aluminum, duranodic, .050" thick, 6" face	919		1.10	47 553.10	1.37	59 298.72
Gravel stop, aluminum, extruded, 4", mill finish, .050" thick	919		0.69	29 586.98	0.86	36 894.96
Roof Openings	1		0.03	1 248.43	0.04	1556.79
Roof hatch, with curb, 1" fiberglass insulation, 2'-6" x 3'-0", galvanized steel, 165 lbs	1		0.03	1 248.43	0.04	1 556.79
Partitions		13.25%	3.10	133 308.66	3.87	166 235.9
Metal partition, 5/8" fire rated gypsum board face, no base, 3 -5/8" @ 24" OC framing, same opposite face, no insulation	7178.66		0.94	40 653.65	1.17	50 695.1
Gypsum board, 1 face only, exterior sheathing, fire resistant, 5/8"	43744.4		1.26	54 304.30	1.57	67 717.46
Add for the following: taping and finishing	43744.4		0.89	38 350.72	1.11	47 823.35
Interior Doors			1.17	50 609.41	1.46	63 109.93

Door, single leaf, kd steel frame, hollow metal, commercial quality, flush, 3'-0" x 7'-0" x 1-3/8"	37.68		1.17	50 609.41	1.46	63 109.93
Fittings			0.21	9 232.48	0.26	11 512.9
Toilet partitions, cubicles, ceiling hung, stainless steel	5.38		0.21	9 232.48	0.26	11 512.9
Wall Finishes			1.81	77 803.81	2.26	97 021.35
Painting, interior on plaster and drywall, walls & ceilings, roller work, primer & 2 coats	43744.4		1.14	49 008.60	1.42	61 113.72
Painting, interior on plaster and drywall, walls & ceilings, roller work, primer & 2 coats	12921.6		0.34	14 476.59	0.42	18 052.31
Floor Finishes			3.51	151 105.19	4.38	188 428.2
Ceramic tile, thin set, 4-1/4" x 4-1/4"	1435.73		0.33	14,318.63	0.41	17855.33
Vinyl, composition tile, maximum	43072		3.51	151,105.19	4.38	188428.2
Ceiling Finishes	43072		9.26	398,777.80	11.55	497275.9
Acoustic ceilings, 3/4" mineral fiber, 12" x 12" tile, concealed 2" bar & channel grid, suspended support	43072		9.26	398,777.80	11.55	497275.9
Plumbing Fixtures		26.81%	38.57	1,661,210.7 2	48.1	2071530
Water closet, vitreous china, tank type, 2 piece close coupled	10.76		2.31	99,704.79	2.88	124331.9
Urinal, vitreous china, wall hung	1		0.46	19,644.55	0.57	24496.75

Lavatory w/trim, vanity top, PE on CI, 20" x 18"	10.76		0.41	17,544.25	0.51	21877.68
Service sink w/trim, PE on CI,wall hung w/rim guard, 24" x 20"	10.76		1.07	45,939.52	1.33	57286.58
Water cooler, electric, wall hung, dual height, 14.3 GPH	5.38		0.34	14,828.59	0.42	18491.25
Domestic Water Distribution			0.67	29,071.48	0.84	36252.14
Gas fired water heater, commercial, 100< F rise, 500 MBH input, 480 GPH	1		0.67	29,071.48	0.84	36252.14
Rain Water Drainage			1.86	79,911.16	2.32	99649.22
Roof drain, CI, soil,single hub, 4" diam, 10' high	26.92		1.86	79,911.16	2.32	99649.22
Roof drain, CI, soil,single hub, 4" diam, for each additional foot add	107.68		0.18	7,577.93	0.22	9449.68
Terminal & Package Units			10.47	450,860.90	13.06	562223.5
Rooftop, single zone, air conditioner, department stores, 10,000 SF, 29.17 ton	43072		10.47	450,860.90	13.06	562223.5
Sprinklers			5.80	249,678.91	7.23	311349.6
Wet pipe sprinkler systems, steel, ordinary hazard, 1 floor, 10,000 SF	43072		5.80	249,678.91	7.23	311349.6
Standpipes			1.33	57,278.76	1.66	71426.61
Wet standpipe risers, class III, steel, black, sch 40, 4" diam pipe, 1 floor	5.38		1.33	57,278.76	1.66	71426.61
Electrical Service/Distribution			0.58	25,154.95	0.72	31368.22

Overhead service installation, includes breakers, metering, 20' conduit & wire, 3 phase, 4 wire, 120/208 V, 400 A	1		0.16	6,927.80	0.2	8638.97
Feeder installation 600 V, including RGS conduit and XHHW wire, 400 A	50		0.13	5,490.25	0.16	6846.34
Switchgear installation, incl switchboard, panels & circuit breaker, 120/208 V, 3 phase, 400 A	1		0.30	12,736.90	0.37	15882.91
Lighting and Branch Wiring			13.60	585,900.36	16.96	730617.8
Receptacles incl plate, box, conduit, wire, 8 per 1000 SF, .9 watts per SF	43072		3.62	156,030.90	4.51	194570.5
Miscellaneous power, 1.5 watts	43072		0.51	21,971.03	0.64	27397.87
Central air conditioning power, 4 watts	55993.6		1.00	43,230.42	1.25	53908.33
Fluorescent fixtures recess mounted in ceiling, 1.6 watt per SF, 40 FC, 10 fixtures @32watt per 1000 SF	53840		8.47	364,668.01	10.56	454741
Communications and Security			1.94	83,649.42	2.42	104310.8
Communication and alarm systems, fire detection, addressable, 25 detectors, includes outlets, boxes, conduit and wire	1.93		1.13	48,545.06	1.41	60535.69
Fire alarm command center, addressable without voice, excl. wire & conduit	5.38		0.82	35,104.35	1.02	43775.12
Equipment & Furnishings		2.19%	3.14	135,406.05	3.92	168851.3
SubTotal	100%	100%	143.87	6196709.67	179.41	7 727 297
Contractor Fees (GC, Overhead, Profit)	10.0%	10.0%	14.39	619670.97	17.94	7 727 297

Architectural Fees	8.0%	8.0%	\$12.66	545310.45	15.79	680 002
User Fees	0.0%	0.0%	0	0	0	0
Total Building Cost			170.92	7 361 691	213.14	9 180 029

Represented cost estimation is based on bare minimum of the building coverage.

Table 5.5.1.2 Hand Calculation according to RSMeansOnline.

Division	Item	Qty	Unit	Unit Cost (O&P)	Total Cost (\$)
Finishes	Gypsum Board	12000	S.F.	2.23	26 786
	Suspended Acoustic Ceiling	16000	S.F.	6.24	99 760
	Vinyl Tile Flooring (VCT)	16000	S.F.	4.05	64 844
	Rubber/Vinyl Sheet Flooring	4000	S.F.	5.99	23 942
Openings	Standard Doors	60	Ea.	2201	132 057
	Double Doors	20	Ea.	3647.48	72 950
	Windows (clear float)	3000	S.F.	11.72	35 165
	Vision Panels (витрины)	4000	S.F.	108.86	435 452
Plumbing	Toilets (floor-mounted)	40	Ea.	1060	42 398
	Urinals	20	Ea.	897.84	17 957
	Lavatories (sinks)	40	Ea.	798.08	31 923
	Janitor Sinks	4	Ea.	1371.7	5 487
	Water Heater (400 gal)	1	Ea.	8105.5	8 106
Electrical	Switchboards (main)	2	Ea.	7232.6	14 465

	LED Lighting Fixtures	500	Ea.	47.76	23 880
	Emergency Exit Signs	50	Ea.	173.33	8 667
	Exterior Lighting (roadway)	20	Ea.	1338.03	26 761
	Landscape Bollards	10	Ea.	1097.36	10 974
HVAC	Rooftop Air Handling Units (RTU) 50–70 ton	4	Ea.	187 050	748 200
	Ductwork + Diffusers	40000	S.F.	8.11	324 220
	Chiller/Boiler System	1	Lot	436 450	436 450
Conveying	Escalators (commercial, 2-story)	4	Ea.	218 225	872 900
	Passenger Elevators (public)	2	Ea.	149 640	299 280
	Freight Elevator	1	Ea.	224 460	224 460
Total					\$3 987 083

Table 5.5.1.3 Full hand calculations with assumed values.

Item	Basis	Cost (USD)
Land Acquisition	(loopnet.com 2025)	5 500 000
Professional & Soft Costs	14% of construction (ULI)	1 285 000
Contingency	10% of building construction	918 000
Construction Insurance	1.3% of building cost	120 000
IT Systems (LAN/CCTV/Wi-Fi)	$\$2.25/\text{SF} \times 146,387$ GLA	329 000

Marketing & Lease-Up	$\$2.25/\text{SF} \times 146,387$ GLA	329 000
Owner O&M Equipment	Allowance	100 000
Geotechnical Investigation	Boreholes, lab tests, report	300 000
Parking Structure (Underground, 130 spaces)	$\$85,000$ per space	11 050 000
Site Preparation and Utilities	3% of construction	275 000
Permitting & Impact Fees	1.5% of construction	138 000
Developer Fee / Overhead	3% of construction	275 000
Financing & Interest Reserve	4% of construction	367 000
Tenant Improvements Allowance	$\$15/\text{SF} \times 50,000 \text{ SF}$	750 000
Total		21 736 000

The values shown for this table have been updated to 2025 Los Angeles price levels, using the same escalation logic as for construction costs shown in the previous subsection. Direct building costs were adjusted by using the RSMeans Historical Cost Index (2018-2022) plus a 4% annual cost of construction inflation (2022-2025), whereas the categories of owner level were taken from external market references (loopnet land pricing, RSMeans 2025 Building Systems, ULI's reports). Items such as foundations, structural steel and exterior walls are classified as "informational" because the cost of these items are already included in the RSMeans building estimate and thus are not added again to prevent double counting.

Table 5.5.1.4 Cost Estimation of the project.

	Amount (USD)
--	--------------

Raw hand calculations	21 736 000
Hand calculations based on RSMeansOnline database	3 987 083
RSMeansOnline calculation report (Total Building Cost)	9 180 029
Total development cost	34 903 112

- Hand jobs (finishes, openings, HVAC, electrical, plumbing, conveying) - \$3 987 083
- RSMeans “Total Building Cost”: \$9 180 029

Any owner & development costs and allowances (aggregate): \$21 736 000

It is about non-building objects that are generally easily moved outside the trade divisions, including land acquisition, underground parking (about 1 level, some 130 spaces), contingently on-building scope, building insurance, owners IT systems (LAN/CCTV/Wi-Fi), marketing and lease-up, owner O and M equipment, geotechnical investigation and other owner-level fees. It is currently considered as part of the building total: (Concrete, steel reinforcement, exterior walls, and site preparation are already a part of the building total and are not added devices a second time).

Interpretation

The sum includes the registered building cost background and all their owner level expenses to present a successful retail asset in Chinatown in Los Angeles. The unitized measures lie within a range within the range of feasibility stage to a multi-anchored, four-layer city mall including having the common parking system and recent MEP systems. Industry benchmarks were thoroughly reviewed to make all estimations.

5.5.2 Cash Inflow

Table 5.5.2.1 Cash inflow of the project.

Revenue Source	Annual Amount (USD)
Rent – Supermarket (anchor)	263 419

Rent – Electronics (anchor)	263 419
Rent – Inline boutiques	885 401
Rent – Food court stalls	266 592
Rent – Cinema (4 small halls)	122 976
Rent – Kiosks / carts	61 980
Rent – Storage	23 436
Percent Rent (all categories)	513 818
CAM Recoveries	1 112 541
Parking Revenue	363 000
Advertising & Signage	120 000
ATM Fees	24 000
Rooftop Lease	24 000
Events & Pop-ups	50 000
Other Income	30 000
Effective Gross Income	4 124 582

The assumptions made in regards to the rent in this project are compared to the recent findings in the Los Angeles retail market. Philippe Properties (2025) report that LA retail lease rates have increased from approximately \$2.90 per square foot in 2022 to a projected \$3.10 per square foot in 2025, which corresponds to average annual growth of about 2.2%. This information was taken from the report by Matthews regarding Real Estate Investment Services in 2025, the report states that average rents are \$36.50 per square foot per year, which means it is \$3.00 per SF per month. In addition, the availability rate is 6.0%. This information was supported by Steven Smith on LinkedIn as well, he claims that average retail asking rents reached about \$3.59 per SF in Q2 of 2025 (Smith, 2025). Taking into account the information obtained from these sources, the inline retail rent range was considered as \$3.10-3.60 per SF per

month for our shopping mall. Supermarket and electronics (anchor tenants) are priced \$2.20, inline boutiques - \$3.50.

Food court units are estimated to be \$4.00 per SF per month, kiosks \$5.00 since these amenities basically pay a premium amount of money due to high visibility (Matthews, 2025). The price of cinema space is deliberately set to be cheaper than inline retail at \$2.00 per square foot per month as per industry norms whereby cinemas tend to pay lower base rent and provide value to the land owner with percentage rent as well as their overall beneficial effect on overall footfall.

5.6 Risk Management

Adaptable risk management to ensure safe and quality construction of the 4-story enclosed shopping mall at 130 W College St. Chinatown, Los Angeles with the management of scope, budget, and schedule. The project has multiple construction, financial, environmental, and external driven problems, including multiple retail tenant coordination, seismic design criteria, and urban constraints. A systematic risk management framework is thus applied to identify, assess and neutralise such risks in a continuous monitoring and proactive planning framework. The project uses a Risk Matrix (3 X 3) to evaluate risks according to a probability and a severity. This matrix uses a combined score to rate each risk under low, moderate or high. Low risks are coloured yellow, moderate risks green, and high risks red. This visual and systematic approach enables focus on important threats like cost overruns, seismic activity, or schedule delays and ensuring that piffling operational issues are handled with standard control measures. The methodology is based on a clear and practical approach to construction management in line with the European Agency for Safety and Health at Work (as well as OHSAS 18001). The risk score is calculated by multiplying the values of Probability (P) and Severity (S):

$$Risk\ Score = P * S$$

Risks were categorised as:

- Low (1-2)
- Moderate (3-4)
- High (6-9)

This framework is useful in bringing the attention of the team to the most critical areas which may include cost escalation, seismic events or the probability of schedule delays but routine problems are managed by normal procedure. The method adheres to broad principles of construction safety and practice involved in project management.

To achieve the same levels of scoring, the project works with three-level scales of probability as well as severity:

Probability:

1. **Rare:** It is not likely to happen throughout the project.
2. **Possible:** May happen in some conditions.
3. **Nearly inevitable:** Expected to happen unless preventative actions are practised.

Severity:

1. **Minor:** small impact on schedule, cost or quality; can be handled by daily measures.
2. **Moderate:** observable effect, which needs corrective action or further coordination.
3. **Serious:** Review entails serious implications on schedule, cost, structural safety, or compliance.

The definitions assist in objective and transparent scoring in placing risks in the matrix.

Table 5.6.1 Risk Matrix 3x3

Probability of a dangerous event	Minor	Moderate	Serious
Rare	Low risk (1)	Low risk (2)	Moderate risk (4)
Possible	Low risk (2)	Moderate risk (4)	High risk (6)
Almost certain	Moderate risk (4)	High risk (6)	High risk (9)

5.6.2 Risk Management Process

The risk management is implemented at the constant points of the project:

1. **Risk Identification:** The risks are identified during the design reviews, site studies, communication with authorities and the comparison with similar business projects in Los Angeles.
2. **Risk Assessment:** The risks are rated on the probability and severity scales and the resultant placed on the 3x3 matrix to ascertain priority.
3. **Risk Response Planning:** High and moderate risks will have mitigation measures, whereas low level risks will be monitored and controlled by normal site practices.
4. **Monitoring and Control:** Risk register is revised every week in the coordination meetings, progress reviews, and safety inspection. The risks can be introduced or new risks can be added as the project develops and the risks can be revised in accordance with the real situation on the site.
5. It is a cycle to ensure proactive control of uncertainties during construction.

Table 5.6.2 Risk Score Table

Risk Code	Category	Risk Description	Possibility	Severity	Risk Score	Mitigation/Management Strategy
PM1	Project Management	Cost overruns due to material price escalation	3	3	9	Implement rigid cost monitoring, negotiate extended supplier deals, keep back-up deals.
PM2	Project Management	Schedule delays from permit or utility approval	2	3	6	Start an early coordination with the city agencies and utility providers; have a permit tracking system.

PM3	Project Managem ent	Insufficient coordination between contractors	2	2	4	Carry out weekly coordination meetings, trade integration through BIM tools.
D1	Design	Incomplete shop drawings or MEP conflicts	2	2	4	Clash detection. Review all the drawings and then build.
D2	Design	Noncompliance with seismic and ADA codes	1	3	3	Code to be reviewed regularly, use structural and accessibility consultants at the early stages.
C1	Construct ion	Site accidents or safety violations	3	2	6	Implementation of strict safety training, daily toolbox talks, and enforcement of PPE.
C2	Construct ion	Supply chain disruption or equipment breakdown	2	2	4	Have backup suppliers and equipment leasing contracts.
C3	Construct ion	Quality control issues during fit-out	2	2	4	Install inspection checklists and involve supervisors of QA/QC.
E1	Environm ental	Weather delays or flooding during excavation	3	2	6	Drainage and cover systems must be planned and adjusted schedule buffers for wet seasons should be presented.
E2	Environm ental	Environmental impact or	2	2	4	CEQA compliances must be followed,

		dust/noise complaints				utilization of dust barriers, and monitoring of air quality.
R1	External	Tenant lease-up risk or low occupancy post-opening	2	3	6	Early marketing, terms of flexible lease, and anchor tenant pre-commitment.
R2	External	Cost inflation or economic downturn	3	3	9	Establish price fix of materials in contracts and maintain cash reserves.
G1	Geotechnical	Unexpected soil conditions or groundwater intrusion	2	3	6	Carry out extensive geotechnical survey and dewatering plan.
G2	Geotechnical	Seismic activity (Los Angeles zone)	3	3	9	Application of reinforced concrete frames with ductile detailing and seismic isolators.

5.6.3 Risk Ownership

In order to achieve accountability, the risk categories will be allocated to the team members who would best handle them:

1. The project risks of project management: Project Manager.
2. Design risks: ARC, structural, and MEP design members.
3. Construction risks: Construction site security and general contractor.
4. Environmental risks: Environmental compliance team.
5. External/marketing risks: Leasing and developer team.

6. Geotechnical/seismic risks: Seismic and structural engineers.

Their responsibilities are to monitor the risks assigned to them, mitigation, and to report changes in the course of the coordination meetings.

Table 5.6.3 Risk Matrix

Probability of a dangerous event	Minor	Moderate	Serious
Rare	D2, C2	–	–
Possible	D1, PM3, C3, E2	PM2, G1, R1	–
Almost certain	–	PM1, C1, E1	R2, G2

Overall, the risk assessment includes high-priority risks, including cost overruns (PM1), seismic hazards (G2), and economic instability (R2) because they are critical to the project success. These are checked by making financial plans beforehand, the strengthening of structural design, and coordination of stakeholders. The moderate risks such as weather disruptions, utility delays, and safety are addressed using scheduling buffers and high supervision on-site. Normal risks are managed under the normal operating procedures. This will be done through continuous monitoring, documentation and periodical review of the risk register so that the project is resilient, compliant and efficient throughout its life cycle.

5.7 Quality Management

Quality Management is the important part of construction management that is responsible for ensuring that the project's processes and outputs meet required standards and specifications. This includes planning quality requirements, carrying out construction work correctly, monitoring and checking the work through quality control and eliminating defects. Good quality management is a positive contributor to the success of the project as it minimizes the rework, costs, enhances safety and makes sure that the work is regulated in accordance with the requirements of the regulations.

Quality Control is the crucial process in construction of ensuring that the project is meeting according to the approved plans, industry and engineering requirements and that all construction stages and the final product comply with the quality standards set by the architect,

engineer, owner and general contractor (Ashokkumar, 2014). Quality control steps for this project were divided into:

1. Quality Control Planning:

Set quality requirements, standards and inspection procedures prior to the beginning of construction.

2. Testing of materials and samples:

Conduct tests of materials and construction components to ensure they are meeting specifications.

3. Training and on-site practices:

Training of workers and supervisors on approved construction methods, quality standards and safety requirements.

4. Inspection plan:

Scheduled and regular inspections at various stages of construction to confirm the quality of work and compliance with standards.

5. Recording:

Document inspection results, test reports, and non-conformances for traceability.

6. Corrective actions:

Identify defects and apply corrective measures to prevent non-conformances from happening again.

A quality checklist is an important tool of quality control and is used to inspect and verify that construction work meets project specifications and standards. It provides a systematic way to check materials and workmanship at each construction stage, helping to ensure consistency and detecting defects.

5.7.1 Quality Metrics

To ensure that the project meets all the required standards, a quality management framework is established centered on seven core metrics. These metrics prioritize design accuracy & structural integrity, financial integrity & budget performance, contractor & vendor qualification, procurement & material quality, safety & regulatory compliance, environmental

sustainability & LEED performance and quality control & field inspection. These metrics help to give measurable quality evaluation for each stage of the project.



Figure 5.7.1 Quality Metrics.

5.7.2 Quality Checklist

A quality checklist is an important tool in construction quality management that helps ensure all project activities, materials and workmanship meet required standards, specifications, and regulatory codes. The use of quality checklists improves consistency, reduces rework and supports the successful delivery of the project to the required standards of quality and performance.

Table 5.7.2. Quality Checklists.

“KHAN TENGRI MALL” 130 W College St, Los Angeles, California. Quality checklist		
Prepared by:		
Date:		
1. Pre-Construction Quality Checklist		
Item	Inspection Criteria	Confirmed?
Design drawings available	Approved	✓ /X
Permits obtained	All legal approvals complete	✓ /X
Material submittals approved	Concrete, steel, finishes approved	✓ /X
Survey control points verified	Coordinates accurate	✓ /X
Qualified subcontractors selected	Licensed and experienced	✓ /X
2. Material Quality Checklist		
Item	Inspection Criteria	Confirmed?
Cement	Proper brand, no moisture damage	✓ /X
Rebar	Correct diameter, grade, rust free	✓ /X
Concrete	Correct mix design, slump tested	✓ /X
Aggregates	Clean and graded	✓ /X
Structural steel	Certified material	✓ /X
Finishing materials	Match approved samples	✓ /X
3. Foundation Works Checklist		

Item	Inspection Criteria	Confirmed?
Excavation depth	Matches drawings	✓ /X
Soil bearing level	Verified by engineer	✓ /X
Pile position	Correct coordinates	✓ /X
Reinforcement cage	Size and spacing correct	✓ /X
Foundation concrete	Proper compaction and curing	✓ /X
4. Structural Concrete Checklist		
Item	Inspection Criteria	Confirmed?
Formwork alignment	Correct dimensions	✓ /X
Rebar spacing	As drawings	✓ /X
Rebar lap length	As code	✓ /X
Embedded items	Installed correctly	✓ /X
Concrete slump	Within approved range	✓ /X
Vibration	Proper compaction	✓ /X
Surface finish	No honeycombing	✓ /X
Curing	Minimum required days	✓ /X
5. Steel / Roof / Framing Checklist		
Item	Inspection Criteria	Confirmed?
Member size	Matches drawings	✓ /X
Bolt tightening	Proper torque	✓ /X
Weld quality	No defects	✓ /X
Protective coating	Applied fully	✓ /X
6. Architectural Finishes Checklist		
Item	Inspection Criteria	Confirmed?

Wall plaster	Smooth and level	✓ /X
Floor	Proper alignment	✓ /X
Ceiling works	Secure installation	✓ /X
Paint works	Uniform finish	✓ /X
Doors/windows	Operable and aligned	✓ /X
Glass installation	No cracks	✓ /X
7. MEP Quality Checklist		
Item	Inspection Criteria	Confirmed?
Electrical conduits	Correct routing	✓ /X
Wiring tests	Passed insulation tests	✓ /X
Plumbing pipes	No leakage	✓ /X
HVAC ducts	Sealed and balanced	✓ /X
Fire alarm system	Functional	✓ /X
Sprinkler system	Pressure tested	✓ /X

5.8 Procurement Planning

Effective procurement planning is the crucial tool of any successful construction project. It provides a structured framework for identifying, sourcing and managing all materials, heavy equipment and subcontractor services required for the build. Establishing vendor prequalification criteria and internal financial controls helps actively mitigates supply chain risks, prevents budget overruns and guarantees that all deliverables meet required quality and safety standards.

5.8.1 Procurement Process

The procurement process for this project is a structured framework to successfully identify, source and manage all necessary materials, heavy equipment, and subcontractor services. Rather than simply purchasing items as needed, it relies on strict vendor

prequalification and internal financial controls to actively mitigate supply chain risks and prevent costly budget overruns.

1. Scope Definition and Resource Allocation

Identification of quantity of material, equipment and services required for this project. Defining structural and architectural specific requirements upfront to prevent budget creep during the bidding phase.

2. Vendor Risk Assessment and Prequalification

Evaluating potential suppliers and subcontractors prior to the bidding phase. This process ensures that only competent and fully capable of executing the project's scope partners are authorized to submit formal proposals.

3. Competitive Bidding and Bid Leveling

Issuing formal Requests for Proposals (RFPs) and Requests for Quotations (RFQs) exclusively to the approved vendor list. Once the bids are received, they will be carefully reviewed and leveled to ensure a direct and fair comparison of costs, material specifications and delivery schedules. This evaluation allows the team to select the most competitive bid.

4. Contract Award and Legal Documentation

Once the optimal bids are selected, the project management team will formally award the contracts to the successful vendors. This phase involves executing binding legal agreements and issuing structured Purchase Orders for all required materials and services. These documents define the scope of work, final pricing and establish fixed delivery schedules.

5. Expediting, Receiving, and Quality Control

Conduct inspections of received materials and completed stages of work. After inspections are approved, complete the financial reconciliation by matching the supplier's final invoice directly against the original purchase order. This step ensures the exact amount and quality of the work meet the original agreement before any money is paid.

5.8.2 Procurement Documents

The procurement process for this project is managed through structured documents designed to maintain a clean audit trail and prevent budget overruns. Each document serves a specific functional role in the lifecycle of a purchase

Table 5.8.2. Procurement Documents.

Procurement Document Item	Description
Purchase Requisition	An internal document used by the project team to request approval for materials or services. It verifies that the requested items align with the established baseline budget before any external vendor is engaged.
Request for Proposal and Request for Quotation	Formal solicitation documents sent to pre-qualified vendors. Request for quotation requests direct pricing on clearly defined materials, while request for proposal asks for comprehensive solutions and pricing for complex subcontractor services.
Bill of Quantities	A detailed, itemized list of all materials, labor, and equipment needed for the build. It provides all bidding vendors with the exact same parameters to ensure a standardized, easily comparable bidding process.
Purchase Order	A binding document issued to the selected supplier, formally authorizing the purchase. It specifies precise quantities, agreed-upon prices, and delivery dates, and serves as a strict financial basis for the order.
Receiving Report	A physical log completed at the construction site upon receipt of materials. It serves as documentary evidence that the correct materials were delivered, in proper condition, and in the requested quantity.

Supplier Invoice	The official bill provided by the vendor for the delivered materials or completed work. To prevent overbilling and maintain a clean audit trail, this document must be strictly reconciled against both the original purchase order and the receiving report before any payment is authorized.
-------------------------	--

5.8.3 Source selection criteria

The other step of the procurement process is establishing clear source selection criteria. Each potential supplier and subcontractor are evaluated by established benchmarks to determine the best option in terms of finances and competitiveness.

Table 5.8.3. Source Selection Criteria.

Source selection criteria	Description
Necessity	Evaluates whether the requested materials or services are strictly required to complete the project. This ensures every purchase directly supports the Work Breakdown Structure and prevents unnecessary spending or scope creep.
Cost and Price Competitiveness	Compares vendor bids against the established baseline budget to find the best financial value. This evaluation goes beyond simply picking the lowest price by factoring in long-term value, transportation costs, and the absence of hidden fees.
Technical Expertise and Quality Standards	Suppliers and subcontractors must be capable of meeting exact project specifications and quality requirements.

Professional Certification	Verifies that vendors have all mandatory industry licenses and qualifications.
-----------------------------------	--

5.8.4 Vendor List

The list of approved suppliers for this project consists of pre-qualified companies categorized by their specific contribution to the construction:

Table 5.8.4. Vendor List.

Company Name	Category	Contact Information
Cemex	Structural Materials - Concrete	Address: 601 Pier 'D' Ave, Long Beach, CA, 90802 Phone: 844-532-3639 Website: https://www.cemexusa.com/
Commercial Metals Company	Structural Materials - Steel	Address: 15050 Santa Ana Ave. Fontana, CA 92337 Phone: 909-873-3060 Website: https://www.cmc.com/
Griffith Company	Earthwork and Foundation	Address: 420 Hindry Ave, Suite D Inglewood, CA 90301 Phone:

		(714) 984-5500 Website: https://griffithcompany.net/ Email: mail@griffithcompany.net
ACCO Engineered Systems	Mechanical, Electrical and Plumbing	Address: 888 E. Walnut Street Pasadena, CA 91101 Phone: 818-244-6571 Website: https://www.accoes.com
Southern California Edison	Utility Providers (Commercial power Supply)	Phone: 1-800-655-4555 Website: www.sce.com Email: economicdevelopment@sce.com
SoCalGas	Utility Providers (Natural gas Supply)	Address: 4619 S Central Ave, Los Angeles, CA 90011, United States Phone: 1-800-427-2000 Website: https://www.socalgas.com/
Santa Clarita Valley Water	Utility Providers (water supply)	Website (Online Registration): https://yourscvwater.com/bid-opportunities

5.9 Construction safety

Considering that the construction industry is one of the most dangerous industries, one of the main goals of construction management is to ensure safety for all workers at the site. Without proper safety measures, accidents can result in serious injuries, loss of life, project delays and financial losses. According to the International Labor Organisation (2021), at least 60,000 deaths occur annually in construction sites worldwide. To ensure safe working conditions, all construction work must be carefully planned and risks must be identified and controlled before work begins. This includes ensuring proper worker training, enforcing the use of personal protective equipment (PPE), conducting regular site inspections, and maintaining clear communication between all team members. Continuous monitoring, reporting, and improvement of safety practices are essential for mitigating risks and protecting the health and well-being of everyone on site.

5.9.1 Hazard Identification and Risk Assessment

Hazard identification and risk assessment are crucial in construction safety management, because it helps to identify potential dangers before work begins. Common hazards include working at heights, heavy machinery operation, electrical risks, and excavation activities. By conducting site analysis and job safety assessments, these risks can be identified and controlled through appropriate measures.

A risk matrix can be used to assess potential hazards by evaluating likelihood and severity.

Table 5.9.1. Risk Matrix Example.

Risk Identification	Likelihood (1-5)	Severity (1-5)	Risk Score
Falls form height	5	5	25
Struck-by objects	4	4	16
Electrical hazards	3	5	15

Equipment and Machinery Accidents	3	4	12
Noise exposure	4	1	4

5.9.2 Hierarchy of controls

Hierarchy of control is a tool to prioritize control measures according to its effectiveness. It outlines five application areas: elimination, replacement, engineering controls, administrative controls, and personal protective equipment, from most effective to least effective.

According to Ajslev et al. (2022), safety efforts are heavily concentrated in lower-level controls, while higher-level controls are rarely implemented. Administrative controls account for approximately 53.6% of implemented measures, while engineering controls represent 35%, whereas higher-level controls such as elimination and substitution constitute only 3.6% in total.

It demonstrates that current safety approaches may not be as effective as possible. To improve safety, more attention needs to be paid to the implementation of highly effective measures.

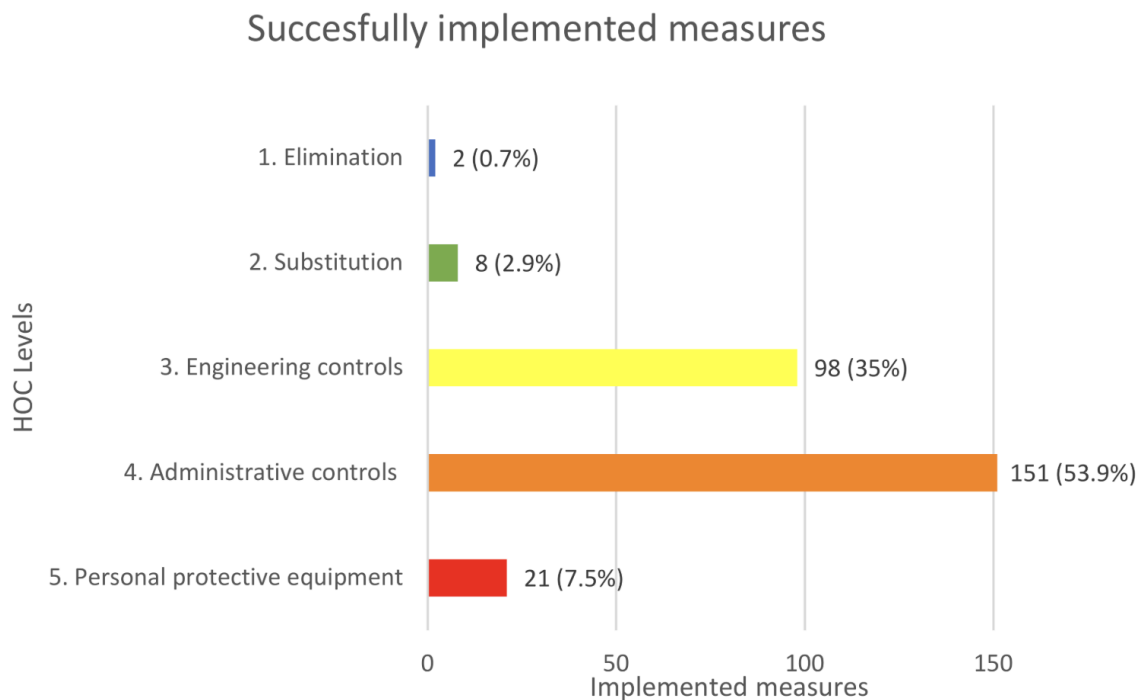


Figure 5.9.2. *Percentage of implemented measures per area.*

5.9.3 Safety Measures

Safety measures and controls are implemented to minimize potential hazards on construction sites to ensure a safe working environment. By applying these control measures, the potential risks can be significantly reduced, improving safety performance on site.

1. Engineering controls: Guardrails, barriers and safety nets.
2. Administrative controls: Safe work procedures, method statements, supervision and scheduling of work activities.
3. Personal Protective Equipment: Helmets, gloves, safety boots and high-visibility clothes.
4. Fall protection systems: Guardrails, safety nets and personal fall arrest systems.
5. Equipment and machinery safety: Regular inspection and maintenance. Operation by trained and authorized workers.
6. Material handling and storage: Safe lifting technique and proper stacking.
7. Monitoring: Regular checks to ensure compliance with safety requirements.

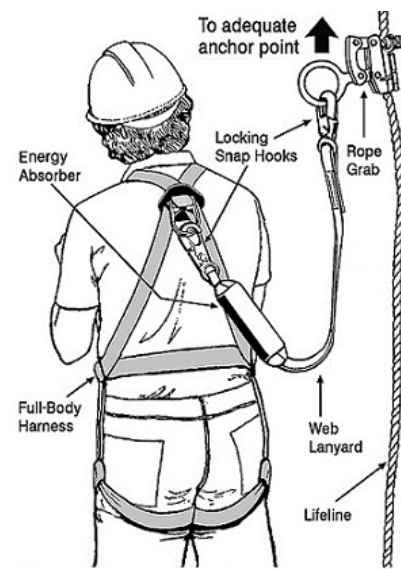


Figure 5.9.3.1. *Fall protection equipment.*



Figure 5.9.3.2. *Personal Protective Equipment.*

5.9.4 Working Training

Construction worker training is a crucial aspect of effective safety management and plays a critical role in reducing accidents, injuries, and productivity losses on construction projects. The training system should begin with mandatory site orientation before work starts and continue through regular toolbox talks, refresher courses and specialized competency training for high-risk tasks.

All workers shall complete mandatory site safety induction before starting work, including PPE use, emergency procedures, hazard communication, fall protection, and Cal/OSHA requirements. In addition, all general construction workers should complete OSHA 10 Hour Construction Training, which provides fundamental knowledge of common construction hazards. Supervisors, foremen, and safety managers should complete OSHA 30 Hour Construction Training, which offers more advanced instruction in hazard recognition, safety responsibilities, accident prevention, and regulatory compliance.

Throughout the construction phases, safety briefings must be held regularly to maintain hazard awareness and reinforce safe work practices among all personnel. A safety briefing should be held daily before each shift, discussing the current work on site, potential risks, weather conditions and preventative measures. Weekly safety meetings should be held to review incidents, near misses, upcoming high-risk tasks and concerns of workers.

Conducting training reduces the likelihood of accidents, injuries, equipment damage and project delays. Most importantly, it fosters a strong safety culture in which all workers understand their responsibilities, openly communicate hazards and actively participate in maintaining a safe workplace.

5.9.5 Inspections

To ensure effective implementation of safety measures, inspection tools such as OSHA construction safety checklists can be used. These checklists provide a systematic approach to identifying hazards and verifying compliance across key aspects such as general safety, scaffolding, excavations and trenches, fall protection, electrical systems, machinery guarding, hazard communication, personal protective equipment, material handling and first-aid and emergency preparedness. These inspections should be performed on a daily basis.

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 5.9.5. OSHA Construction Safety Inspection Template. Adapted from Safetymint. OSHA Construction Safety Inspection Checklist.

5.10 Construction Site Planning

Site planning is an important part of construction management as it determines the location of various facilities during the construction process. The effective planning positively affects the quality and speed of construction as well as safety.

The construction site planning for this project is located between the intersection of W College and N Alameda and intersection of N Main and Bruno streets. Office buildings, canteen, WC and locker rooms are located on the other side of W College street. Both office buildings and construction site are surrounded by a fence. Gates with a checkpoint makes sure that only authorized people have access.

The building itself is located on the north side of the site. There are two construction cranes on the east and west sides of the building, covering the entire area of the building. Excavated soil and waste material stocks are located on the west side of the site. The equipment, reinforcement stockpile and material stockpile areas are located next to each other on the east side of the construction site. This arrangement allows for efficient movement of vehicles and workers.

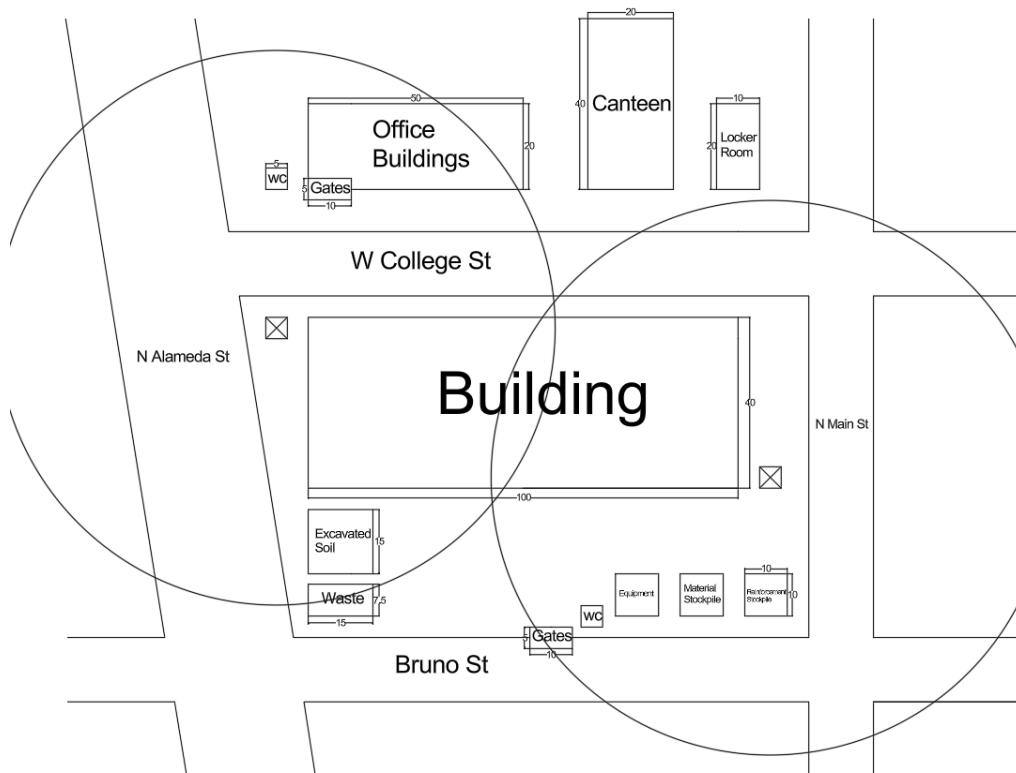


Figure. 5.10. Construction Site Plan.

6. Environmental engineering

6.1 Waste management options

6.1.1 Recycling potential

The waste generation of the proposed Los Angeles shopping mall is 970 kg/day (354,050 kg/year or 354 tons/year). The composition of the waste stream was broken down into the following major material categories: food waste, paper/cardboard, plastics, glass, metals, textiles/wood, and fines.

To develop a sustainable and feasible waste management strategy that complies with California regulatory standards, different waste treatment options need to be evaluated. This assessment takes into account the practical constraints such as recycling rates, contamination and system inefficiencies - not conceptually looking just at diversion. Therefore, this section defines the upper bounds of the amount of kilograms that can be realistically recycled, composted, or used to recover energy.

The practical recycling rates for the selected streams consider sorting and processing inefficiencies, and are based on the 50 States of Recycling report. It is important to note that while glass makes up part of the waste stream, it is not included in the recycling potential and revenues because it usually generates low or no revenue due to its low market value, transportation costs, and contamination problems. The realistic recycling potential and estimated revenues for the mall's high-value streams are shown in Table 6.1.1.

Table 6.1.1. Realistic recycling potential and revenue

Material type	Daily Weight (kg)	Recycle rate (%)	Yearly Recycled Weight (tons)	Market Value (\$/year)
Mixed paper	81.5	60	17.84	535.20
OCC	122.2	60	26.77	2,677.00
PET plastic	17.5	56	3.57	892.50
HDPE plastic	34.9	24	3.06	1,300.50
Aluminum can	7.3	77	2.04	3,060.00
Non-Aluminum metal	41.2	29	4.36	654.00

Total	304.6		57.64	\$9,119.20
--------------	--------------	--	--------------	-------------------

Technical Limitations of Material Recovery

Recycling rates shown in Table 6.1.1 are not 100% since the potential generation does not translate into actual recovery. The material recovery efficiency in the commercial sector is highly compromised by contamination of the material (e.g., liquids on mixed paper or food on HDPE/PET bottles) which instantly declassifies the material to residual waste to landfill. The 50 States of Recycling report also factors in "yield losses" at the Material Recovery Facility (MRF) level, where optical sorter and manual picking lines are only 80-85% efficient. As such, the practical recycling potential considers a conservative 15-20% yield loss at the processing stage and 10-15% at the source-separation stage, thus justifying the targeted recovery weights and avoiding unrealistic and over-optimistic financial returns.

6.1.2 Composting potential

According to the waste composition summary, the shopping mall generates 368.6 kg/day of food waste or 134,539 kg/year (134.5 tons/year). California's SB 1383 classifies food waste as an organic waste which needs to be diverted from landfills.

The composting potential calculation is based on a 75% diversion rate, which is in line with the state goal of SB 1383 to reduce the rate of organic waste disposal by 75% by 2025. The diverted organics will be off-loaded off-site by licensed waste haulers working for the LASAN RecycLA franchise.

Table 6.1.2. Composting potential

Source	Annual food waste (tons)	Diversion rate	Annual diverted (tons)
Food court	134.5	75%	100.9

6.1.3 Incineration potential

Incineration (also termed as waste-to-energy) is the combustion of solid waste. Incineration can provide a volume reduction of up to 90%, and so it is an option where landfill capacity is scarce. Not all solid wastes are suitable for incineration; combustible is defined as having a heat value. Combustibles are generally organic (carbon-containing) materials, and non-combustibles are inorganic (metal, glass, etc.) materials without energy potential.

According to the mall's waste generation data, the total mass of the major combustible materials (food waste, paper and plastics) is 272,619 kg/year. The Higher Heating Values (HHV) of the materials are 4.2 MJ/kg for food, 15.8 MJ/kg for mixed paper and 32.8 MJ/kg for mixed plastics.

The total energy recoverable is calculated as shown in equation:

$$\text{Energy (MJ)} = \text{Mass (kg)} \times \text{HHV (MJ/kg)}$$

Table 6.1.3 Incineration potential

Material	Annual Mass (kg/year)	HHV (MJ/kg)	Energy Potential (MJ/year)
Food waste	134,539	4.2	565,064
Paper (mixed + OCC)	74,351	15.8	1,174,746
Plastics	63,729	32.8	2,090,311
Total	272,619		3,830,121

In practice, waste-to-energy plants convert this thermal energy to electricity with an efficiency of roughly 20% for small-to-medium scale facilities. The net electricity generation is obtained as follows:

$$3,830,121 \text{ MJ/year} \times 0.2 \times 0.2778 \text{ kWh/MJ} = 212,801 \text{ kWh/year}$$

Evaluation of Incineration Viability

Although the incineration energy potential (212,801 kWh/year) is high, waste-to-energy (incineration) is ultimately de-prioritized in the Integrated Solid Waste Management plan. Moisture in the mall's main waste stream, food waste from the grocery and food court (about 70% moisture content by weight), severely impairs the combustion efficiency of an incinerator. The energy required to boil this water (latent heat of vaporization) reduces the net energy yield that can be extracted from the Higher Heating Value (HHV). Further, incineration undermines the ISWM's intent to reduce greenhouse gas emissions, since the combustion of plastics emits fossil CO₂ and potentially harmful dioxins, while recycling displaces the need to produce new plastics and retains their embodied carbon.

6.2 Waste management plan

The 4-storey mall produces an annual solid waste of around 354 tons. The core functional spaces which generate solid waste are the grocery and food court (highly perishable organic waste) and the retail/electronics/cinema (mainly paper, plastics and residuals). The system will focus on the segregation of recyclables, organics and residuals, as per the local regulations.

6.2.1 Central bin allocation

Volumetric sizing and density justification

Recycleables: The recyclable stream presents a unique volumetric challenge due to its exceptionally low bulk density. The mall generates approximately 350 kg/day of total recyclables. If left uncompacted, mixed commercial recyclables (plastics, aluminum, mixed paper, and cardboard) have an average bulk density of roughly 50 kg/m³ (U.S. EPA, 2016).

- **Daily volume calculation:** $\frac{350 \text{ kg/day}}{50 \text{ kg/m}^3} = 7 \text{ m}^3/\text{day}$
- **Conversion to imperial:** $7 \text{ m}^3/\text{day} \times 1.308 = 9.15 \text{ yd}^3/\text{day}$.

On average, the mall would produce 32 yd³ of uncompacted recyclables (assuming a twice-weekly hauler collection schedule, allowing for up to 3.5 days worth of recyclables to accumulate). Using only standard 3 yd³ bins would require placing eleven bins in the central enclosure, which is inefficient and costly.

This spatial constraint mathematically supports the proposed 60 inch OCC vertical baler. Old Corrugated Cardboard (OCC) comprises about 122.2 kg of the daily volume, and by sending it to the baler, it is removed from the bin set-up. The OCC is then baled at a density of more than 400 kg/m³. The remaining mixed recyclables (227.8 kg/day) is then reduced to 5.9 yd³/day. Over the course of 3.5 days this equates to 20 yd³ which is efficiently housed in seven 3 yd³ commercial recycling dumpsters, while the OCC is stored on pallets.

Organic waste: Organic waste from the food court and recyclables found in the retail and visitor areas, means that central loading docks will need to be commercial sized. The sizing of commercial 3-cu yd bins is based upon the standard bulk density of normal non-compacted commercial waste. For instance, 368.6 kg/day of heavy food waste is generated from the grocery and food court. The bulk density of food waste is around 450 kg/m³ (U.S. EPA, 2016).

- **Daily volume calculation:** $\frac{368.6 \text{ kg/day}}{450 \text{ kg/m}^3} = 0.819 \text{ m}^3/\text{day}$
- **Conversion to imperial:** $0.819 \text{ m}^3/\text{day} \times 1.308 = 1.07 \text{ yd}^3/\text{day}$

The facility will generate about 3.21 cubic yards of organic waste over a maximum of three days from the previous pickup (e.g., Friday to Monday). Thus, two 3-cu yd green bins will provide 6 cubic yards of capacity, and a volume safety factor of close to 2.0 will prevent overflow, health risks and pest problems during busy weekend periods. Similar volume-to-density calculations were performed for the uncompacted blue and black bins to determine their bin quantities.

Landfill: The residual stream consists of heavily mixed dry waste, sweepings, non-recyclable films, and contaminated materials. The estimated generation rate for this residual fraction is 251.4 kg/day. The standard bulk density of commercial uncompacted residual waste is approximately 100 kg/m³ (U.S. EPA, 2016).

- **Daily volume calculation:** $\frac{251.4 \text{ kg/day}}{100 \text{ kg/m}^3} = 2.514 \text{ m}^3/\text{day}$
- **Conversion to imperial:** $2.514 \text{ m}^3/\text{day} \times 1.308 = 3.28 \text{ yd}^3/\text{day}$

For a twice-weekly municipal collection, the uncompacted accumulation reaches 11.48 cubic yards per cycle. While this could theoretically be managed by four standard 3 yd³ dumpsters, utilizing a 2 yd³ stationary compactor is far more efficient. Commercial stationary compactors easily achieve a compaction ratio of 4:1 for general municipal solid waste.

- **Compacted volume calculation:** $\frac{11.48 \text{ yd}^3}{4} = 2.87 \text{ yd}^3$

This calculation proves that routing the residual stream through a 2yd³ or 3yd³ stationary compactor bin allows the facility to easily bridge the 3.5-day gap between hauler pickups without the risk of overflow. It drastically reduces the physical footprint of the residual waste enclosure and minimizes the required frequency of expensive black-bin hauler visits.

Table 6.1.2. Central waste enclosure bins

Bin color	Waste type	Bin size	Collection frequency	Location
Green	Food waste (grocery & food court)	Two 3-cu yd	Thrice Weekly	Central enclosure
Blue	Recyclables (retail/visitor)	Three 3-cu yd	Twice Weekly	Central enclosure

Black	Landfill (visitor/residual)	Two 3-cu yd	Twice Weekly	Central enclosure
-------	-----------------------------	-------------	--------------	-------------------

6.2.2 Floor-level bin placement

At-grade collection stations will be placed strategically on the four retail plates. Green bins will only be located where food is prepared or eaten to avoid extreme contamination. All bins will comply with SB 1383's color codes (green for organics, blue for recyclables, black for landfill) and will carry educational labels with information on what can be disposed of in each bin.

Table 6.2.2. Floor-level bin placement

Location	Stations	Bins per Station	Total Bins	Bin Colors
Food Court (3rd Floor)	6	3	18	Green, Blue, Black
Supermarket (1st Floor)	2	3	6	Green, Blue, Black
Retail Corridors	12	2	24	Blue, Black
Restrooms	8	1	8	Black
Administrative Offices	3	2	6	Blue, Black
Total			62	

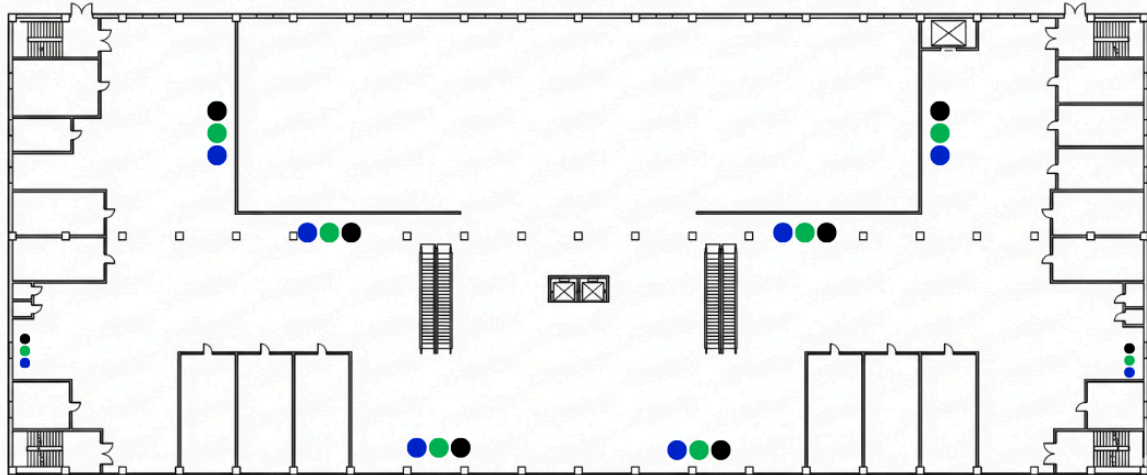


Figure 6.2.2. Bin placement for one of the floors

6.2.3 Collection schedule

The outdoor collection process is designed to ensure hygiene, prevent spillage and to increase the value of clean recyclables. On collection days, the dock attendant is responsible for making the bins available to local franchise haulers before 6:00 AM.

Table 6.2.3. Collection schedule

Waste stream	Collection days	Provider
Food organics (Green bins)	Monday, Wednesday, Friday	LASAN/Private Hauler
Retail recyclables (Blue bins)	Monday, Thursday	LASAN
Landfill (Black bins)	Tuesday, Friday	LASAN

6.2.4 Bin quantity verification

To ensure the size of the bins is adequate to the building scale, the minimum total storage area is calculated as per the mall's total floor space (approximately 160000 ft²). The area is sufficient to accommodate the 7 proposed 3-cu yd central bins, and maintain sufficient clearance for haulers and custodial carts.

Table 6.2.4. Minimum required collection space calculation

Component	Calculation	Result (ft ²)
Base requirement (first 25000 ft ²)	Base	312
Additional space	$\frac{160000-25000}{1000} \times 16$	2160
Total Minimum Space Required		2160

6.3 Waste collection plan

6.3.1 Internal logistics and custodial routing

970 kg of waste to be collected per day in a very busy retail area demands a proper routing exercise to avoid the spread of odors and impact of collection on the customer experience.

- **Midday collection (2:00 PM):** Collection is strictly focused on putrescible waste areas (Grocery and Food Court), the collection for this shift collects only putrescible waste. Custodians will use heavy-duty, leak-proof tilt trucks (1-cu yd standard capacity) to transport the heavy organics. For safety and floor protection, carts will not be overloaded (no more than 150 kg allowed during transport). Carts will be sent to the logistic elevator, avoiding all public and passenger elevators.
- **End-of-day collection (10:00 PM):** This is a major sweep for the dry waste streams (retails, offices, cinemas). Given that the bulk density of dry recyclables (paper, plastic) is much lower (30-50 kg/m³), custodians will be equipped with high-capacity, multi-stream custodial carts that will enable the simultaneous collection of the blue and black bins while avoiding cross-contamination.

6.3.2 Contamination management at the dock

Once transported to the central enclosure at ground level, the waste does not just get dumped. The dock attendant is obliged to conduct a "top level visual audit". If the attendant identifies more than 10% apparent organic contamination in a blue (recycling) cart (e.g., a leaking food container over the paper), then he/she must divert that particular cart's contents into the black (landfill) compactor. This lowers the recycling diversion rate, but it is essential to achieve quality control and avoid having the entire 3-cu yd external recycling bin rejected by the hauler and incurred with fines for contamination.

6.3.3 Waste flow and personnel responsibilities

- **Waste flow:** Green waste transport is vertical dependent. The mall has 4 retail spine escalators and 2 guest elevators, but solid waste will be solely transported from the upper floors by 1 logistic elevator. This guarantees that recycling carts do not interfere with the customer flow.
- **Personnel:**
 - Custodial Staff: Based on an industry standard of 28,000 ft² per custodian, the 160,000 ft² facility requires approximately 6.15 Full-Time Equivalents (FTEs). Therefore, a team of 6 to 7 custodians will be divided between day and night shifts.
 - Dock Attendant (1-2 part-time): Assigned to monitor the central enclosure, check for severe contamination, and unlock gates for external haulers.
 - Facility Manager (1): Responsible for enforcing the waste management plan, conducting periodic waste audits, and supervising staff compliance.

6.3.4 Staff training requirements

Source separation is critical for the success of an integrated solid waste management system. The mall manager is mandated (SB 1383, California Code of Regulations, Section 18984.10) to provide training.

- New employee orientation:: All employees, particularly those working in organic waste areas such as the grocery and food court, are required to take a waste-sorting orientation during the first 14 days on the job.
- Annual refresher training: Annual training for all mall employees to reiterate the color system for green, blue and black bins and the importance of the economic/environmental costs of cross-contamination.

6.4 Integrated Solid Waste Management (ISWM) plan

6.4.1 System goal and elements

The main objective of the integrated solid waste management (ISWM) plan for the Khan Tengri Mall is to divert waste from landfill to minimize residual waste. This design pushes the diversion of all organic waste to composting and maximises the collection of recyclables to preserve their material value. Any other impact of this ISWM system, such as economic gains from recyclables or meeting California's SB 1383 and AB 341 standards, are considered as positive side effects of this primary environmental goal.

For the ISWM plan to be successful in protecting the purity of the diverted streams and achieving this diversion-oriented objective, it includes the following elements:

- **Bin location:** Concentrated sorting stations on the 1st floor of the grocery and 3rd floor of the food court to capture the greatest amount of organic waste.
- **Strict color-coding:** Universal compliance with the conventional green (organics), blue (recyclables) and black (landfill) code to enable quick visual sorting and avoid contamination.
- **Staff training:** Initial and annual training of custodial and tenant staff.
- **Pick-up frequency:** Regular high frequency external pick-ups (particularly for green bins) to avoid overfill, and subsequent spillover (and incorrect disposal) of organics into the black stream.
- **Hauler selection:** Use of LASAN and private franchise haulers that have the certified infrastructure to transfer streams to commercial composting facilities and Material Recovery Facilities (MRFs) instead of landfills.

6.4.2 Plan modifications based on ISWM goal

In order to reach the newly established first goal of maximum diversion and minimum residual stream footprint, the following technical upgrades have been incorporated into the waste stream management plan:

1. Installation of an OCC vertical baler: Since Old Corrugated Cardboard (OCC) generates the best return on the market, a vertical baler will be installed in the central loading dock. This will compact OCC into mill-ready bales, avoiding contamination with other streams, optimising storage capacity and reducing the carbon emissions associated with hauling.
2. Implementation of a residual waste compactor: To reduce the volume of the residual (landfill) stream, a stationary compactor will be used. This reduces the number of required black-bin collections, and indirectly encourages staff to accurately divert organics and recyclables to their uncompacted, high-frequency collection bins.

6.4.3 ISWM flowchart

The flowchart below illustrates the material movement of the ISWM system - from source to off-site disposal. This process is tailored to separate, process and secure the revenue streams.

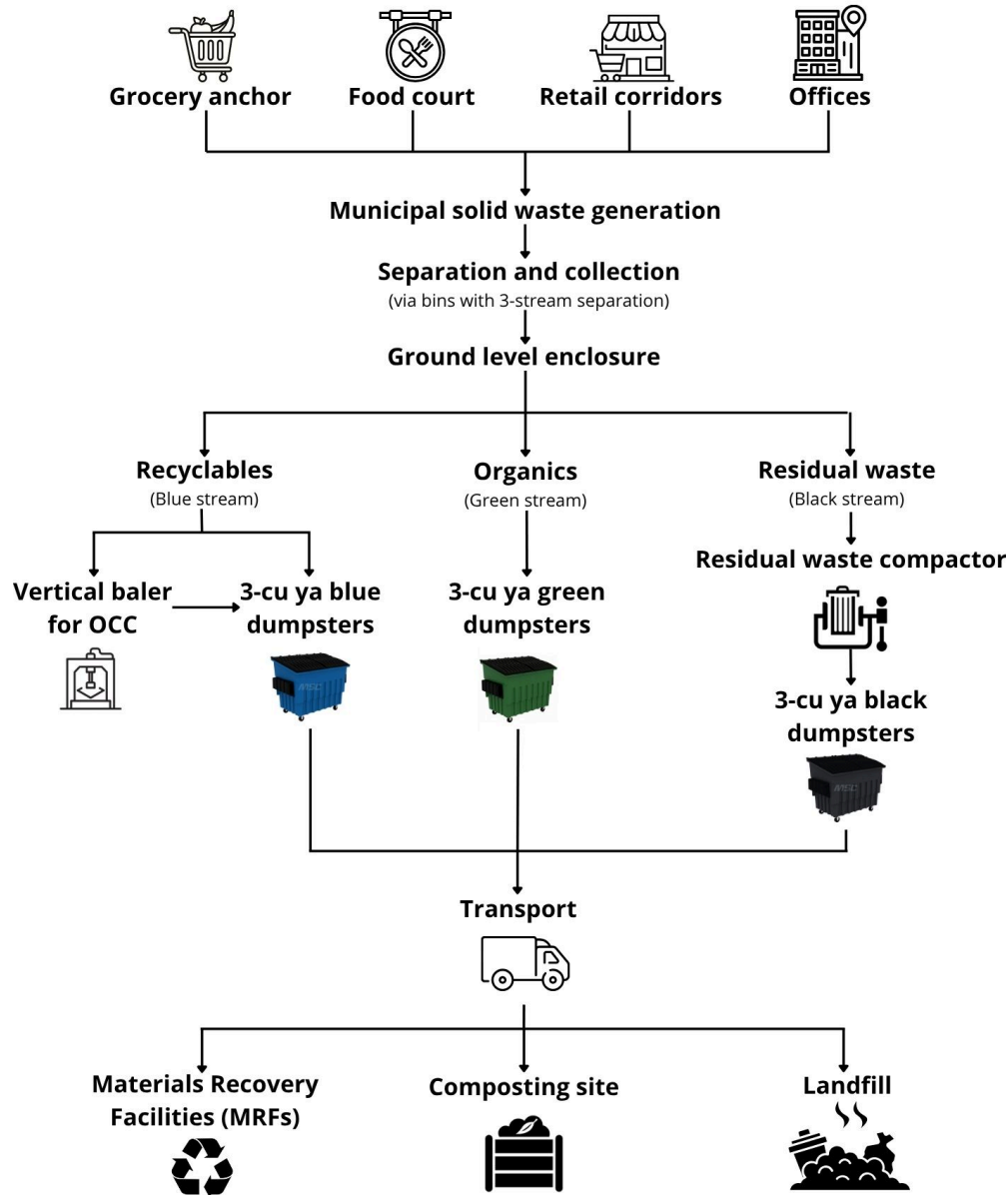


Figure 6.4.3. ISWM flowchart

6.5 Operational finances of waste management plan

This section discusses the financial aspects of the Khan Tengri Mall Integrated Solid Waste Management (ISWM) plan. The financial analysis is based on the start-up capital costs, the annual operating costs (labor and hauling), and the expected revenues from the sale of sorted recyclables.

The capital investments are the initial purchase of collection bins and heavy-duty solid waste processing equipment required for a large-scale commercial enterprise (160000 ft²). Using simple bins is not enough to achieve an efficient commercial system; thus, processing equipment is incorporated to comply with industry practice (Tchobanoglous et al., 1993).

- **Central enclosure bins:** Seven 3-cu yd metal dumpsters. Estimated at \$800 per unit = \$5,600.
- **Floor-level sorting stations:** 62 commercial-grade stations. To reflect realistic commercial pricing (e.g., Rubbermaid Commercial or similar), these are estimated at \$300 per 3-bin station = \$18,600.
- **Vertical baler (OCC):** One standard 60-inch vertical baler for cardboard processing \$12,000 (CalRecycle equipment market estimates).
- **Stationary compactor:** One 2-cu yd stationary compactor for residual waste \$=\$ \$18,000 (CalRecycle equipment market estimates).

Total Initial Capital Cost: \$54,200.

To accurately reflect the annualized burden of these capital investments in the operational budget, the costs are amortized. Assuming a standard useful lifespan of 10 years for heavy solid waste equipment (U.S. EPA, 1997) and an interest rate of 5%, the Capital Recovery Factor (CRF) is utilized to find the Annualized Capital Cost (A):

$$A = P \frac{i(1+i)^n}{(1+i)^n - 1}$$

Where,

- P = 54,200,
- i = 0.05,
- n = 10

$$A = 54200 \times \frac{0.05(1+0.05)^{10}}{(1+0.05)^{10} - 1} \approx \$7000/\text{year}$$

2.5.2 Operational costs (OPEX)

Operational costs represent the comprehensive annual expenses required to maintain the ISWM system. This analysis expands beyond basic wages to include employee benefits, utilities, and equipment upkeep.

- **Internal labor costs:** The waste management logistics require approximately 1.5 Full-Time Equivalents (FTEs). The mean annual wage in the Los Angeles metropolitan area is roughly \$35,000. Factoring in a standard 30% for fringe benefits (insurance, payroll taxes), the true labor cost is \$45,500 per FTE.

- *Calculation:* $\$1.5 * \$45,500 = \$68,250/\text{year}$.
- **External hauling fees:** Based on Los Angeles Sanitation and Environment (LASAN) commercial franchise rates:
 - Organics (Thrice weekly): \$4,500/year
 - Recyclables (Twice weekly): \$3,500/year
 - Landfill (Twice weekly): \$4,000/year
 - *Total Hauling:* \$12,000/year.
- **Equipment Maintenance & Supplies:** Routine maintenance for the baler and compactor (estimated at 5% of their combined \$30,000 capital cost), plus heavy-duty liners and baling wire. Estimated at \$3,500/year.
- **Utilities:** Electrical consumption for the baler and compactor. Estimated at 5,000 kWh/year at the LA commercial rate of \$0.20/kWh = \$1,000/year.

Total Annual Operational Cost (OPEX): \$84,750/year.

2.5.3 Projected revenue

The rigorous source separation and use of an OCC baler protects the purity and marketability of the recyclables. Utilizing the 50 States of Recycling report commodity values, the estimated annual revenue is:

- Old Corrugated Cardboard (OCC): \$2,677.00
- High-Density Polyethylene (HDPE): \$1,300.50
- Polyethylene Terephthalate (PET): \$892.50
- Mixed Paper: \$535.20
- Aluminum Cans: \$3,060.00
- Non-Aluminum Metals: \$654.00

Total Annual Revenue: ≈\$9,100/year.

2.5.4 Net cost summary

The final economic performance of the ISWM system calculates the true net financial impact by combining OPEX and Annualized CAPEX, offset by material revenue.

- **Total Annualized Capital Cost (Amortization):** \$7000/year
- **Total Annual Operational Cost (OPEX):** \$84,750.00/year
- **Gross Annual System Cost:** \$91,750/year
- **Total Annual Revenue:** \$9,100/year
- **Net Operational Cost:** $\$91,750 - \$9,100 = \$82,650/\text{year}$

While the ISWM system operates at a net cost of \$82,650 annually, this represents a highly detailed, realistic utility profile for a 160000 ft² commercial facility. By introducing proper

compaction and baling infrastructure, the mall meets its environmental objective of residual minimization while successfully recovering over 10% of its waste management expenditures through protected recyclable sales.

Reference List

- ASCE Hazard Tool. (n.d.). *ASCE 7 Hazard Tool*. American Society of Civil Engineers. Retrieved October 3, 2025, from <https://ascehazardtool.org>
- American Concrete Institute. (2019). ACI 318-19: Building Code Requirements for Structural Concrete. ACI.
- American Society of Civil Engineers. (2010). ASCE/SEI 7-10: Minimum Design Loads for Buildings and Other Structures. ASCE.
- American Society of Civil Engineers. (2022). ASCE/SEI 7-22: Minimum Design Loads and Associated Criteria for Buildings and Other Structures. ASCE.
- City of Los Angeles. (2025). *Sewer pipes (GIS dataset)*. Los Angeles GeoHub. <https://geohub.lacity.org/datasets/sewer-pipes>
- Gershunov, A., Guzman-Morales, J., & Ilana, W. (2021). Hot and cold flavors of Santa Ana winds: Their causes, trends and impacts. *Climate Dynamics*, 57, 2765–2785. <https://doi.org/10.1007/s00382-021-05802-z>
- Google Maps. (2025). *130 W College St, Los Angeles, CA 90012*. Retrieved October 3, 2025, from <https://www.google.com/maps>
- LA County Public Works. (2025). *Storm drain system map*. Los Angeles County Flood Control District. <https://pw.lacounty.gov/fcd/StormDrain>
- LAFD (Los Angeles Fire Department). (2025). *Fire Zone Map*. Los Angeles Fire Department. <https://lafd.org/fire-prevention/brush/fire-zone/fire-zone-map>
- SunCalc. (2025). *Sun path and solar position calculator for 130 W College St, Los Angeles, CA*. <https://www.suncalc.org>
- Western Regional Climate Center (WRCC). (n.d.). *Station daily climate summary for Los Angeles, CA (Station CA5115)*. Desert Research Institute. Retrieved October 3, 2025, from <https://wrcc.dri.edu/cgi-bin/cliMAIN.pl?ca5115>
- Bilodeau, W. L., Bilodeau, S. W., Gath, E. M., Osborne, M., & Proctor, R. J. (2007). Geology of Los Angeles, California, United States of America. *Environmental and Engineering Geoscience*, 13(2), 99–160. <https://doi.org/10.2113/gseegeosci.13.2.99>

California Department of Resources Recycling and Recovery (CalRecycle). (n.d.). *Estimated Solid Waste Generation Rates*.

<https://www2.calrecycle.ca.gov/wastecharacterization/general/rates>

CalRecycle. (n.d.). Short-Lived Climate Pollutants (SLCP): Organic Waste Methane Emissions Reductions (SB 1383). State of California.

City of Los Angeles Department of Public Works. (2002). *Solid Waste Existing Conditions*.

https://planning.lacity.gov/eir/2k_Ave_of_Stars/text/solid.pdf

Tchobanoglous, G., Theisen, H., & Vigil, S. (1993). *Integrated Solid Waste Management: Engineering Principles and Management Issues*. McGraw-Hill.

Biocycle. (2024). *State of Disposal and Recycling in California*. Retrieved from

<https://www.biocycle.net/ca-disposal-and-recycling/>

US Environmental Protection Agency (EPA). (2023). *Municipal Solid Waste Generation, Recycling, and Disposal in the United States: Facts and Figures for 2021*.

<https://www.epa.gov/facts-and-figures-about-materials-waste-and-recycling/national-overview-facts-and-figures-materials>

Los Angeles Sanitation. (2023). *Recycling and Waste Management in Los Angeles*.

<https://www.lacitysan.org>

Planning Los Angeles. (2002). *Solid Waste Management and Disposal in Los Angeles*.

https://planning.lacity.gov/eir/2k_Ave_of_Stars/text/solid.pdf

California Integrated Waste Management Board. (2024). *Waste Management and Recycling Resources for Commercial Buildings*.

City of Los Angeles Sanitation (LASAN). (n.d.). *RecycLA Commercial Waste and Recycling Franchise System*.

Ball Corporation & Eunomia Research & Consulting. (2023). *The 50 States of Recycling: A state-by-state assessment of U.S. packaging recycling rates*. Ball Corporation.

https://www.ball.com/getmedia/eb3620b7-e8af-44af-83cd-fb8606753600/50-STATES_2023-V12.pdf

LA Sanitation & Environment. (n.d.). Home. City of Los Angeles. Retrieved December 1, 2025, from <https://sanitation.lacity.gov/>

United States Environmental Protection Agency (USEPA). (2022). *Energy Recovery from the Combustion of Municipal Solid Waste (MSW)*.

Minimum design loads and associated criteria for buildings and other structures. (2017). In

American Society of Civil Engineers eBooks. <https://doi.org/10.1061/9780784414248>

Wolnitzek, R. (2012, April 12). *Scuppers*. Architekwiki.

<https://www.architekwiki.com/wiki/scuppers>

GZ Industrial Supplies Limited. (2025, June 13). *Terms of use*. GZ Industrial Supplies.

<https://www.gz-supplies.com/terms-of-use>. [gz-supplies.com](https://www.gz-supplies.com)

Concrete Reinforcing Steel Institute. (2025). *CRSI product catalog — 3rd edition (Spring 2025)*.

CRSI. (Publication). members.crsi.org

Funk, S. W. / SW Funk Industrial. (2025, July 7). *Standard industrial plant design: Tips for maximum efficiency*. SW Funk Industrial — News.

<https://swfunk.com/news/standard-industrial-plant-design-tips/> . swfunk.com

MDPI. (2024). *MDPI 2024 Annual Achievements Report*. MDPI.

https://mdpi-res.com/data/mdpi_annual_achievements_report_2024.pdf

Reclamation cost escalation Factor for 2025: 4.22%. (2024). Department of Natural Resources
Division of Oil, Gas and

Mining. <https://ogm.utah.gov/wp-content/uploads/2024/12/Escalation-Memo-2025-signed.pdf>

2025 RSMeans data Cost Book Updates. (n.d.).

<https://www.rsmeans.com/landing-pages/2025-rsmeans-cost-index>

Edzarenski. (2024, February 28). *Construction inflation 2022*. Construction Analytics.

<https://edzarenski.com/2022/02/11/construction-inflation-2022>

Urban Land Institute. (2023). Dollars & cents of shopping centers & malls.

<https://knowledge.uli.org/-/media/files/emerging-trends/2026/uli-pwc-emerging-trends-in-real-estate-us-canada-2026.pdf?rev=f54dee472276423aa181e523f7cd3cb2&hash=68A2BC47E7CCB1D1F1AE3E36BF4453A6>

1272-1276 w Court St. (n.d.). www.loopnet.com. Retrieved November 23, 2025, from

<https://www.loopnet.com/viewer/pdf?file=https%3a%2f%2fimages1.loopnet.com%2fd2%2fv1OTnU4MXXurv0z4p2fj1n2Vf29WpSmWljKRyaIFnLc%2f127212741276%2520W%2520CourtOM.pdf>

- Philippe Properties. (2025). Los Angeles Commercial Real Estate Market Q1 2025: Office, Retail, Multifamily.
<https://www.philippeproperties.com/blog/los-angeles-commercial-real-estate-market-2025-trends-office-industrial-retail-multifamily>
- Smith, S. (2025, September 15). Retail rents keep climbing in LA; average almost \$3.1/sf. LinkedIn.
https://www.linkedin.com/posts/steven-smith-88427a43_retail-rents-keep-climbing-in-la-average-activity-7369523943059161088-P8re
- Matthews Real Estate Investment Services. (2025). Los Angeles Retail Market Report Q3 2025.
https://www.matthews.com/market_insights/los-angeles-retail-market-report-q325
- Urban Land Institute (ULI). (2023). Reshaping Retail.
<https://www.scribd.com/document/470656913/ULI-Europe-Reshaping-Retail-Final-pdf>
- Felten, B. (2025, October 5). ICSC released compiled numbers by retail activity. LinkedIn.
https://www.linkedin.com/posts/brentfelten_icsc-released-compiled-numbers-by-retail-activity-7374810543565615104-J8P1
- CrossTown LA. (2025, March 19). Drop in parking tickets leads to widening city deficit.
<https://xtown.la/2025/03/19/drop-in-parking-tickets-leads-to-widening-city-deficit/>
- FlashParking. (2023). National Parking Association announces findings in post-pandemic road to recovery index.
<https://www.flashparking.com/blog/national-parking-association-announces-findings-in-post-pandemic-road-to-recovery-index/>
- D.Ashokkumar. (2014). Study of Quality Management in Construction Industry. *International Journal of Innovative Research in Science Engineering and Technology*.
https://www.researchgate.net/publication/375379774_Study_of_Quality_Management_in_Construction_Industry

Occupational safety and health management in the construction sector. (2021, January 29).

International Labour Organization.

<https://www.ilo.org/meetings-and-events/occupational-safety-and-health-management-construction-sector-3>

Ajslev, J. Z. N., Møller, J. L., Andersen, M. F., Pirzadeh, P., & Lingard, H. (2022). The hierarchy of Controls as an approach to visualize the impact of occupational safety and health coordination. *International Journal of Environmental Research and Public Health*, 19(5), 2731. <https://doi.org/10.3390/ijerph19052731>

Safetymint (2024). OSHA Construction Safety Inspection Checklist. *Checklists - Safety Audit and Inspection Checklists*.

<https://www.safetymint.com/checklists/osha-construction-safety-inspection/>

Appendix

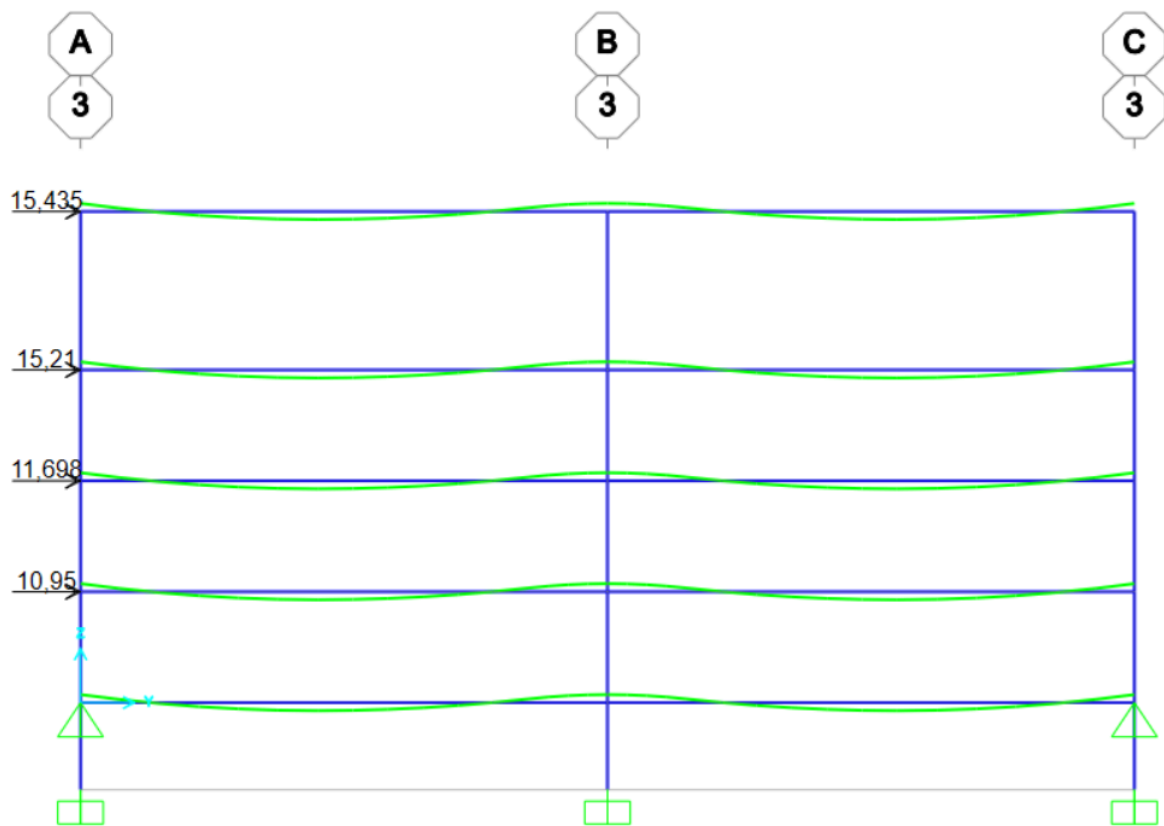


Figure 9.2 Load Assignment for Case II Wind Load (Frame C)

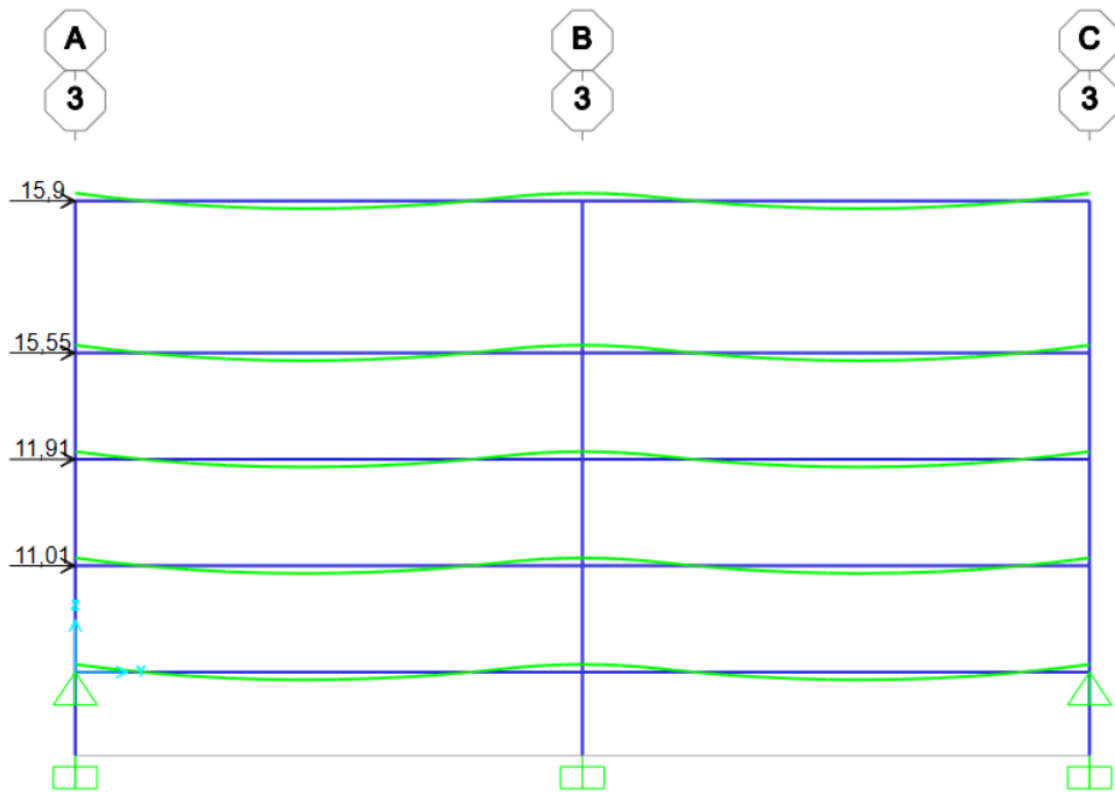


Figure 9.3 Load Assignment for Case III Wind Load (Frame C)

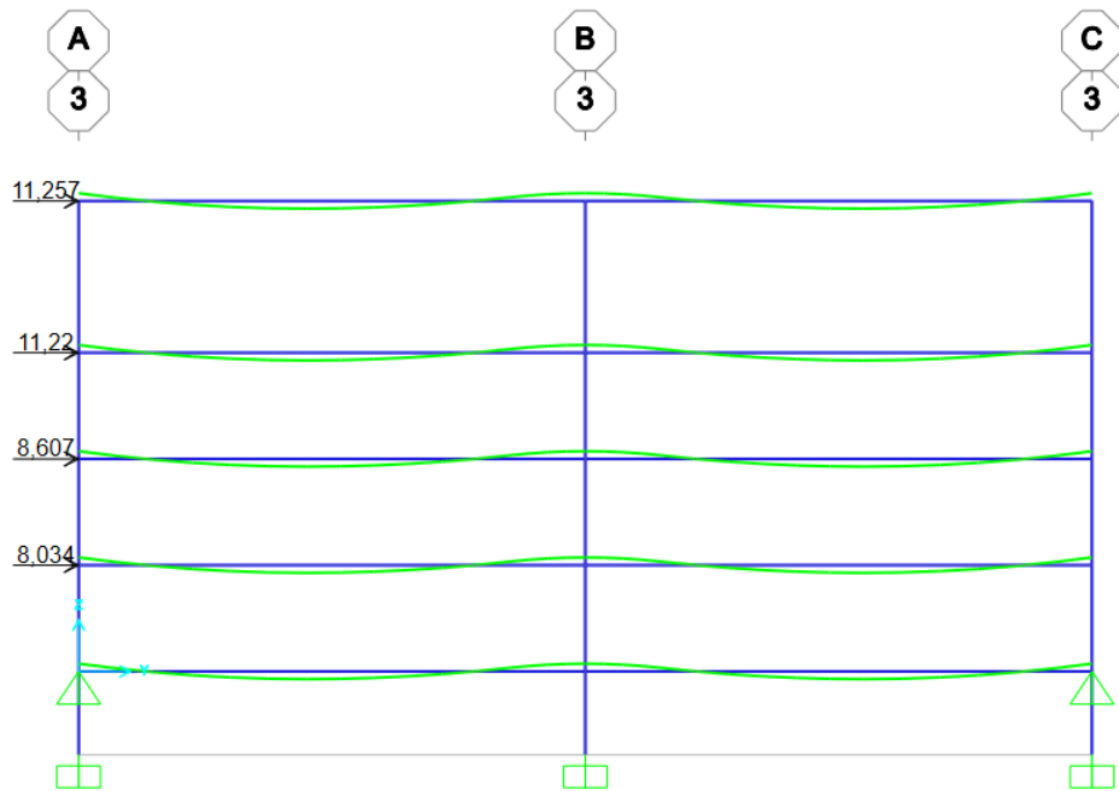


Figure 9.4 Load Assignment for Case IV Wind Load (Frame C)

Table 9.1. Load calculation for exterior columns in typical span

Exterior Column (1 - 4 floor) - 5 Span- Concrete Wall							
Floor	DL	RL	LL	SL	AT	Factored Load	R(kN)
Basement Slab	6.08	0	2.75	0	50	11.68	584.19
Floor 1 Slab	6.14	0	2.75	0	50	11.75	587.74
Floor 2 Slab	6.10	0	2.75	0	50	11.71	585.42
Floor 3 Slab	6.73	0	2.75	0	50	12.47	623.48
Floor 4 Slab	4.08	0.624	0.00	0	50	5.21	260.64

Table 9.2. Load calculation for exterior columns in typical span including large span beam self-weight

20 Beam Self Weight					
Floor	B Self Weight	wb	Rb	Add Rcol	Factored Load
Basement Slab	122.50	6.13	61.25	61.25	73.50
Floor 1 Slab	122.50	6.13	61.25	61.25	73.50
Floor 2 Slab	122.50	6.13	61.25	61.25	73.50
Floor 3 Slab	122.50	6.13	61.25	61.25	73.50
Floor 4 Slab	122.50	6.13	61.25	61.25	73.50

Table 9.3. Load calculation for exterior columns in typical span including typical span beam self-weight

5 Beam Self Weight					
Floor	B Self Weight	wb	Rb	Add Rcol	Factored Load
Basement Slab	14.14	2.83	7.07	14.14	16.97
Floor 1 Slab	14.14	2.83	7.07	14.14	16.97
Floor 2 Slab	14.14	2.83	7.07	14.14	16.97
Floor 3 Slab	14.14	2.83	7.07	14.14	16.97
Floor 4 Slab	14.14	2.83	7.07	14.14	16.97

Table 9.4. Load calculation for exterior columns in typical span including typical and minor beam self-weight

Exterior + Beams + Walls + Minor Beams+Parapets						
Floor	Wall Self Weight	wb	Rb	Add Rcol	Factored Load	Total R col
Basement Slab	84.55	16.91	42.27	84.55	101.46	791.68

Floor 1 Slab	84.55	16.91	42.27	84.55	101.46	795.24
Floor 2 Slab	84.55	16.91	42.27	84.55	101.46	792.92
Floor 3 Slab	121.94	24.39	60.97	121.94	146.33	875.85
Floor 4 Slab	0.00	0.00	0.00	0.00	0.00	373.43
parapets	-	1.12	2.81	5.62	6.75	
Minor Beam SW	8.65	1.73	4.32	12.97	15.57	

Table 9.5. Iterative method for minimum exterior columns in typical span size estimation

Exterior Column Size Estimation 5 span											
Basement											
Iteration	b (mm)	H (mm)	p (kN/m^3)	W1 (kN)	W1+axial	φ	fc (MPa)	fy	Ag	b'	Final
1	1000.00	3300.00	25.00	99.00	3728.11	0.65	40.00	420.00	189367.11	435.16	500.00
2	435.16	3300.00	25.00	18.75	3647.86	0.65	40.00	420.00	168147.34	410.06	
3	410.06	3300.00	25.00	16.65	3645.75	0.65	40.00	420.00	168050.50	409.94	
Floor 1											
Iteration	b (mm)	H (mm)	p (kN/m^3)	W1 (kN)	W1+axial	φ	fc (MPa)	fy	Ag	b'	Final
1	1000.00	4200.00	25.00	126.00	2963.43	0.65	40.00	420.00	136598.70	369.59	450.00
2	369.59	4200.00	25.00	17.21	2854.64	0.65	40.00	420.00	131584.10	362.75	
3	362.75	4200.00	25.00	16.58	2854.01	0.65	40.00	420.00	131554.98	362.71	
Floor 2											
Iteration	b (mm)	H (mm)	p (kN/m^3)	W1 (kN)	W1+axial	φ	fc (MPa)	fy	Ag	b'	Final
1	1000.00	4200.00	25.00	126.00	2168.19	0.65	40.00	420.00	99942.34	316.14	400.00
2	316.14	4200.00	25.00	12.59	2054.78	0.65	40.00	420.00	94714.84	307.76	
3	307.76	4200.00	25.00	11.93	2054.12	0.65	40.00	420.00	94684.48	307.71	
Floor 3											
Iteration	b (mm)	H (mm)	p (kN/m^3)	W1 (kN)	W1+axial	φ	fc (MPa)	fy	Ag	b'	Final
1	1000.00	4200.00	25.00	126.00	1375.27	0.65	40.00	420.00	63392.99	251.78	300.00
2	251.78	4200.00	25.00	7.99	1257.26	0.65	40.00	420.00	57953.22	240.73	
3	240.73	4200.00	25.00	7.30	1256.57	0.65	40.00	420.00	57921.63	240.67	

Floor 4											
Iteration	b (mm)	H (mm)	p (kN/m^3)	W1 (kN)	W1+axial	ϕ	fc (MPa)	fy	Ag	b'	Final
1	1000.00	6000.00	25.00	180.00	553.43	0.65	40.00	420.00	25510.14	159.72	300.00
2	159.72	6000.00	25.00	4.59	378.02	0.65	40.00	420.00	17424.72	132.00	
3	132.00	6000.00	25.00	3.14	376.56	0.65	40.00	420.00	17357.64	131.75	

Table 9.6 Load calculation for exterior columns in large span

Exterior Column (1 - 4 floor) - 20 Span Concrete Wall							
Floor	DL	RL	LL	SL	AT	Factored Load	R(kN)
Basement Slab	6.08	0	2.75	0	50	11.68	584.19
Floor 1 Slab	6.14	0	2.75	0	50	11.75	587.74
Floor 2 Slab	6.10	0	2.75	0	50	11.71	585.42
Floor 3 Slab	6.73	0	2.75	0	50	12.47	623.48
Floor 4 Slab	4.08	0.624	0.00	0	50	5.21	260.64

Table 9.7. Load calculation for exterior columns in large span including large span beam self-weight

20 Beam Self Weight					
Floor	B Self Weight	wb	Rb	Add Rcol	Factored Load
Basement Slab	122.50	6.13	61.25	122.50	147.00
Floor 1 Slab	122.50	6.13	61.25	122.50	147.00

Floor 2 Slab	122.50	6.13	61.25	122.50	147.00
Floor 3 Slab	122.50	6.13	61.25	122.50	147.00
Floor 4 Slab	122.50	6.13	61.25	122.50	147.00

Table 9.8. Load calculation for exterior columns in large span including typical span beam self-weight

5 Beam Self Weight					
Floor	B Self Weight	wb	Rb	Add Rcol	Factored Load
Basement Slab	14.14	2.83	7.07	7.07	8.49
Floor 1 Slab	14.14	2.83	7.07	7.07	8.49
Floor 2 Slab	14.14	2.83	7.07	7.07	8.49
Floor 3 Slab	14.14	2.83	7.07	7.07	8.49
Floor 4 Slab	14.14	2.83	7.07	7.07	8.49

Table 9.9. Load calculation for exterior columns in large span including typical and minor beam self-weight

Exterior + Beams + Walls + Minor Beams						
Floor	B Self Weight	wb	Rb	Add Rcol	Factored Load	Total R col
Basement Slab	338.18	16.91	169.09	338.18	405.82	1161.06
Floor 1 Slab	338.18	16.91	169.09	338.18	405.82	1164.62
Floor 2 Slab	338.18	16.91	169.09	338.18	405.82	1162.30
Floor 3 Slab	487.75	24.39	243.88	487.75	585.30	1379.83
Floor 4 Slab	0.00	0.00	0.00	0.00	0.00	458.69

parapets	-	1.12	11.25	22.50	26.99	
Minor Beam SW	8.65	1.73	4.32	12.97	15.57	

Table 9.10. Iterative method for minimum exterior columns in large span size estimation

Exterior Column Size Estimation 20 span											
Basement											
Iteration	b (mm)	H (mm)	p (kN/m^3)	W1 (kN)	W1+axial	φ	fc (MPa)	fy	Ag	b'	Final
1	1000.00	3300.00	25.00	99.00	5425.49	0.65	40.00	420.00	275584.85	524.96	600.00
2	524.96	3300.00	25.00	27.28	5353.78	0.65	40.00	420.00	246781.52	496.77	
3	496.77	3300.00	25.00	24.43	5350.93	0.65	40.00	420.00	246650.08	496.64	
Floor 1											
Iteration	b (mm)	H (mm)	p (kN/m^3)	W1 (kN)	W1+axial	φ	fc (MPa)	fy	Ag	b'	Final
1	1000.00	4200.00	25.00	126.00	4291.43	0.65	40.00	420.00	197812.95	444.76	550.00
2	444.76	4200.00	25.00	24.92	4190.36	0.65	40.00	420.00	193153.89	439.49	
3	439.49	4200.00	25.00	24.34	4189.77	0.65	40.00	420.00	193126.83	439.46	
Floor 2											
Iteration	b (mm)	H (mm)	p (kN/m^3)	W1 (kN)	W1+axial	φ	fc (MPa)	fy	Ag	b'	Final
1	1000.00	4200.00	25.00	126.00	3126.82	0.65	40.00	420.00	144130.10	379.64	450.00
2	379.64	4200.00	25.00	18.16	3018.98	0.65	40.00	420.00	139159.25	373.04	
3	373.04	4200.00	25.00	17.53	3018.35	0.65	40.00	420.00	139130.38	373.00	
Floor 3											
Iteration	b (mm)	H (mm)	p (kN/m^3)	W1 (kN)	W1+axial	φ	fc (MPa)	fy	Ag	b'	Final
1	1000.00	4200.00	25.00	126.00	1964.52	0.65	40.00	420.00	90554.28	300.92	350.00
2	300.92	4200.00	25.00	11.41	1849.93	0.65	40.00	420.00	85272.26	292.01	

350.00

3	292.01	4200.00	25.00	10.74	1849.26	0.65	40.00	420.00	85241.58	291.96	
Floor 4											
Iteration	b (mm)	H (mm)	p (kN/m ³)	W1 (kN)	W1+axial	φ	f _c (MPa)	f _y	A _g	b'	Final
1	1000.00	6000.00	25.00	180.00	638.69	0.65	40.00	420.00	29440.16	171.58	300.00
2	171.58	6000.00	25.00	5.30	463.99	0.65	40.00	420.00	21387.35	146.24	
3	146.24	6000.00	25.00	3.85	462.54	0.65	40.00	420.00	21320.54	146.02	

Table 9.11 Load calculation for corner columns in typical span

Corner Column							
Floor	DL	RL	LL	SL	AT	Factored Load	R(kN)
Basement Slab	6.08	0	3.39	0	25	12.71	317.74
Floor 1 Slab	6.14	0	3.39	0	25	12.78	319.52
Floor 2 Slab	6.10	0	3.39	0	25	12.73	318.36
Floor 3 Slab	6.73	0	3.39	0	25	13.50	337.39
Floor 4 Slab	4.08	0.888	0	0	25	5.34	133.62

Table 9.12. Load calculation for corner columns including large span beam self-weight

20 Beam Self Weight					
Floor	B Self Weight	w _b	R _b	Add R _{col}	Factored Load
Basement Slab	122.50	6.13	61.25	61.25	73.50
Floor 1 Slab	122.50	6.13	61.25	61.25	73.50
Floor 2 Slab	122.50	6.13	61.25	61.25	73.50
Floor 3 Slab	122.50	6.13	61.25	61.25	73.50
Floor 4 Slab	122.50	6.13	61.25	61.25	73.50

Table 9.13. Load calculation for corner columns including typical span beam self-weight

5 Beam Self Weight					
Floor	B Self Weight	wb	Rb	Add Rcol	Factored Load
Basement Slab	14.14	2.83	7.07	7.07	8.49
Floor 1 Slab	14.14	2.83	7.07	7.07	8.49
Floor 2 Slab	14.14	2.83	7.07	7.07	8.49
Floor 3 Slab	14.14	2.83	7.07	7.07	8.49
Floor 4 Slab	14.14	2.83	7.07	7.07	8.49

Table 9.14. Load calculation for corner columns including typical and minor beam self-weight

5 Beam Self Weight					
Floor	B Self Weight	wb	Rb	Add Rcol	Factored Load
Basement Slab	14.14	2.83	7.07	7.07	8.49
Floor 1 Slab	14.14	2.83	7.07	7.07	8.49
Floor 2 Slab	14.14	2.83	7.07	7.07	8.49
Floor 3 Slab	14.14	2.83	7.07	7.07	8.49
Floor 4 Slab	14.14	2.83	7.07	7.07	8.49

Table 9.15. Load calculation for corner columns including exterior wall self-weight in typical span

5 Wall Self Weight					
Floor	B Self Weight	wb	Rb	Add Rcol	Factored Load
Basement Slab	84.55	16.91	42.27	42.27	50.73

Floor 1 Slab	84.55	16.91	42.27	42.27	50.73
Floor 2 Slab	84.55	16.91	42.27	42.27	50.73
Floor 3 Slab	121.94	24.39	60.97	60.97	73.16
Floor 4 Slab	0.00	0.00	0.00	0.00	0.00
parapets	-	1.12	2.81	2.81	3.37

Table 9.16. Load calculation for corner columns including exterior wall self-weight in large span

20 Wall Self Weight + Minor Beams						
Floor	B Self Weight	wb	Rb	Add Rcol	Factored Load	Total R col
Basement Slab	338.18	16.91	169.09	169.09	202.91	661.15
Floor 1 Slab	338.18	16.91	169.09	169.09	202.91	662.93
Floor 2 Slab	338.18	16.91	169.09	169.09	202.91	661.76
Floor 3 Slab	487.75	24.39	243.88	243.88	292.65	792.97
Floor 4 Slab	0.00	0.00	0.00	0.00	0.00	240.26
parapets	-	1.12	11.25	11.25	13.50	
Minor Beam SW	8.65	1.73	4.32	6.49	7.78	

Table 9.17. Iterative method for minimum corner columns size estimation

Corner Column Size Estimation Typical											
Basement											
Iteration	b (mm)	H (mm)	p (kN/m ³)	W1 (kN)	W1+axial	φ	fc (MPa)	fy	Ag	b'	Final
1	1000.00	3300.00	25.00	99.00	3118.07	0.65	40.00	420.00	158380.46	397.97	450.00
2	397.97	3300.00	25.00	15.68	3034.75	0.65	40.00	420.00	139886.22	374.01	
3	374.01	3300.00	25.00	13.85	3032.92	0.65	40.00	420.00	139801.82	373.90	

Floor 1											
Iteration	b (mm)	H (mm)	p (kN/m^3)	W1 (kN)	W1+axial	φ	fc (MPa)	fy	Ag	b'	Final
1	1000.00	4200.00	25.00	126.00	2357.92	0.65	40.00	420.00	108687.97	329.68	400.00
2	329.68	4200.00	25.00	13.69	2371.62	0.65	40.00	420.00	109319.23	330.63	
3	330.63	4200.00	25.00	13.77	2371.69	0.65	40.00	420.00	109322.89	330.64	
Floor 2											
Iteration	b (mm)	H (mm)	p (kN/m^3)	W1 (kN)	W1+axial	φ	fc (MPa)	fy	Ag	b'	Final
1	1000.00	4200.00	25.00	126.00	1820.99	0.65	40.00	420.00	83938.47	289.72	350.00
2	289.72	4200.00	25.00	10.58	1705.57	0.65	40.00	420.00	78618.03	280.39	
3	280.39	4200.00	25.00	9.91	1704.90	0.65	40.00	420.00	78587.13	280.33	
Floor 3											
Iteration	b (mm)	H (mm)	p (kN/m^3)	W1 (kN)	W1+axial	φ	fc (MPa)	fy	Ag	b'	Final
1	1000.00	4200.00	25.00	126.00	1159.23	0.65	40.00	420.00	53434.52	231.16	300.00
2	231.16	4200.00	25.00	6.73	1039.96	0.65	40.00	420.00	47936.92	218.95	
3	218.95	4200.00	25.00	6.04	1039.27	0.65	40.00	420.00	47904.99	218.87	
Floor 4											
Iteration	b (mm)	H (mm)	p (kN/m^3)	W1 (kN)	W1+axial	φ	fc (MPa)	fy	Ag	b'	Final
1	1000.00	6000.00	25.00	180.00	420.26	0.65	40.00	420.00	19371.86	139.18	300.00
2	139.18	6000.00	25.00	3.49	243.75	0.65	40.00	420.00	11235.52	106.00	
3	106.00	6000.00	25.00	2.02	242.28	0.65	40.00	420.00	11168.01	105.68	

Table 9.18. Wind Force Calculations for first floor in both directions including the torsional effect (Case 2)

1st floor	
Case 2 (Y-axis)	Case 2 (X-axis)

Frame	Ftorsion (kN)	Fdirect (kN)	Ftotal (kN)	Frame	Ftorsion (kN)	Fdirect (kN)	Ftotal (kN)
1	-0.07	11.01	10.94	A	-0.79	8.13	7.34
2	-0.06	11.01	10.95	B	0.00	8.13	8.13
3	-0.06	11.01	10.95	C	0.79	8.13	8.92
4	-0.05	11.01	10.96				
5	-0.04	11.01	10.97				
6	-0.04	11.01	10.97				
7	-0.03	11.01	10.98				
8	-0.02	11.01	10.99				
9	-0.01	11.01	10.99				
10	-0.01	11.01	11.00				
11	0.00	11.01	11.01				
12	0.01	11.01	11.02				
13	0.01	11.01	11.02				
14	0.02	11.01	11.03				
15	0.03	11.01	11.04				
16	0.04	11.01	11.04				
17	0.04	11.01	11.05				
18	0.05	11.01	11.06				
19	0.06	11.01	11.06				
20	0.06	11.01	11.07				
21	0.07	11.01	11.08				

Table 9.19. Wind Force Calculations for second floor in both directions including the torsional effect (Case 2)

2nd floor							
Case 2 (Y-axis)				Case 2 (X-axis)			
Frame	Ftorsion (kN)	Fdirect (kN)	Ftotal (kN)	Frame	Ftorsion (kN)	Fdirect (kN)	Ftotal (kN)

1	-0.269	11.914	11.645	A	-1.094	8.974	7.880
2	-0.242	11.914	11.672	B	0.000	8.974	8.974
3	-0.215	11.914	11.698	C	1.094	8.974	10.067
4	-0.189	11.914	11.725				
5	-0.162	11.914	11.752				
6	-0.135	11.914	11.779				
7	-0.108	11.914	11.806				
8	-0.081	11.914	11.833				
9	-0.054	11.914	11.860				
10	-0.027	11.914	11.887				
11	0.000	11.914	11.914				
12	0.027	11.914	11.941				
13	0.054	11.914	11.968				
14	0.081	11.914	11.995				
15	0.108	11.914	12.022				
16	0.135	11.914	12.049				
17	0.162	11.914	12.075				
18	0.189	11.914	12.102				
19	0.215	11.914	12.129				
20	0.242	11.914	12.156				
21	0.269	11.914	12.183				

Table 9.20. Wind Force Calculations for fourth floor in both directions including the torsional effect (Case 2)

4th floor							
Case 2 (Y-axis)				Case 2 (X-axis)			
Frame	Ftorsion (kN)	Fdirect (kN)	Ftotal (kN)	Frame	Ftorsion (kN)	Fdirect (kN)	Ftotal (kN)
1	-0.588	15.905	15.317	A	-1.584	13.312	11.728

2	-0.529	15.905	15.376	B	0.000	13.312	13.312
3	-0.470	15.905	15.435	C	1.584	13.312	14.896
4	-0.411	15.905	15.493				
5	-0.353	15.905	15.552				
6	-0.294	15.905	15.611				
7	-0.235	15.905	15.670				
8	-0.176	15.905	15.728				
9	-0.118	15.905	15.787				
10	-0.059	15.905	15.846				
11	0.000	15.905	15.905				
12	0.059	15.905	15.964				
13	0.118	15.905	16.022				
14	0.176	15.905	16.081				
15	0.235	15.905	16.140				
16	0.294	15.905	16.199				
17	0.353	15.905	16.257				
18	0.411	15.905	16.316				
19	0.470	15.905	16.375				
20	0.529	15.905	16.434				
21	0.588	15.905	16.492				

Table 9.21. Wind Force Calculations for first floor in both directions including the torsional effect (Case 4)

1st floor							
Case 4 (Y-axis)				Case 4 (X-axis)			
Frame	Ftorsion (kN)	Fdirect (kN)	Ftotal (kN)	Frame	Ftorsion (kN)	Fdirect (kN)	Ftotal (kN)
1	-0.287	8.264	7.977	A	-2.767	6.102	3.336
2	-0.259	8.264	8.005	B	0.000	6.102	6.102

3	-0.230	8.264	8.034	C	2.767	6.102	8.869
4	-0.201	8.264	8.063				
5	-0.172	8.264	8.092				
6	-0.144	8.264	8.120				
7	-0.115	8.264	8.149				
8	-0.086	8.264	8.178				
9	-0.057	8.264	8.206				
10	-0.029	8.264	8.235				
11	0.000	8.264	8.264				
12	0.029	8.264	8.293				
13	0.057	8.264	8.321				
14	0.086	8.264	8.350				
15	0.115	8.264	8.379				
16	0.144	8.264	8.408				
17	0.172	8.264	8.436				
18	0.201	8.264	8.465				
19	0.230	8.264	8.494				
20	0.259	8.264	8.523				
21	0.287	8.264	8.551				

Table 9.22. Wind Force Calculations for second floor in both directions including the torsional effect (Case 4)

2nd floor							
Case 4 (Y-axis)				Case 4 (X-axis)			
Frame	Ftorsion (kN)	Fdirect (kN)	Ftotal (kN)	Frame	Ftorsion (kN)	Fdirect (kN)	Ftotal (kN)
1	-0.421	8.943	8.522	A	-4.052	6.736	2.684
2	-0.379	8.943	8.564	B	0.000	6.736	6.736
3	-0.337	8.943	8.607	C	4.052	6.736	10.788

4	-0.295	8.943	8.649	
5	-0.253	8.943	8.691	
6	-0.211	8.943	8.733	
7	-0.168	8.943	8.775	
8	-0.126	8.943	8.817	
9	-0.084	8.943	8.859	
10	-0.042	8.943	8.901	
11	0.000	8.943	8.943	
12	0.042	8.943	8.985	
13	0.084	8.943	9.028	
14	0.126	8.943	9.070	
15	0.168	8.943	9.112	
16	0.211	8.943	9.154	
17	0.253	8.943	9.196	
18	0.295	8.943	9.238	
19	0.337	8.943	9.280	
20	0.379	8.943	9.322	
21	0.421	8.943	9.364	

Table 9.23. Wind Force Calculations for fourth floor in both directions including the torsional effect (Case 4)

4th floor							
Case 4 (Y-axis)				Case 4 (X-axis)			
Frame	Ftortion (kN)	Fdirect (kN)	Ftotal (kN)	Frame	Ftortion (kN)	Fdirect (kN)	Ftotal (kN)
1	-0.853	11.939	11.086	A	-8.210	9.993	1.783
2	-0.768	11.939	11.171	B	0.000	9.993	9.993
3	-0.682	11.939	11.257	C	8.210	9.993	18.202
4	-0.597	11.939	11.342				

5	-0.512	11.939	11.427	
6	-0.426	11.939	11.513	
7	-0.341	11.939	11.598	
8	-0.256	11.939	11.683	
9	-0.171	11.939	11.769	
10	-0.085	11.939	11.854	
11	0.000	11.939	11.939	
12	0.085	11.939	12.024	
13	0.171	11.939	12.110	
14	0.256	11.939	12.195	
15	0.341	11.939	12.280	
16	0.426	11.939	12.366	
17	0.512	11.939	12.451	
18	0.597	11.939	12.536	
19	0.682	11.939	12.622	
20	0.768	11.939	12.707	
21	0.853	11.939	12.792	

Table 9.24. Stiffness Center Calculations

							Transverse	Longitudinal
Floor	Column size	I _{c,cr} , m ⁴	I _{b,cr} , m ⁴	Minor b, I _{b,cr}	D, kN/m	h, m	C _f , kN/rad	C _f , kN/rad
1	0.65	1.04E-02	1.70E-03	0	6041	4.2	507456	50746
2	0.55	5.34E-03	1.70E-03	0	5420	4.2	455295	45529
3	0.5	3.65E-03	1.70E-03	0	4937	4.2	414703	41470
4	0.5	3.65E-03	1.70E-03	0	2159	6	259065	25907

Table 9.25. Seismic Load Calculations for first floor in both directions including the torsional effect

1st floor Frame #	Y axis			X axis		
	Ftorsion, kN	Fdirect, kN	Ftotal, kN	Ftorsion, kN	Fdirect, kN	Ftotal, kN
Frame 1/A	-6.10	98.31	92.21	-111.37	98.31	-13.06
Frame 2/B	-5.49	98.31	92.82	0.00	98.31	98.31
Frame 3/C	-4.88	98.31	93.43	111.37	98.31	209.68
Frame 4	-4.27	98.31	94.04			
Frame 5	-3.66	98.31	94.65			
Frame 6	-3.05	98.31	95.26			
Frame 7	-2.44	98.31	95.87			
Frame 8	-1.83	98.31	96.48			
Frame 9	-1.22	98.31	97.09			
Frame 10	-0.61	98.31	97.70			
Frame 11	0.00	98.31	98.31			
Frame 12	0.61	98.31	98.92			
Frame 13	1.22	98.31	99.53			
Frame 14	1.83	98.31	100.14			
Frame 15	2.44	98.31	100.74			
Frame 16	3.05	98.31	101.35			
Frame 17	3.66	98.31	101.96			
Frame 18	4.27	98.31	102.57			
Frame 19	4.88	98.31	103.18			
Frame 20	5.49	98.31	103.79			
Frame 21	6.10	98.31	104.40			

Table 9.26. Seismic Load Calculations for second floor in both directions including the torsional effect

	Y axis	X axis
2nd floor		

Frame #	Ftortion, kN	Fdirect, kN	Ftotal, kN	Ftortion, kN	Fdirect, kN	Ftotal, kN
Frame 1/A	-12.37	197.77	185.40	-221.52	1384.39	1162.87
Frame 2/B	-11.13	197.77	186.64	0.00	1384.39	1384.39
Frame 3/C	-9.90	197.77	187.88	221.52	1384.39	1605.92
Frame 4	-8.66	197.77	189.11			
Frame 5	-7.42	197.77	190.35			
Frame 6	-6.18	197.77	191.59			
Frame 7	-4.95	197.77	192.82			
Frame 8	-3.71	197.77	194.06			
Frame 9	-2.47	197.77	195.30			
Frame 10	-1.24	197.77	196.53			
Frame 11	0.00	197.77	197.77			
Frame 12	1.24	197.77	199.01			
Frame 13	2.47	197.77	200.24			
Frame 14	3.71	197.77	201.48			
Frame 15	4.95	197.77	202.72			
Frame 16	6.18	197.77	203.95			
Frame 17	7.42	197.77	205.19			
Frame 18	8.66	197.77	206.43			
Frame 19	9.90	197.77	207.67			
Frame 20	11.13	197.77	208.90			
Frame 21	12.37	197.77	210.14			

Table 9.27. Seismic Load Calculations for fourth floor in both directions including the torsional effect

4th floor Frame #	Y axis			X axis		
	Ftortion, kN	Fdirect, kN	Ftotal, kN	Ftortion, kN	Fdirect, kN	Ftotal, kN

Frame 1/A	-45.955	701.90	655.94	-736.70	4913.28	4176.58
Frame 2/B	-41.359	701.90	660.54	0.00	4913.28	4913.28
Frame 3/C	-36.764	701.90	665.13	736.70	4913.28	5649.97
Frame 4	-32.168	701.90	669.73			
Frame 5	-27.573	701.90	674.32			
Frame 6	-22.977	701.90	678.92			
Frame 7	-18.382	701.90	683.51			
Frame 8	-13.786	701.90	688.11			
Frame 9	-9.191	701.90	692.71			
Frame 10	-4.595	701.90	697.30			
Frame 11	0.000	701.90	701.90			
Frame 12	4.595	701.90	706.49			
Frame 13	9.191	701.90	711.09			
Frame 14	13.786	701.90	715.68			
Frame 15	18.382	701.90	720.28			
Frame 16	22.977	701.90	724.87			
Frame 17	27.573	701.90	729.47			
Frame 18	32.168	701.90	734.06			
Frame 19	36.764	701.90	738.66			
Frame 20	41.359	701.90	743.26			
Frame 21	45.955	701.90	747.85			

Table 9.28 Detailed Major Beam Reinforcement Design for Roof in the Exterior Frame

5m Major Beam for Roof for Exterior frame			
Variables	SAP2000		
	End	Mid	End
SAP 2000 Tension Area, mm ²	650.3	510.8	710.25

Area, in ²	1.008	0.792	1.101
SAP 2000 Compression Area, mm ²	250.15	180.45	280.5
Area, in ²	0.388	0.280	0.435
Tension Bars	3#6	3#5	3#6
Compression Bars	2#5	2#4	2#5
Shear Reinforcement	#3 at 200 mm	#3 at 250 mm	#3 at 200 mm
Reinforcement Ratio	0.00695	0.00470	0.00695

Table 9.29 Detailed Major Beam Reinforcement Design for Floor 3 in the Exterior Frame

5m Major Beam for Floor 3 for Exterior frame			
Variables	SAP2000		
	End	Mid	End
SAP 2000 Tension Area, mm ²	980.75	860.5	1050.2
Area, in ²	1.520	1.334	1.628
SAP 2000 Compression Area, mm ²	370.3	260.9	400.45
Area, in ²	0.574	0.404	0.621
Tension Bars	4#6	3#6	4#6
Compression Bars	2#6	2#5	2#6
Shear Reinforcement	#3 at 150 mm	#3 at 200 mm	#3 at 150 mm
Reinforcement Ratio	0.00950	0.00695	0.00950

Table 9.30 Detailed Major Beam Reinforcement Design for Floor 2 in the Exterior Frame

5m Major Beam for Floor 2 for Exterior frame			
Variables	SAP2000		
	End	Mid	End
SAP 2000 Tension Area, mm ²	1250.8	950.2	1320.45

Area, in2	1.939	1.473	2.047
SAP 2000 Compression Area, mm2	430.6	310.75	460.25
Area, in2	0.667	0.482	0.713
Tension Bars	4#7	4#6	4#7
Compression Bars	2#6	2#5	2#6
Shear Reinforcement	#3 at 150 mm	#3 at 200 mm	#3 at 150 mm
Reinforcement Ratio	0.0118	0.0085	0.0118

Table 9.31 Detailed Major Beam Reinforcement Design for Floor 1 in the Exterior Frame

5m Major Beam for Floor 1 for Exterior frame			
Variables	SAP2000		
	End	Mid	End
SAP 2000 Tension Area, mm2	1380.5	1020.75	1450.6
Area, in2	2.140	1.582	2.248
SAP 2000 Compression Area, mm2	480.3	350.2	510.45
Area, in2	0.744	0.543	0.791
Tension Bars	4#7	4#6	4#7
Compression Bars	2#6	2#5	2#6
Shear Reinforcement	#3 at 150 mm	#3 at 200 mm	#3 at 150 mm
Reinforcement Ratio	0.0118	0.0085	0.0118

Table 9.32 Detailed Major Beam Reinforcement Design for Roof in the Interior Frame

5m Major Beam for Roof for Interior frame			
Variables	SAP2000		
	End	Mid	End
SAP 2000 Tension Area, mm2	720.4	580.6	780.3

Area, in2	1.117	0.900	1.209
SAP 2000 Compression Area, mm2	280.200	200.500	310.700
Area, in2	0.434	0.311	0.482
Tension Bars	3#6	3#5	3#6
Compression Bars	2#5	2#4	2#5
Shear Reinforcement	#3 at 200 mm	#3 at 250 mm	#3 at 200 mm
Reinforcement Ratio	0.00695	0.00470	0.00695

Table 9.33 Detailed Major Beam Reinforcement Design for Floor 4 in the Interior Frame

5m Major Beam for Floor 4 for Interior frame			
Variables	SAP2000		
	End	Mid	End
SAP 2000 Tension Area, mm2	850	830.5	910.2
Area, in2	1.318	1.287	1.411
SAP 2000 Compression Area, mm2	340.5	250.3	370.8
Area, in2	0.528	0.388	0.575
Tension Bars	4#5	2#7	3#8
Compression Bars	2#5	2#4	2#5
Shear Reinforcement	#3 at 180 mm	#3 at 250 mm	#3 at 180 mm
Reinforcement Ratio	0.00659	0.00570	0.01065

Table 9.34 Detailed Major Beam Reinforcement Design for Floor 3 in the Interior Frame

5m Major Beam for Floor 3 for Interior frame			
Variables	SAP2000		
	End	Mid	End

SAP 2000 Tension Area, mm2	1050.3	920.7	1120.5
Area, in2	1.628	1.427	1.737
SAP 2000 Compression Area, mm2	400.2	290.4	430.6
Area, in2	0.620	0.450	0.667
Tension Bars	4#6	3#7	4#6
Compression Bars	2#6	2#5	2#6
Shear Reinforcement	#3 at 150 mm	#3 at 200 mm	#3 at 150 mm
Reinforcement Ratio	0.00950	0.00864	0.00950

Table 9.35 Detailed Major Beam Reinforcement Design for Floor 2 in the Interior Frame

5m Major Beam for Floor 2 for Interior frame			
Variables	SAP2000		
	End	Mid	End
SAP 2000 Tension Area, mm2	1320.8	1020.4	1400.6
Area, in2	2.047	1.582	2.171
SAP 2000 Compression Area, mm2	460.3	340.8	490.5
Area, in2	0.713	0.528	0.760
Tension Bars	4#7	4#6	4#7
Compression Bars	2#6	2#5	2#6
Shear Reinforcement	#3 at 150 mm	#3 at 200 mm	#3 at 150 mm
Reinforcement Ratio	0.0118	0.0085	0.0118

Table 9.36 Detailed Major Beam Reinforcement Design for Floor 1 in the Interior Frame

5m Major Beam for Floor 1 for Interior frame			
Variables	SAP2000		
	End	Mid	End

SAP 2000 Tension Area, mm2	1450.9	1090.5	1530.8
Area, in2	2.249	1.690	2.373
SAP 2000 Compression Area, mm2	510.6	380.3	540.7
Area, in2	0.791	0.589	0.838
Tension Bars	4#7	4#6	4#7
Compression Bars	2#6	2#5	2#6
Shear Reinforcement	#3 at 150 mm	#3 at 200 mm	#3 at 150 mm
Reinforcement Ratio	0.0118	0.0085	0.0118

Table 9.37 Detailed Minor Beam Reinforcement Design for Roof

5m Minor Beam for Roof			
Variables	SAP2000		
	End	Mid	End
SAP 2000 Tension Area, mm2	628.47	628.47	628.47
Area, in2	0.974	0.974	0.974
SAP 2000 Compression Area, mm2	628.47	628.47	628.47
Area, in2	0.974	0.974	0.974
Tension Bars	2#7	2#7	2#7
Compression Bars	2#7	2#7	2#7
Shear Reinforcement	#3 at 300 mm	#3 at 350 mm	#3 at 300 mm
Reinforcement Ratio	0.01032	0.01032	0.01032

Table 9.38 Detailed Minor Beam Reinforcement Design for Floor 4

5m Minor Beam for Floor 4			
Variables	SAP2000		
	End	Mid	End

SAP 2000 Tension Area, mm2	662.6	662.6	678.2
Area, in2	1.027	1.027	1.051
SAP 2000 Compression Area, mm2	615.7	615.7	615.7
Area, in2	0.954	0.954	0.954
Tension Bars	2#7	2#7	2#7
Compression Bars	2#7	2#7	2#7
Shear Reinforcement	#3 at 200 mm	#3 at 250 mm	#3 at 200 mm
Reinforcement Ratio	0.01032	0.01032	0.01032

Table 9.39 Detailed Minor Beam Reinforcement Design for Floor 3

5m Minor Beam for Floor 3			
Variables	SAP2000		
	End	Mid	End
SAP 2000 Tension Area, mm2	692.3	615.7	615.7
Area, in2	1.073	0.954	0.954
SAP 2000 Compression Area, mm2	615.7	615.7	615.7
Area, in2	0.954	0.954	0.954
Tension Bars	2#7	2#7	2#7
Compression Bars	2#7	2#7	2#7
Shear Reinforcement	#3 at 200 mm	#3 at 250 mm	#3 at 200 mm
Reinforcement Ratio	0.01032	0.01032	0.01032

Table 9.40 Detailed Minor Beam Reinforcement Design for Floor 2

5m Minor Beam for Floor 2			
Variables	SAP2000		
	End	Mid	End
SAP 2000 Tension Area, mm2	653.7	615.7	652.9

Area, in2	1.013	0.954	1.012
SAP 2000 Compression Area, mm2	615.7	615.7	615.7
Area, in2	0.954	0.954	0.954
Tension Bars	2#7	2#7	2#7
Compression Bars	2#7	2#7	2#7
Shear Reinforcement	#3 at 200 mm	#3 at 250 mm	#3 at 200 mm
Reinforcement Ratio	0.01032	0.01032	0.01032

Table 9.41 Detailed Minor Beam Reinforcement Design for Floor 1

5m Minor Beam for Floor 1			
Variables	SAP2000		
	End	Mid	End
SAP 2000 Tension Area, mm2	651.5	615.7	651
Area, in2	1.010	0.954	1.009
SAP 2000 Compression Area, mm2	615.7	615.7	615.7
Area, in2	0.954	0.954	0.954
Tension Bars	2#7	2#7	2#7
Compression Bars	2#7	2#7	2#7
Shear Reinforcement	#3 at 200 mm	#3 at 250 mm	#3 at 200 mm
Reinforcement Ratio	0.01032	0.01032	0.01032

Table 9.42. Final Structural Detailing for Prestressed Beam in the Floor 1-4

Location	End	Mid	End
Top selected bars	6#11	3#10	6#11
Bottom selected bars	5#8	6#11	5#11
Shear reinforcement	#4 @ 200 mm	#4 @ 250 mm	#4 @ 200 mm