ANALYSIS AND DESIGN OF DIAGRID STRUCTURAL SYSTEM FOR HIGH RISE STEEL BUILDINGS

BY

Khushbu D. Jani

10MCLC04

DEPARTMENT OF CIVIL ENGINEERING

AHMEDABAD-382481

May 2012

ANALYSIS AND DESIGN OF DIAGRID STRUCTURAL SYSTEM FOR HIGH RISE STEEL BUILDINGS

Major Project

Submitted in partial fulfillment of the requirements

For the degree of

Master of Technology in Civil Engineering

(Computer Aided Structural Analysis and Design)

By

Khushbu D. Jani

10MCLC04

NIRMA UNIVERSITY

INSTITUTE OF TECHNOLOGY

DEPARTMENT OF CIVIL ENGINEERING

AHMEDABAD-382481

May 2012

Declaration

This is to certify that

a. The major project comprises my original work towards the Degree of Master of Technology in Civil Engineering (Computer Aided Structural Analysis and Design) at Nirma University and has not been submitted elsewhere for a degree.

b. Due acknowledgement has been made in the text to all other material used.

Khushbu D. Jani

Certificate

This is to certify that the Major Project entitled "ANALYSIS AND DESIGN OF DIAGRID STRUCTURAL SYSTEM FOR HIGH RISE STEEL BUILDINGS" submitted by Ms. Khushbu D. Jani (10MCLC04), towards the partial fulfillment of the requirement for the degree of Master of Technology in civil Engineering (Computer Aided Structural Analysis and Design) of Nirma University, Ahmedabad, is the record of work carried out by her under my supervision and guidance. In my opinion, the submitted work has reached a level required for being accepted for examination. The results embodied in this major project, to the best of my knowledge, haven't been submitted to any other university or institution for award of any degree or diploma.

Dr. P. V. Patel
Guide and Professor,
Department of Civil Engineering,
Institute of Technology,
Nirma University, Ahmedabad.

Dr. P. H. Shah
Professor and Head,
Department of Civil Engineering,
Institute of Technology,
Nirma University, Ahmedabad.

Dr K Kotecha
Director,
Institute of Technology,
Nirma University,
Ahmedabad.

Examiner

Date of Examination

Abstract

Advances in construction technology, materials, structural systems and analytical methods for analysis and design facilitated the growth of high rise buildings. Structural design of high rise buildings is governed by lateral loads due to wind or earthquake. Lateral load resistance is provided by interior structural system or exterior structural system. Usually shear wall core, braced frame and their combination with frames are interior system where lateral load is resisted by centrally located elements. While framed tube, braced tube structural system resist lateral loads by elements provided on periphery of structure. It is very important that the selected structural system is such that the structural elements are utilized effectively while satisfying design requirements.

Recently diagrid structural system is adopted in tall buildings due to its structural efficiency and flexibility in architectural planning. Compared to closely spaced vertical columns in framed tube, diagrid structure consists of inclined columns on the exterior surface of building. Due to inclined columns lateral loads are resisted by axial action of the diagonal compared to bending of vertical columns in framed tubular structure. Diagrid structures generally do not require core because lateral shear can be carried by the diagonals on the periphery of building.

The aim of present study is to understand the behavior of diagrid structure under gravity and lateral loading. The methodology of preliminary design of diagrid system based on shear and bending stiffness is presented in this study. Design of 36 storey diagrid steel structural system having external columns at 74.5 slope is illustrated. Comparison of results of approximate analysis considering stiffness based approach and exact analysis using ETABS software is carried out for 36 storey diagrid building. Analysis results in terms of quantity of steel in diagrid members and top storey displacements are compared as obtained by approximate analysis and exact analysis. The influence of factor 'S' (Ratio of bending deformation to shear deformation) on

quantity of steel is also explored. It is found that approximate analysis is useful for preliminary design of diagonal column of diagrid structures. Further comparison of diagrid and tubular structural system is carried out to understand the advantages of diagrid structural system. Perimeter diagrid structural system saves approximately 14 percent structural steel compared to a conventional tubular structure for 36 storey building considered in this study.

Optimal angle of inclined columns in diagrid structural system is evaluated using stiffness based methodology. For this purpose 40, 50, 60, 70 and 80 storey tall buildings are considered with different angle of inclination of diagonal column on periphery like 50.2, 67.4, 74.5, 78.2, and 80.5 degrees with 2, 4, 6, 8, and 10 storey modules respectively. It is observed that the optimal angle of 40, 50, 60, 70 and 80 storey diagrid structure ranges between 65 degrees to 75 degrees.

Complete analysis and design of 36 storey diagrid steel building considering all load combinations is presented. A regular floor plan of 36 m x 36 m size is considered. ETABS software is used for modeling and analysis of structural members. All structural members are designed as per IS 800:2007 considering all load combinations. Dynamic along wind and across wind are considered for analysis and design of the structure. Load distribution in diagrid system is also studied for 36 storey building. Design and detailing of diagrid nodal connection is illustrated for 36 storey building. Similarly, analysis and design of 50, 60, 70 and 80 storey diagrid structures is carried out. Comparison of analysis results in terms of time period, top storey displacement and inter-storey drift is presented in this report. The design of diagonal column for all buildings is carried out and final dimensions of perimeter column and interior column are presented. Finally effects of sequential loading on analysis results are presented.

Acknowledgement

I would like to thank my guide **Dr. Paresh V. Patel**, whose keen interest and knowledge base helped me to carry out the major project work. His constant support and guidance during my project work equipped me with a great understanding of different aspects of the project work. He has shown keen interest in this work right from beginning and has been a great motivating factor in outlining the flow of my work.

My sincere thanks and gratitude to **Prof. N. C. Vyas**, Professor, Department of Civil Engineering, **Shri Himmat Solanki**, Visiting Faculty, Department of Civil Engineering, **Dr. U. V. Dave**, Professor, Department of Civil Engineering, **Dr. S. P. Purohit**, Senior Associate Professor, Department of Civil Engineering, Institute of Technology, Nirma University, Ahmedabad for their continual kind words of encouragement and motivation throughout the major project work.

I further extend my thanks to **Dr. P. H. Shah**, Head, Department of Civil Engineering and **Dr K Kotecha**, Director, Institute of Technology, Nirma University,Ahmedabad for providing all kind of required resources during my study.

Finally, I would like to thank all my friends and family members, for supporting and encouraging me in all possible ways throughout the major project work.

- Khushbu D. Jani

10MCLC04

Abbreviation, Notation and Nomenclature

$A_{d,w}$ .... Area of Each Diagonal on the Web

$A_{d,f}$ .... Area of Each Diagonal on the Flange

V .... Shear force

M .... Moment

L_d .... Length of Diagonal

E_d .... Modulus of Elasticity of Steel

Θ .... Angle of Diagonal Member

γ .... Transverse Shear Strain

χ .... Curvature

N_w .... Number of Diagonal on Each Web Plane

N_f .... Number of Diagonal on Each Flange Plane

δ .... Contribution of Web Diagonal for Bending Rigidity

B .... Building Width in direction of Applied Force

h .... Height of Module

H .... Total Height of Building

Δu .... Displacement

$\Delta\beta$ .... Rotation

K_T .... Transverse Shear Stiffness

K_B .... Bending Stiffness

S .... Ratio of Bending and Shear Displacement

Contents

Declaration **iii**

Certificate **iv**

Abstract **v**

Acknowledgement **vii**

Abbreviation, Notation and Nomenclature **viii**

List of Tables **xii**

List of Figures **xiv**

1 Introduction **1**
- 1.1 General 1
- 1.2 History of Diagrid System 5
- 1.3 Examples 6
- 1.4 Benefits 14
- 1.5 Limitations 15
- 1.6 Objective of Study 15
- 1.7 Scope of Work 16
- 1.8 Organization of Report 16

2 Literature Review **18**
- 2.1 General 18
- 2.2 Literature Review 18
  - 2.2.1 Diagrid System 18
  - 2.2.2 Analysis and Design Aspects and Optimal Configuration . . . 19
  - 2.2.3 Case Study of Diagrid System 23
  - 2.2.4 Across Wind Loading 24
- 2.3 Summary 24

3 Behavior of Diagrid Structural System **26**
- 3.1 General 26
- 3.2 Behavior under Gravity Loading 27

3.3 Behavior under Lateral Loading 28
3.4 Methodology of Preliminary Design of Diagrid System 30
3.4.1 Shear Stiffness and Bending Stiffness 30
3.4.2 Displacement due to Shear and Bending 34
3.5 Design Example of 36 Storey Building 37
3.5.1 Determining Optimal Value of ‘S’ 41
3.6 Comparison of Diagrid and Tubular Structural System 42
3.6.1 General 42
3.6.2 Building Configuration 42
3.6.3 Loading Data 44
3.6.4 Results and Discussion 45
3.7 Summary 49

4 Optimal Angle of Diagrid Structural System 50
4.1 General 50
4.2 Building Configuration 50
4.3 Design Examples of 40, 50, 60, 70 and 80 Storey Buildings 53
4.4 Results and Discussion 59
4.5 Summary 61

5 Analysis and Design of Diagrid Structural System 62
5.1 General 62
5.2 Building Configuration 62
5.3 Loading Data 63
5.4 Modelling and Analysis 67
5.4.1 Analysis Results of 36 Storey Building 68
5.4.2 Load Distribution in Diagrid System 70
5.5 Design of 36 Storey Building 72
5.5.1 Composite Slab 72
5.5.2 Beam 78
5.5.3 Column 83
5.6 Design and Detailing of Connection 88
5.6.1 General 88
5.6.2 Design Calculations 91
5.7 Analysis and Design of 50, 60, 70, 80 storey Buildings 95
5.8 Effect of Sequential Loading on Analysis Result 99
5.9 Summary 104

6 Summary and Conclusion 105
6.1 Summary 105
6.2 Conclusion 106
6.3 Future Scope of Work 109

References 110

A Wind Loading Calculations 113

B Preliminary Design of Diagrid Structures **121**

C List of Paper Published / Communicated **137**

List of Tables

3.1 Preliminary Member Sizing for the 36-Storey Diagrid Structure . . . 39
3.2 Comparison between Approximate and Exact Analysis 40
3.3 Size of Typical Members of Structural System 40
3.4 Base Shear due to Static Wind Loading 44
3.5 Base Shear due to Static Earthquake Loading 45
3.6 Load Combinations for Static Loads 45
3.7 Size of Typical Members for Structural System 46
3.8 Structural Weight of System 48

4.1 Preliminary Member Sizing and Diagonal Steel Mass for the 70-storey Diagrid Structure with ‘S’=4 (67.4 degrees) 55
4.2 Diagrid Diagonal Steel Mass (Ton) 60
4.3 Size of Typical Members of Structural System 61

5.1 Base Shear due to Wind Loading 64
5.2 Base Shear due to Static Earthquake Loading 66
5.3 Load Combinations 66
5.4 Load Distribution in 36 Storey Diagrid System 71
5.5 Design forces for Beams 78
5.6 Design forces for Columns 83
5.7 First Mode Time Period of 36, 50, 60, 70 and 80 Storey Diagrid Structure 95
5.8 Top Displacement of 36, 50, 60, 70 and 80 Storey Diagrid Structure . 95
5.9 Inter-Storey Drift of 36, 50, 60, 70 and 80 Storey Diagrid Structure . 97
5.10 Size of Typical Members of 36, 50, 60, 70 and 80 Storey Diagrid Structure 98
5.11 Sequential and One Step Analysis Results for Diagonal Columns (36 Storey) 102
5.12 Sequential and One Step Analysis Results for Diagonal Columns (60 Storey) 103
5.13 Sequential and One Step Analysis Results for Diagonal Columns (80 Storey) 104

A.1 Along Wing Load for 36 Storey 115
A.2 Across Wing Load for 36 Storey 119

B.1 Preliminary Member Sizing for the 40-Storey Diagrid Structure (S=1,2,3) 122
B.2 Preliminary Member Sizing for the 50-Storey Diagrid Structure (S=1,2,4) 125

B.3 Preliminary Member Sizing for the 60-Storey Diagrid Structure (S=1,3,5) 128
B.4 Preliminary Member Sizing for the 70-Storey Diagrid Structure (S=1,4,8) 131
B.5 Preliminary Member Sizing for the 80-Storey Diagrid Structure (S=1,5,9) 134

List of Figures

1.1 Interior Structural Systems . . . 2
1.2 Exterior Structural Systems . . . 3
1.3 Diagrid Structural System . . . 4
1.4 IBM Building . . . 5
1.5 John Hancock Building . . . 6
1.6 Swiss Re Building . . . 6
1.7 Plan of Swiss Re Building . . . 7
1.8 Hearst Tower . . . 8
1.9 Plan of Hearst Tower . . . 8
1.10 Central China Television (CCTV) . . . 9
1.11 Uniform Bracing System . . . 9
1.12 Cyclone Tower . . . 10
1.13 Lotte Super Tower . . . 11
1.14 Jinling Tower . . . 12
1.15 Capital Gate Tower . . . 13
1.16 Connection Detail of Capital Gate Tower . . . 13
1.17 Bow Tower . . . 14

3.1 Effect of Gravity Load on Diagrid Module . . . 27
3.2 Effect of Shear Force on Diagrid Module . . . 28
3.3 Effect of Overturning Moment on Diagrid Module . . . 29
3.4 6-Storey Diagrid Module . . . 30
3.5 Deformed Shape of Diagrid Module due to Lateral Load . . . 31
3.6 Uniform Bending and Shear Deformation of Tall Buildings . . . 35
3.7 Typical Floor Plan and Elevation of 36 Storey Building . . . 37
3.8 Preliminary Member Sizing for the 36-Storey Diagrid Structure . . . 39
3.9 Hollow Pipe and Interior Column Section . . . 40
3.10 Preliminary Member Sizing for the 36-Storey Diagrid Structures with Various ‘S’ . . . 41
3.11 Diagrid Steel Mass for 36-Storey . . . 42
3.12 Typical Floor Plan 36 Storey Buildings . . . 43
3.13 Typical Elevation of 36 Storey Buildings . . . 43
3.14 Interior Column Section . . . 46
3.15 Time Period of Structural System . . . 47
3.16 Displacement of Structural System . . . 47

3.17 Inter-Storey Drift of Structural System 48

4.1 Typical Floor Plan of Diagrid Structure 51
4.2 Elevation of 40 Storey Model with (a) 50.2, (b) 67.4, (c) 74.5 , (d) 78.2; (e) 80.5 degrees Inclination 51
4.3 Elevation of 50 Storey Model with (a) 50.2, (b) 67.4, (c) 74.5 , (d) 78.2; (e) 80.5 degrees Inclination 51
4.4 Elevation of 60 Storey Model with (a) 50.2, (b) 67.4, (c) 74.5 , (d) 78.2; (e) 80.5 degrees Inclination 52
4.5 Elevation of 70 Storey Model with (a) 50.2, (b) 67.4, (c) 74.5 , (d) 78.2; (e) 80.5 degrees Inclination 52
4.6 Elevation of 80 Storey Model with (a) 50.2, (b) 67.4, (c) 74.5 , (d) 78.2; (e) 80.5 degrees Inclination 53
4.7 Preliminary Member Sizing for the 70-storey Diagrid Structure with various 'S' (67.4 degrees) 56
4.8 Diagrid Steel Mass for 40-Storey Structure (67.4 degrees) 57
4.9 Diagrid Steel Mass for 50-Storey Structure (67.4 degrees) 57
4.10 Diagrid Steel Mass for 60-Storey Structure (67.4 degrees) 57
4.11 Diagrid Steel Mass for 70-Storey Structure (67.4 degrees) 58
4.12 Diagrid Steel Mass for 80-Storey Structure (67.4 degrees) 58
4.13 Diagrid Diagonal Steel Mass of 40, 50, 60, 70 and 80 Storey Model . . 59

5.1 Typical Floor Plan and Elevation of 36 Storey Buildings 63
5.2 Static Along Wind Loading: Lateral Force and Storey Shear 64
5.3 Dynamic Along Wind Loading: Lateral Force and Storey Shear . . . 65
5.4 Across Wind Loading: Lateral Force and Storey Shear 65
5.5 36 Storey Diagrid Building Model (74.5^0) 68
5.6 Time Period of 36 Storey Structural System 69
5.7 Storey Shear of 36 Storey Structural System 69
5.8 Displacement of 36 Storey Structural System 70
5.9 Inter-Storey Drift of 36 Storey Structural System 70
5.10 Load Distribution in Exterior and Interior Frame 71
5.11 Composite Slab Cross-Section 74
5.12 Cross-section of Metal Deck Sheeting 75
5.13 Profiled Sheeting during Construction 76
5.14 Plan of Beam Location 81
5.15 Plan of Column Location 85
5.16 Load Path at Node 88
5.17 Location of Connection 89
5.18 Load Considered for Connections (a) WL (b) 1.5(DL+LL) 89
5.19 Load combination for Connections (c) 1.2(DL+LL+WL) (d) 1.5(DL+WL) 90
5.20 Load combination for Connections (e) 0.9DL+1.5WL (f) 1.2(DL+LL)+0.6WL 90
5.21 Connection between Diagrid, Gusset Plate and Stiffener 91
5.22 Detail of Gusset Plate and Stiffener 92
5.23 Web Plate With Web Stiffener 93

5.24 Nodal Connection Detail 94
5.25 Displacement of 50 and 60 Storey Structural System 96
5.26 Displacement of 70 and 80 Storey Structural System 96
5.27 Inter-Storey Drift of 50 and 60 Storey Structural System 97
5.28 Inter-Storey Drift of 70 and 80 Storey Structural System 97
5.29 Interior Column Section for 36, 50 and 60 Storey 98
5.30 Interior Column Section for 70 and 80 Storey 98
5.31 Stage-Wise Construction 99
5.32 Sequential Loading 100
5.33 Sequential Loading 101
5.34 Run Construction Sequence Analysis 101
5.35 Elevation for Diagonal Columns No. 102

A.1 Typical Floor Plan and Elevation of Diagrid Structure 113
A.2 Lateral Load Along the Height: Along Wind 116
A.3 Lateral Load Along the Height: Across wind 120

B.1 Preliminary Member Sizing for the 40-Storey Diagrid Structure (S=1,2,3) 123
B.2 Diagrid Steel Mass for the 40-Storey Structure 124
B.3 Preliminary Member Sizing for the 50-Storey Diagrid Structure (S=1,2,4) 126
B.4 Diagrid Steel Mass for the 50-Storey Structure 127
B.5 Preliminary Member Sizing for the 60-Storey Diagrid Structure (S=1,3,5) 129
B.6 Diagrid Steel Mass for the 60-Storey Structure 130
B.7 Preliminary Member Sizing for the 70-Storey Diagrid Structure (S=1,4,8) 132
B.8 Diagrid Steel Mass for the 70-Storey Structure 133
B.9 Preliminary Member Sizing for the 80-Storey Diagrid Structure (S=1,5,9) 135
B.10 Diagrid Steel Mass for the 80-Storey Structure 136

Chapter 1

Introduction

1.1 General

The advances in construction technology, evolution of efficient structural system and scarcity of urban land caused the development of the high rise buildings around the world. Lateral loading due to wind or earthquake are governing in design of high rise buildings along with gravitational loading. The widely used lateral load resisting systems in high rise buildings are: rigid frame, shear wall, wall-frame, braced tube system, outrigger system and tubular system. Recently, the diagrid -Diagonal Grid- structural system is widely used for tall steel buildings due to its structural efficiency and aesthetic potential provided by the unique geometric configuration of the system [13].

A classification for the structural system of a high-rise was introduced in 1969 by Fazlur-Khan and was extended to incorporate interior and exterior structures. The primary lateral load-resisting systems are defined based on whether a structural system is an interior or exterior one. Fig.1.1 and Fig.1.2 show the interior and exterior structural systems respectively for varying number of storeys [12].
The following are the interior structural systems:

- Rigid frame

- Braced frame
- Shear-wall structures
- Wall-frame structures
- Outrigger structures

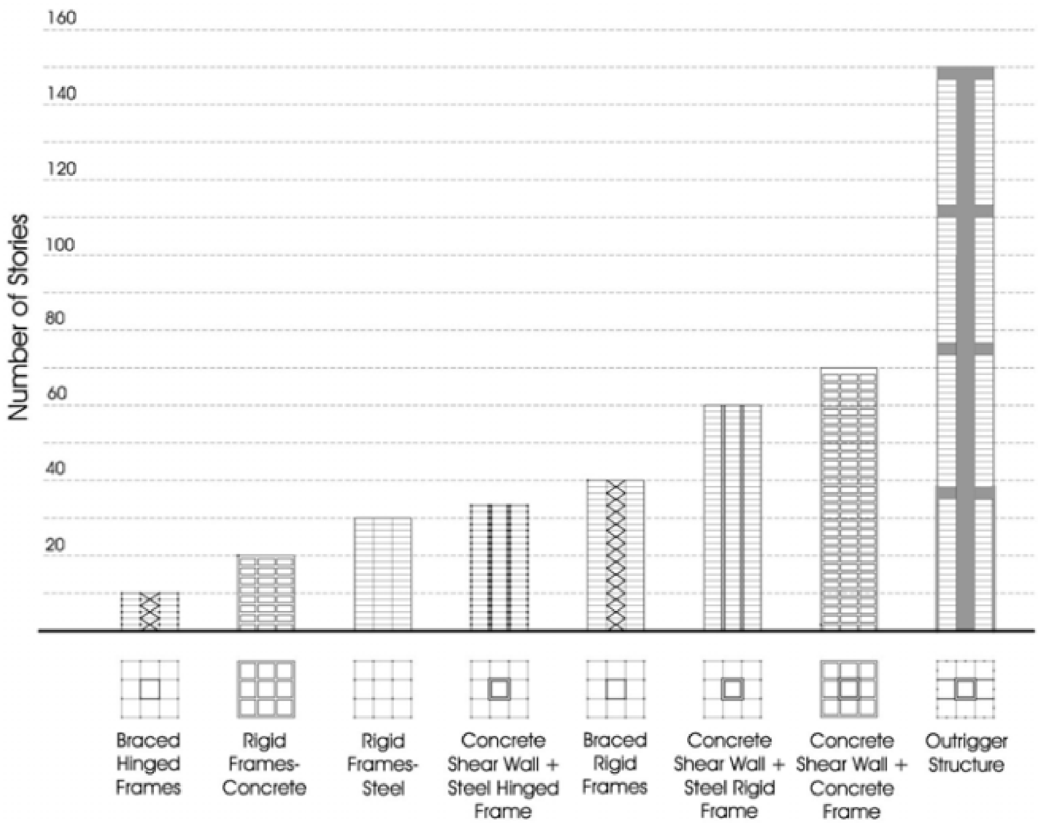


Figure 1.1: Interior Structural Systems

The following are the exterior structural systems:

- Tube
- Diagrid
- Exo-skeleton
- Space Truss Structure
- Super Frame

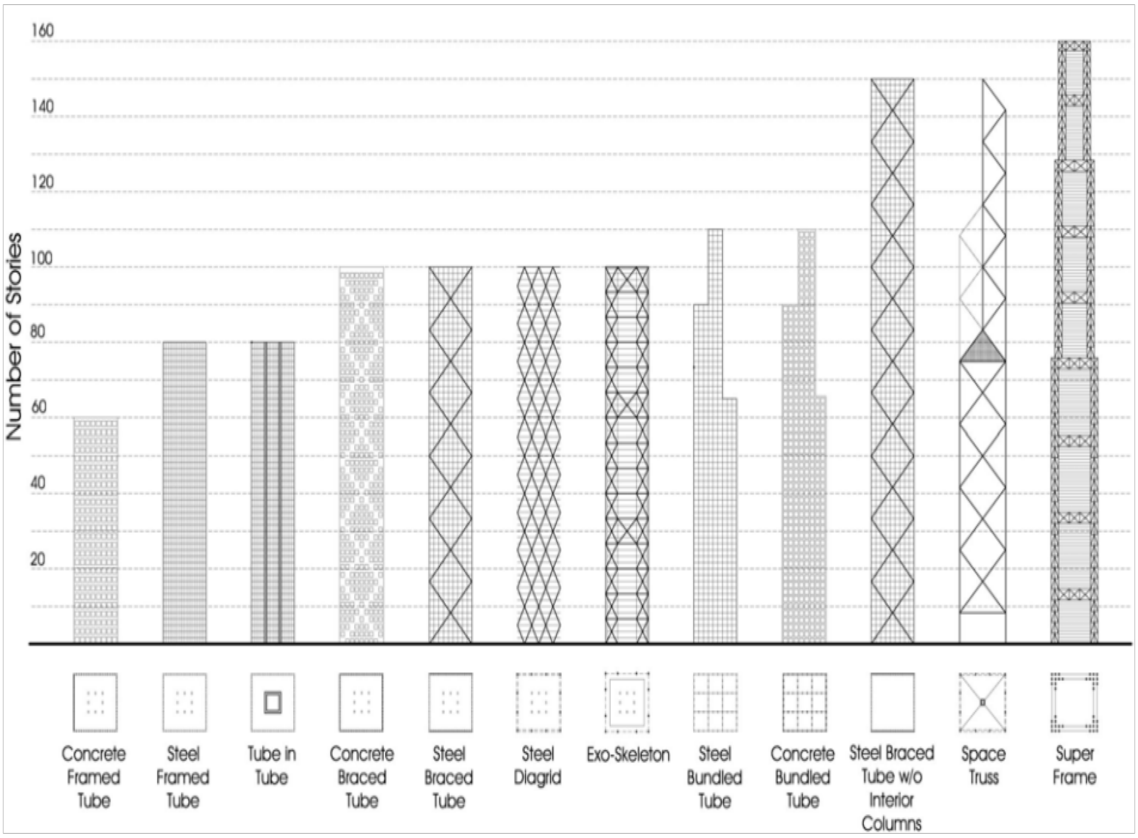


Figure 1.2: Exterior Structural Systems

Diagrid is a particular form of space truss as shown in Fig.1.3. It consists of multiple diagonal elements that form a diagonal grid on the periphery of the structure and perimeter grid form the series of triangulated truss system. In diagrid structures, almost all conventional columns are eliminated. The diagonal grid makes the structure stable even without having any vertical columns [13].

Diagrid is formed by intersecting the diagonal and horizontal components. The examples of diagrid structure all around the world are the Swiss Re in London, Hearst Tower in New York, Lotte Super Tower in Seoul, Cyclone Tower in Asan (Korea), Capital Gate Tower in Abu Dhabi, Bow Tower in Canada and Jinling Tower in China. The new headquarter for Central China Television (CCTV) in Beijing is also one of the examples of utilization of diagrid structural system to support the challenging shape [13].

Figure 1.3: Diagrid Structural System

Diagrid has good appearance and it is easily recognized. The configuration and efficiency of a diagrid system reduce the number of structural elements required on the facade of the buildings, therefore less obstruction to the outside view. The structural efficiency of diagrid system also helps in avoiding interior and corner columns, therefore allowing significant flexibility with the floor plan. Perimeter diagrid system saves approximately 20 percent of the structural steel weight when compared to a conventional moment-frame structure [13].

The diagonal members in diagrid structural systems can carry gravity loads as well as lateral forces due to their triangulated configuration. Diagrid structures are more effective in minimizing shear deformation because they carry lateral shear by axial action of diagonal members. Diagrid structures generally do not need high shear rigidity cores because shear can be carried by the diagonal members located on the perimeter [3]. In diagrid structures, diagonals carry both storey shear and moment. Thus, the optimal angle of diagonals is highly dependent upon the building height [6].

1.2 History of Diagrid System

The 13 storey IBM Building in Pittsburgh that was completed in 1963 is an early example of successful implementation of diagrid system as shown in Fig.1.4. Diagonal bracing members are very efficient structural element in resisting lateral loads. Thus, to resist the lateral load effectively, most of the high rise buildings consist of steel frame with diagonal bracing of different configuration like X, K and eccentric etc. Generally, diagonal bracings are embedded in the building cores which are usually located inside the building [14].

Figure 1.4: IBM Building

A major departure from this design approach occurred when braced tubular structures were introduced in the late 1960s. For the 100 storey tall John Hancock Buildings in Chicago as shown in Fig.1.5, the diagonals were located along the entire exterior perimeter surfaces of the building in order to maximize their structural effectiveness and to capitalize on the aesthetic innovation. Diagrid structural system is used in recent tall buildings due to its structural efficiency and flexibility in architectural planning. Compared to closely spaced vertical columns in framed tube, diagrid structure consists of inclined columns on the exterior surface of building [14].

Figure 1.5: John Hancock Building

1.3 Examples

Followings are the examples of diagrid structural system in high rise buildings around the world.

Swiss Re Building

Swiss Re Building is located in London as shown in Fig.1.6 and also known as 30 St. Mary Axe. It is the first modern application and the most representative example of diagrid structural system. It is designed by Sir Norman Foster, with 40 stories and interstorey height of 4.15 m. The total height of tower is 180 m [16].

Figure 1.6: Swiss Re Building

The plan of Swiss Re Building is shown in Fig.1.7. The building is circular in plan with diameter changing along elevation. The diagrid is generated by a series of steel triangles, two-storey high and 9 m wide, with an intermediate tie connecting the two diagonals. The diagonals are circular hollow section (CHS) members, with cross section between 508 x 40 mm at the lowest floors and 273 x 12.5 mm at the top, while the chord members have rectangular hollow section (RHS), 250 x 300 mm with wall thickness of 25 mm. The circular central core, which has constant diameter along elevation, does not contribute to the lateral resistance and rigidity, being a simple frame structure [16].

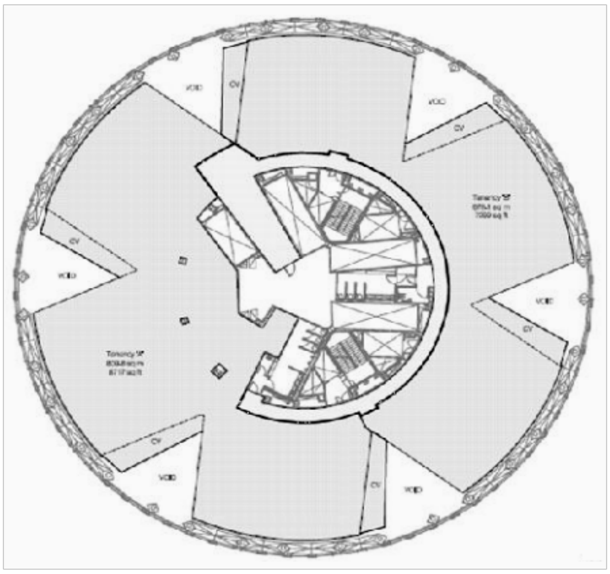


Figure 1.7: Plan of Swiss Re Building

Hearst Tower

Hearst Tower is located in New York as shown in Fig.1.8. It is designed by Sir Norman Foster, with 46 stories and 183 m tall, has a prismatic and rectangular floor plan, 48 x 37 m as shown in Fig.1.9 and is built on an existent 6-storey building. The diagrid structure creating the diamond effect in the facade. The diagrid module is 12.25 m wide and 16.54 m high, and covers 4 stories. The diagonal cross section is I shape, with the maximum size W14 x 370 at the base of the diagrid. The mega column between the tenth and the ground level are concrete filled box section of 1100 x 1100 x 10 mm [16].

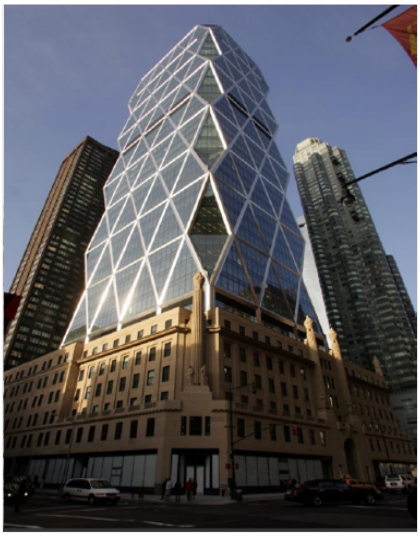


Figure 1.8: Hearst Tower

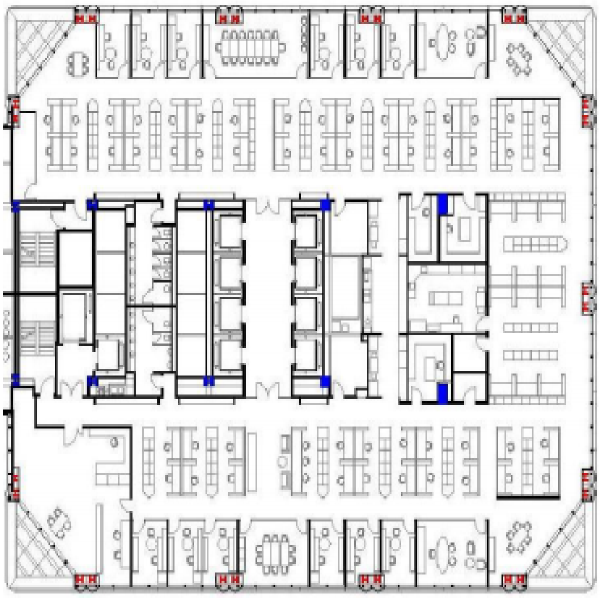


Figure 1.9: Plan of Hearst Tower

Central China Television (CCTV)

Central China Television Tower is located in Beijing, China as shown in Fig.1.10 and is a good example of utilization of diagrid structural system to support building with challenging shape. CCTV is designed by the Rem Loolhass of OMA (Office for Metropolitan Architecture). The 234m tall tower redefines the form of skyscraper, with the primary system comprised of a continuous structural tube of columns, beams and braces around the entire skin of the building. Uniform bracing pattern of CCTV Tower is shown in Fig.1.11 [19].

Figure 1.10: Central China Television (CCTV)

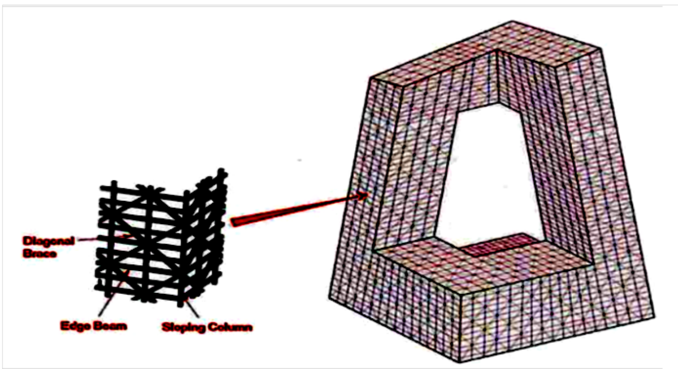


Figure 1.11: Uniform Bracing System

Cyclone Tower

Cyclone Tower is located in Asan, Korea as shown in Fig.1.12 and it is a mixed development that consists of the following buildings:

- Two similar residential towers of 45 stories with six levels of podium car park and two levels of basement

- A residential tower of 66 stories with six levels of retail podium and five levels of basement
- An office tower of 51 stories with seven levels of commercial podium and seven levels of basement
- A department store of 11 stories

The total height of building is 243.7 m and office tower story height is 3.5 m. In this building, the main structure is composed of an RC core and the steel diagrid column. These two structural elements act together and make the building stiff. The diagrid is composed of sloping columns modularized in every four stories. It acts as a brace tube and maximizes the moment resisting capacity of the building. 800 mm steel pipe section is used for diagrid column members and their thickness is 16, 25 and 40 mm. 1300 mm steel pipe section is used for corner column and for perimeter girders H-500 x 200 x 10 x 16 is considered [8].

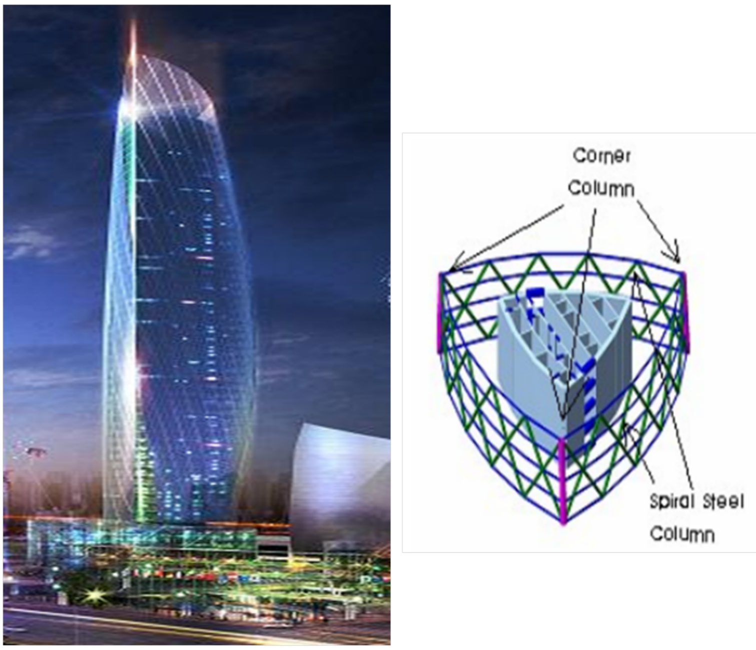


Figure 1.12: Cyclone Tower

Lotte Super Tower

Lotte Super Tower is located in Seoul, Korea as shown in Fig.1.13 It is designed by

Skidmore, Owings and Merril, with 123 stories and 555 m height. In this building, a dual system comprised of an exterior steel diagrid and an interior reinforced concrete core is utilized for lateral resistance to wind and seismic effect. This building is under construction and expected to complete at the end of 2015 [20].

Figure 1.13: Lotte Super Tower

Jinling Tower

Jinling Tower is located in Nanjing, China as shown in Fig.1.14. It is a structural system to create an iconic form on the Nanjing skyline. The 320 m tall, 80 storey building consists four vertical quadrants, each floor gradually rotating relative to the floor below for a total of 90 degrees over the building height. The bottom floor plates are square to provide the efficient plans. Cruciform shaped floor plans are used in the middle two quadrants housing luxury apartments. At the top of the building, the floor plates become square once again to house a six-star hotel. The tower is enclosed in a diagonal mesh tubular frame and is combined with the central core. The system provides efficient resistance to seismic and wind loads [21].

Figure 1.14: Jinling Tower

Capital Gate Tower

Capital Gate Tower is located in Abu Dhabi as shown in Fig.1.15. It is 35-storey iconic tower of 160 meter, part of the capital gate development around the Abu Dhabi national exhibition center. It features a presidential-style luxury hotel and office spaces. Leaning towards one direction to an angle of 18 degrees, the tower has been nominated for the consideration of guinness book of world record as the world's most inclined buildings. External diagonal are of rectangular hollow tubes whereas the internal diagonal members are of circular hollow pipes. Connection details is shown in Fig.1.16.

More than 13,000 tons of material is consumed for main structural steel work in addition to the metal decking. Capital Gate is constructed on the top of the 2 meter deep concrete base filled with an incredibly mesh of reinforced steel. This base sits above an intensive distribution of 490 piles which have been drilled 30 meters underground to accommodate the gravitational and lateral pressure caused by the lean of the building [22].

Figure 1.15: Capital Gate Tower

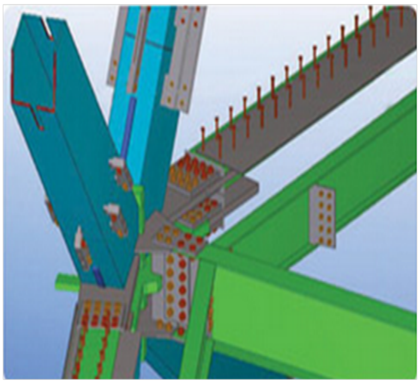


Figure 1.16: Connection Detail of Capital Gate Tower

Bow Tower

The Bow Tower is located in Canada as shown in Fig.1.17. The new 59 storey Bow project for Encana Corporation is the tallest building in Calgary and Western Canada at nearly 247 meter high. The building is one of the largest commercial office developments in Canada with 180,000 square meters of total floor space. The diagrid system takes advantage of the structural efficiencies of its natural curved form to reduce overall required steel quantity when compared to a building with a conventional braced core or rigid frame perimeter tube structures. The perimeter diagonals of the diagrid form equilateral triangles which are the primary elements for the gravity as well as wind and seismic load resisting systems. The diagrid node placement occurs every six floors up the face of the building. This perimeter diagrid system with limited interior bracing towers saves approximately 20 percent of the structural steel weight compared to a conventional moment-frame structures [7].

Figure 1.17: Bow Tower

1.4 Benefits

Followings are the advantages of diagrid structural system:

- Diagrid system offers mostly column free exterior and interior space.
- Generous amounts of day lighting due to less number of interior columns and structure.
- Perimeter diagrid system saves approximately 20 percent of a structural steel when compared to a conventional moment-frame structure.
- Simple connection techniques although they need to be perfected.

- Diagrid system has higher torsional rigidity than the others.
- Full exploitation of the structural material.
- Free and clear, unique floor plans are possible.
- Aesthetically dominated and expressive.

1.5 Limitations

Followings are some of the limitations of diagrid structural system:

- Less construction experience in creating a diagrid skyscraper.
- The diagrid can dominate aesthetically, which can be an issue depending upon design intent.
- It is difficult to design window that creates a regular language from floor to floor.
- The diagrid system becomes difficult if not executed properly.
- Complicated joints.

1.6 Objective of Study

Following are the main objectives of study:

- To understand the behavior of diagrid structural system for high rise buildings.
- To evaluate the efficiency of diagrid structural system compared to other structural system, like frame tube.
- To understand the advantages of diagrid structural system for high rise buildings.
- To derive optimal configuration of diagrid system.
- To understand design methodology of diagrid members and connections.

1.7 Scope of Work

The scope of present work is as follows:

- The study of high-rise steel buildings, around the world where diagrid structural system used.
- Understanding the behavior of diagrid system under the gravity loading.
- Understanding the behavior of diagrid system under the lateral loading.
- Study of the methodology for preliminary design of diagrid system.
- Comparison of behavior of diagrid structural system with the tubular structural system.
- Analysis and design of 36 storey diagrid structural system using ETABS software as per IS:800-2007.
- Evaluation of the optimal angle of diagrid structural system for 40, 50, 60, 70 and 80 storey structures.
- Consideration of effect of construction sequence in design of diagrid structural system.
- Design of structural members and connections as per IS 800:2007.

1.8 Organization of Report

The contents of the major project report are divided in to the various chapters as follows:

An introduction of diagrid structural system is discussed in **Chapter 1**. An overview on diagrid structural system for tall buildings is discussed. The case studies, benefits and limitations of diagrid structural system are also discussed in this chapter. It also includes the objective of study and scope of work.

Literature review is presented in **Chapter 2**. It includes review of literature related to diagrid structural system.

The behavior of diagrid structural system is discussed in **Chapter 3**. Behavior of diagrid structural system under gravity loading and under lateral loading is presented in this chapter. The methodology of preliminary design of diagrid system is also discussed. Design example of 36 storey diagrid structure is illustrated in this chapter. Comparison of diagrid and tubular structural system is carried out to understand the advantages of diagrid structural system.

Optimal angle of diagrid structural system is evaluated in **Chapter 4**. In this chapter, optimal angle of inclined columns in diagrid structural system is evaluated. For this purpose 40, 50, 60, 70 and 80 storey tall buildings are considered with different angle of inclination of diagonal column on periphery like 50.2, 67.4, 74.5, 78.2, and 80.5 degrees with 2, 4, 6, 8, and 10 storey modules respectively. The angle of inclination is kept uniform throughout the height. The stiffness based design methodology is presented for 40 to 80 storey diagrid structures in this chapter.

Analysis and design of diagrid structural system is carried out in **Chapter 5**. In this chapter, the modeling, analysis and design of diagrid system are carried out using ETABS software. The structural elements are designed as per IS: 800-2007. The effect of sequential loading in analysis is also discussed.

Summary of major project, conclusion and future scope of work are presented in **Chapter 6**.

Chapter 2

Literature Review

2.1 General

Literature review related to diagrid structural system is presented in this chapter. Various research papers have been refereed to understand the basic behavior, advantages, limitations, modeling, analysis and optimal angle of diagrid structural system. Literature also presents importance and effectiveness of diagrid structural system compare to other system.

2.2 Literature Review

2.2.1 Diagrid System

Toreno et al.[16] discussed the structural performance of diagrid structure in tall buildings. They presented the characteristics of diagrid systems, focusing attention on the structural behavior under gravity and lateral load and reviewing strength-based and stiffness-based design criteria. They also carried out the comparative analysis of the structural performance of some recent diagrid tall buildings, namely Swiss Re building in London, Hearst Headquarters in New York and West Tower in Guangzhou.

Ali and Moon[12] discussed the various structural system for tall buildings and the technological driving forces behind tall building developments. They presented the classification of interior structural system and exterior structural system for tall buildings. The most representative structural systems for tall buildings like outrigger system and diagrid system were discussed.

2.2.2 Analysis and Design Aspects and Optimal Configuration

Kim and Lee[3] investigated the seismic performance of buildings with typical diagrid structural system. They analyzed and designed the 36 storey diagrid structural system with various inclination of diagrid columns (50.20^0, 61.00^0, 67.40^0, 71.60^0, 74.50^0 and 79.50^0). The building was 36 m x 36 m in plan and their seismic response were evaluated using nonlinear static and dynamic analysis. The structural response analyses were carried out using the SAP 2000 software. According to the analysis results the diagrid structures showed higher overstrength with smaller ductility compared with tubular structure. The brace angle between 60 to 70 degrees seemed to be most efficient in resisting lateral as well as gravity loads. It was also observed that as the slopes of braces increased, the shear lag effect increased and lateral strength decreased.

Moon[14] presented the diagrid structural systems as structurally efficient as well as architecturally pleasing structural system for high-rise buildings. He discussed the behavior and a preliminary design methodology for diagrid structural system. A preliminary design methodology is used to determine the preliminary member sizes of diagrid structural system. He also studied the optimal range of angles for high-rise buildings ranging from 20 to 60 stories. He also discussed the architectural and constructability of diagrid structural system. The structural analysis of buildings were carried out using SAP2000 software. The diagrid angles used were 34, 53, 63, 69, 76, 82 and 90 degrees. According to analysis results, for 60 storey diagrid structures having aspect ratio of about 7, the optimal range of diagrid angles was from about

65 degrees to 75 degrees and for 42 storey buildings having the aspect ratio 5, the range was lower by 10 degrees because the importance of bending to the total lateral displacement was reduced as the building height decreased.

Leonard[13] presented the shear lag effect in high rise buildings with diagrid structural system. A 60 storey building model with a typical floor height 3.6 m and 50 psf uniformly distributed wind load was considered in the work. Also, corner columns were eliminated from the model as it was the general preference of architect and developers. The structural analysis of 60 storey building were carried out using SAP2000 software. The diagrid angles used were 31, 45, 63.4, 71.6 and 80.5 degrees. He concluded that in 60 storey building, the optimal angle for diagrid system range between 63.4 degrees to 71.6 degrees and the diagrid system perform 3 times better than framed tube buildings in shear lag ratio and lateral deflection. He also concluded that shear lag effect does not influence the lateral deflection of high-rise buildings and diagrid structural system. He demonstrated high efficiency in carrying lateral load in high-rise buildings.

Moon[6] discussed the stiffness-based design methodologies for high-rise buildings with diagonals such as braced tubes and most recently developed diagrid structures. The stiffness-based methodologies were used for determining member sizes for structural systems. He presented the guidelines to determine the shear and bending deformation for optimal design, which used the least amount of structural material to meet the stiffness requirements. The difference between conventional braced tube structures and diagrid structures was that, for diagrid structures, almost all conventional vertical columns were eliminated. For braced tube and diagrid structures 40, 50, 60, 70, 80, 90 and 100 storey building with the aspect ratio of which ranges from 4.3 to 10.8 were considered. The building plan dimension was 36 m x 36 m and its typical storey height was 3.9 m. The structural analysis were carried out using the SAP 2000 software. It was found that, for braced tube structures, the typically adopted angles in practice ranging from about 40 to 50 degrees were closer to the optimal angle. In diagrid structures, as a building becomes taller, the optimal angle also becomes steeper within the typical range of about 60 to 70 degrees. According to analysis results the diagrid structures used less material than conventional braced tube structures even up to 100 storey structures.

Moon[4] discussed the stiffness-based design methodologies for braced-tubes high-rise buildings. Braced-tubes are efficient structural systems for tall buildings and have been continuously used for tall buildings since their emergence in the late 1960s. This methodology was applied to a set of braced tube building ranging from 40 to 80 stories tall, with the height-to-width aspect ratio ranging from 4.3 to 8.6, and parameters for most economical design in terms of material usage were generated for representative design loadings. The building plan dimension was 36 m x 36 m with an 18 m x 18 m gravity core at the center and its typical storey height was 3.9 m. The diagonals run 10 stories and created an angle of 47.3 degrees. The structural having these "Preliminary" design member were analyzed using the SAP 2000 software. Comparison was done between targeted maximum displacements and the displacements computed using SAP 2000. He concluded that, the stiffness based methodology can be used for preliminary member design. He also concluded that, the optimal angle for braced tube structure range between 40 degrees to 50 degrees.

Moon[1] discussed the structural performance of diagrid system for tall buildings with various complex geometries such as twisted, tilted and free-form tower. For each complex shaped, 60 stories tall buildings were designed with diagrid system and structural efficiency was also studied. The building plan dimension was 36 m x 36 m with an 18 m x 18 m gravity core at the center and its typical storey height was 3.9 m. The structures having these "Preliminary" design members were analyzed using the SAP 2000 software. Comparison was made between the targeted maximum displacements and the displacements computed using SAP2000.

He concluded that, as the rate of twisting increased, the buildings lateral stiffness decreased, and, consequently, its deflection increased. He also concluded that, as the rate of twisting increased, the diagrid angle deviated more from its original optimal condition, which resulted in substantial reduction of the lateral stiffness of the tower.

Moon[15] presented various sustainable design strategies for high-rise buildings. Material-saving design methodologies for tall buildings structural system were investigated. Analysis and design of 60 storey diagrid system with aspect ratio of about 6 and having various diagonal angles ranging from 53 to 76 degrees were carried out. Structural steel masses required for 60 storey building was evaluated. The diagrid structure configured with a diagonal angle 69 degrees met the design requirement with the minimum amount of steel. Diagrid structures of various heights ranging from 40 to 100 stories were studied in the same way. It was found that an angle of 63 degrees was near optimal angle for the 40 and 50 storey diagrid structures. Study results suggested that for 60 storey and taller diagrid structures the angle of 69 degrees was the near optimal angle.

Kim et al.[2] investigated the cyclic behavior of the diagrid nodes under lateral loads such as wind load or seismic load. They suggested the details and welding methods of nodes for application to an actual building based on the test results. The diagrid nodes were examined experimentally and installed in the Lotte Super Tower located in Seoul, South Korea. The Lotte Super Tower is a unique form that is transformed from a 70 meter square footprint to a 39 meter diameter circle at 555 meters above the ground. In this building, a dual system comprised of an exterior steel diagrid and an interior reinforced concrete core was utilized for lateral resistance to wind and seismic effect. Two types of specimens were developed to investigate the cyclic performance of the diagrid nodes and testing of diagrid nodes were carried out. One was open type section for the node and another was box type section for the node. Results are developed in terms of initial stiffness, strength, ductility and energy absorption. Based on the analysis results box type specimens developed superior structural performance over the open type specimens under the cyclic loading.

Kim et al.[5] investigated the cyclic behavior of the diagrid nodes with H-Section Braces under lateral loads such as wind load or seismic load. They discussed the cyclic performance for diagrid nodes with an emphasis on the hysteresis characteristics, welding methods, and failure modes. They selected the key parameters to investigate the effect on the structural performance of the specimens as the overlapped length of the side stiffener and welding methods. Five diagrid nodes were designed and constructed for testing. Results in terms of initial stiffness, strength, ductility and energy absorption were presented.

2.2.3 Case Study of Diagrid System

Charnish and McDonnell[7] presented the new 59 storey Bow project for EnCana Corporation, the tallest building in Calgary and Western Canada at nearly 247 meters high. The bow shaped EnCana Tower was composed of a unique diagrid structural system. The diagrid system takes advantage of the structural efficiencies of its natural curved form to reduce overall required steel quantity when compared to a building with a conventional braced core or rigid frame perimeter tube structures. This paper described the structural design consideration of lateral system, some construction methods, and the unique sustainable features incorporated into the building.

Soo et al.[8] presented the Penta-Port project located in Asan, is a tall steel building. The total height of the building is 243.7 m. The building has variations of the floor plan in each floor, making the exterior of the building curvilinear. Comparisons and analysis were performed to select the most efficient structural system and to maximize the uniqueness of the external appearances. They proposed three structural system to select most efficient system, outrigger with belt truss system, diagrid system and super column with super brace system. Based on analysis results, the diagrid system was recommended for architectural planning, structural efficiency and stability. In this building, the main structure is composed of RC core and exterior steel diagrid column. The diagrid system becomes a landmark because of its unique external appearance and structural system.

Rahimian and Eilon[9] presented the structural system of 46-storey Hearst Tower located in New York. The total height of the building is 183 meter. This highly efficient structural system required 20 percent less structural steel compared to conventional moment frame structure. The wide flange rolled steel sections were used for diagonal members with all nodes being prefabricated and installed at site using bolted connection. Steel erection was completed in 2005 and Hearst Headquarters was open in September 2006.

2.2.4 Across Wind Loading

Quan and Gu[17] presented the equivalent static across-wind loads and responses of high-rise buildings. The across-wind equivalent static wind loads and responses were evaluated by combining resonant component and background component. The resonant component was computed according to the power spectral density of base moment and the background component was computed according to the base moment coefficients. The present method of equivalent static wind load was compared with AIJ recommended method for verification.

Quan and Gu[18] presented the across wind effect for high rise buildings. In this paper, 15 tall building models with different cross-sections and aspect ratios from 4 to 9 were tested with high-frequency force balance technique in a wind tunnel to obtain their first-mode generalized across-wind dynamic forces. They derived the formulas of the power spectra of the across-wind dynamic forces, the coefficients of base moment and shear force. Comparison was done between present formulas and literature for verification.

2.3 Summary

In this chapter, literature review related to diagrid system is presented. The review of literature includes behavior of diagrid system, analysis of diagrid system, optimal configuration of diagrid system, construction sequence and connection details of diagrid structural system.

Chapter 3

Behavior of Diagrid Structural System

3.1 General

Behavior of diagrid structure under gravity and lateral loading is presented in this chapter. Also methodology of preliminary design of diagrid system is discussed. Design of 36 storey diagrid structure is illustrated.

Diagrid is a system of triangulated beams, straight or curved, and horizontal rings that together make up a structural system for a skyscraper.

Diagrid structures are more effective in minimizing shear deformation because they carry shear by axial action of diagonal members while conventional framed tubular structures carry shear by the bending of the vertical columns. Diagrid structures generally do not need high shear rigidity cores because shear can be carried by the diagrids located on the perimeter [3].

Diagrid system is the most efficient, because the diagrid forms an exterior tube that can maximize the moment arm to resist overturning. Diagrid system has higher torsional rigidity than the other structural systems. Applying the diagrid system could make the building a landmark because of its unique structural system and external appearance [8].

3.2 Behavior under Gravity Loading

The analysis of the diagrid structures can be carried out in a preliminary stage by dividing the building elevation into a group of stacking floors each corresponding to a diagrid module. As shown in Fig.3.1, the diagrid module under the gravity load N_G is subjected to vertical downward force $N_{G,mod}$, which cause the compression in two diagonals and tension in horizontal chord. Internal forces in a diagrid elements due to gravity load can be determined using equations 3.1 and 3.2. It has been assumed that the external load is transferred to the diagrid module only at the apex joint of the module itself [16].

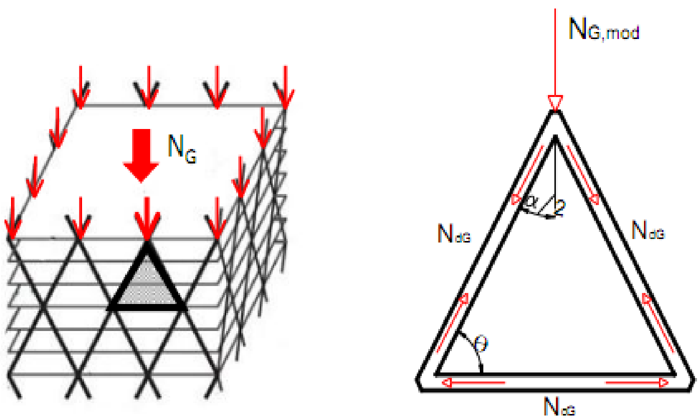


Figure 3.1: Effect of Gravity Load on Diagrid Module

$$N_{dG} = \frac{N_{G,mod}}{2\ \cos(\frac{\alpha}{2})} \tag{3.1}$$

$$N_{cG} = N_{dG} \sin \frac{\alpha}{2} \tag{3.2}$$

3.3 Behavior under Lateral Loading

Due to lateral load W, the shear force V_w and the overturning moment M_w generate in the structure.

Under the horizontal load W, the shear V_w causes a horizontal force in the apex joint of the diagrid modules $V_{w,mod}$. The shear force V_w is mainly resisted by the web panels of the diagrid module. As shown in Fig.3.2, direction and intensity of $V_{w,mod}$ depends on the position of diagrid module with respect to the direction of wind load. The horizontal force $V_{w,mod}$, which cause the compression in one diagonal and tension in another diagonal. Internal forces in a diagrid elements due to lateral load can be determined using equation 3.3. It has been assumed that the external load is transferred to the diagrid module only at the apex joint of the module itself [16].

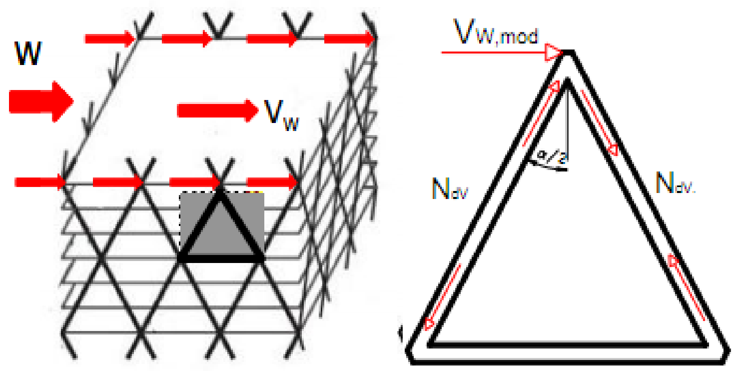


Figure 3.2: Effect of Shear Force on Diagrid Module

$$N_{dV} = \frac{V_{w,mod}}{2\ \sin(\frac{\alpha}{2})} \tag{3.3}$$

The horizontal load W and the overturning moment M_w causes vertical forces in the apex joint of the diagrid module $N_{w,mod}$. As shown in Fig.3.3, direction and intensity of $N_{w,mod}$ depends on the position of the module with respect to the direction of wind load. The maximum intensity of upward and downward force develop in the modules located on the windward and leeward sides, respectively, and gradually decreasing values in modules located on the web sides. The vertical upward force $N_{w,mod}$, causes the tension in two diagonals and compression in horizontal chord. The vertical downward force $N_{w,mod}$, causes the compression in two diagonals and tension in horizontal chord. Internal forces in a diagrid elements due to lateral load can be determined using equations 3.4 and 3.5. It has been assumed that the external load is transferred to the diagrid module only at the apex joint of the module itself [16].

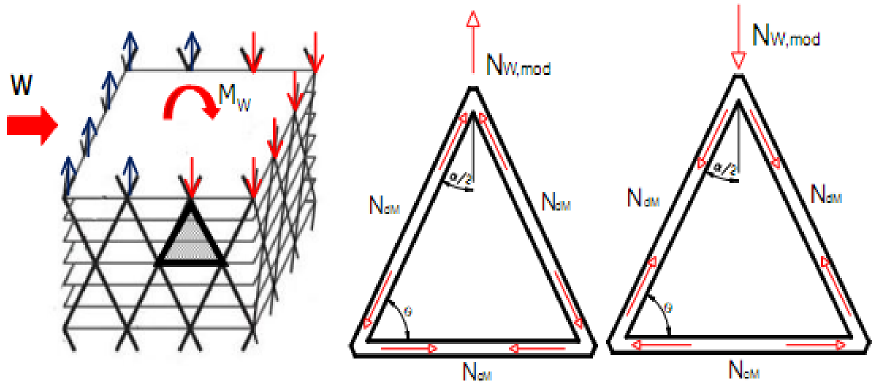


Figure 3.3: Effect of Overturning Moment on Diagrid Module

$$N_{dM} = \frac{N_{W,mod}}{2\ \cos(\frac{\alpha}{2})} \tag{3.4}$$

$$N_{cM} = N_{dM} \sin\frac{\alpha}{2} \tag{3.5}$$

3.4 Methodology of Preliminary Design of Diagrid System

This Methodology is based on stiffness and can be applied to diagrid structures of various heights and aspect ratios.

3.4.1 Shear Stiffness and Bending Stiffness

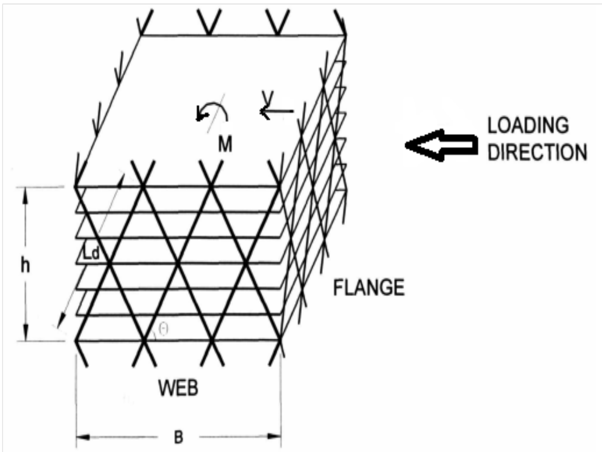


Figure 3.4: 6-Storey Diagrid Module

Each module is defined by a single level of diagrids that extend over n stories. 6-storey diagrid module is shown in Fig.3.4. Depending upon the direction of loading, the faces act as either web or flange elements. The diagonal members are assumed as truss element, and therefore the transverse shear and moment are resisted through only axial action. With this idealization, the design problem reduces to determining the cross-sectional area of typical web and flanges members for each module. These quantities are establishes with a stiffness based approach. Fig.3.5 shows the deformed shape of diagrid module due to transverse shear and bending moment.

The shear force, V, and bending moment, M, are expressed in terms of the relative displacement and rotation measures, Δu and $\Delta\beta$, for the module as

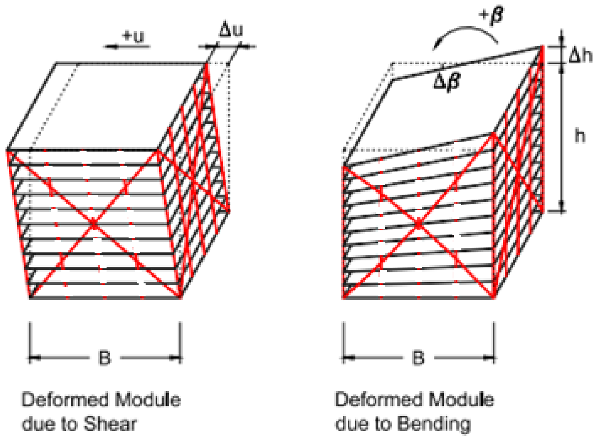


Figure 3.5: Deformed Shape of Diagrid Module due to Lateral Load

$$V = K_T \Delta\ u \tag{3.6}$$

$$M = K_B \Delta \beta \tag{3.7}$$

Where the K_T is the transverse shear stiffness and K_B is the bending stiffness.

The displacement and rotation due to transverse shear and bending moment are given by,

$$\Delta\ u = \gamma\ h \tag{3.8}$$

$$\Delta\ \beta = \chi\ h \tag{3.9}$$

Where the h is the height of the module, γ is the transverse shear strain and χ is the curvature.

The shear force V and bending moment M can also be expressed in terms of the diagonal member forces by,

$$V = N_w \ F_{d,w} \ \cos\Theta \tag{3.10}$$

$$M = (N_f + \delta)BF_{d,f} \ \sin\Theta \tag{3.11}$$

Where N_w is the number of diagonals extending over the full module height in one web plane. N_f is the numbers of diagonals for one flange plane and δ is the addition to this number to take the web diagonals into accounts.

Assuming linear elastic behavior, the member forces are also related to the diagonal strains, $\varepsilon_{d,w}$ and $\varepsilon_{d,f}$ respectively, by

$$F_{d,w} = A_{d,w}\sigma_{d,w} = A_{d,w}E\varepsilon_{d,w} \tag{3.12}$$

$$F_{d,f} = A_{d,f}\sigma_{d,f} = A_{d,f}E\varepsilon_{d,f} \tag{3.13}$$

The extensional strain in the diagonals on the web, $\varepsilon_{d,w}$, due to the relative lateral displacement between adjacent nodes is a function of Δu and Θ.

$$\varepsilon_{d,w} = \frac{e_{d,w}}{L_d} = \frac{\Delta u \cos\Theta}{\frac{h}{\sin\Theta}} = \frac{\Delta u \cos\Theta \sin\Theta}{h} \tag{3.14}$$

Where L_d is the diagonal length of module and $e_{d,w}$ is the change in diagonal length of module.

Similarly, The extensional strain in the diagonals on the flange, $\varepsilon_{d,f}$, is related to the angle change, $\Delta\beta$

$$\varepsilon_{d,f} = \frac{e_{d,f}}{L_d} = \frac{\Delta h}{L_d} = \frac{B\Delta\beta \sin\Theta}{2L_d} \tag{3.15}$$

Combining above equations results in the following expression for the module stiffness measures.

$$K_T = 2N_w(\frac{A_{d,w}E\cos^2\Theta}{L_d}) \tag{3.16}$$

$$K_B = N_f(\frac{B^2 A_{d,f}E\sin^2\Theta}{2L_d}) \tag{3.17}$$

With V and M, one specifies the desired transverse shear and bending deformation, γ and χ, and determines the required stiffness using equations 3.6 and 3.7.

$$K_T = \frac{V}{\gamma h} \tag{3.18}$$

$$K_B = \frac{M}{\chi h} \tag{3.19}$$

Finally, substituting for the stiffness terms, one obtains expressions for the typical areas in the web and flange [14].

$$A_{d,w} = \frac{VL_d}{2N_w E_d h\gamma \cos^2 \Theta} \tag{3.20}$$

$$A_{d,f} = \frac{2ML_d}{(N_f + \delta)B^2 E_d h\chi \sin^2 \Theta} \tag{3.21}$$

3.4.2 Displacement due to Shear and Bending

Optimal design can be obtain through deformation corresponds to a state of uniform shear and bending deformation under the design loading. Uniform deformation states are possible only for statically determinate structure. Assuming the diagrid structure is modeled as a cantilever beam, the deflection at the top is given by,

$$u(H) = \gamma H + \frac{\chi H^2}{2} \tag{3.22}$$

Where H is the height of building, γ H is the contribution from shear deformation and $(\chi H^2)/2$ is the contribution from bending as shown in Fig.3.6.

The design starts from specifying the bending deformation and shear deformation of the structure. In order to specify the relative contribution of shear versus bending deformation, a dimensionless factor 'S' is introduced, which is equal to the ratio of

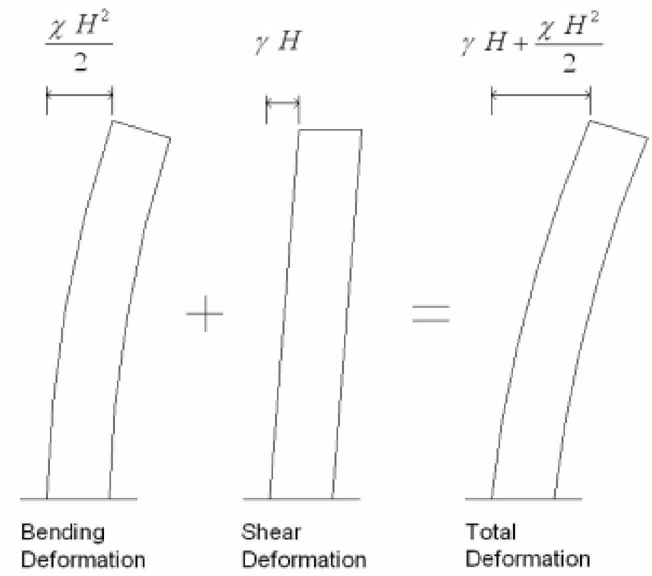


Figure 3.6: Uniform Bending and Shear Deformation of Tall Buildings

the displacement at the top of the structure due to bending and the displacement due to shear. The 'S' value can be calculated by,

$$S = \frac{\frac{\chi H^2}{2}}{\gamma H} = \frac{H\chi}{2\gamma} \tag{3.23}$$

As a building become taller and its height-to-width aspect ratio increases, the building naturally tends to act more like a bending beam, and a larger bending to shear deformation ratio 'S', is a reasonable choice for an economical design. The choice of 'S' value for the least amount of structural material usage depends on not only the building's aspect ratio but also the structural system employed for the building, because each building has its own unique behavioral characteristics. The maximum allowable displacement is usually expressed as a fraction of the total building height.

$$u(H) = \frac{H}{\alpha} \tag{3.24}$$

Noting Equation 3.22 and 3.23, equation 3.24 is expands to

$$u(H) = (1 + S)\gamma h = \frac{H}{\alpha} \tag{3.25}$$

Then,

$$\gamma = \frac{1}{(1 + S)\alpha} \tag{3.26}$$

Also, χ is determined using Equation 3.23.

$$\chi = \frac{2\gamma S}{H} = \frac{2S}{H(1 + S)\alpha} \tag{3.27}$$

Typical value for α are is about 500 [14].

3.5 Design Example of 36 Storey Building

The stiffness based methodology discussed in previous section is applied to 36-storey diagrid structure. The 36 storey building is having 36m x 36m square plan with diagonals at slope 74.5 degrees as external braces as shown in Fig.3.7. The plan and side view of the building are shown in Fig.3.7. The inclined columns are provided at six meter spacing along the perimeter. The model structure has the typical story height of 3.6m. The interior frame of the diagrid structures is designed only for gravity load and is modeled as pin-connected. Design wind pressure of 4.5 kN/m^2 is considered uniform throught the height.

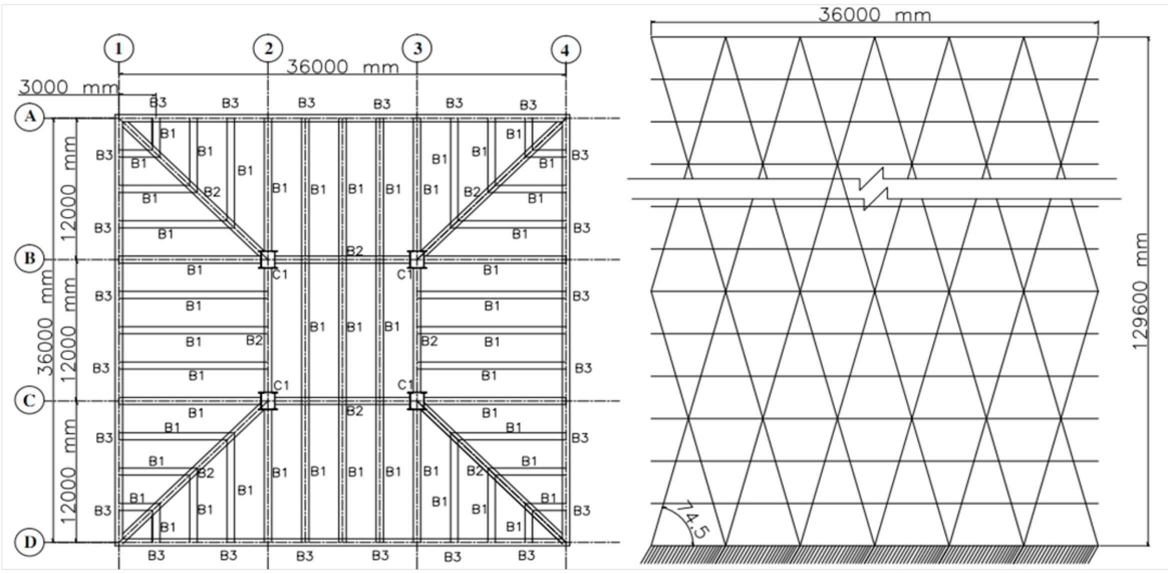


Figure 3.7: Typical Floor Plan and Elevation of 36 Storey Building

The first step is to divide the structure into appropriate modules. For 36 storey building, every 6-storey segment is considered as structural module. The second step is to calculate shear force and bending moment considering wind load as mentioned. For each six storey structural module, shear forces and bending moments are shown in Table.3.1. Preliminary member sizes are calculated for a 36 storey structure with an angle of 74.5 degrees. The parameter for this case is S=1. Member sizes for the modules are computed using the equations 3.28 and 3.29. Preliminary member sizes are shown in Table.3.1. As the wind can blow in either direction, the role of a plane can be either a flange or a web. Preliminary design value for the module is taken as the larger of the two values. Profiles of the required areas for the typical diagonals in the web and flange are plotted in Fig.3.8. From the Table.3.1 and Fig.3.8, it is observed that the transverse shear is governing for 36 storey diagrid structure. The building is only design for shear force.

Following are the calculation of area of diagonal and steel quantity for Module

Sample Calculation of Area of Diagonal for Shear ($A_{d,w}$) (1^{st} Module):

$$A_{d,w} = \frac{VL_d}{2N_w E_d h\gamma \cos^2\Theta} = \frac{17232.867 \times 22.41}{2 \times 12 \times (2 \times 10^8) \times 21.6 \times 0.001 \times \cos^2 74.5} \tag{3.28}$$

where, $\gamma = \frac{1}{(1+S)\alpha} = \frac{1}{(1+1)\times 500} = 0.001$

$A_{d,w} = 0.0517\ m^2$

Sample Calculation of Steel Quantity for Shear (1^{st} Module):

Total Steel of Module = Steel Density$\times L_d \times$Number of diagonals in Module$\times A_{d,w}$

˜ $= 78.5 \times 22.41 \times 48 \times 0.0517$

˜ $= 4366.8$ kN

Sample Calculation of Area of Diagonal for Bending ($A_{d,f}$) (1^{st} Module):

$$A_{d,f} = \frac{2ML_d}{(N_f + \delta)B^2 E_d h\chi \sin^2\Theta} = \frac{2 \times 1071589.103 \times 22.41}{(12+2) \times 36^2 \times (2 \times 10^8) \times 21.6 \times 0.0000154 \times \sin^2 74.5} \tag{3.29}$$

where, $\chi = \frac{2S}{H(1+S)\alpha} = \frac{2\times 1}{129.6\times(1+1)\times 500} = 0.0000154$

$A_{d,f} = 0.0429\ m^2$,

Sample Calculation of Steel Quantity for Bending (1^{st} Module):

Total Steel of Module = Steel Density $\times$ L_d $\times$ Number of diagonals in Module$\times$ $A_{d,f}$

˜ $= 78.5 \times 22.41 \times 48 \times 0.0429$

˜ $= 3613.0$ kN

Similarly steel quantity of other modules are calculated and shown in Table.3.1.

Table 3.1: Preliminary Member Sizing for the 36-Storey Diagrid Structure

Storey	V (kN)	M (kN-m)	$A_{d,w}$ (m^2) (Shear)	$A_{d,f}$ (m^2) (Bending)	Steel (kN) (Shear)	Steel (kN) (Bending)
31st- 36th	2654.325	21934.079	**0.0080**	0.0009	**672.6**	73.9
25th-30th	5570.034	105906.481	**0.0167**	0.0042	**1411.4**	357.0
19th-24th	8485.742	252858.184	**0.0255**	0.0101	**2150.2**	852.5
13th-18th	11401.451	462789.189	**0.0342**	0.0185	**2889.1**	1560.3
7th-12th	14317.159	735699.495	**0.0430**	0.0294	**3627.9**	2480.5
1st-6th	17232.867	1071589.103	**0.0517**	0.0429	**4366.8**	3613.0
Total Steel (kN)					**15118**	

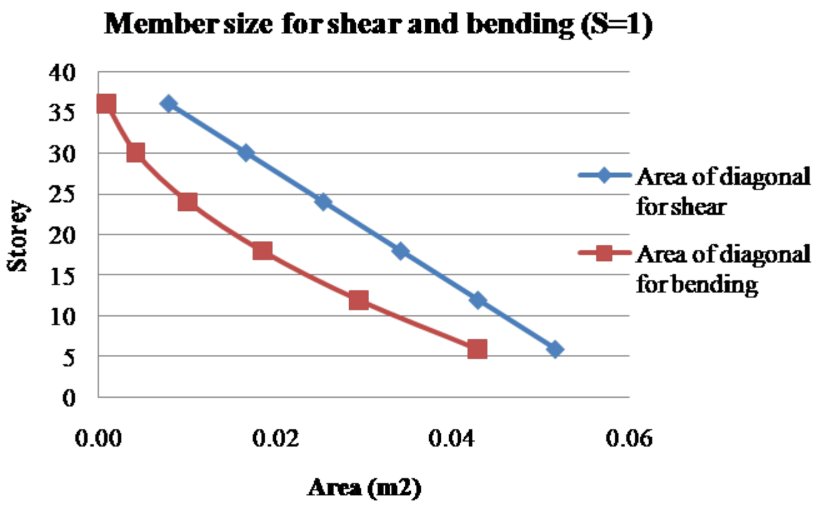


Figure 3.8: Preliminary Member Sizing for the 36-Storey Diagrid Structure

The structure with the preliminary design is analyzed using ETABS to assess the accuracy of the preliminary sizing methodology. For analysis the beams and columns are modelled by beam elements and the external braces are modelled by truss elements. Comparison of approximate analysis considering stiffness based approach and exact analysis using ETABS software is carried out. Table.3.2 compares the results for the top storey displacement and steel required. There is 13.95 % reduction in displacement of diagrid structure in x-direction due to wind load in exact analysis. Exact analysis requires 4.5 % higher steel compared to approximate analysis.

Table 3.2: Comparison between Approximate and Exact Analysis

Structure	Height	Width	S	Method	Top storey Displacement	Steel in Diagonal
36 storey	129.6 m	36 m	1	Approximate Analysis	0.258 m	1511.8 ton
				Exact Analysis	0.222 m	1580.96 ton

The sizes of typical members of diagrid structure are shown in Table.3.3.

Table 3.3: Size of Typical Members of Structural System

Elements of Diagrid System	Section Size
Diagonals	375 mm Pipe sections with 12 mm thickness(from 31st to 36th storey) 450 mm Pipe sections with 12 mm thickness(from 25th to 30th storey) 450 mm Pipe sections with 20 mm thickness(from 19th to 24th storey) 450 mm Pipe sections with 25 mm thickness(from 13th to 18th storey) 600 mm Pipe sections with 25 mm thickness(from 7th to 12th storey) 675 mm Pipe sections with 25 mm thickness(from 1st to 6th storey)
Beams B1 Beams B2 Beams B3	ISMB 600 ISWB 600 with top and bottom cover plate of 220 x 50 mm ISWB 600
Interior Columns	1500 mm x 1500 mm

Hollow pipe and interior column section are shown in Fig.3.9.

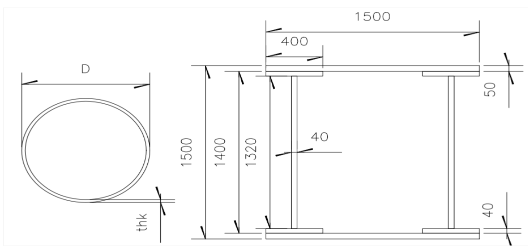


Figure 3.9: Hollow Pipe and Interior Column Section

3.5.1 Determining Optimal Value of 'S'

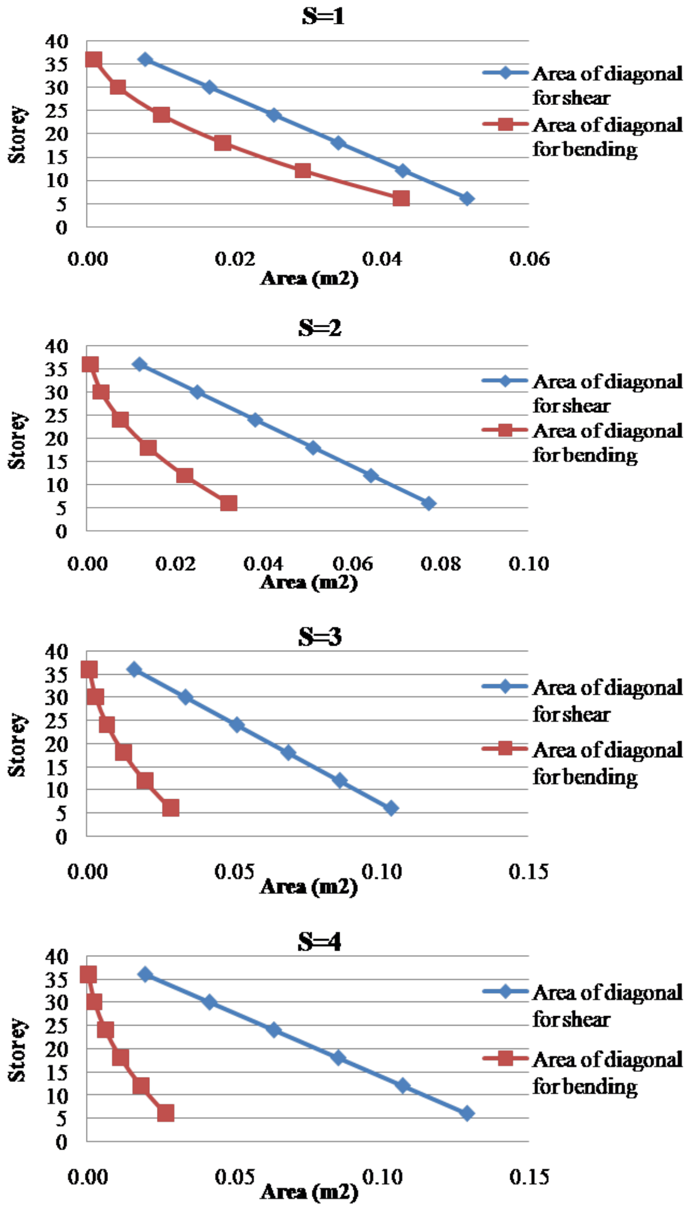


Figure 3.10: Preliminary Member Sizing for the 36-Storey Diagrid Structures with Various 'S'

In Table.3.1 and Table.3.2 steel quantity is calculated based on values of 'S' as 1.0. In order to understand effect of 'S' on steel quantity, various values of 'S' i.e. 2, 3, 4 are considered. Fig.3.10 shows the sets of diagonal area for the 36 storey building, with value of 'S' varying from 1 to 4. Fig.3.11 shows total steel quantity for 36 storey structures with varying values of 'S'. A optimal value of 'S' can be obtained by comparing the steel required for each design. From Fig.3.11 the optimal value for 'S' is observed as 1. The steel quantity include weight of perimeter diagonal only. The internal columns and beams are same for all buildings. So, they are not considered for comparison.

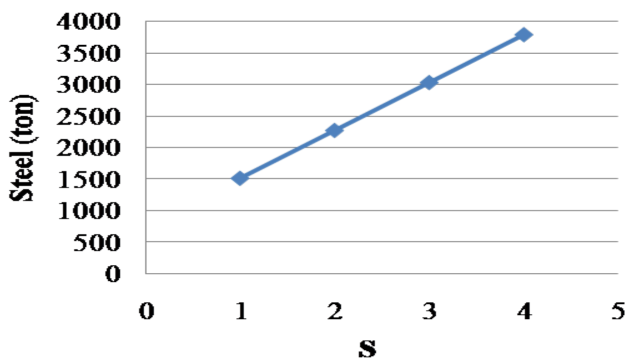


Figure 3.11: Diagrid Steel Mass for 36-Storey

3.6 Comparison of Diagrid and Tubular Structural System

3.6.1 General

To illustrate the advantages of diagrid structural system over tubular structural system 36 storey steel building is presented in this chapter. Modeling, analysis and design of diagrid and framed tube structure are carried out using ETABS software. All structural members are designed using IS 800:2007 [23] considering all load combinations. The static analysis is carried out using wind loads and earthquake loads.

3.6.2 Building Configuration

The 36 storey building is having 36m x 36m square plan is shown in Fig.3.12. The side view of diagrid and tubular system are shown in Fig.3.13. The inclined columns are provided at six meter spacing along the perimeter. In diagrid structures pair of braces is located on the periphery of the building. In tubular structure external vertical columns are spaced at three meters along the periphery. Typical storey height is 3.6 m and total height of building is 129.6 m. For analysis the beams and columns is modeled by beam elements and braces are modeled by truss elements.

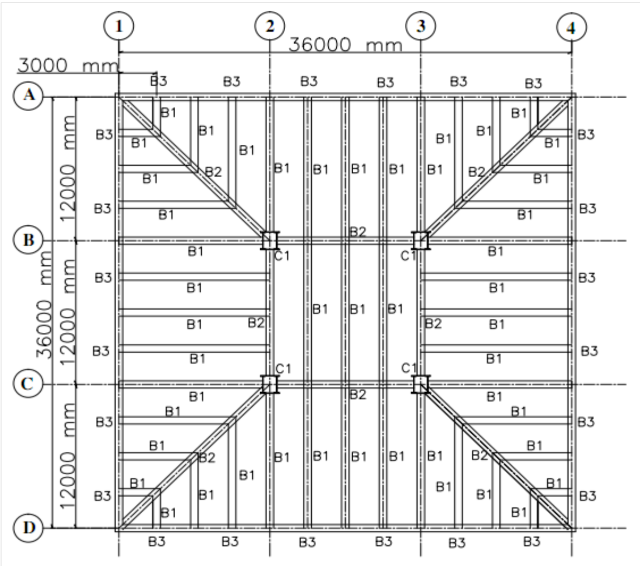


Figure 3.12: Typical Floor Plan 36 Storey Buildings

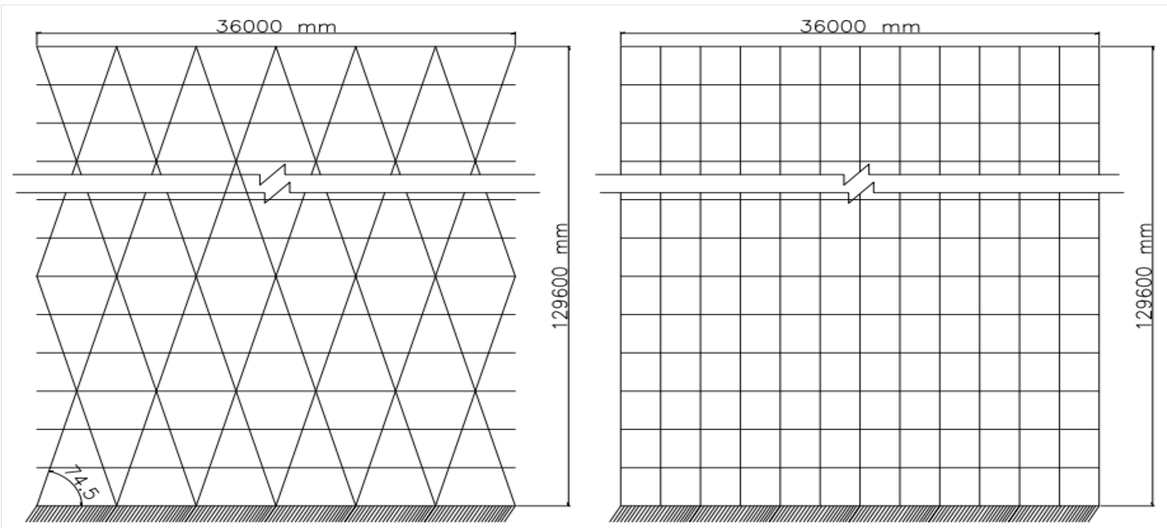


Figure 3.13: Typical Elevation of 36 Storey Buildings

3.6.3 Loading Data

Following loading are considered for the analysis and design of structure;

Dead Load : Dead load of slab 3.75 kN/m^2

self-weight of the structural members.

Live Load : 2.5 kN/m^2

Wind Load:

Basic wind speed : 30 m/sec

Terrain category : III

Class : C

The base shear for along wind load in X-direction and in Y-direction as per IS:875(III)-1987 [24] are shown in Table.3.4

Table 3.4: Base Shear due to Static Wind Loading

Loading	Base Shear of Diagrid System (kN)	Base Shear of Tubular System (kN)
X-direction	3231.26	3100.63
Y-direction	3231.26	3100.63

Earthquake Load:

Location : Ahmedabad

Zone factor : III

Importance factor : 1

Response reduction factor : 5

Soil condition : Medium

The base shear for earthquake load in X-direction and in Y-direction as per IS:1893-2002 [25] are shown in Table.3.5

Table 3.5: Base Shear due to Static Earthquake Loading

Loading	Base Shear of Diagrid System (kN)	Base Shear of Tubular System (kN)
X-direction	1732.29	1451.08
Y-direction	1733.17	1451.08

Load Combinations: Table.3.6 shows the load combinations for the analysis and design of structure as per IS:800-2007 [23].

Table 3.6: Load Combinations for Static Loads

Sr No.	Limit state of collapse	Limit state of serviceability
1.	1.5 (DL + LL)	1.0 (DL + LL)
2.	1.2 (DL + LL ± WLx)	1.0 DL + 0.8 LL ± 0.8 WLx
3.	1.2 (DL + LL ± WLy)	1.0 DL + 0.8 LL ± 0.8 WLy
4.	1.5 (DL ± WLx)	1.0 DL ± 1.0 WLx
5.	1.5 (DL ± WLy)	1.0 DL ± 1.0 WLy
6.	0.9 DL ± 1.5 WLx	1.0 DL + 0.8 LL ± 0.8 EQx
7.	0.9 DL ± 1.5 WLy	1.0 DL + 0.8 LL ± 0.8 EQy
8.	1.2 (DL + LL) ± 0.6 WLx	1.0 DL ± 1.0 EQx
9.	1.2 (DL + LL) ± 0.6 WLy	1.0 DL ± 1.0 EQy
10.	1.2 (DL + LL ± EQx)	
11.	1.2 (DL + LL ± EQy)	
12.	1.5 (DL ± EQx)	
13.	1.5 (DL ± EQy)	
14.	0.9 DL ± 1.5 EQx	
15.	0.9 DL ± 1.5 EQy	
16.	1.2 (DL + LL) ± 0.6 EQx	
17.	1.2 (DL + LL) ± 0.6 EQy	

3.6.4 Results and Discussion

The results in terms of time period, displacement, inter-storey drift and structural steel quantity are compared to understand the advantages of diagrid structural system over the tubular steel structures. Tubular sections are considered for the design of diagrid building and beam sections are considered for the design of framed tube structure. The dimensions of typical structural members after design are shown in Table.3.7.

Table 3.7: Size of Typical Members for Structural System

Structural System	Elements of Structure	Section Size
Diagrid Structural System	Diagonals	375 mm Pipe sections with 20 mm thickness (from 13th to 36th storey) 450 mm Pipe sections with 20 mm thickness (from 1st to 12th storey)
	Beams B1 Beams B2 Beams B3	ISMB 600 ISWB 600 with top and bottom cover plate of 220x50 mm ISWB 600
	Interior Columns	1500 mm x 1500 mm
Tubular Structural System	Perimeter Columns	2ISWB550 (from 13ht to 36th storey) 2ISWB600 (from 1st to 12th storey)
	Beams B1 Beams B2 Beams B3	ISMB 600 ISWB 600 with top and bottom cover plate of 220x50 mm ISWB 600
	Interior Columns	1500 mm x 1500 mm

Interior columns section is shown in Fig.3.14.

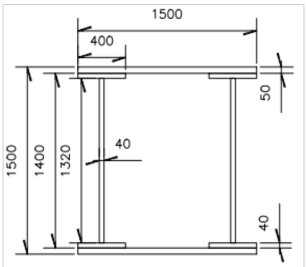


Figure 3.14: Interior Column Section

Fig.3.15 shows the comparison of time period of the diagrid structure and tubular structure. The first mode time period of diagrid system is 49.69 % less than that of tubular structure which indicates more stiffness of diagrid structure.

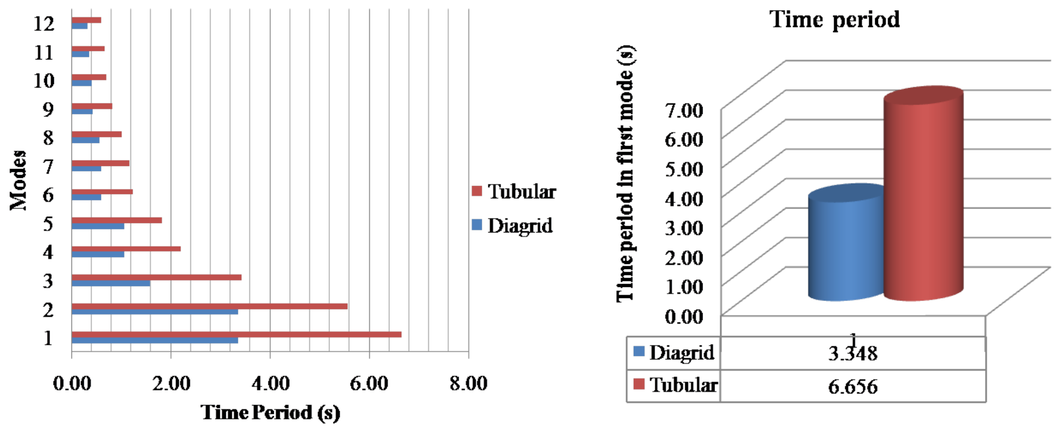


Figure 3.15: Time Period of Structural System

Fig.3.16 shows the comparison of displacement of the diagrid structure and tubular structure. There is 71.78 % and 65.65 % reduction in displacement of diagrid system in x-direction due to wind load and earthquake load respectively. Also 60.29 % and 51.91 % reduction in displacement of diagrid system in y-direction is observed due to wind load and earthquake load respectively in comparison of tubular structure.

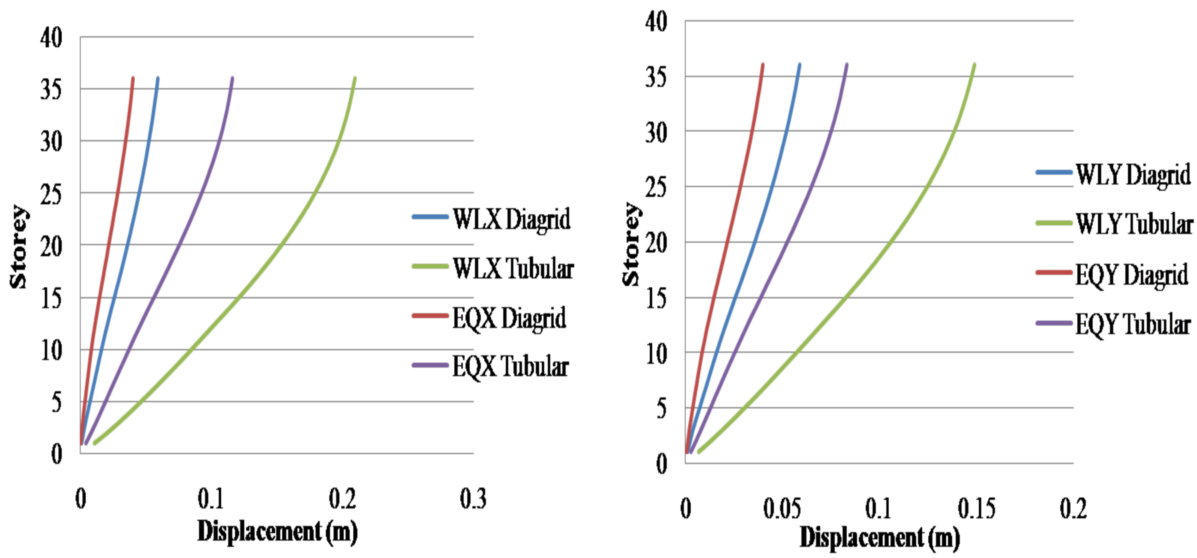


Figure 3.16: Displacement of Structural System

Fig.3.17 shows the comparison of inter-storey drift of the diagrid structure and tubular structure. There is 37.50 % and 38.96 % reduction in inter-storey drift of diagrid system in x-direction due to wind load and earthquake load respectively. Also 29.42 % and 26.59 % reduction in inter-storey drift of diagrid system in y-direction is observed due to wind load and earthquake load respectively in comparison of tubular structure.

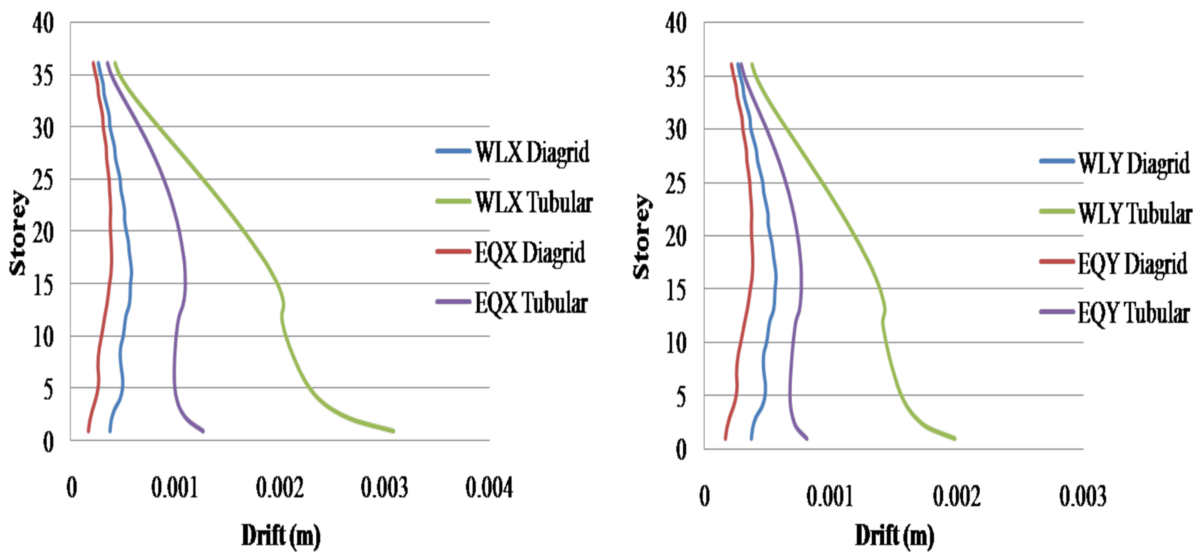


Figure 3.17: Inter-Storey Drift of Structural System

Table.3.8 shows the comparison of structural weight of diagrid and framed tube structural system. Diagrid structural system requires 14 % less steel compared to framed tube system.

Table 3.8: Structural Weight of System

Building Story Height	Tubular Structure Weight (kN) for Perimeter Columns	Diagrid Structure Weight (kN) for Diagonals
36 Storey (129.6 m)	15348.37	13200.87

3.7 Summary

In this chapter, behavior of diagrid structure under gravity and lateral loading is presented. Also methodology of preliminary design of diagrid system is discussed.

Design of 36 storey diagrid steel structural system having external columns at 74.5 slope is illustrated. Approximate analysis considering stiffness based approach and exact analysis using ETABS software is carried out for 36 storey diagrid building. Analysis results in terms of quantity of steel in diagrid members and top storey displacements are compared as obtained by approximate analysis and exact analysis. The influence of factor 'S' (Ratio of bending deformation to shear deformation) on quantity of steel is also explored.

Further comparison of diagrid and tubular structural system is carried out to understand the advantages of diagrid structural system. Perimeter diagrid structural system saves approximately 14 % structural steel compared to a conventional tubular structure for 36 storey building considered in this study.

Chapter 4

Optimal Angle of Diagrid Structural System

4.1 General

In order to study the optimal angle of diagrid structural system 40, 50, 60, 70 and 80 storey steel buildings are considered in this chapter. To find optimal design five different cases having angles of 50.2, 67.4, 74.5, 78.2, and 80.5 degrees with 2, 4, 6, 8, and 10 storey modules respectively are considered for each diagrid building. The stiffness based design methodology is used for 40 to 80 storey diagrid structures. In diagrid structures, diagonals resist both storey shear and moment. Thus, the optimal angle of diagonals is highly dependent on the building height.

4.2 Building Configuration

The 40, 50, 60, 70 and 80 storey tall building is having 36 m $\times$ 36 m square plan as shown in Fig.4.1. In diagrid structures, pair of braces is located on the periphery of the building. The different angle of inclination of diagonal column on periphery like 50.2, 67.4, 74.5, 78.2, and 80.5 degrees with 2, 4, 6, 8, and 10 storey modules respectively are shown in Fig.4.2 to Fig.4.6. The angle of inclination is kept uniform throught the height. The inclined columns are provided at six meter spacing along the perimeter. The interior frame of the diagrid structures is designed only for gravity load.

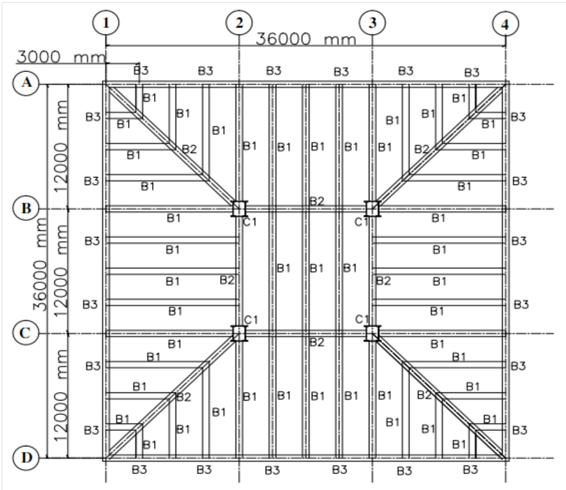


Figure 4.1: Typical Floor Plan of Diagrid Structure

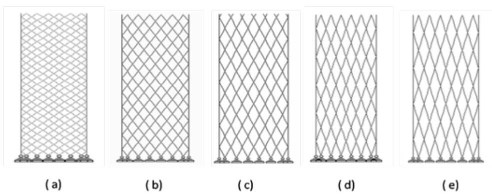


Figure 4.2: Elevation of 40 Storey Model with (a) 50.2, (b) 67.4, (c) 74.5 , (d) 78.2; (e) 80.5 degrees Inclination

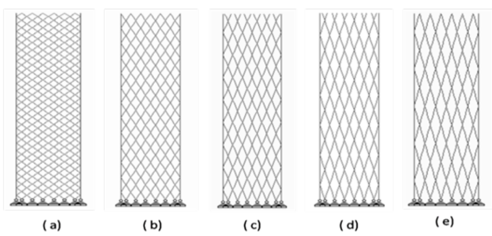


Figure 4.3: Elevation of 50 Storey Model with (a) 50.2, (b) 67.4, (c) 74.5 , (d) 78.2; (e) 80.5 degrees Inclination

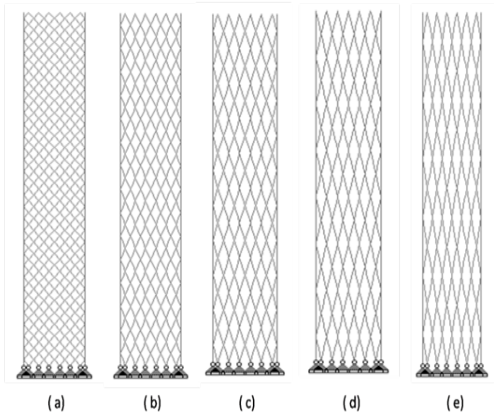


Figure 4.4: Elevation of 60 Storey Model with (a) 50.2, (b) 67.4, (c) 74.5 , (d) 78.2; (e) 80.5 degrees Inclination

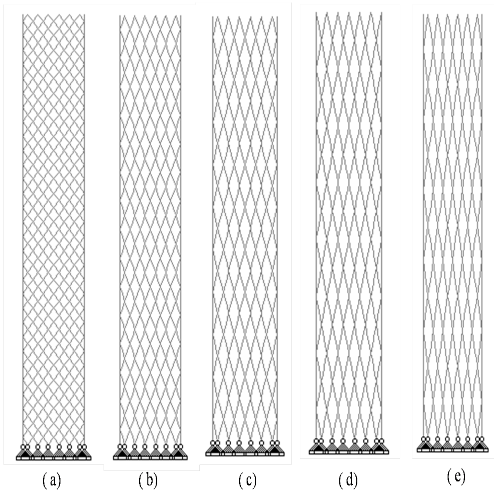


Figure 4.5: Elevation of 70 Storey Model with (a) 50.2, (b) 67.4, (c) 74.5 , (d) 78.2; (e) 80.5 degrees Inclination

Following structural data are considered for study:

No. of storey = 40, 50, 60, 70 and 80

Typical height of storey = 3.6 m

Total height of 40, 50, 60, 70 and 80 storey building = 144 m, 180 m, 216 m, 252 m and 288 m

Wind Load: Dynamic along wind loading is calculated as per IS:875(III)-1987 [24] (Gust factor method) and calculations are presented in the Appendix A.

Basic Wind Speed = 30 m/s, Terrain Category = III

Two percent damping is assumed for the calculation for the gust effect factor.

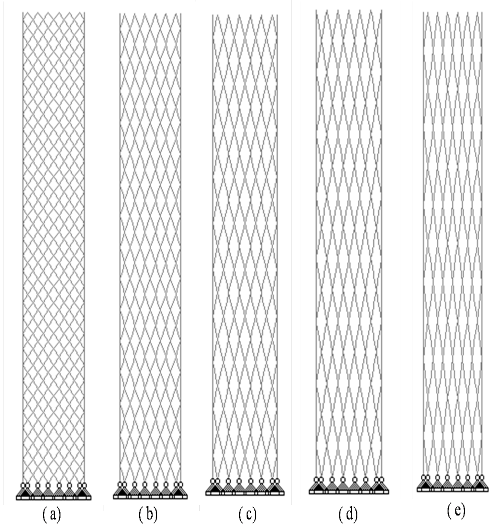


Figure 4.6: Elevation of 80 Storey Model with (a) 50.2, (b) 67.4, (c) 74.5 , (d) 78.2; (e) 80.5 degrees Inclination

4.3 Design Examples of 40, 50, 60, 70 and 80 Storey Buildings

The stiffness based approach discussed in previous chapter is applied to 40, 50, 60, 70 and 80 storey diagrid structures, with an aspect ratio of 4 to 8. The optimal design study for each storey height diagrid building is carried out with five different inclination of columns. The required lateral stiffness is allocated to the perimeter diagrids, and consequently core structures are assumed to carry only gravity load in this study. The maximum lateral displacement, $\frac{H}{500}$ is used as a deformation parameters. Shear forces and bending moments are calculated using given loadings. Member sizes for the each modules are calculated using equations 4.1 and 4.2

Following are the calculation of area of diagonal and steel quantity for Module
(70 Storey with 67.4 degrees)

Sample Calculation of Area of Diagonal for Shear ($A_{d,w}$) (1^{st} Module):

$$A_{d,w} = \frac{VL_d}{2N_w E_d h\gamma \cos^2\Theta} \tag{4.1}$$

where, $\gamma = \frac{1}{(1+S)\alpha} = \frac{1}{(1+4)\times 500} = 0.0004$

$A_{d,w} = \frac{7833.40\times 15.6}{2\times(12)\times(2\times 10^8)\times 14.4\times 0.0004\times\cos^2 67.4}$

$A_{d,w} = 0.02887\ m^2$

Sample Calculation of Steel Quantity for Shear (1^{st} module):

Total Steel of Module = Steel Density$\times L_d\times$Number of diagonals in Module$\times A_{d,w}$

˜ $= 78.5\times 15.6\times 48\times 0.02887$

˜ $= 1696.98$ kN

Sample Calculation of Area of Diagonal for Bending ($A_{d,f}$) (1^{st} Module):

$$A_{d,f} = \frac{2ML_d}{(N_f+\delta)B^2 E_d h\chi \sin^2\Theta} \tag{4.2}$$

where, $\chi = \frac{2S}{H(1+S)\alpha} = \frac{2\times 4}{252\times(1+4)\times 500} = 0.0000127$

$A_{d,f} = \frac{2\times 1101633.4\times 15.6}{(12+2)\times 36^2\times(2\times 10^8)\times 14.4\times 0.0000127\times\sin^2 67.4}$

$A_{d,f} = 0.06116\ m^2$

Sample Calculation of Steel Quantity for Bending (1^{st} module):

Total Steel of Module = Steel Density $\times$ L_d $\times$ Number of diagonals in Module $\times$ $A_{d,f}$

˜ $= 78.5\times 15.6\times 48\times 0.06116$

˜ $= 3595.16$ kN

For design study, both N_w (Number of Diagonal on Each Web Plane) and N_f (Number of Diagonal on Each Flange Plane) are taken as 12. To consider the contribution of the web diagonals to the bending rigidity one extra diagonal on each flange is considered, resulting in $\delta\approx 2$. Shear forces, bending moments, preliminary member sizing and diagonal steel mass for the 70-storey diagrid structure having 67.4 degrees with ‘S’=4 are shown in Table.4.1.

Table 4.1: Preliminary Member Sizing and Diagonal Steel Mass for the 70-storey Diagrid Structure with ‘S’=4 (67.4 degrees)

Storey	V (kN)	M (kN-m)	Ad (m^2) (Shear)	Ad (m^2) (Bending)	Steel (kN) (Shear)	Steel (kN) (Bending)
69th-70th	227.20	285.48	**0.00084**	0.00002	**24.61**	0.466
65th-68th	811.72	6654.10	**0.00299**	0.00037	**175.84**	21.71
61st-64th	1385.11	21380.96	**0.00510**	0.00119	**300.06**	69.77
57th-60th	1947.48	44306.40	**0.00718**	0.00246	**421.89**	144.59
53rd-56th	2497.87	75270.08	**0.00921**	0.00418	**541.12**	245.64
49th-52nd	3033.98	114084.32	**0.01118**	0.00633	**657.26**	372.31
45th-48th	3555.92	160543.53	**0.01311**	0.00891	**770.33**	523.93
41st-44th	4063.72	214444.56	**0.01498**	0.01191	**880.34**	699.83
37th-40th	4555.13	275574.14	**0.01679**	0.01530	**986.80**	899.33
33rd-36th	5029.61	343690.37	0.01854	**0.01908**	1089.58	**1121.62**
29th-32nd	5487.45	418550.95	0.02022	**0.02324**	1188.77	**1365.93**
25th-28th	5924.57	499907.85	0.02183	**0.02775**	1283.46	**1631.44**
21st-24th	6333.33	587409.51	0.02334	**0.03261**	1372.02	**1917.00**
17th-20th	6714.53	680650.87	0.02475	**0.03779**	1454.60	**2221.29**
13th-16th	7068.19	779240.13	0.02605	**0.04326**	1531.21	**2543.03**
9th-12th	7383.02	882728.39	0.02721	**0.04901**	1599.42	**2880.77**
5th-8th	7645.32	990503.10	0.02818	**0.05499**	1656.24	**3232.49**
1st-4th	7833.40	1101633.40	0.02887	**0.06116**	1696.98	**3595.16**
Total Steel Quantity (kN)					**25267.06**	

The required areas for the typical diagonals in the web and flange along the height for 70 storey diagrid structure having 67.4 degrees with ‘S’= 1, 4 and 8 are shown in Fig.4.7. For 70 storey diagrid structure having 67.4 degrees with ‘S’= 4 gives the least amount of steel quantity. As, wind can blow in either direction, the role of a plane can be either a flange or a web. So, preliminary design value for the module is taken as the larger of the two values.

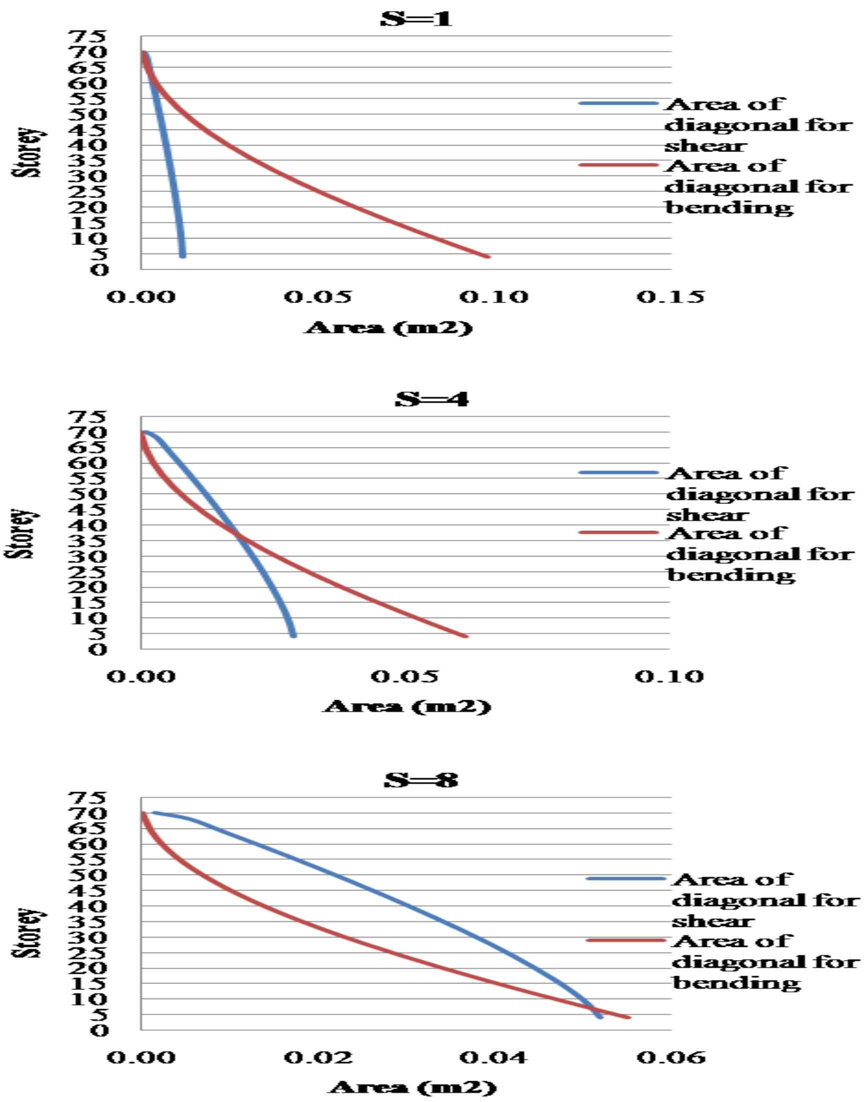


Figure 4.7: Preliminary Member Sizing for the 70-storey Diagrid Structure with various 'S' (67.4 degrees)

To find out the optimal design of 40 to 80 storey diagrid structure five different diagonal angles are considered. For examples, for 70 storey diagrid structures, five different angles are considered with various 'S' values. Among all the designs within the five sets, the one which requires the least amount of material is considered as optimal. The angle and 'S' value for that particular design is recorded as the optimal angle for a 70 storey diagrid structure and as the optimal 'S' which produced the most desirable deformation mode respectively. Similar studies are carried out for 40, 50, 60 and 80 storey diagrid structure. It is found that 67.4 degrees angle is near the optimal angle for the 40 to 80 storey diagrid structure.

Variation in diagonal steel quantity with different values of 'S' for 40, 50, 60, 70, and 80 storey diagrid structure is shown in Fig.4.8 to Fig.4.12.

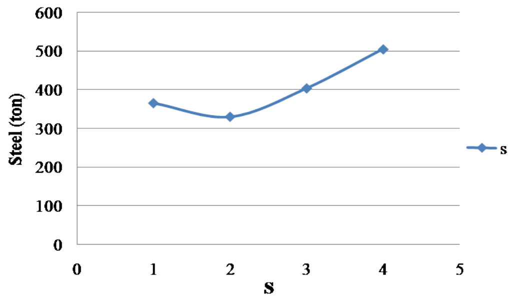


Figure 4.8: Diagrid Steel Mass for 40-Storey Structure (67.4 degrees)

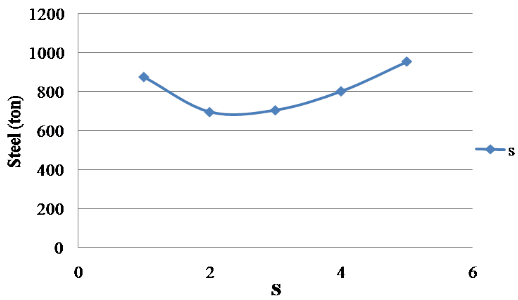


Figure 4.9: Diagrid Steel Mass for 50-Storey Structure (67.4 degrees)

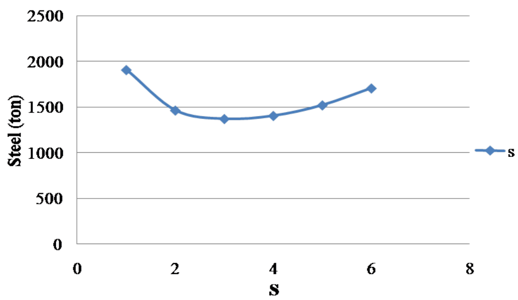


Figure 4.10: Diagrid Steel Mass for 60-Storey Structure (67.4 degrees)

It can be observed that, as the building become taller, optimal 'S' values based on material usage become higher.

The optimal value of 'S' for 40, 50, 60, 70 and 80 storey is 2, 2, 3, 4 and 5 respectively. From the results, it can be seen that, bending mode is governing compared to the shear mode as building become taller.

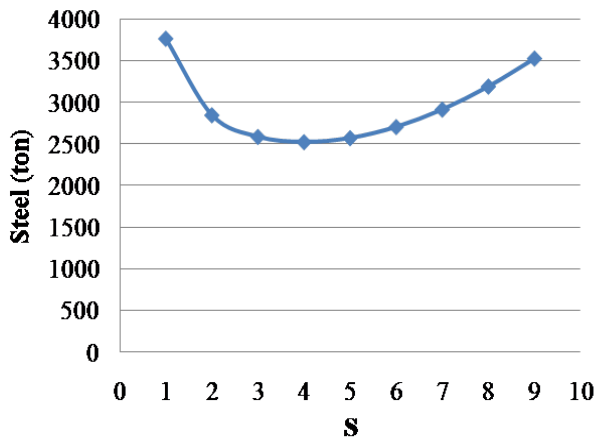


Figure 4.11: Diagrid Steel Mass for 70-Storey Structure (67.4 degrees)

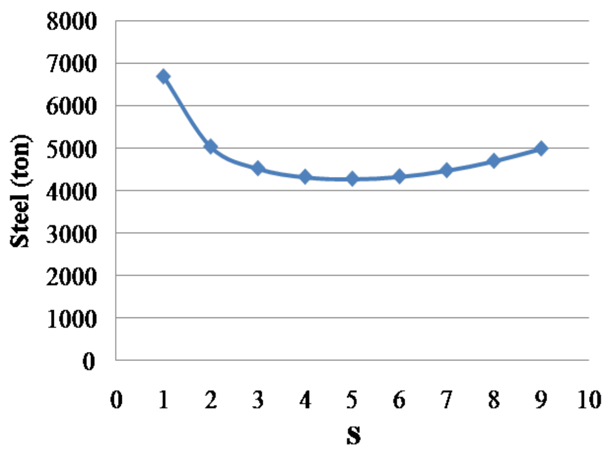


Figure 4.12: Diagrid Steel Mass for 80-Storey Structure (67.4 degrees)

4.4 Results and Discussion

Results in terms of structural steel quantity of perimeter diagonal are compared for various buildings to find the optimal angle.

Table.4.2 shows the comparison of structural steel usage for diagonals of the 40, 50, 60, 70 and 80 storey diagrid structures. It is observed that lowest structural steel required of 40, 50, 60, 70 and 80 storey diagrid structural system is 330.6 ton, 696.6 ton, and 1371.4 ton, 2526.7 ton and 4278.2 ton for 67.4 degrees respectively as shown in Fig.4.13. Thus, considering structural steel usage the optimal angle of 40, 50, 60, 70 and 80 storey diagrid structural system is 67.4 degrees.

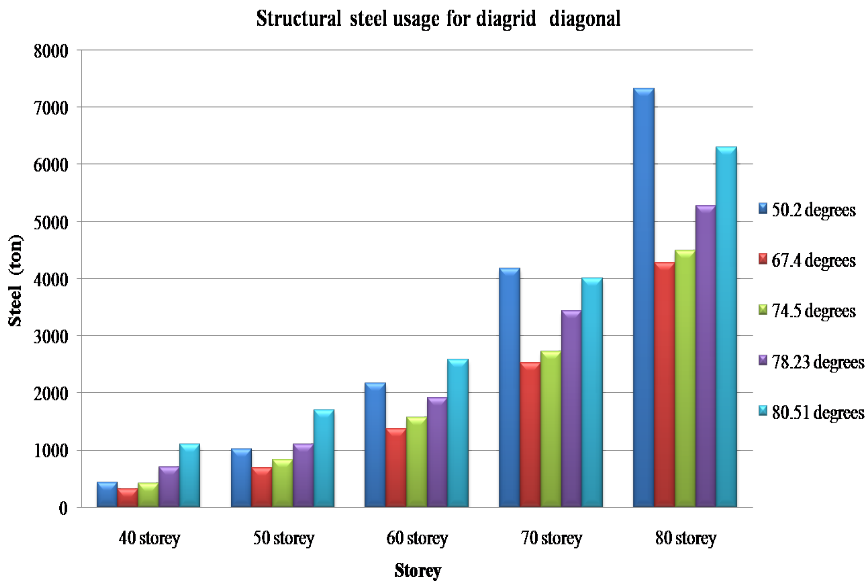


Figure 4.13: Diagrid Diagonal Steel Mass of 40, 50, 60, 70 and 80 Storey Model

Table 4.2: Diagrid Diagonal Steel Mass (Ton)

Storey	**Degrees**				
	50.2	67.4	74.5	78.23	80.51
40	679.8 (S=1)	365.9 (S=1)	**426.9** **(S=1)**	**708.8** **(S=1)**	**1102.5** **(S=1)**
	441.3 **(S=5)**	**330.6** **(S=2)**	616.0 (S=2)	1063.2 (S=2)	1653.8 (S=2)
	531.2 (S=9)	404.2 (S=3)	821.4 (S=3)	1417.6 (S=3)	2205.1 (S=3)
50	1691.9 (S=1)	875.1 (S=1)	**843.1** **(S=1)**	**1102.2** **(S=1)**	**1707.8** **(S=1)**
	1024.1 **(S=7)**	**696.4** **(S=2)**	960.5 (S=2)	1638.7 (S=2)	2561.7 (S=2)
	1157.9 (S=12)	802.3 (S=4)	1280.7 (S=3)	2184.9 (S=3)	3415.6 (S=3)
60	3758.2 (S=1)	1909.3 (S=1)	1731.7 (S=1)	**1919.1** **(S=1)**	**2589.0** **(S=1)**
	2176.5 **(S=9)**	**1371.4** **(S=3)**	**1579.2** **(S=2)**	2486.7 (S=2)	3883.6 (S=2)
	2229.9 (S=12)	1707.3 (S=6)	1955.8 (S=3)	3315.7 (S=3)	5178.1 (S=3)
70	7444.8 (S=1)	3763.1 (S=1)	3329.9 (S=1)	**3436.8** **(S=1)**	**4018.7** **(S=1)**
	4188.3 **(S=11)**	**2526.7** **(S=4)**	**2737.0** **(S=2)**	3621.0 (S=2)	5532.4 (S=2)
	4193.9 (S=12)	3191.6 (S=8)	2940.6 (S=3)	4805.0 (S=3)	7376.6 (S=3)
80	13307.4 (S=1)	6686.3 (S=1)	5818.9 (S=1)	5826.0 (S=1)	**6309.6** **(S=1)**
	7330.3 **(S=13)**	**4278.2** **(S=5)**	**4495.1** **(S=3)**	**5284.9** **(S=2)**	7483.5 (S=2)
	7362.3 (S=16)	4989.9 (S=9)	4943.6 (S=4)	6515.0 (S=3)	9978.1 (S=3)

For the diagrid members steel pipes of varying diameters from 825 mm to 375 mm are used along the height of the building. Their thickness varies from 50 mm to 12 mm. The sizes of typical diagonal members of 40 to 80 storey diagrid structure are shown in Table.4.3.

Table 4.3: Size of Typical Members of Structural System

Storey	Diagonals Section Size
40	375 mm Pipe sections with 12 mm thickness
50	375 mm Pipe sections with 12 mm thickness to 450 mm Pipe sections with 20 mm thickness
60	375 mm Pipe sections with 12 mm thickness to 750 mm Pipe sections with 25 mm thickness
70	375 mm Pipe sections with 12 mm thickness to 825 mm Pipe sections with 25 mm thickness
80	375 mm Pipe sections with 12 mm thickness to 825 mm Pipe sections with 50 mm thickness

4.5 Summary

In this chapter, optimal angle of inclined columns in diagrid structural system is evaluated using stiffness based methodology. For this purpose 40, 50, 60, 70 and 80 storey tall buildings are considered with different angle of inclination of diagonal column on periphery like 50.2, 67.4, 74.5, 78.2, and 80.5 degrees with 2, 4, 6, 8, and 10 storey modules respectively.

It is observed that the optimal angle of 40, 50, 60, 70 and 80 storey diagrid structure ranges between 65 degrees to 75 degrees.

Chapter 5

Analysis and Design of Diagrid Structural System

5.1 General

In this chapter, analysis and design of 36 storey diagrid structural system is presented. Modeling, analysis and design of diagrid structure is carried out using ETABS software. All structural members are designed using IS 800:2007 [23] considering all load combinations. The static analysis is carried out considering wind loads and earthquake loads on structures. Dynamic along wind analysis of structure is performed as per IS:875(III)-1987 [24]. Across wind static load is also calculated. Similarly structural design of 50, 60, 70 and 80 storey buildings is carried out. Important analysis results in terms of time period, top storey displacement and interstorey drift are presented for all buildings. Further sizes of diagonal members for all buildings are presented. The effect of sequential loading on design forces of diagrid column is also discussed. Typical design of diagrid connection is illustrated in this chapter.

5.2 Building Configuration

The 36 storey building is having 36 m $\times$ 36 m square plan with diagonals at slope 74.5 degrees as external braces as shown in Fig.5.1. The plan and side view of the building are shown in Fig.5.1. The pair of inclined columns are provided at six meter spacing along the perimeter.

Design data:

No. of story = 36 nos.

Total height of building= 129.6 m

Plan area of building = 36 m x 36 m

Typ. Floor height= 3.6 m

Diagrid structure with slope of 74.50^0

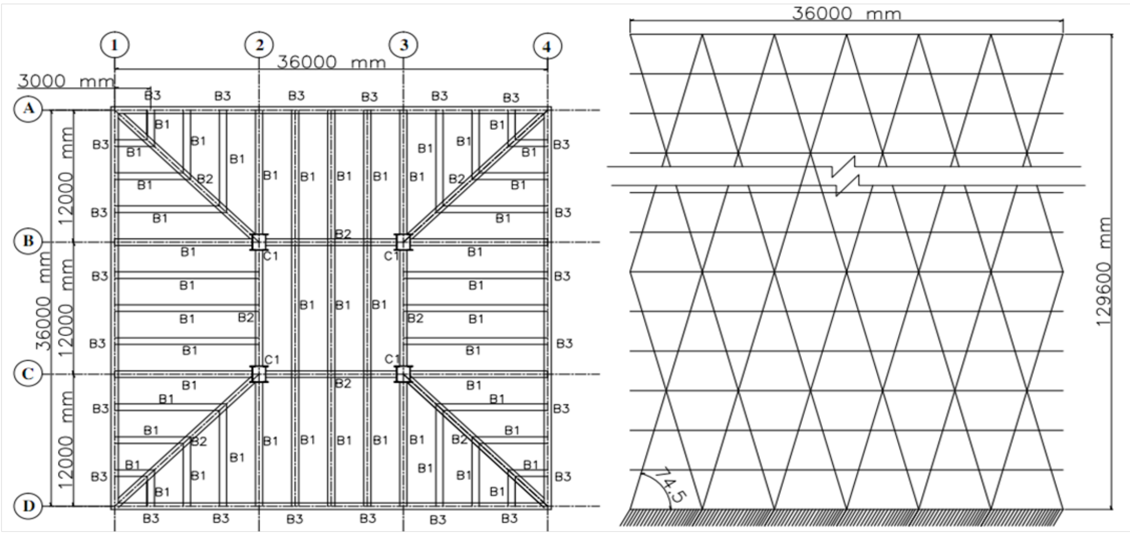


Figure 5.1: Typical Floor Plan and Elevation of 36 Storey Buildings

5.3 Loading Data

Following loadings are considered for the analysis and design of structure;

Dead Load : Dead load of slab 3.75 kN/m^2

Self-weight of the structural members.

Live Load : 2.5 kN/m^2

Wind Load:

Static wind loading is calculated as per IS: 875(III)-1987[24]. Dynamic along wind loading is calculated using gust factor method as per IS: 875(III)-1987[24]. Calculation of across wind equivalent static load is presented in Appendix A.

Basic wind speed : 30 m/sec, Terrain category : III

Class : C

Base shear for static Along wind, dynamic Along wind and Across wind loading is shown in Table.5.1. Dynamic Along wind and across winds are applied simultaneously to the building for analysis.

Table 5.1: Base Shear due to Wind Loading

Loading	Base Shear of Diagrid System (kN)
Static Along Wind Load (IS:875(III)-1987)	3231.26
Dynamic Along Wind Load (IS:875(III)-1987)	3227.60
Across Wind Load	721.87

Fig.5.2 shows the static along wind loading lateral force and storey shear variation along the height of building. Fig.5.3 shows the dynamic along wind loading lateral force and storey shear variation along the height of building, calculated using gust factor method. Across wind storey shear variation along the height of building is shown in Fig.5.4.

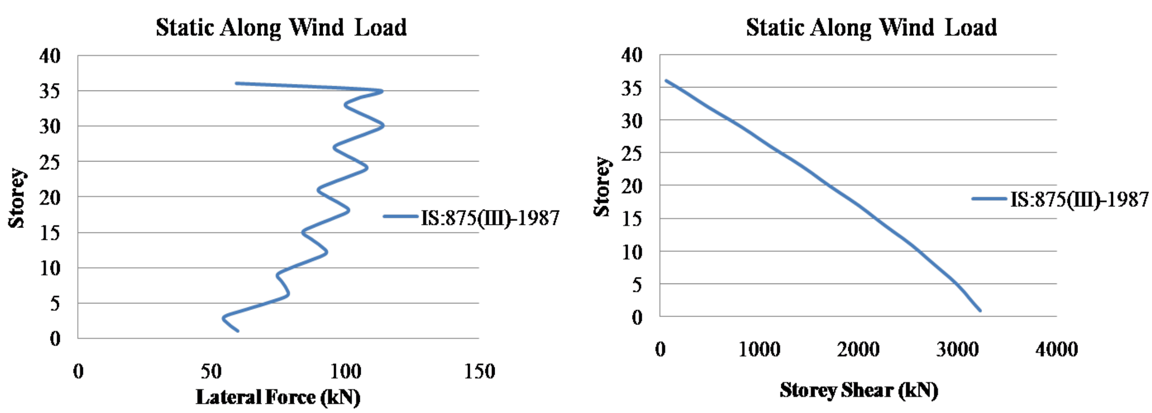


Figure 5.2: Static Along Wind Loading: Lateral Force and Storey Shear

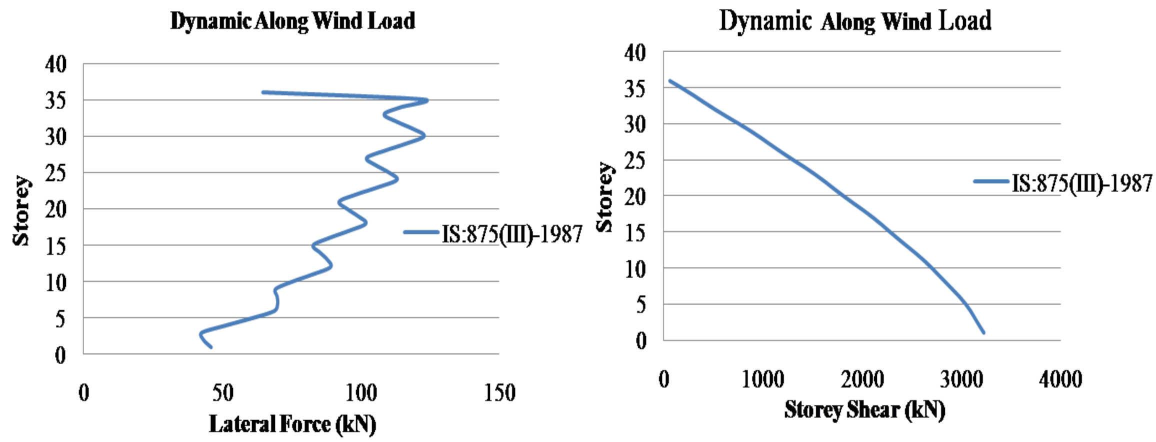


Figure 5.3: Dynamic Along Wind Loading: Lateral Force and Storey Shear

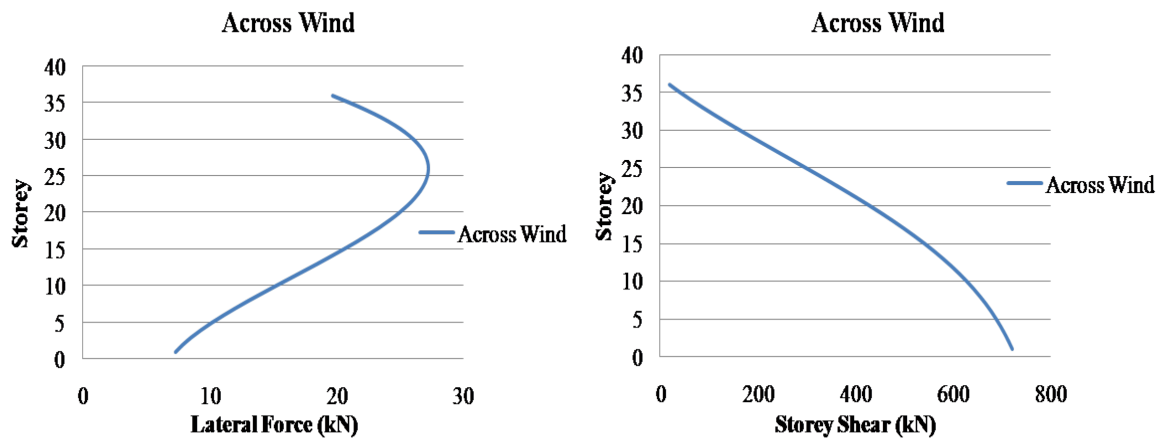


Figure 5.4: Across Wind Loading: Lateral Force and Storey Shear

Earthquake Load:

Location : Ahmedabad

Zone factor : III

Importance factor : 1

Response reduction factor : 5

Soil condition : Medium

The base shear for earthquake load in X-direction and in Y-direction as per IS:1893-2002 [25] are shown in Table.5.2

Table 5.2: Base Shear due to Static Earthquake Loading

Loading	Base Shear of Diagrid System (kN)
X-direction	1814.85
Y-direction	1816.24

Load Combinations: Load combinations are considered as per IS:800-2007 [23] for design of structure. Table.5.3 shows the load combinations for the analysis and design of structure as per IS:800-2007 [23].

Table 5.3: Load Combinations

Sr No.	Limit state of collapse (for Static Loads)	Limit state of serviceability (for Static Loads)
1.	1.5 (DL + LL)	1.0 (DL + LL)
2.	1.2 (DL + LL ± WLx)	1.0 DL + 0.8 LL ± 0.8 WLx
3.	1.2 (DL + LL ± WLy)	1.0 DL + 0.8 LL ± 0.8 WLy
4.	1.5 (DL ± WLx)	1.0 DL ± 1.0 WLx
5.	1.5 (DL ± WLy)	1.0 DL ± 1.0 WLy
6.	0.9 DL ± 1.5 WLx	1.0 DL + 0.8 LL ± 0.8 EQx
7.	0.9 DL ± 1.5 WLy	1.0 DL + 0.8 LL ± 0.8 EQy
8.	1.2 (DL + LL) ± 0.6 WLx	1.0 DL ± 1.0 EQx
9.	1.2 (DL + LL) ± 0.6 WLy	1.0 DL ± 1.0 EQy
10.	1.2 (DL + LL ± EQx)	
11.	1.2 (DL + LL ± EQy)	
12.	1.5 (DL ± EQx)	
13.	1.5 (DL ± EQy)	
14.	0.9 DL ± 1.5 EQx	
15.	0.9 DL ± 1.5 EQy	
16.	1.2 (DL + LL) ± 0.6 EQx	
17.	1.2 (DL + LL) ± 0.6 EQy	

	Limit state of collapse (for Dynamic Loads)	Limit state of serviceability (for Dynamic Loads)
18.	1.2 (DL + LL ± DyWLx)	1.0 DL + 0.8 LL ± 0.8 DyWLx
19.	1.2 (DL + LL ± DyWLy)	1.0 DL + 0.8 LL ± 0.8 DyWLy
20.	1.5 (DL ± DyWLx)	1.0 DL ± 1.0 DyWLx
21.	1.5 (DL ± DyWLy)	1.0 DL ± 1.0 DyWLy
22.	0.9 DL ± 1.5 DyWLx	
23.	0.9 DL ± 1.5 DyWLy	
24.	1.2 (DL + LL) ± 0.6 DyWLx	
25.	1.2 (DL + LL) ± 0.6 DyWLy	

5.4 Modelling and Analysis

In this section, modelling and analysis of diagrid structure are discussed. For analysis the beams and columns are modeled by beam elements and diagonal columns or braces are modeled by truss elements. Analysis results of 36 storey building are presented.

The steps followed for modelling and analysis of structure is as follows:

- Prepare the three dimensional structural model as shown in Fig 5.5 .
- Assign the support conditions. The building model, shown in Fig 5.5, is supported by hinged supports.
- Apply the loads to the structure as mentioned in the section 5.3.
- Assign the section properties to the elements.
- Analyze the Model.
- From analysis results structural elements are designed and if required, sections are modified and analysis is carried out again.

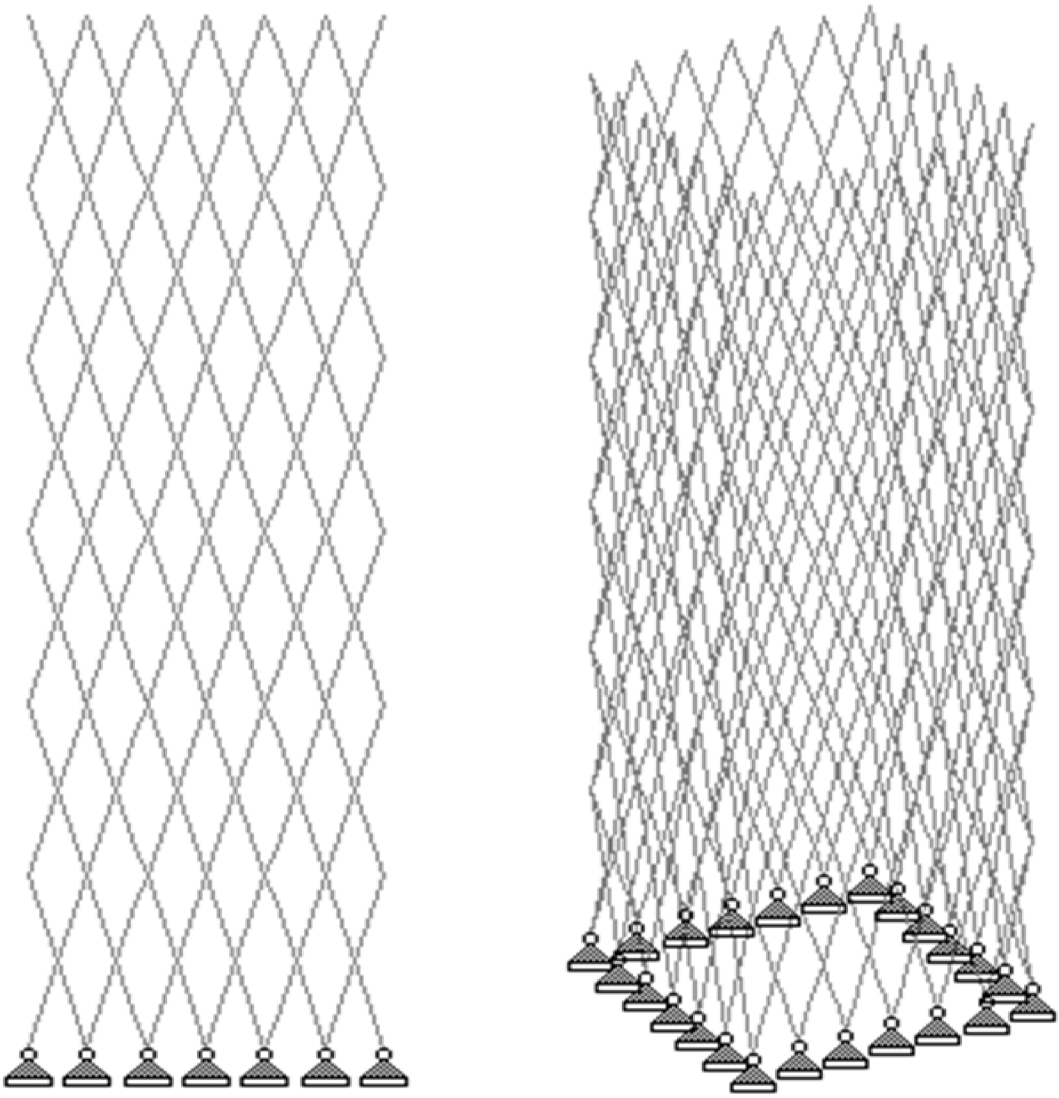


Figure 5.5: 36 Storey Diagrid Building Model (74.5^0)

5.4.1 Analysis Results of 36 Storey Building

The analysis results in terms of Time period, Storey shear, Displacement, Inter-storey Drift are presented.

Time Period:

The time period of 36 storey diagrid structure is shown in Fig 5.6. The first mode time period of diagrid system is 3.14 seconds.

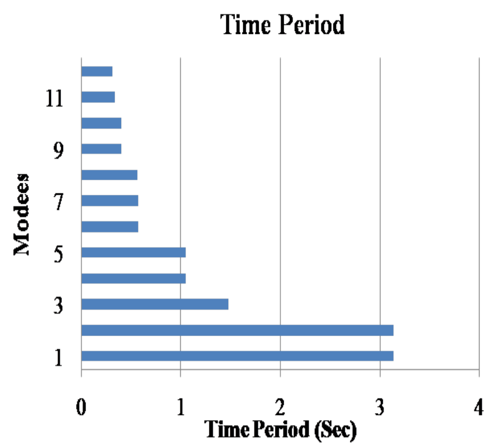


Figure 5.6: Time Period of 36 Storey Structural System

Storey Shear:

The storey shear of 36 storey diagrid structure is shown in Fig 5.7. It is observed that storey shear in x-direction and y-direction due to dynamic wind load is higher compared to earthquake load.

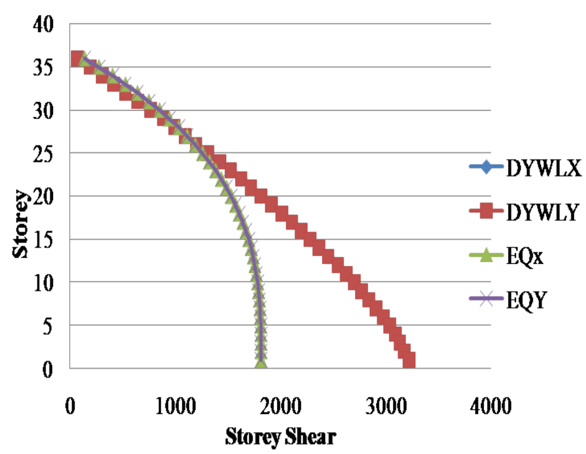


Figure 5.7: Storey Shear of 36 Storey Structural System

Displacement:

The displacement of 36 storey diagrid structure is shown in Fig 5.8. It is observed that displacement in x-direction and y-direction due to dynamic wind load is higher compared to earthquake load.

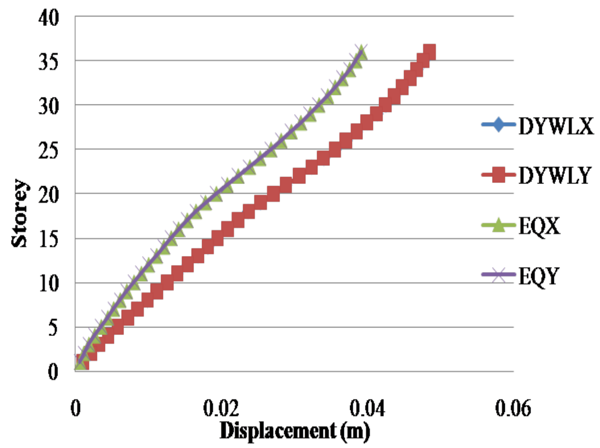


Figure 5.8: Displacement of 36 Storey Structural System

Inter-Storey Drift:

The inter-storey drift of 36 storey diagrid structure is shown in Fig 5.9. It is observed that inter-storey drift in x-direction and y-direction due to dynamic wind load is higher compared to earthquake load.

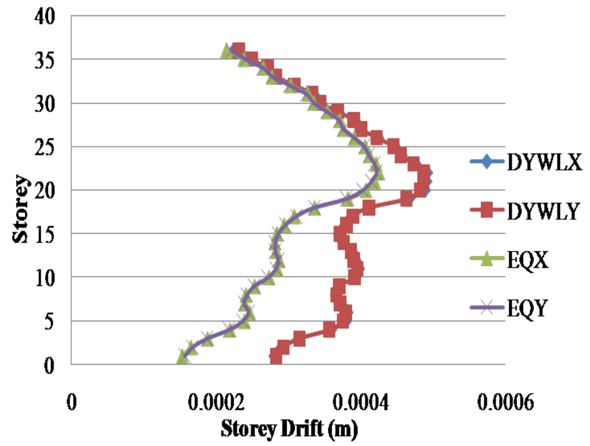


Figure 5.9: Inter-Storey Drift of 36 Storey Structural System

5.4.2 Load Distribution in Diagrid System

There are mainly two type of loading act on the building i.e. gravity load and seismic or wind load. The base shear of wind load is higher compare to earthquake load.

Thus, wind load is governing for the design of structure.

Table.5.4 shows the gravity load and lateral load distribution in exterior frame and interior frame. It is observed that, 97.68 % and 2.31 % lateral load is resisted by exterior and interior frame respectively. While 51.62 % and 48.38 % gravity load is resisted by exterior and interior frame respectively. Thus, lateral load is mainly resisted by exterior frame (Diagonal Columns) and gravity load is resisted by both the exterior frame (Diagonal Columns) and interior frame.

Table 5.4: Load Distribution in 36 Storey Diagrid System

Loading	Total load (kN) on Diagrid System	Loading on Perimeter Diagonals (kN)	Loading on Interior Columns (kN)
Gravity Load	3,51,264.24 kN	1,81,332.12 kN	1,69,932.12 kN
Lateral Load	3227.60 kN	3152.92 kN	74.68 kN

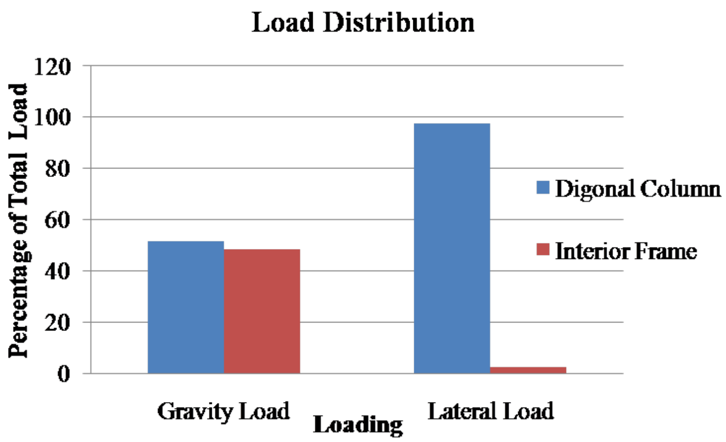


Figure 5.10: Load Distribution in Exterior and Interior Frame

From the results, it can be observed that interior frame is mainly resisting gravity loading.

5.5 Design of 36 Storey Building

In this section, design of composite slab, beam and column are illustrated. Eurocode 4 is used for design of composite slab. IS:800-2007 is used for design of beams and columns. Deck slab, beams and columns are the part of the interior frame. Thus, they are designed only for gravity load.

5.5.1 Composite Slab

The slab is designed as a composite metal deck. As there is no Indian standard covering profiled decking, Eurocode 4 (EC4) is refereed for guidance. The following are the design steps of the metal deck.

1. Select the deck profile and list the decking sheet data (preferably from manufacturer's data).

2. Find out the loading acting on the slab.

3. Design the profiled sheeting as shuttering.
   If, profiled deck sheet is propped during construction then, effective span is calculated using the formula. (This rule is taken from BS 5950:Part 4 as there is no provision in Eurocode).

   - Calculate the effective length l_e of the span
     $l_e = (l - B + d_{ap})/2$
     where,
     l = Actual span of the composite floor
     B = Width of top flanges of the steel beams
     d_{ap} = The depth of the sheeting

   - Calculate factored moments and shear acting of profiled sheeting

   - Check for moment
     Moment capacity (M_{pa}/γ_{ap}) > factored moment.
     where,

M_{pa} = plastic moment of resistance of sheet

γ_{ap} = partial safety factor

- Check for vertical shear

  Shear capacity $(V_{pa}/\gamma_{ap}) >$ factored shear.

- Check for deflection

4. Design of composite slab

- Calculate the effective length l_e of the span.

  Effective length = Center to center between supports or clear distance between supports + effective depth of slab whichever is less.

- Calculate factored moments and shear.

- Check for moment

  Bending resistance $M_{p.Rd} = N_{cf}(d_p - 0.42x) >$ factored moment

  where,

  $N_{cf} = A_p f_{yp}/\gamma_{ap}$

  A_p = Effective area per meter width

  f_{yp} = yield strength of steel

  d_p = effective depth of slab

  x = depth of neutral axis = $N_{cf}/b(0.36 f_{ck})$

- Check for vertical shear

  Shear resistance $V_{v.Rd} = (b_0/b) d_p \tau_{Rd} k_v (1.2 + 40\rho) >$ factored shear

  Where,

  b = one typical wavelength of profiled sheeting

  b_0 = average width of trough

  τ_{Rd} = basic shear strength of concrete

  $k_v = (1.6 - d_p) \geq 1$

  $\rho = A_p/(b_0 d_p)$

  A_p = effective area of sheeting within width b_0

- Check for longitudinal shear

Longitudinal shear is checked by m-k method. The m-k method gives the vertical shear resistance $V_{i.Rd}$ as

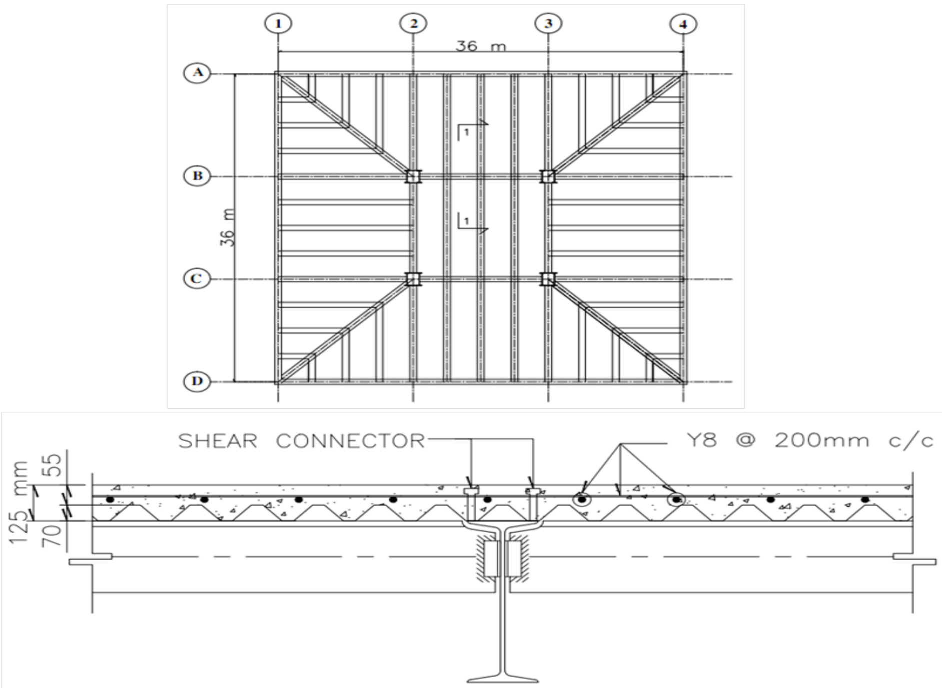
$V_{i.Rd} = bd_p \left[\frac{mA_p}{bl_s} + k\right] / \gamma_{vs} >$ factored shear

m and k values are generally provided by manufacturers of profiled sheet. This values are based on experimental results.

- Check for deflection

The design of composite metal deck slab

The cross section of composite metal deck is shown in Fig.5.11



Section 1-1

Figure 5.11: Composite Slab Cross-Section

Span of deck	:	3.0	m
γ_{ap}	:	1.15	
Concrete grade	:	25	MP_a
Total depth of composite slab	:	125	mm

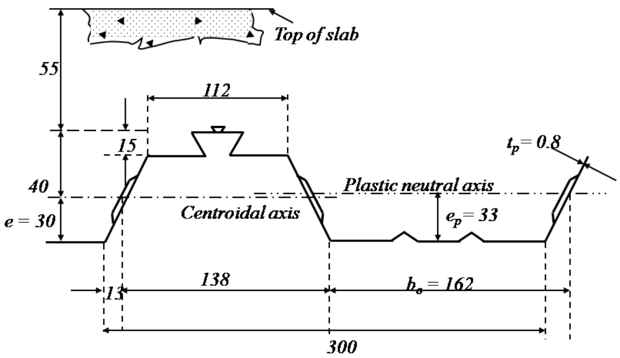


Figure 5.12: Cross-section of Metal Deck Sheeting

Decking Sheet Data:

Yield strength of steel	f_{yp}	:	280	N/mm^2
Design thickness of sheet	t_p	:	0.8	mm
Effective area of cross section	A_p	:	1185	mm^2/m
Moment of inertia	I_p	:	560000	mm^4/m
Plastic moment of resistance	M_{pa}	:	4.92	kNm/m
Distance of centroid above base	e	:	30	mm
Distance of plastic neutral axis above base	e_p	:	33	mm
Resistance to vertical shear	V_{pa}	:	49.2	kN/m
For resistance to longitudinal shear	m	:	184	N/mm^2
	k	:	0.053	N/mm^2
Modulus of elasticity of steel	Ea	:	200000	N/mm^2
Depth of the sheeting		:	70	mm

Load Data	Loads	Factored loads
	kN/m^2	kN/m^2
Dead load of slab	3.75	3.75×1.35=5.0625
Imposed load	2.5	2.5×1.5=3.375
Construction load	1.5	1.5×1.5=2.25

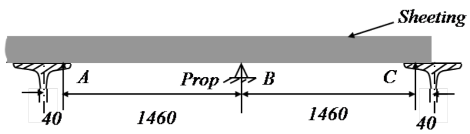


Figure 5.13: Profiled Sheeting during Construction

Profiled steel sheeting as shuttering

1. Effective length of the span

The profiled deck sheet is propped at the center

Assume top flange of supporting steel beams	:	0.15	m
Effective length l_e	:	1460	mm

2. Factored moments and vertical shears

Assume end supports are unrestrained

Moments

Sagging moment	:	1.379	kNm/m
Hogging moment	:	1.612	kNm/m

Shear

Shear force at A	:	5.388	kN/m
Shear force at B	:	6.406	kN/m

3. Check for moment

Design moment = M_{pa}/γ_{ap}	:	4.278	kNm/m

Design moment > 1.612 kNm/m

$\therefore$ profiled deck is safe in flexure at construction stage

4. Check for shear

Design shear = V_{pa}/γ_{ap}	:	42.783	kN/m

Design shear > 6.406 kN/m

$\therefore$ profiled deck is safe in shear at construction stage

5. Check for deflection

Design load at construction stage = DL of slab + construction load	:	5.25	kN/m^2
Max deflection	:	1.15	mm

Deflection is very less $\therefore$ O.K.

Composite Slab

1. Effective span (propping is removed)

Effective length Le	:	2.95	m

2. Flexure and vertical shear:

The design ultimate loading w = (w_u DL + w_u LL)	:	8.813	kN/m^2
Mid span BM M_{sd} = w ${L_e}^2$ / 8	:	9.55	kNm/m
For vertical shear consider e_{ff} span as	:	2.95	m
Vertical shear at support V_{sd}	:	12.98	kN/m

Check for moment:

$N_{cf} = A_p \times f_{yp} \ / \ \gamma_{ap}$	:	288.52	kN/m
design com strength of concrete $f_{ck,cu} = 0.36 \times 25$	:	9.00	N/mm^2
X = $N_{cf}/(\text{b} \times f_{ck,cu})$	:	32.06	mm
hc	:	55.00	
$M_{p,Rd} = N_{cf}$ (d_p - 0.42X)	:	23.52	kNm/m

$M_{p,Rd} > M_{sd}$

$\therefore$ Bending resistance is sufficient

Check for vertical shear:

b_o	:	162.00	mm
b	:	300.00	mm
d_p	:	95	mm
Ap effective area of sheeting within width bo	:	174	mm^2
ρ = Ap / (bo×dp)	:	0.009	
kv = (1.6-dp)	:	1.505	m
Basic shear strength of concrete τ_{RD}	:	0.30	N/mm^2
V_{rd} = (bo/b)×dp×τ_{RD}×kv×(1.2+40ρ)	:	36.09	kN/m

Vrd > Vsd

shear resistance is OK

Check for longitudinal shear:

It is checked by 'm-k' method. It gives the vertical shear resistance

γ_{vs}	:	1.25	
Ls = L/4	:	736.25	mm
V_{iRd}	:	26.54	kN

$V_{iRd} > V_{sd} \therefore$ O.K.

Check for serviceability:

Provide 0.4% reinforcement to avoid cracking

$A_{st} = (0.4 \times 1000 \times 55)/100 = 220\ mm^2$

Provide 8 mm dia. @ 200 mm c/c

5.5.2 Beam

Design forces for beams are shown in Table.5.5

Table 5.5: Design forces for Beams

No.	Moment (kN-m)	Shear (kN)	Governing Load Case
B1	524	157	1.5(DL+LL)
B2	2223	584	1.5(DL+LL)
B3	140	115	1.5(DL+LL)

The design of beams are carried out as per IS: 800-2007[23]. Following are the steps of design of flexural members.

1. Determine the factored loads, bending moments and shear force.

2. Calculate the required plastic section modulus.

$$Z_p = \frac{M \times \gamma_{mo}}{f_y} \tag{5.1}$$

   where,

   γ_{mo} = Partial Safety factor

   f_y = Yield Stress of the material

3. Select suitable section.

4. Classification of section based on b/t_f and d/t_w ratio.(i.e. plastic, compact or semi-compact).

5. Checked for shear. (clause 8.4 of the code)

   Design Strength

$$V_d = \frac{A_v \times f_y}{\sqrt{3} \times \gamma_{mo}} \tag{5.2}$$

   if $V_u < 0.6V_d$

$$M_d = \frac{\beta_b \times Z_p \times f_y}{\gamma_{mo}} \leqslant \frac{1.2 \times Z_e \times f_y}{\gamma_{mo}} \tag{5.3}$$

   if $V_u > 0.6V_d$

   For Plastic and Compact section

$$M_{dv} = M_d - \beta(M_d - M_{fd}) \leqslant \frac{1.2 \times Z_e \times f_y}{\gamma_{mo}} \tag{5.4}$$

   For Semi compact section

$$M_{dv} = \frac{Z_e \times f_y}{\gamma_{mo}} \tag{5.5}$$

   if $M_u > M_d$ or M_{dv} then modify the section.

6. Check for web buckling.

   Web buckling strength $F_{wb} = (b_1 + n_1)t_w f_{cd} > V_u$

   Where,

   b_1 = width of stiff bearing on the flange

   n_1 = dispersion of the load through the web at 45^0

   t_w = web thickness

   f_{cd} = allowable compressive stress

7. Check for web crippling.

   Web crippling strength $F_{wc} = (b_1 + n_2)t_w f_y/\gamma_{mo} > V_u$

   Where,

   $n_2 = 2.5{\times}(t_f + R_1)$

   t_f = flange thickness

   R_1 = root radius

8. Check for deflection.

   Deflection (which is a serviceability limit state) must be calculated on the basis of the unfactored impose loads.

   Allowable maximum deflection, $\delta_{max} = L/300$

The design of flexural member

The design of B1 beam is carried out as shown in Fig 5.14.

Span of Beam	= 12	mm
Design Moment M_u	= 524	kNm
Design Shear V_u	= 157	kN

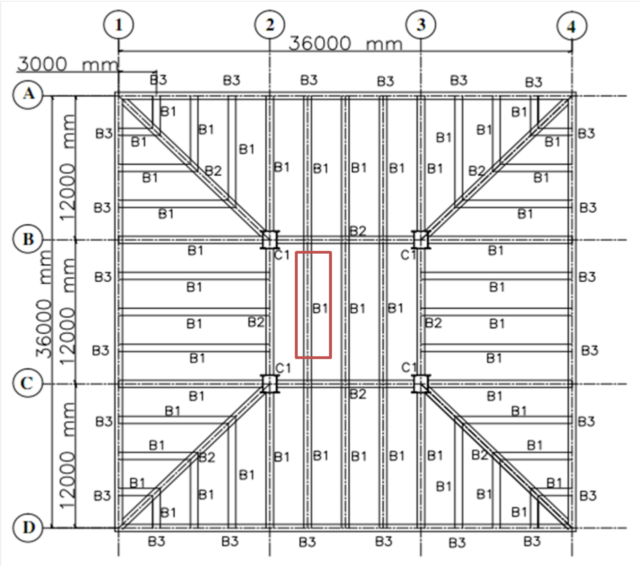


Figure 5.14: Plan of Beam Location

f_y	$= 250$	N/mm^2
γ_{mo}	1.1	
Z_{pz} required	2.30E+06	mm^3
Selected section	ISMB550	
h = 550 mm	I_{ez} = 6.48E+08 mm^4	r_x = 221.6 mm
b_f = 190 mm	I_{ey} = 1.83E+07 mm^4	r_y = 37.3 mm
t_f = 19.3 mm	Z_{ez} = 2.35E+06 mm^3	
t_w = 11.2 mm	Z_{ey} = 1.93E+05 mm^3	
R_1 = 18 mm	Z_{pz}= 2.71E+06 mm^3	
	Z_{py}= 3.64E+05 mm^3	

Section

Classification

ε	1	
b/t_f	4.92	$< 9.4\varepsilon$
d/t_w	42.44	$< 84\varepsilon$

Section is Plastic

Shear check

$V_u \leq V_d$

Shear area	Av	6160 mm^2	clause 8.4.1.1
Design strength	V_d	840 kN	$\geq V_u$
$\therefore$ Shear strength is	O.K.		
(clause 8.2.1.2)			
$0.6 \times V_d$	504	kN	$> V_u$

Moment check

$M_d = \beta_b Z_p f_y / \gamma_{m0}$	$<$	$1.2 Z_e f_y / \gamma_{m0}$	
Design bending strength	M_d	616.35 kN	$<$ 643.58 kN
$M_u \leq M_d$			
$\therefore$ Bending strength is	O.K.		
d/t_w	45.66		67ε

Web buckling check

Assume bearing length b_1	100	mm	
n_1 = h/2	275	mm	
(From table-10) h/b_f	2.89 mm	>1.2	
(From table-10) t_f	19.30 mm	<40	
Class a			
λ	106.11		
(From table-9(a)) f_{cd}	122	N/mm^2	
Buckling resistance = $f_{cd} \times A_b$	512.40	kN	$> V_u$

$\therefore$ Section safe in buckling resistance

Web crippling check

$n_2 = 2.5\times(R_1 + t_f)$	93.25	mm	
$F_w = (b_1+n_2)\times t_w\times f_y/\gamma_{mo}$	491.909	kN	$> V_u$

$\therefore$ Section safe in bearing resistance

Deflection check

Allowable maximum deflection	δ_{max}	$= L/300$	$= 40$ mm
Actual deflection	δ_{actual}	$= 5wl^4/384EI$	$= 39$ mm
δ_{actual}	$<$	δ_{max}	

$\therefore$ Section safe defection

5.5.3 Column

Design forces for Column are shown in Table.5.6

Table 5.6: Design forces for Columns

No.	Axial force P kN	Moment M_z kNm	Moment M_y kNm	Shear force V_z kN	Shear force V_y kN	Governing Load
C1	63700	1290	1120	404	443	1.5(DL+LL)

The design of beam-columns are carried out as per IS: 800-2007[23]. The design of the beam-columns involves a trial-and-error procedure. A trial section is selected by some process and is then checked with the appropriate interaction formulas. Following are the steps of the design of beam-columns.

1. Determine the factored loads and moments acting on the beam-column.

2. Choose an initial section and calculate the necessary section properties.

3. Classify the cross section (Plastic, Compact or Semi-compact) as per clause 3.7 of code.

4. Calculate the bending strength of the cross section about the major and minor axis of the member (as per clause 8.2.1.2 of the code).

5. (1) Determine the shear resistance of the cross section (as per clause 8.4.1 of the code). When the design shear force exceed $0.6V_d$, then the design bending strength must be reduced as given in clause 9.2.2 of the code.

   (2) Check whether shear buckling has to be taken into account (as per clause 8.4.2 of the code).

6. Calculate the reduced plastic flexural strength (as per clause 9.3.1.2 of the code), if the section is plastic or compact.

7. Check the interaction equation for cross-section resistance for biaxial bending (clause 9.3.1.1 of code for plastic and compact section and clause 9.3.1.3 of code for semi-compact section). If not satisfied go to step 2.

8. Calculate the design compressive strength P_{dz} and P_{dy} (clause 7.1.2 of the code) due to axial force.

9. Calculate the design bending strength by lateral-torsional buckling (clause 8.2.2 of the code).

10. Calculate the moment amplification factors (clause 9.3.2.2 of the code).

11. Check with the interaction equation for buckling resistance (clause 9.3.2.2 of the code). If the interaction equation is not satisfied or when it is over design then go to step 2 and revise the section.

The design of column

The design of C1 beam-column is carried out as shown in Fig 5.15.

Axial force	Pu	= 63700 kN	
Moment	Mu	@ Z kNm	@ Y kNm
at Base		= 1290	1120
at top		= 173	173

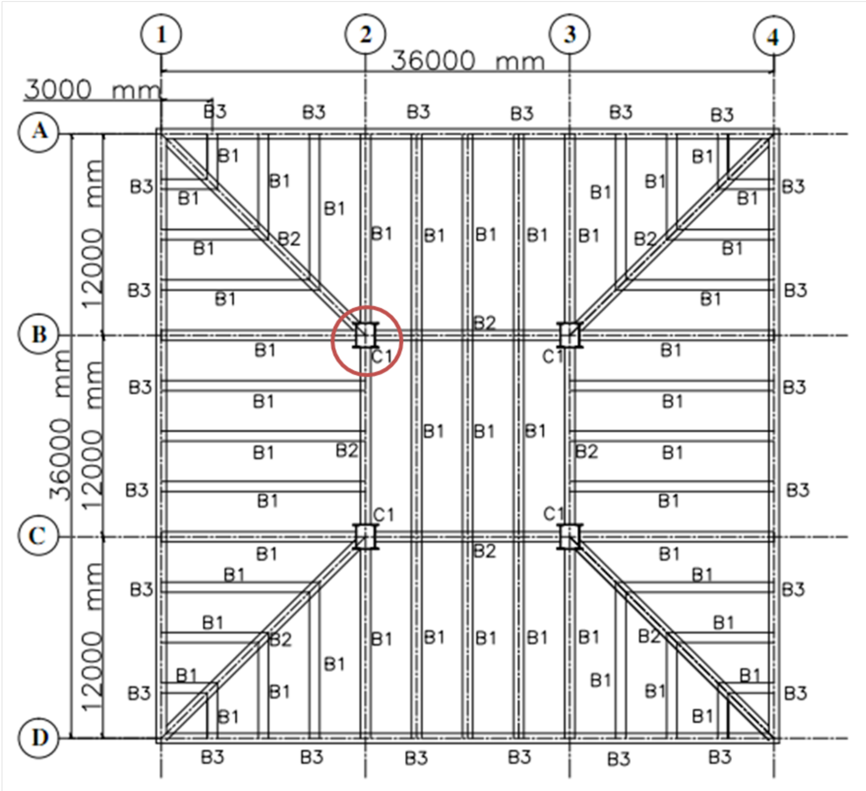


Figure 5.15: Plan of Column Location

Shear	Vu		@ Z kN	@ Y kN
			404	443
Partial safety factor	γ_{mo}	=	1.1	
Length of member	L	=	3.6	m
Yield stress	fy	=	250	Mpa
Modulus of Elasticity	E	=	200000	Mpa

Builtup Section 1500x1500

A = 319600.00 mm^2	$I_{ez} = 1.23\times10^{11}\ mm^4$	$Z_{pz} = 1.87\times10^{8}\ mm^3$
h = 1500 mm	$I_{ey} = 8.02\times10^{10}\ mm^4$	$Z_{py} = 1.75\times10^{8}\ mm^3$
b = 1500 mm	$Z_{ez} = 1.65\times10^{8}\ mm^3$	r_{zz} = 622.40 mm
	$Z_{ey} = 1.07\times10^{8}\ mm^3$	r_{yy} = 501.23 mm

Effective Length @ z, $L_{ez} = K_y$ L	3.60 m		
Effective Length @ y, $L_{ey} = K_z$ L	3.60 m		

1. C/s Classification		**clause 3.7**	
$\epsilon = (250/f_y)^{0.5}$		1	
IS:800-2007 Table 2, page no-18			
Internal element of compression flange	b/t_f	$8.33 < 42$	
d/t_w		$33 < 84$	
∴Cross-section is classified as		Plastic	

2. C/s resistance for yielding		**Clause(9.3.1.2)**	
Factored applied axial force	P	63700	kN
Design strength = $A_g \ f_y/\gamma_{mo}$	N_d	69478.26	kN
$M_d = \beta_b Z_p f_y/\gamma_{mo}$	M_{dz}	42526.82	kNm
Design strength in Bending	M_{dy}	39950	clause 8.2.1.2
$(P/N_d) + (M_y/M_{dy}) + (M_z/M_{dz})$		0.99	$\leqslant 1$
∴ Section is O.K.			

3. Buckling resistance in compression		**clause(7.1.2)**	
$\lambda_z = K_z L/r_z$		5.78	
$\lambda_y = K_y L/r_y$		7.18	
$\lambda_1 = \pi(E/f_y)^{0.5}$		88.81	
Non dimensional effective slenderness ratio			clause 7.1.2.1
$\lambda_z = (\lambda_z/\lambda_1)$		0.0651	
$\lambda_y = (\lambda_y/\lambda_1)$		0.0808	
Imperfection factor	α_z	0.490	
	α_y	0.490	
ϕ-values	ϕ_z	0.469	
	ϕ_y	0.474	

χ-values	χ_z	1.07	
	χ_y	1.06	
Design Compressive strength	$P_d =$	$A_e \times X \times f_y/\gamma_{mo}$	
	$P_{dz} =$	77801.61	$> P_u$
	$P_{dy} =$	77172.77	$> P_u$

IS:800-2007 clause 7.1.2, page no-34

$\therefore$ Section is O.K.

4. Buckling resistance in bending clause(8.2.2)

IS:800-2007 clause 8.2.2.1, page no-54

Elastic lateral buckling moment	M_{cr}	8.658×10^{12}	Nmm
Non dimensional slenderness ratio	λ_{LT}	0.069	
Imperfection parameter	α_{LT}	0.490	
	ϕ_{LT}	0.488	
Bending stress reduction factor	X_{LT}	1.0	
Bending stress reduction factor	f_{bd}	233.73	N/mm^2
Lateral torsional buckling-	M_{dz}	38584.62	kN/m
resistance	M_{dy}	24332.24	kN/m

$\therefore$ Section is O.K.

5. Buckling resistance in bending and axial comp. clause(9.3.2.2)

$\psi_z = 0.134$

$\psi_y = 0.154$

Equi uniform moment factor	C_{mz}	0.654	Table 18
	C_{my}	0.662	Table 18
Equi uniform moment factor for lateral torsional buckling	C_{mLT}	0.654	
$n_y = P/P_{dy}$		0.825	
$n_z = P/P_{dz}$		0.819	

$K_y = 1+(\lambda_y-0.2)n_y \leq (1+0.8n_y)$	0.902	≤ 1.663
$K_z = 1+(\lambda_z-0.2)n_z \leq (1+0.8n_z)$	0.890	≤ 1.658
K_{LT}	0.986	≥ 0.797

Check No 1

$P/P_{dy} + K_y(C_{my}M_y)/M_{dy} + K_{LT}M_z/M_{dz}$ 0.856 ≤ 1

Check No 2

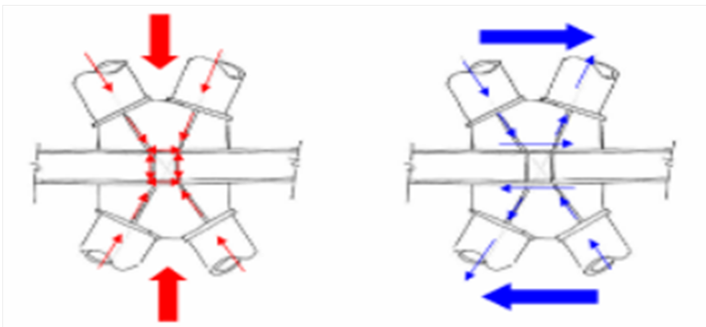
$P/P_{dz} + 0.6K_y(C_{my}M_y)/M_{dy} + K_zC_{mz}M_z/M_{dz}$ 0.837 ≤ 1

$\therefore$ **Section is O.K.**

5.6 Design and Detailing of Connection

5.6.1 General

Generally connection of inclined column in diagrid structures are more complicated than conventional orthogonal connection and tend to be more expensive, required skilled labor and prefabrication of node is essential. In diagrid structures connections, are designed for two main loading type i.e., vertical load and horizontal load are shown in Fig 5.16. The vertical load will be transferred in the form of axial load in the diagonal members above the node to the gusset plate and stiffener then to the diagonal below the nodes as shown Fig 5.16. The horizontal shear will be transferred in the form of axial load in the diagonal members above the node with one in compression and one in tension to the gusset plate and stiffener.



(a) Under Vertical Load (b) Under Horizontal Shear

Figure 5.16: Load Path at Node

Design of connection for 36 storey structure is illustrated for one joint as shown in Fig 5.17.

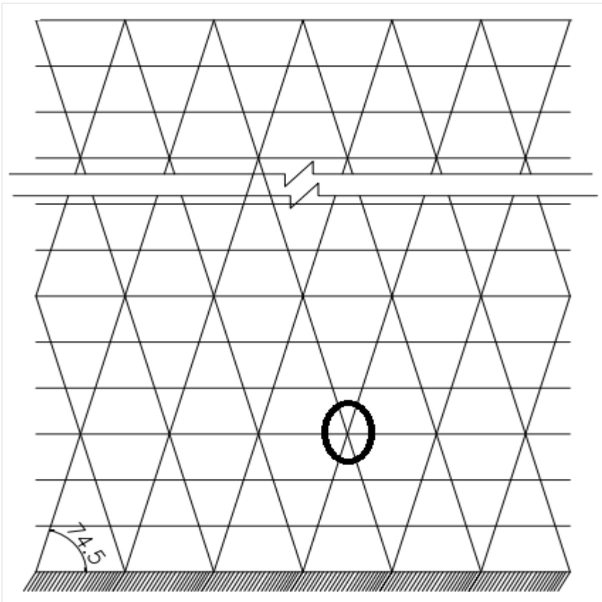
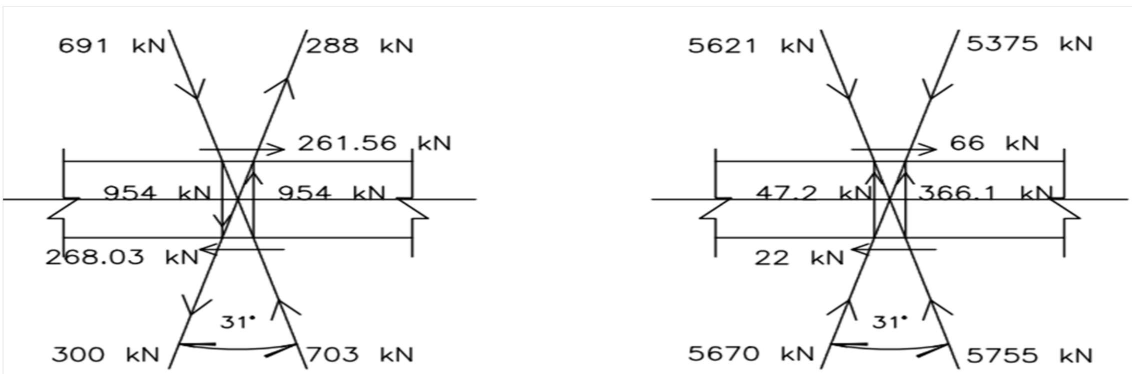


Figure 5.17: Location of Connection

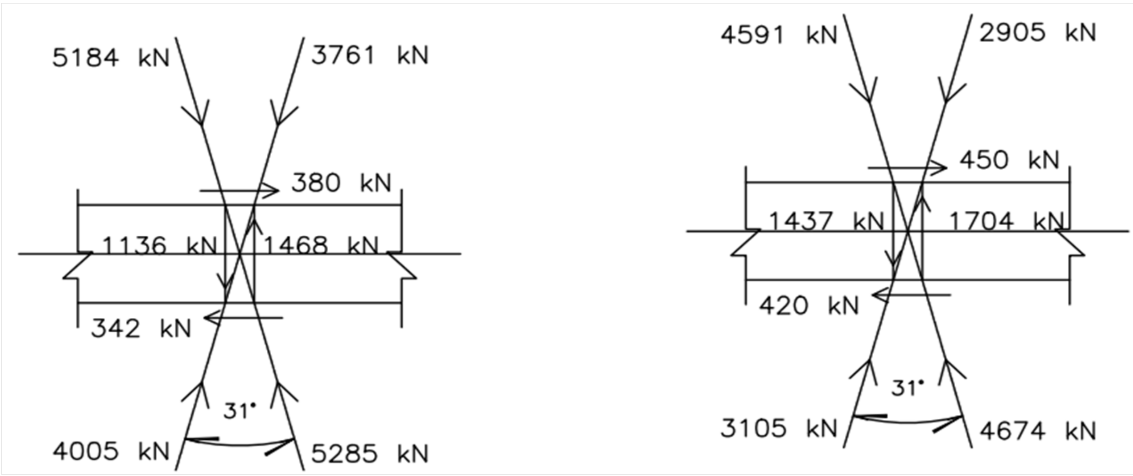
Various load cases are considered for design of nodal connection as shown in Fig 5.18, Fig 5.19 and Fig 5.20. Among the all load cases shown, governing load is considered for nodal connection design.



(a) WL

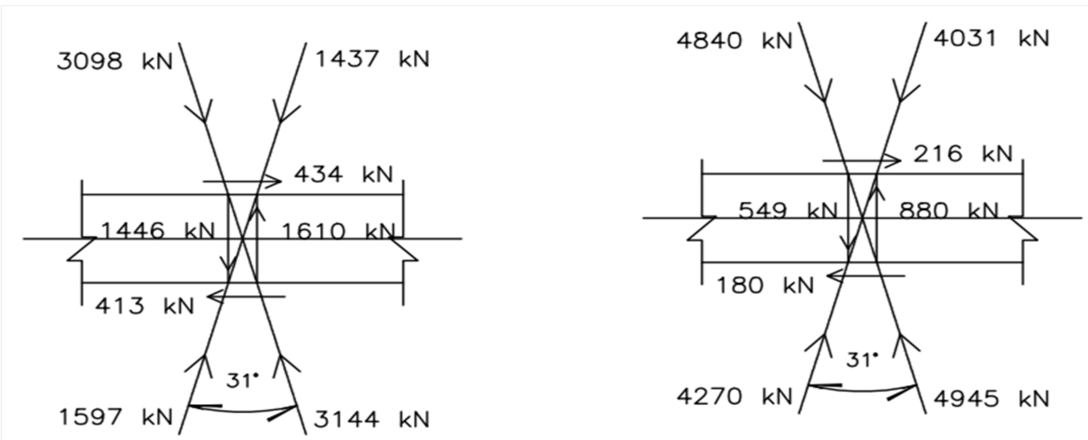
(b) 1.5(DL+LL)

Figure 5.18: Load Considered for Connections (a) WL (b) 1.5(DL+LL)



(c) 1.2(DL+LL+WL)   (d) 1.5(DL+WL)

Figure 5.19: Load combination for Connections (c) 1.2(DL+LL+WL) (d) 1.5(DL+WL)



(e) 0.9 DL+ 1.5 LL   (f) 1.2(DL+LL) + 0.6WL

Figure 5.20: Load combination for Connections (e) 0.9DL+1.5WL (f) 1.2(DL+LL)+0.6WL

5.6.2 Design Calculations

Maximum Axial force in diagonal column = 5750 kN [1.5(DL+LL)]

Design of Bolts between

Diagrid, Gusset and Stiffener

Nominal diameter of the bolt d	=	36 mm (HSFG 8.8 Grade)
Ultimate Tensile Stress F_u	=	800 MPa
Stress Area of Bolt A_{nb}	=	817 mm^2
Tension capacity of bolt	=	$(0.9 \times f_u \times A_{nb})/1.25$
	=	$(0.9 \times 800 \times 817)/1.25$
	=	470 kN
Required numbers of bolts	=	$5750/470 = 12.23 \approx 16$ bolts
Minimum edge distance	=	$1.5 \times d = 54$ mm ≈ 60 mm

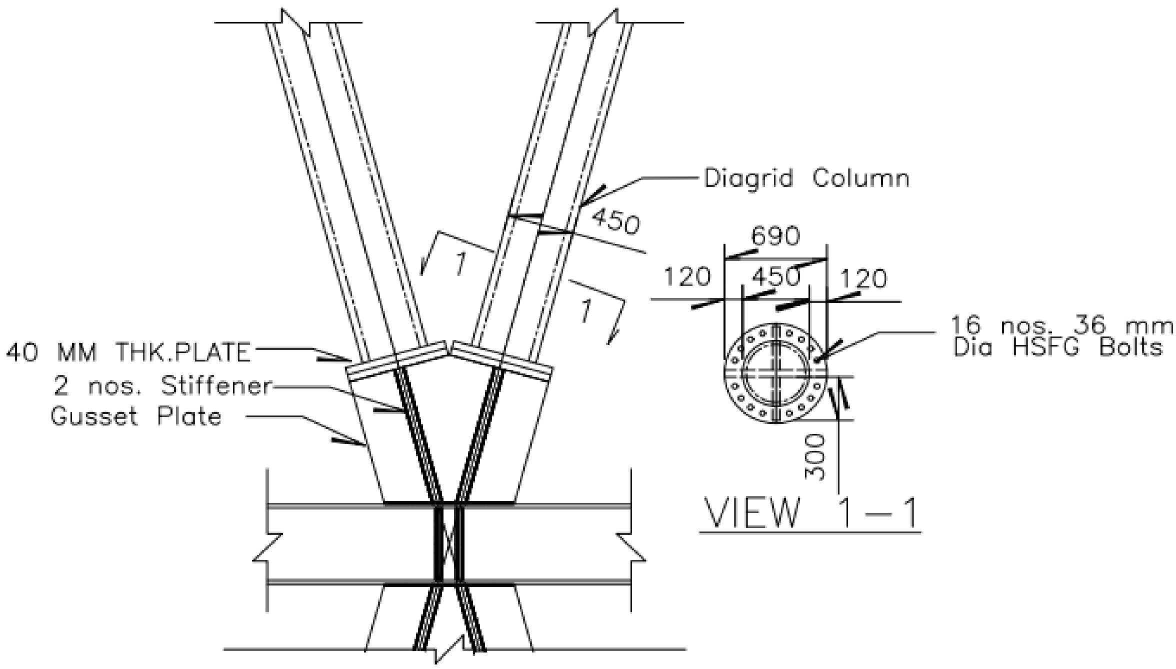


Figure 5.21: Connection between Diagrid, Gusset Plate and Stiffener

Design of Gusset and Stiffener

Capacity of Gusset plate	=	$(A_v \times F_{yw})/(\gamma_{mo} \times \sqrt{3})$
	=	$((950 \times 40) \times 250)/(1.1 \times 1000 \times \sqrt{3})$
	=	4986.20 kN
Load to be resisted by Stiffener	=	5750 - 4986.20 = 763.79 kN
Capacity of Stiffener	=	$(A_v \times F_{yw})/(\gamma_{mo} \times \sqrt{3})$
	=	$((2 \times 300 \times 40) \times 250)/(1.1 \times 1000 \times \sqrt{3})$
	=	3149.18 kN $>$ 763.79 kN

Adopted size of stiffener is fine

Design of Weld between

Gusset Plate and Stiffener

Considering size of weld	=	12 mm
Effective throat thickness	=	$0.7 \times 12 = 8.4mm$
Required length of the weld	=	$(5750 \times 10^3 \times \sqrt{3} \times 1.25)/(410 \times 12 \times 0.7)$
	=	3614.70 mm
Provided length of the weld	=	3800 mm (All four sides)

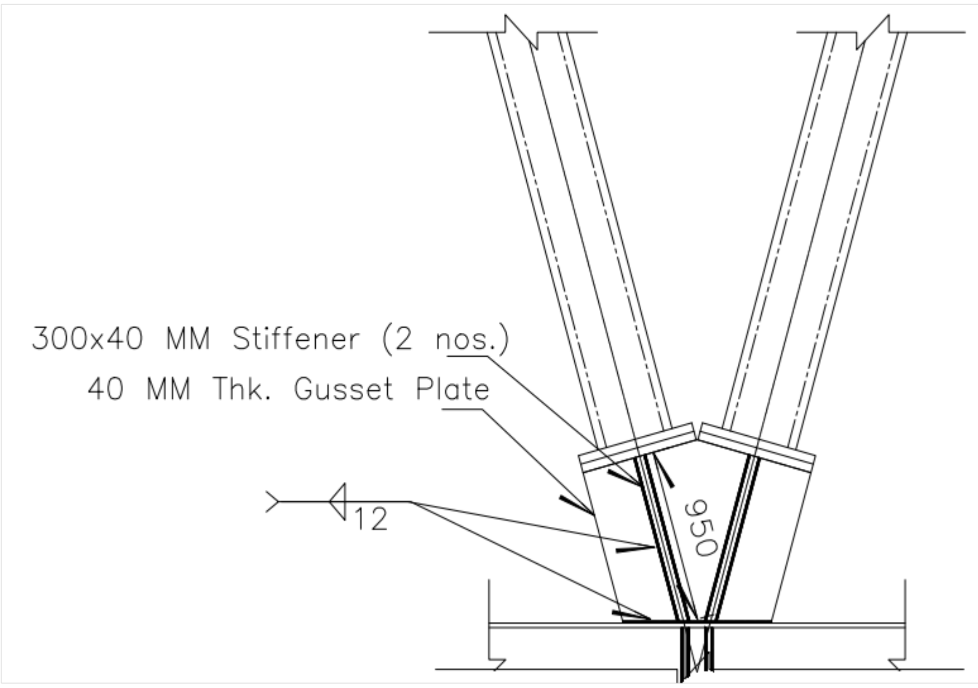


Figure 5.22: Detail of Gusset Plate and Stiffener

Design of Web plate

and Web Stiffener

Maximum Factored Load	=	1704 kN ≈ 1800 kN [1.5(DL+WL)]
Moment	=	$1800 \times 0.110 = 200$ kN.m [1.5(DL+WL)]
Moment Capacity		
$M_d = \beta_b Z_p f_y / \gamma_{m0}$	<	$1.2 Z_e f_y / \gamma_{m0}$
Plastic modules Z_p	=	$2 \times b_f \times t_f \times (D - t_f)/2 + t_w \times d^2/4$
	=	$2 \times 600 \times 25 \times (550 - 25)/2 + 25 \times 500^2/4$
	=	9.43×10^6
M_d	=	$1 \times 9.43 \times 10^6 \times 250/1.1$
M_d	=	2144.88 kN.m > 200 kN.m
Safe to carry the applied moment		
Shear Capacity of Web	=	$(A_v \times F_{yw})/(\gamma_{mo} \times \sqrt{3})$
	=	$((500 \times 25) \times 250)/(1.1 \times 1000 \times \sqrt{3})$
	=	1640.199 kN
Load to be resisted by Stiffener	=	1800 - 1640.19 = 159.80 kN

Provided two stiffener plates of size $300mm \times 25mm$ on either side of the web.

Effective Area	=	$(b_{eff} \times t_{web}) + (2 \times b_s \times t_s)$
	=	$(100 \times 25) + (2 \times 300 \times 25)$
	=	$17500\ mm^2$
Moment of Inertia of Stiffener	=	$(b_s \times (d_s)^3)/12$
	=	$450 \times 10^6 mm^4$

Assuming c/d > 1.5 (clause 8.7.2.4)

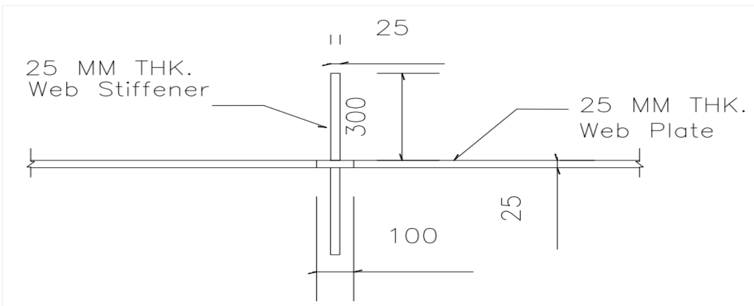


Figure 5.23: Web Plate With Web Stiffener

Minimum I_S	$=$	$0.75d(t_w)^3$
	$=$	$0.75 \times (500) \times 25^3$
	$=$	$5.85 \times 10^6 < 450 \times 10^6 mm^4$

The adopted size of stiffener is appropriate.

Check for Buckling Resistance

I	$=$	$450 \times 10^6 + (2 \times 50 \times 25^3/12)$
	$=$	$450.13mm^4$
A	$=$	$17500\ mm^2$
r	$=$	$\sqrt{\frac{I}{A}} = 507.16mm$
KL	$=$	$0.7 \times 500 = 350$ mm
λ	$=$	KL/r =.609 mm
Table 9(c); f_{cd}	$=$	227 Mpa
Buckling Resistance of Stiffener	$=$	$227 \times 17500/10000$
	$=$	3972.50 kN $>$ 161.60 kN

Stiffener is safe

The final detail of diagonal connection is shown Fig 5.24.

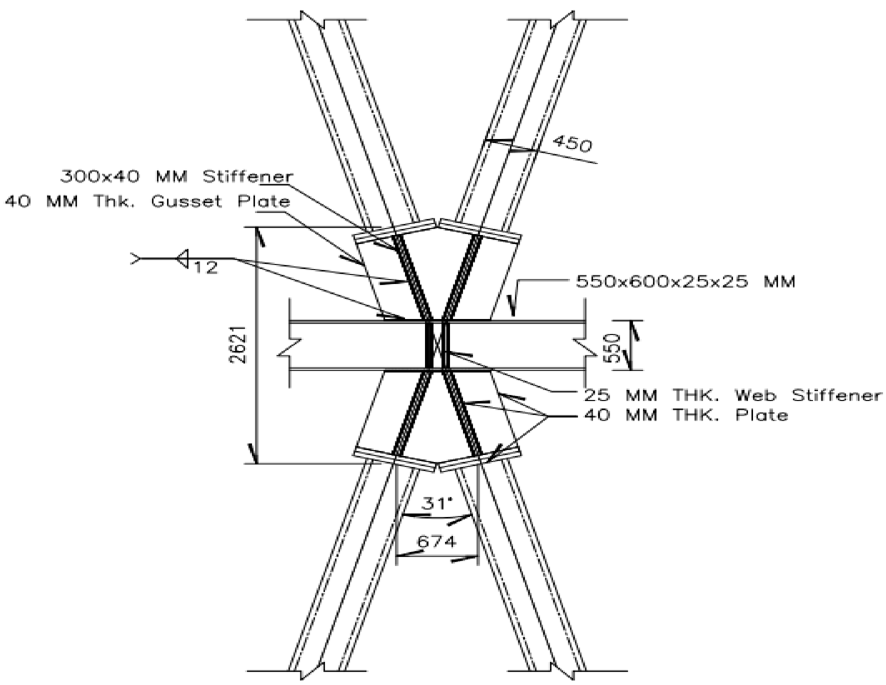


Figure 5.24: Nodal Connection Detail

5.7 Analysis and Design of 50, 60, 70, 80 storey Buildings

As discussed in section 5.2 to 5.6, structural design of 50, 60, 70 and 80 storey buildings is carried out. Analysis results in terms of time period, top storey displacement and drift and member sizes for 50, 60, 70 and 80 storey are presented in this section. Allowable lateral displacement of top storey is considered Height/500.

The first mode time period of 36, 50, 60, 70 and 80 storey diagrid structure are shown in Table 5.7.

Table 5.7: First Mode Time Period of 36, 50, 60, 70 and 80 Storey Diagrid Structure

Storey	First Mode Time Period (sec)
36 storey	3.16
50 storey	4.46
60 storey	5.01
70 storey	5.09
80 storey	5.86

The maximum top displacement of 36, 50, 60, 70 and 80 storey diagrid structure in X-direction and Y-direction due to dynamic wind load are shown in Table 5.8.

Table 5.8: Top Displacement of 36, 50, 60, 70 and 80 Storey Diagrid Structure

Storey	Top Displacement in X-direction due to DYWLX (m)	Top Displacement in Y-direction due to DYWLY (m)
36 storey	0.048	0.048
50 storey	0.102	0.102
60 storey	0.136	0.136
70 storey	0.143	0.143
80 storey	0.192	0.192

Storey displacement along the height of buildings are shown in Fig 5.25 and Fig 5.26 for 50, 60, 70 and 80 storey building respectively.

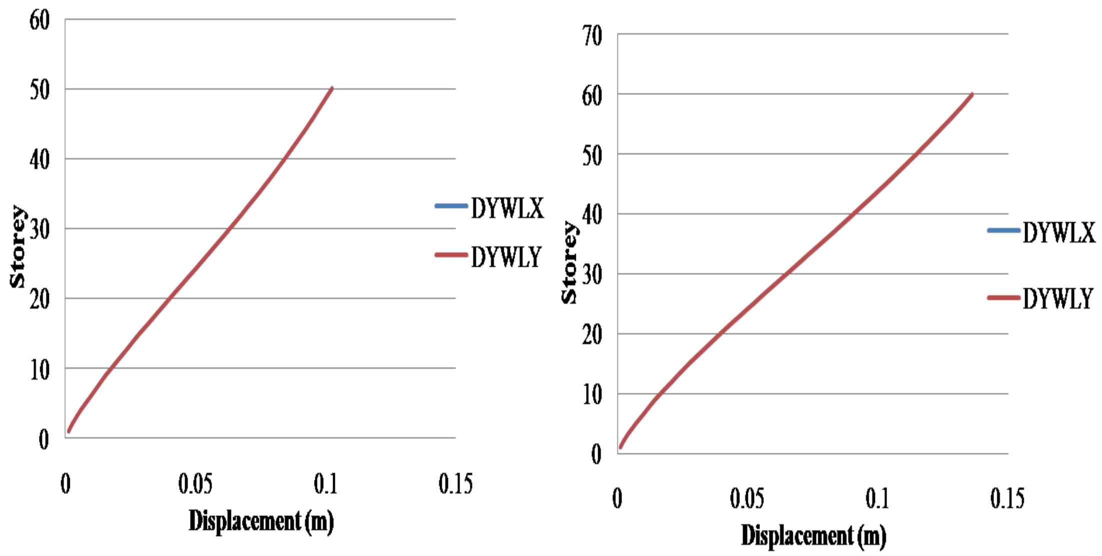


Figure 5.25: Displacement of 50 and 60 Storey Structural System

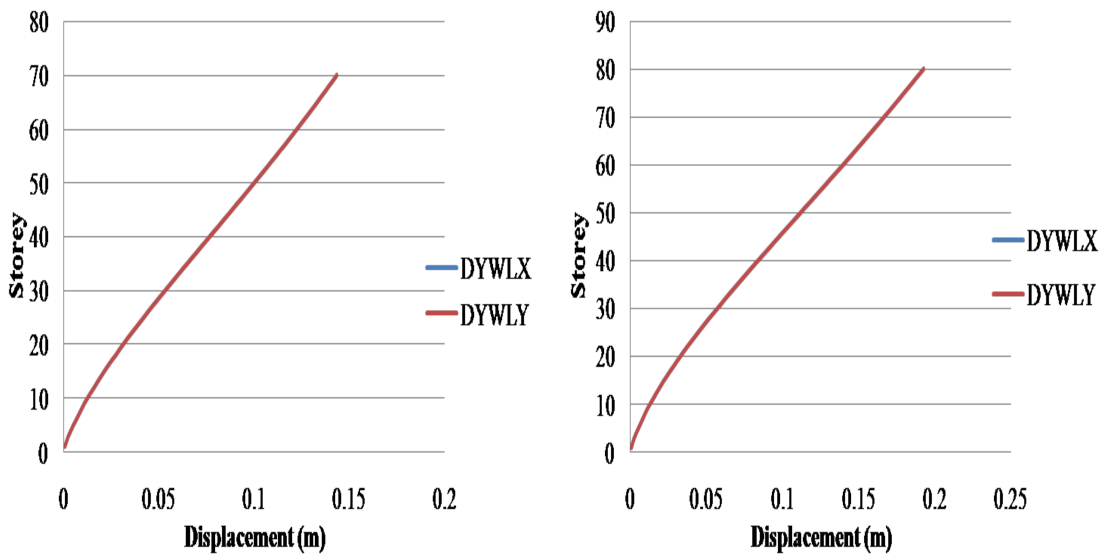


Figure 5.26: Displacement of 70 and 80 Storey Structural System

The maximum inter-Storey drift of 36, 50, 60, 70 and 80 storey diagrid structure in X-direction and Y-direction due to dynamic wind load are shown in Table 5.9.

Table 5.9: Inter-Storey Drift of 36, 50, 60, 70 and 80 Storey Diagrid Structure

Storey	Inter-Storey Drift in X-direction due to DYWLX (m)	Inter-Storey Drift in Y-direction due to DYWLY (m)
36 storey	0.000224	0.000232
50 storey	0.000480	0.000492
60 storey	0.000561	0.000572
70 storey	0.000566	0.000574
80 storey	0.000717	0.000725

The variation of inter-storey drift along the height of 50, 60, 70 and 80 storey buildings are shown in Fig 5.27 and Fig 5.28.

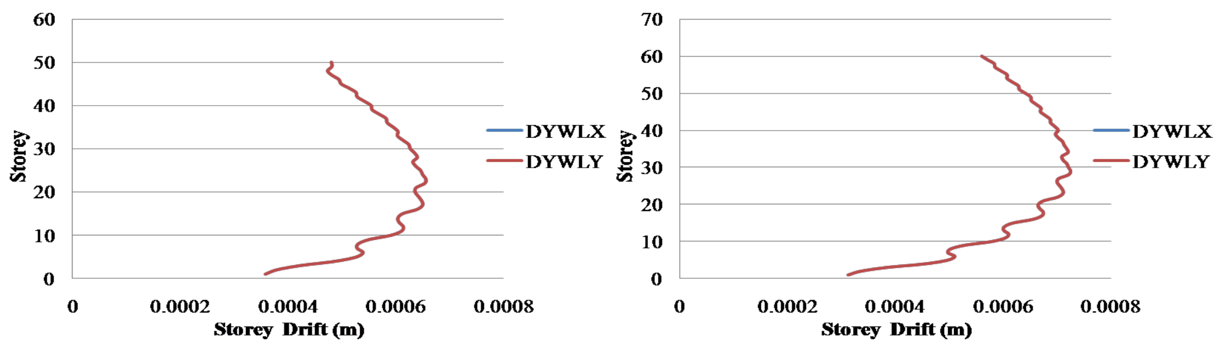


Figure 5.27: Inter-Storey Drift of 50 and 60 Storey Structural System

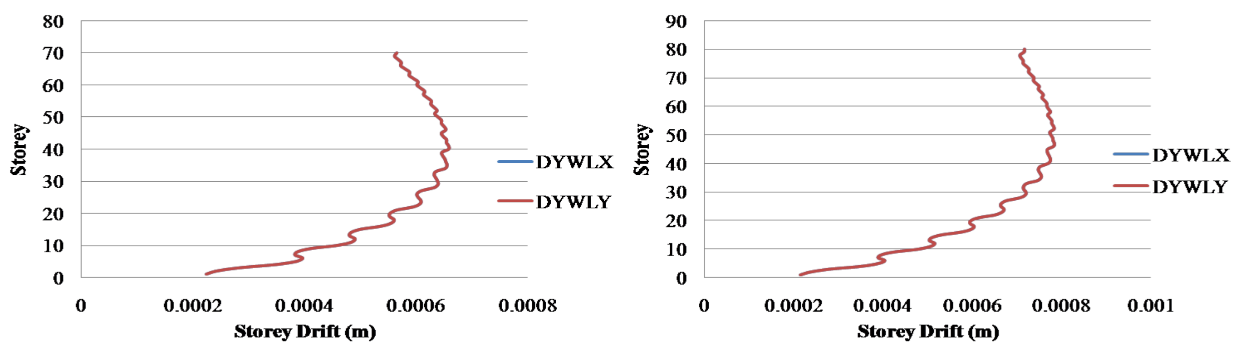


Figure 5.28: Inter-Storey Drift of 70 and 80 Storey Structural System

The sizes of typical members of 36, 50, 60, 70 and 80 Storey diagrid structure are shown in Table.5.10.

Table 5.10: Size of Typical Members of 36, 50, 60, 70 and 80 Storey Diagrid Structure

Storey	Diagonal Columns	Interior Columns
36 storey	375 mm Pipe sections with 12 mm thickness (from 19th to 36th storey) 450 mm Pipe sections with 25 mm thickness (from 1st to 18th storey)	1500 mm X 1500 mm
50 storey	525 mm Pipe sections with 25 mm thickness	1650 mm X 1650 mm
60 storey	750 mm Pipe sections with 25 mm thickness	1800 mm X 1800 mm
70 storey	675 mm Pipe sections with 50 mm thickness	2000 mm X 2000 mm
80 storey	825 mm Pipe sections with 50 mm thickness	2200 mm X 2200 mm

The details of internal column resisting mainly gravity loads is shown in Fig. 5.29 and Fig. 5.30.

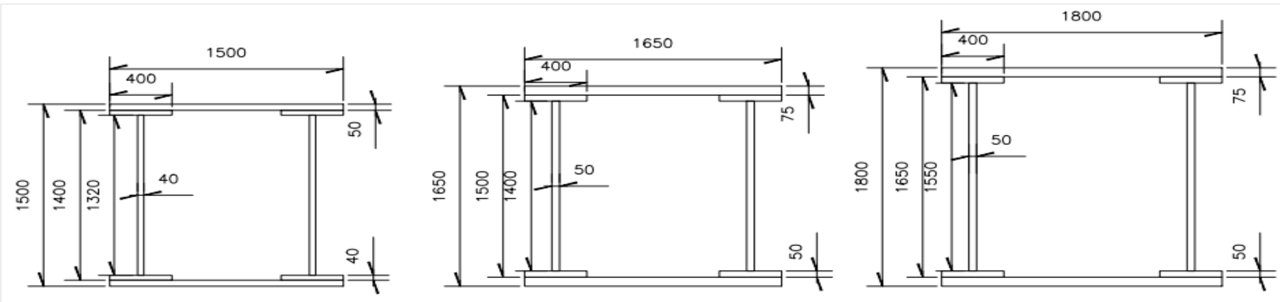


Figure 5.29: Interior Column Section for 36, 50 and 60 Storey

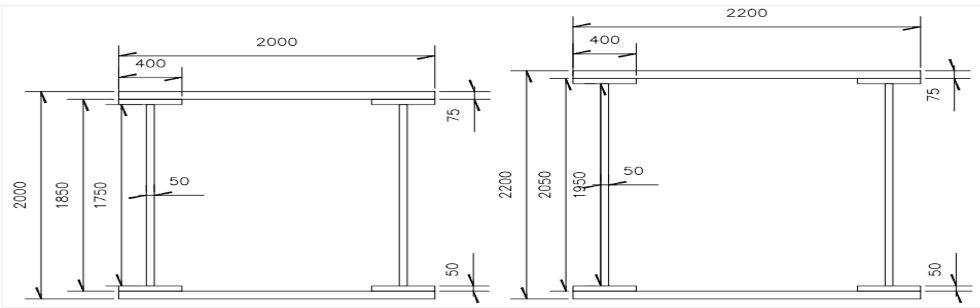


Figure 5.30: Interior Column Section for 70 and 80 Storey

5.8 Effect of Sequential Loading on Analysis Result

Generally, structures are analyzed by conventional one-step method, which assumes that all the design loads are imposed only after the frame has been completed up to its roof level. Generally, sequential loading are not considered for analysis and design purpose of structures. But in actual, loads are applied on structure in stages as the construction of frame proceeds, whereas the remaining part of it is imposed after completion of the frame. In this section, effects of sequential loading on analysis results are presented. Stage-wise construction is shown in Fig 5.31.

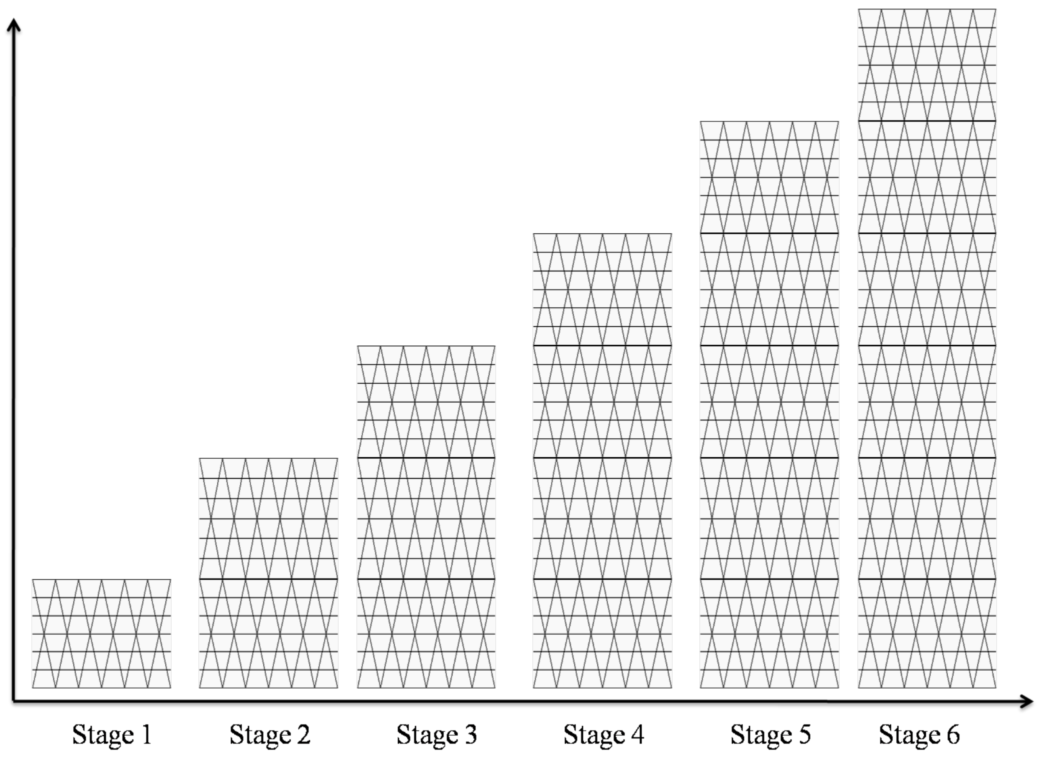


Figure 5.31: Stage-Wise Construction

To understand the importance of sequential loading 36 storey building is considered for analysis. The construction gravity load is applied to the building. The construction gravity load is divided into dead load and live load. The dead load covers mainely the weight of concrete slab. The live load is categorized into sustained and extraordinary live loads. The sustained live load includes the weight of forms/shores, reshores and braces. The extraordinary live load include the construction live load such as the crew and construction equipments. ACI Committee 347 [26] recommends that construction live load during placement should be minimum 2.5 kN/m^2. Structure are analyzed based on two load cases: dead load and combined dead and live load. ETABS software is used for analysis purpose.

The steps for analysis are as follows:

[1] Prepared the modelled.

[2] Assign the support condition.

[3] Assign the section properties/member sizes.

[4] Apply the gravity load to the structure.

[5] Apply the sequential loading for dead load and combined dead and live load. Sequential Loading is assigned as shown in Fig 5.32 and Fig 5.33.

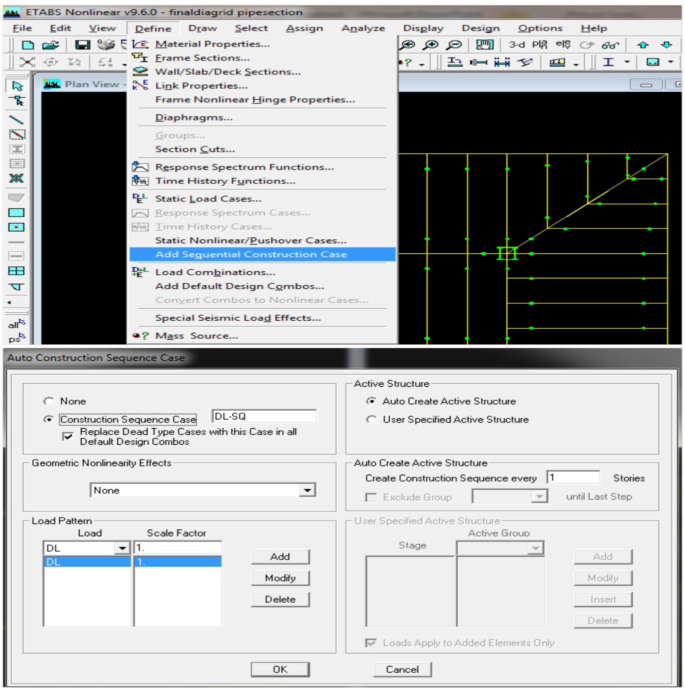


Figure 5.32: Sequential Loading

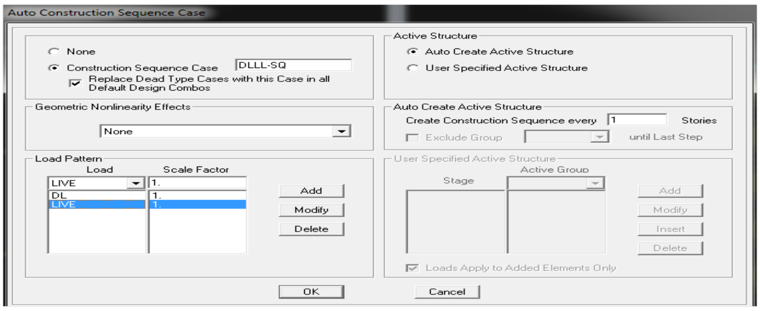


Figure 5.33: Sequential Loading

[6] Analyze by applying Run Analysis Command.

[7] Construction sequence analysis is carried out by applying Run Construction Sequence Analysis Command. Fig. 5.34 shows the Run Construction Sequence Analysis.

[8] Results are taken.

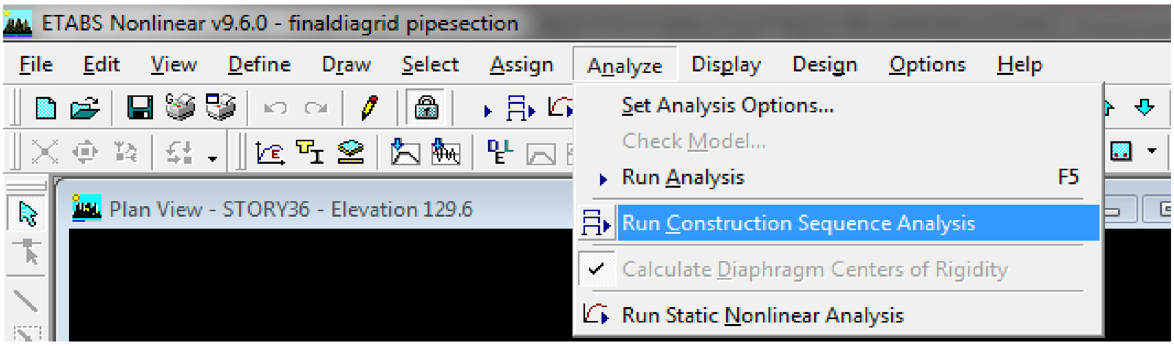


Figure 5.34: Run Construction Sequence Analysis

The diagonal columns are designed as truss elements. Thus, only axial forces comparison is done for loads. Axial forces of diagonal members due to dead load and combined dead and live are obtained. Sequential and one step analysis results due to dead load and combined dead and live of diagonal columns for bottom storey of 36 storey building are shown in Table.5.11. Diagonal column numbers DC1 to DC12 are shown in Fig.5.35

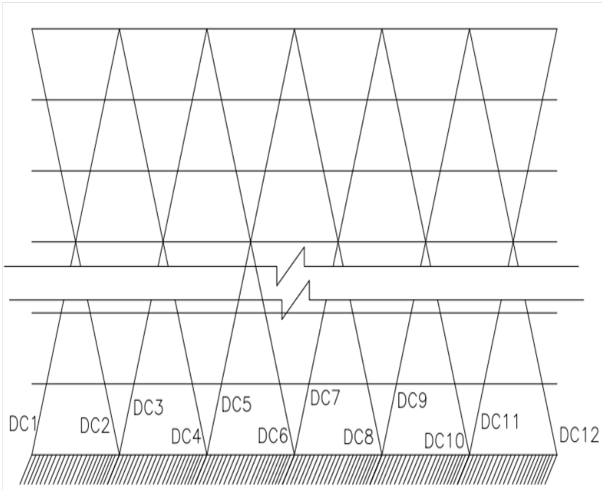


Figure 5.35: Elevation for Diagonal Columns No.

Table 5.11: Sequential and One Step Analysis Results for Diagonal Columns (36 Storey)

Diagonal Columns No.	Ratio of Sequential and One step Analysis for Diagonal Column Axial force (DL)	Ratio of Sequential One step Analysis for Diagonal Column Axial force (DL+LL)
DC1	1.015	1.011
DC2	0.960	0.956
DC3	0.983	0.983
DC4	0.992	0.993
DC5	1.021	1.024
DC6	1.023	1.026
DC7	1.023	1.026
DC8	1.021	1.024
DC9	0.992	0.993
DC10	0.983	0.983
DC11	0.960	0.956
DC12	1.015	1.011

The ratio of axial force in diagonal column due to sequential analysis and one step analysis indicates additional axial force developed in column due to sequential loading. From the results, It is observed that there is 1.0 to 2.5 % increase in axial force under sequential loading is observed when only dead load is considered. While 1.0 to 3.0 % higher axial force in diagonal columns due to sequential loading is observed compared to one step analysis for combined dead and live load. Same procedure is carried out for 60 and 80 storey buildings. For 60 and 80 storey building similar results are observed. Sequential and one step analysis results due to dead load and combined dead and live of diagonal columns for bottom storey of 60 and 80 storey building are shown in Table.5.12 and Table.5.13 respectively. This indicates, in tall building effects of sequential loading need to be considered for analysis and design.

Table 5.12: Sequential and One Step Analysis Results for Diagonal Columns (60 Storey)

Diagonal Columns No.	Ratio of Sequential and One step Analysis for Diagonal Column Axial force (DL)	Ratio of Sequential One step Analysis for Diagonal Column Axial force (DL+LL)
DC1	1.019	1.021
DC2	0.964	0.952
DC3	0.987	0.986
DC4	0.995	0.996
DC5	1.027	1.028
DC6	1.026	1.029
DC7	1.026	1.029
DC8	1.027	1.028
DC9	0.995	0.996
DC10	0.987	0.986
DC11	0.964	0.952
DC12	1.019	1.021

Table 5.13: Sequential and One Step Analysis Results for Diagonal Columns (80 Storey)

Diagonal Columns No.	Ratio of Sequential and One step Analysis for Diagonal Column Axial force (DL)	Ratio of Sequential One step Analysis for Diagonal Column Axial force (DL+LL)
DC1	1.022	1.025
DC2	0.970	0.965
DC3	0.993	0.980
DC4	0.998	0.995
DC5	1.031	1.035
DC6	1.033	1.037
DC7	1.033	1.037
DC8	1.031	1.035
DC9	0.993	0.995
DC10	0.993	0.980
DC11	0.970	0.965
DC12	1.022	1.025

5.9 Summary

In this chapter, complete analysis and design of 36 storey diagrid steel building is presented. A regular floor plan of 36 m x 36 m size is considered. ETABS software is used for analysis and design of structural members. All structural members are designed using IS 800:2007 considering all load combinations. Load distribution in diagrid system is also studied for 36 storey building. Diagrid nodal connection is designed and detailed for 36 storey building. Also, the analysis and design results of 50, 60, 70 and 80 storey diagrid structures are presented. Finally effects of sequential loading on analysis results are presented.

Chapter 6

Summary and Conclusion

6.1 Summary

In present report introduction of diagrid structural system, history of diagrid system, study of important buildings with diagrid structural system, advantages and limitations are discussed.

Behavior of diagrid structure under gravity and lateral loading is presented. Also methodology of preliminary design of diagrid system based on shear and bending stiffness is discussed. Design of 36 storey diagrid steel structural system having external columns at 74.5 slope is illustrated. Comparison of results of approximate analysis considering stiffness based approach and exact analysis using ETABS software is carried out for 36 storey diagrid building. Analysis results in terms of quantity of steel in diagrid members and top storey displacements are compared as obtained by approximate analysis and exact analysis. The influence of factor 'S' (Ratio of bending deformation to shear deformation) on quantity of steel is also explored. Further comparison of diagrid and tubular structural system is carried out to understand the advantages of diagrid structural system.

Optimal angle of inclined columns in diagrid structural system is evaluated using stiffness based methodology. For this purpose 40, 50, 60, 70 and 80 storey tall buildings are considered with different angle of inclination of diagonal column on periphery like 50.2, 67.4, 74.5, 78.2, and 80.5 degrees with 2, 4, 6, 8, and 10 storey modules respectively.

Complete analysis and design of 36 storey diagrid steel building is presented. A regular floor plan of 36 m x 36 m size is considered. ETABS software is used for modeling and analysis of structural members. All structural members are designed as per IS 800:2007 considering all load combinations. Dynamic along wind and across wind are considered for analysis and design of the structure. Load distribution in diagrid system is also studied for 36 storey building. Design and detailing of diagrid nodal connection is illustrated for 36 storey building. Similarly, analysis and design of 50, 60, 70 and 80 storey diagrid structures is carried out. Comparison of analysis results in terms of time period, top storey displacement and inter-storey drift is presented in this report. The design of diagonal column for all buildings is carried out and final dimensions of perimeter column and interior column are presented. Finally effects of sequential loading on analysis results are presented.

6.2 Conclusion

Based on study of diagrid structural system carried out in this project following conclusions are derived:

- From the comparison of results of approximate analysis considering stiffness based approach and exact analysis using ETABS software for 36 storey diagrid building, following observations are made.
  - There is 13.95 % reduction in displacement of diagrid structure in x-direction due to wind load in exact analysis.
  - Based on exact analysis 4.5 % higher steel is required compared to approx-

imate analysis.

- Optimal value of 'S' for design of diagonal members of 36 storeys building is 1.
- Approximate analysis can be used for preliminary design of diagonal column of diagrid structures.

- From the comparison of analysis and design results of 36 storey diagrid and tubular structural system to understand the advantages of diagrid structural system, following observations are made.

  - The time period of this diagrid structural system is 3.34 sec and tubular structural system is 6.65 sec. Thus 49.69 % reduction in time period of diagrid structural system, indicates its higher stiffness compared to tubular structures.
  - There is 71.78 % and 65.65 % reduction in displacement of diagrid system in x-direction due to wind load and earthquake load respectively. Also 60.29 % and 51.91 % reduction in displacement of diagrid system in y-direction is observed due to wind load and earthquake load respectively in comparison of tubular structure.
  - There is 37.50 % and 38.96 % reduction in inter-storey drift of diagrid system in x-direction due to wind load and earthquake load respectively. Also 29.42 % and 26.59 % reduction in inter-storey drift of diagrid system in y-direction is observed due to wind load and earthquake load respectively in comparison of tubular structure.
  - Perimeter "Diagrid" system saves approximately 14 % of a structural steel when compared to a conventional tubular structure.

- The optimal value of 'S' (ratio of the displacement at the top of the structure due to bending and the displacement due to shear) for 40, 50, 60, 70 and 80 storey is 2, 2, 3, 4 and 5 respectively. From the results, it can be seen that, bending mode is governing compared to the shear mode as building become taller.

- It is observed that the optimal angle of 40, 50, 60, 70 and 80 storey diagrid structure ranges between 65 degrees to 75 degrees.

- As diagrid columns resist gravity and lateral loads through axial action, the performance of diagrid structural system is better compared to tubular or framed structure.

- The perimeter diagrid columns are to be modelled as truss member to induce axial forces under gravity and lateral loading.

- Most of the lateral load is resisted by diagrid columns on the periphery, while gravity load is resisted by both the internal columns and peripherial diagonal columns. So, internal columns need to be designed for vertical load only. Due to increase in lever arm of peripherial diagonal columns, diagrid structural system is more effective in lateral load resistance.

- Design and detailing of diagrid node is difficult compared to regular frame structures, due to intersection of large diameter pipes at a joint and inclination of members. Design of joint is to be carried out considering maximum axial forces in all members meeting at a joint from the all load combinations.

- It is observed that there is 1.0 to 2.5 % increase in axial force under sequential loading is observed when only dead load is considered. While 1.0 to 3.0 % higher axial force in diagonal columns due to sequential loading is observed compared to one step analysis for combined dead and live load. For 60 and 80 storey building similar results are observed.

6.3 Future Scope of Work

The present work can be extended in future to include following aspects:

- In the present study plan dimension is kept constant throughout the height. In future, parametric study considering variation in plan dimension along the height of diagrid structural system can be carried out.
- Use of diagrid structural system in other plan shapes like rectangular, circular etc can be explored.
- Design of foundation system for diagrid structural system of various heights.
- Study of shear leg effect for diagrid structural system.

References

[1] Moon K.S., "Diagrid Structures for Complex-Shaped Tall Buildings", Procedia Engineering 2011, Vol. 14, pp.1343-1350.

[2] Kim Y.J., Kim M.H., Jung I.Y., Ju Y.K. and Kim S.D, "Experimental Investigation of the Cyclic Behavior of Nodes in Diagrid Structures", Engineering Structures 2011, Vol. 33, No. 10, pp. 2134-2144.

[3] Kim J., Jun Y. and Lee Y.H., "Seismic Performance Evaluation of Diagrid System Buildings", 2^{nd} Specialty Conference on Disaster Mitigation, Manitoba, June 2010.

[4] Moon K.S., "Stiffness-Based Design Methodology for Steel Braced Tube Structures: A Sustainable Approach", Engineering Structures 2010, Vol. 32, No. 10, pp. 3163-3170.

[5] Kim Y.J., Jung I.Y., Ju Y.K., Park S.J. and Kim S.D., "Cyclic Behavior of Diagrid Nodes with H-section Braces", Journal of Structures Engineering 2010, Vol. 136, No. 9, pp. 1111-1122.

[6] Moon K.S., "Material-Saving Design Strategies for Tall Building Structures", CTBUH 8th World Congress, Dubai, March 2008.

[7] Charnish B. and McDonnell T., "The bow: Unique Diagrid Structural System for a Sustainable Tall Building", CTBUH 8th World Congress, Dubai, March 2008.

[8] Soo k.J., Sik K.Y. and Hee L.S., "Structural Schematic Design of Tall Building in Asan using the Diagrid System", CTBUH 8th World Congress, Dubai, March 2008.

[9] Rahimian A. and Eilon Y., "Hearst Headquarters: Innovation and Heritage in Harmony", CTBUH 8th World Congress, Dubai, March 2008.

[10] Moon K.S., "Optimal Grid Geometry of Diagrid Structures for Tall Buildings", Architectural Science Review, 2008.

[11] Fu X.Y., Wu B., Chen X.C., Meng M.L., Sun C., Jiang H.B., Gao Y. and Li J.W., "Research on Structural Design of A Super High-Rise Building in Qartar", The IES Journal Part A: Civil and Journal of Structural Engineering 2008, Vol. 1, No. 3, pp. 186-197.

[12] Ali M. and Moon K.S., "Structural Developments in Tall Buildings: Current Trends and Future Prospects", Architectural Science Review 2007, Vol. 50, No. 3, pp. 205-223.

[13] Leonard J., "Investigation of Shear Lag Effect in High-Rise Buildings with Diagrid System", M.S. thesis, Massachusetts Institute of Technology, 2007.

[14] Moon K.S., "Dynamic Interrelationship between Technology and Architecture in Tall Buildings", Ph.D. dissertation, Department of Architecture, Massachusetts Institute of Technology, 2005.

[15] Moon K.S., "Sustainable Ddesign of Tall Building Structures and Facades".

[16] Toreno M., Arpino R., Mele E., Brandonisio G. and Luca A., "An Overview on Diagrid Structures for Tall Buildings"

[17] Quan Y.and Gu M., "Across-wind Equivalent Static Wind Loads and Responses of High-rise Buildings with Rectangle Sections".

[18] Gu M.ang Quan Y., "Across-Wind Loads of typical Tall Buildings", Journal of Wind engineering and Industrial Aerodynamics 2004, pp.1147-1165.

[19] Carroll C., Cross P., Duan X., Gibbons C., Ho G., Kwok M., Lawson R., Lee A., Li R., Luong A., McGowan R. and Pope C., "Case Study: CCTV Building", CTBUH Journal, 2008.

[20] http://www.archdaily.com/38768/lotte-super-tower-kpf/

[21] Sarkisian M., Mathias N., Long E., Mazeika A., Gordan J. and Chakar J., "Jin Mao Tower's Influence on China's New Innovative Tall Buildings", Shanghai International Seminar of Design and Construction Technologies of Super High-Rise Buildings, Shanghai, Chaina, May 2006.

[22] http://www.tekla.com/ae/solutions/references/Pages/Capital-Gate-Tower-UAE.aspx

[23] IS: 800-2007. General construction in steel - code of practice. Bureau of Indian Standard, New Delhi.

[24] IS: 875(Part-3)-1987. Code of practice for design loads (other than earthquake) for buildings and structures, wind loads. Bureau of Indian Standard, New Delhi.

[25] IS: 1893(Part-I)-2002. Criteria for Earthquake Resistant Design of Structures. Bureau of Indian Standard, New Delhi.

[26] ACI 347.2R-05. Guide for Shoring/Reshoring of Concrete Multistory Buildings.

Appendix A

Wind Loading Calculations

Dynamic Along Wind loading by Gust factor method as per IS:875(III)-1987

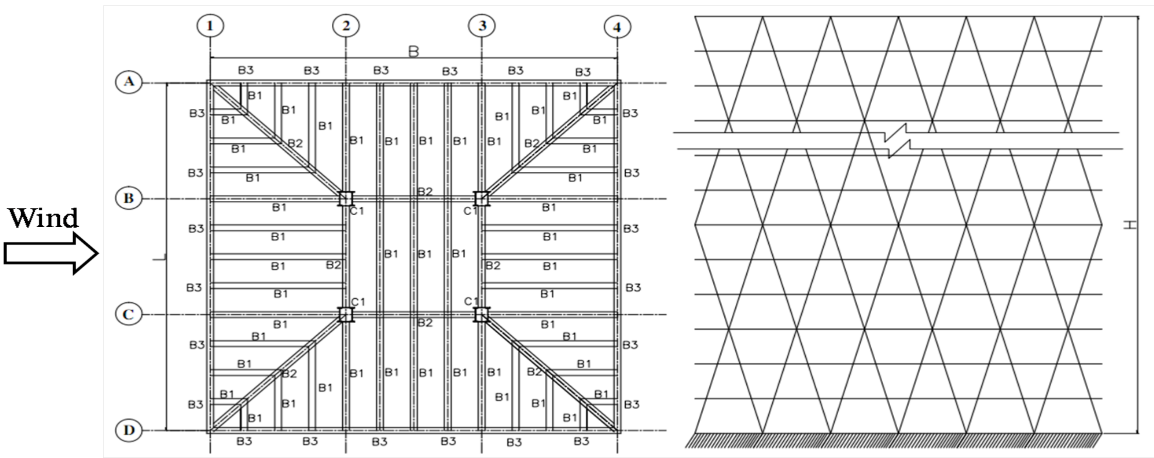


Figure A.1: Typical Floor Plan and Elevation of Diagrid Structure

Following are the steps of wind load calculation by gust factor method:

<u>**Building Configuration**</u>

No. of storey	= 36
Width of building (B)	= 36 m
Length of building (L)	= 36 m
Total height of building (H)	= 129.6 m
Typical height of storey	= 3.6 m

Basic Wind speed V_b	$=$ 30 m/s
Terrain category	$=$ III
k_1 (Probability factor)	$=$ 1
k_3 (Topography factor)	$=$ 1
Design wind speed V_z=V_bxk_1xk_2xk_3	
<u>Along wind load F</u> $=$ C_f**x**A_e**x**p_z**xG**	
a/b	$=$ 1
h/b	$=$ 3.6
C_f (Form the Fig. 4)	$=$ 1.35
$g_f.r$ (From Fig. 8, For height 129.6m and category III)	$=$ 1.17
$L_{(h)}$	$=$ 1625
C_y (Lateral correlation constant)	$=$ 10.00
C_z (Longitudinal correlation constant)	$=$ 12.00
λ	$=$ 0.2314
$C_z.h/L_{(h)}$	$=$ 0.9570
f_0 (Natural frequency)	$=$ 0.514 Hz
B (Background factor from Fig. 9.)	$=$ 0.65
V_h (Hourly mean wind speed at height z)	$=$ 24.588 m/sec
F_0 (Reduced frequency)	$=$ 32.536
S (Size reduction factor from Fig. 10.)	$=$ 0.038
$\emptyset$	$=$ 0
$f_o.L_{(h)}/V_h$	$=$ 33.969
E (Gust Energy factor from Fig. 11.)	$=$ 0.05
β (Damping coefficient for welded steel structure)	$=$ 0.01
G (Gust factor)	$=$ 2.072

$$G = 1 + g_f r \left[B(1+\phi)^2 + \frac{SE}{\beta} \right] \tag{A.1}$$

Table A.1: Along Wing Load for 36 Storey

Storey	Height (m)	k_1	k_2	k_3	V_z (m/sec)	P_z=0.6 Vz^2 (kN/m^2)	F_z (kN)
36	129.6	1	0.820	1	24.588	0.363	64.837
35	126	1	0.816	1	24.480	0.360	123.106
34	122.4	1	0.812	1	24.372	0.356	114.844
33	118.8	1	0.809	1	24.264	0.353	108.493
32	115.2	1	0.805	1	24.156	0.350	112.818
31	111.6	1	0.802	1	24.048	0.347	118.799
30	108	1	0.798	1	23.940	0.344	122.929
29	104.4	1	0.794	1	23.832	0.341	116.675
28	100.8	1	0.791	1	23.724	0.338	108.819
27	97.2	1	0.785	1	23.547	0.333	102.176
26	93.6	1	0.778	1	23.354	0.327	105.454
25	90	1	0.772	1	23.160	0.322	110.188
24	86.4	1	0.766	1	22.966	0.316	113.126
23	82.8	1	0.759	1	22.771	0.311	106.519
22	79.2	1	0.753	1	22.577	0.306	98.549
21	75.6	1	0.746	1	22.382	0.301	92.319
20	72	1	0.740	1	22.188	0.295	95.184
19	68.4	1	0.732	1	21.964	0.289	99.098
18	64.8	1	0.727	1	21.798	0.285	101.915
17	61.2	1	0.720	1	21.605	0.280	95.886
16	57.6	1	0.714	1	21.410	0.275	88.629
15	54	1	0.707	1	21.216	0.270	82.948
14	50.4	1	0.701	1	21.022	0.265	85.440
13	46.8	1	0.690	1	20.712	0.257	88.125
12	43.2	1	0.680	1	20.388	0.249	89.157
11	39.6	1	0.669	1	20.064	0.242	82.697
10	36	1	0.658	1	19.740	0.234	75.339
9	32.4	1	0.647	1	19.416	0.226	69.470
8	28.8	1	0.634	1	19.020	0.217	69.944
7	25.2	1	0.616	1	18.480	0.205	70.155
6	21.6	1	0.598	1	17.940	0.193	69.032
5	18	1	0.574	1	17.220	0.178	60.915
4	14.4	1	0.544	1	16.320	0.160	51.495
3	10.8	1	0.508	1	15.240	0.139	42.800
2	7.2	1	0.500	1	15.000	0.135	43.502
1	3.6	1	0.500	1	15.000	0.135	46.221
Total Base Shear (kN)							**3227.60**

Lateral load along the height for dynamic along wind loading calculated as per IS:875(III)-1987 is shown in Fig A.2.

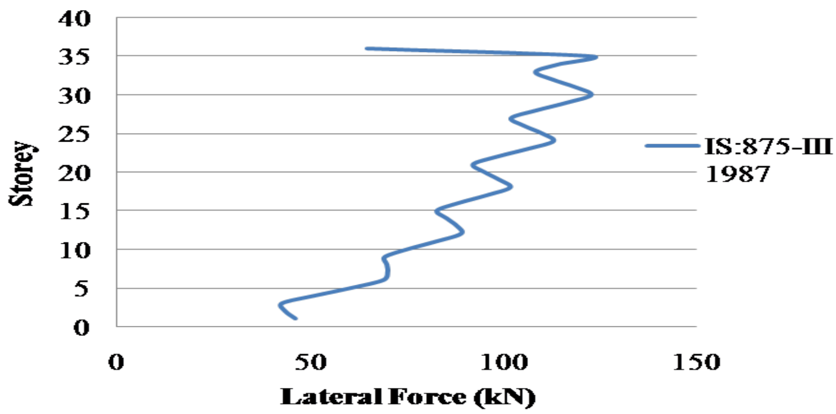


Figure A.2: Lateral Load Along the Height: Along Wind

Across Equivalent Static Wind Load: Static across wind analysis carried out as per [17] and [18]. Following are the steps to calculate across equivalent static wind load, in transverse direction of flow of wind.

1. Find out structural parameters and geometrical parameters of the building

H	=	Height of the building
B	=	Width of the building
D	=	Depth of the building
f	=	frequency
	=	1 / (0.015H) for RC building
	=	1 / (0.02H) for Steel building
β	=	mode shape
m(z)	=	mass per unit height
$M_1{}^*$	=	1^{st} generalized mass

2. Calculate the wind field parameters

I_H	=	Turbulence density
U_H	=	mean wind speed at the top of the building
w_H	=	Design wind pressure $(0.6{\times}U_H^2)$

3. Calculate the background base moment coefficient

$C_{M-BO} =$ $(0.002{\alpha_w}^2$-$0.017\alpha_w$-$1.4)(0.056\alpha_{db}^2$-$0.16\alpha_{db}$+$0.03)(0.03{\alpha_{ht}}^2$-$0.622\alpha_{ht}$+$4.357)$

where,

α_w = 4.2 - $4e^{3.7-60I_H}$

α_{db} = D / B

α_{ht} = H / T (T = min(B,D))

α_{hr} = H / $\sqrt{BD}$

4. Calculate the peak factor for resonant response

gR $= \sqrt{(2ln(600f))} + \frac{0.5772}{\sqrt{(2ln(600f))}}$

5. Calculate the reduced base moment power spectral density

${S_M}^*(n) = S_p\ \eta\ (n/f_p)^\lambda\ /\ [\{1-(n/f_p)^2\}^2 + \eta\ (n/f_p)^2]$

where,

f_p = location parameter

$= 10^{-5}(191 - 9.48\alpha_w + 1.28\alpha_{hr} + \alpha_{hr}\alpha_w)(68 - 21\alpha_{db} + 3\alpha_{db}^2)$

S_p = amplitude parameter

$= (0.1\alpha_w^{-0.4} - 0.0004e^{\alpha_w})(0.84\alpha_{hr} - 2.12 - 0.05\alpha_{hr}^2)(0.422 + \alpha_{db}^{-1} - 0.08\alpha_{db}^{-2})$

η = band width parameter

$= (1 + 0.00473e^{1.7\alpha_w})(0.065 + e^{1.26-0.63\alpha_{hr}})e^{1.7-(3.44/\alpha_{db})}$

λ = deflection parameter

$= (-0.8 + 0.06\alpha_w + 0.0007e^{\alpha_w})(-\alpha_{hr}^{0.34} + 0.00006e^{\alpha_{hr}})(0.414\alpha_{db} + 1.67{\alpha_{db}}^{-1.23})$

6. Calculate the aerodynamic damping ratio

$$\xi_a = \frac{0.0025\left(1-\left(\frac{U^*}{9.8}\right)^2\right)\left(\frac{U^*}{9.8}\right) + 0.000125\left(\frac{U^*}{9.8}\right)^2}{\left(1-\left(\frac{U^*}{9.8}\right)^2\right)^2 + 0.291\left(\frac{U^*}{9.8}\right)^2}$$

where,

U^* = reduced wind speed

$= U_H/(fB)$

7. Calculate peak across-wind equivalent static wind load

F $= \sqrt{p_B(z)^2 + p_R(z)^2}$

where,

$p_B(z)$ = background component

= (0.38 + 0.75h + 4.2h^2 - 4.5h^3) g_B C_{M-BO} w_H B

h = z/H

z = height from the base

g_B = peak factor for background component

$p_R(z)$ = resonant component

$$= \frac{Hm(z)}{M_1{}^*} B w_H \left(\frac{z}{H}\right)^{\beta} g_R \sqrt{\frac{\pi \phi S_M^*}{4(\xi_{s1} + \xi_{s2})}}$$

Height	H	129.6	m
Width	D	36	m
Length	B	36	m
Frequency	f1	0.386	for steel building
Structural damping ratio	ξ_s	2%	
Turbulence density	I_H	11%	
Mean wind velocity	U_H	30	m/sec
Design wind pressure	W_H	0.54	kN/m^2

α_w	= 3.98,	α_{ht}	= 3.6	C_{M-BO}	= 0.266,	g_R	= 3.475
α_{db}	= 1,	α_{hr}	= 3.6	β	= 1,	ϕ	= 1

Wind Tunnel Parameters

Location parameter	f_P	0.0861
Amplitude parameter	S_p	0.0124
Band width parameter	η	0.3852
Deflection parameter	λ	1.684
reduced frequency	n	0.463
reduced power spectral density	$S_M{}^*(n)$	0.00010

Aerodynamic damping ratio

reduced wind speed	U^*	2.160
damping ratio	ξ_a	0.059
peak factor background component	g_B	3.5

Table A.2: Across Wing Load for 36 Storey

Storey	Height from base z	h=z/H	$G_B(h)$	$G_R(h)$	F kN
36	129.6	1.0000	0.774	0.657	19.73
35	126	0.9722	0.880	0.639	21.14
34	122.4	0.9444	0.973	0.621	22.43
33	118.8	0.9167	1.054	0.602	23.60
32	115.2	0.8889	1.123	0.584	24.61
31	111.6	0.8611	1.181	0.566	25.46
30	108	0.8333	1.228	0.548	26.14
29	104.4	0.8056	1.265	0.529	26.66
28	100.8	0.7778	1.293	0.511	27.02
27	97.2	0.7500	1.311	0.493	27.23
26	93.6	0.7222	1.321	0.475	27.29
25	90	0.6944	1.323	0.456	27.21
24	86.4	0.6667	1.318	0.438	26.99
23	82.8	0.6389	1.305	0.420	26.65
22	79.2	0.6111	1.286	0.402	26.20
21	75.6	0.5833	1.262	0.383	25.63
20	72	0.5556	1.232	0.365	24.98
19	68.4	0.5278	1.197	0.347	24.23
18	64.8	0.5000	1.158	0.329	23.41
17	61.2	0.4722	1.116	0.310	22.51
16	57.6	0.4444	1.070	0.292	21.56
15	54	0.4167	1.022	0.274	20.57
14	50.4	0.3889	0.972	0.256	19.53
13	46.8	0.3611	0.920	0.237	18.47
12	43.2	0.3333	0.867	0.219	17.38
11	39.6	0.3056	0.814	0.201	16.29
10	36	0.2778	0.761	0.183	15.21
9	32.4	0.2500	0.708	0.164	14.13
8	28.8	0.2222	0.657	0.146	13.08
7	25.2	0.1944	0.607	0.128	12.07
6	21.6	0.1667	0.560	0.110	11.10

Storey	Height from base z	h=z/H	$G_B(h)$	$G_R(h)$	F kN
5	18	0.1389	0.516	0.091	10.18
4	14.4	0.1111	0.475	0.073	9.33
3	10.8	0.0833	0.437	0.055	8.57
2	7.2	0.0556	0.404	0.037	7.89
1	3.6	0.0278	0.377	0.018	7.33
Total Base Shear (kN)					**721.87**

Lateral force along the height for Across wind loading is shown in Fig A.3.

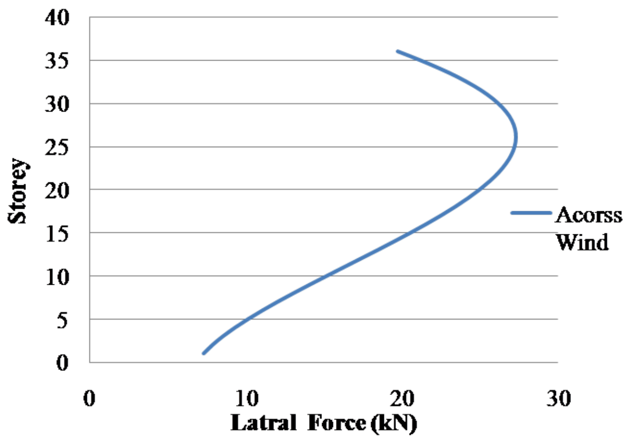


Figure A.3: Lateral Load Along the Height: Across wind

Along wind and across wind effects are combined for analysis and design of tall structures.

Appendix B

Preliminary Design of Diagrid Structures

40 Storey Diagrid Structure with 67.4 degress

Storey shear and bending moment due to wind load at each module is calculated and presented in Table.B.1. The area of diagonal to resist shear and bending are also shown in Table.B.1 and Fig. B.1, for various values of 'S'= 1,2,3.

Table B.1: Preliminary Member Sizing for the 40-Storey Diagrid Structure (S=1,2,3)

Storey	V	M	S=1		S=2		S=3	
	(kN)	(kN-m)	A_{dw} (Shear) (m^2)	A_{df} (Bend ing) (m^2)	A_{dw} (Shear) (m^2)	A_{df} (Bend ing) (m^2)	A_{dw} (Shear) (m^2)	A_{df} (Bend ing) (m^2)
37th-40th	444	2034	**0.0006**	0.0001	**0.00098**	0.00008	**0.00131**	0.00007
33rd-36th	930	11016	**0.0013**	0.0005	**0.00206**	0.00042	**0.00274**	0.00037
29th-32nd	1399	26909	**0.0020**	0.0013	**0.00310**	0.00102	**0.00413**	0.00091
25th-28th	1847	49460	**0.0027**	0.0025	**0.00409**	0.00188	**0.00545**	0.00167
21st-24th	2266	78309	0.0033	**0.0039**	**0.00501**	0.00298	**0.00668**	0.00265
17th-20th	2657	113040	0.0039	**0.0057**	**0.00588**	0.00430	**0.00783**	0.00383
13th-16th	3019	153252	0.0044	**0.0077**	**0.00668**	0.00583	**0.00890**	0.00519
9th-12th	3342	198484	0.0049	**0.0100**	0.00739	**0.00756**	**0.00985**	0.00672
5th-8th	3611	248110	0.0053	**0.0125**	0.00799	**0.00945**	**0.01065**	0.00840
1st-4th	3803	301174	0.0056	**0.0152**	0.00841	**0.01147**	**0.01122**	0.01019

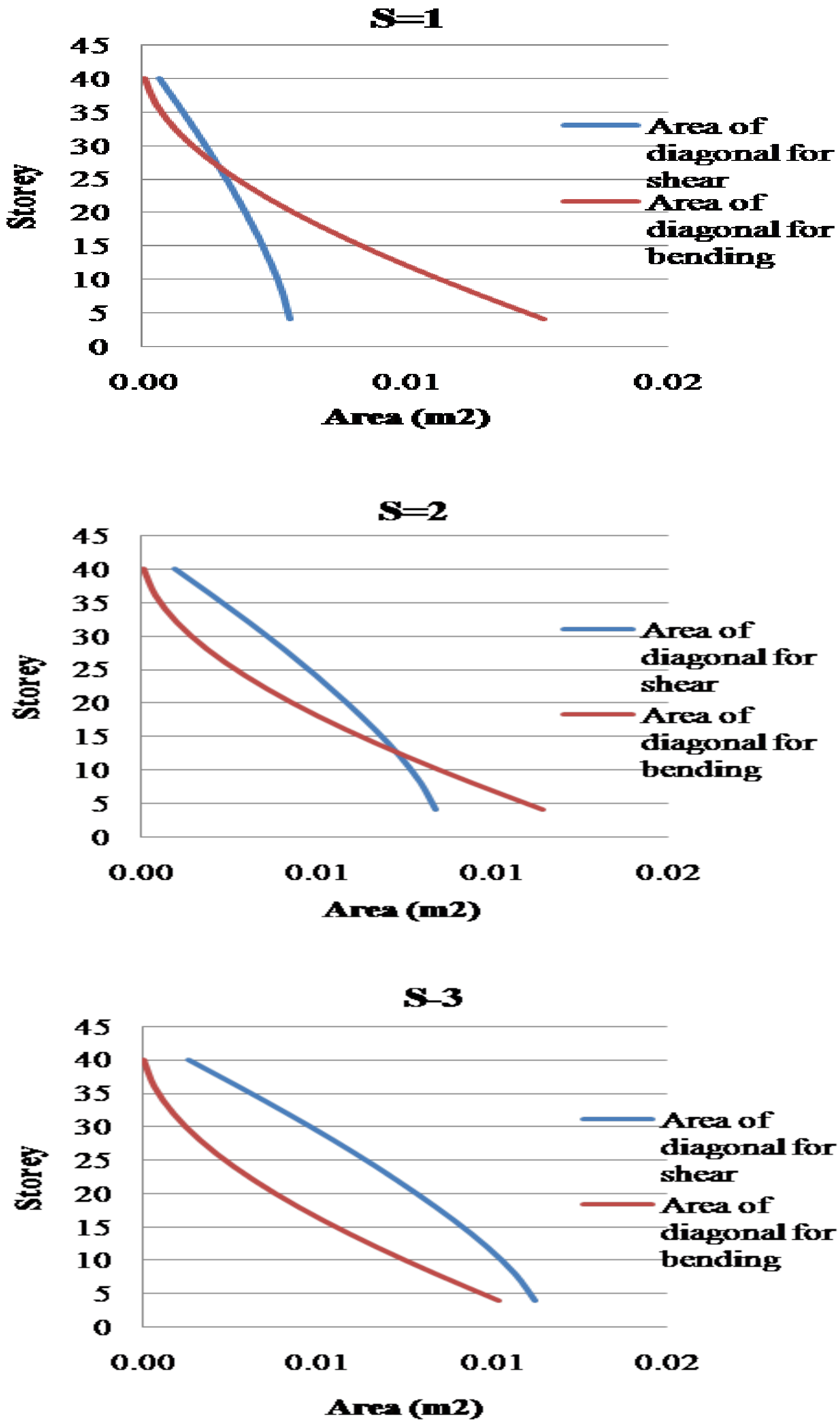


Figure B.1: Preliminary Member Sizing for the 40-Storey Diagrid Structure (S=1,2,3)

From the area of diagonal total steel required for diagonal columns are calculated for varying values of 'S' = 1,2,3 and is presented in Fig. B.2.

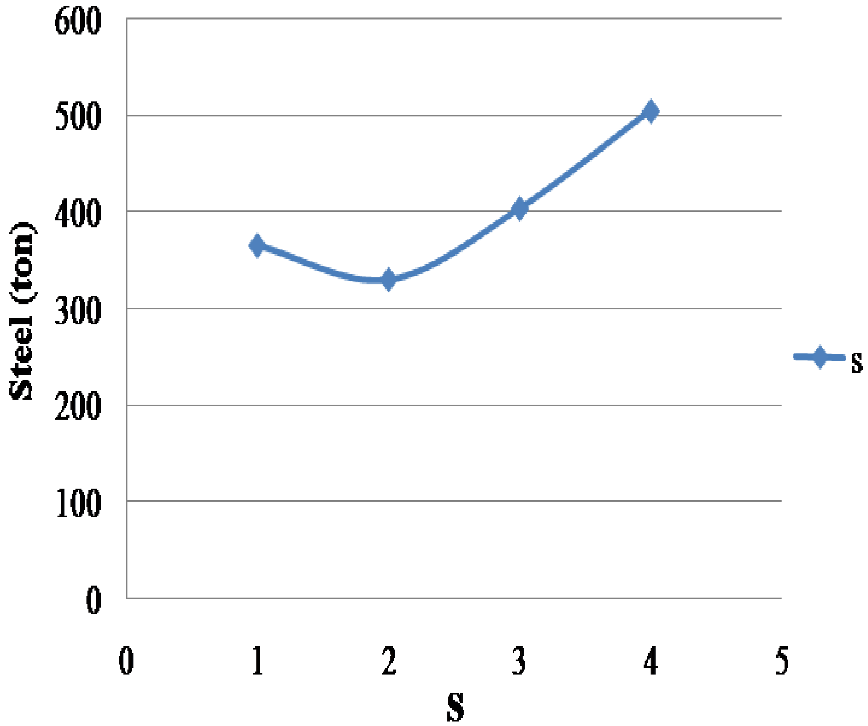


Figure B.2: Diagrid Steel Mass for the 40-Storey Structure

50 Storey Diagrid Structure with 67.4 degress

Storey shear and bending moment due to wind load at each module is calculated and presented in Table.B.2. The area of diagonal to resist shear and bending are also shown in Table.B.2 and Fig. B.3, for various values of 'S'= 1,2,4.

Table B.2: Preliminary Member Sizing for the 50-Storey Diagrid Structure (S=1,2,4)

Storey	V	M	S=1		S=2		S=4	
	(kN)	(kN-m)	A_{dw} (Shear) (m^2)	A_{df} (Bending) (m^2)	A_{dw} (Shear) (m^2)	A_{df} (Bending) (m^2)	A_{dw} (Shear) (m^2)	A_{df} (Bending) (m^2)
49th-50th	198	249	**0.0002**	0.00002	**0.0004**	0.00001	**0.0007**	0.00001
45th-48th	705	5799	**0.0010**	0.00037	**0.0015**	0.00028	**0.0026**	0.00023
41st-44th	1199	18582	**0.0017**	0.00118	**0.0026**	0.00088	**0.0044**	0.00074
37th-40th	1677	38393	**0.0024**	0.00244	**0.0037**	0.00183	**0.0061**	0.00152
33rd-36th	2138	64995	0.0031	**0.00412**	**0.0047**	0.00309	**0.0078**	0.00258
29th-32nd	2583	98154	0.0038	**0.00623**	**0.0057**	0.00467	**0.0095**	0.00389
25th-28th	3008	137628	0.0044	**0.00873**	**0.0066**	0.00655	**0.0110**	0.00546
21st-24th	3405	183075	0.0050	**0.01162**	0.0075	**0.00871**	**0.0125**	0.00726
17th-20th	3776	234102	0.0055	**0.01485**	0.0083	**0.01114**	**0.0139**	0.00928
13th-16th	4120	290328	0.0060	**0.01842**	0.0091	**0.01382**	**0.0151**	0.01151
9th-12th	4426	351316	0.0065	**0.02229**	0.0097	**0.01672**	**0.0163**	0.01393
5th-8th	4681	416471	0.0069	**0.02643**	0.0103	**0.01982**	**0.0172**	0.01652
1st-4th	4863	484889	0.0071	**0.03077**	0.0107	**0.02308**	0.0179	**0.01923**

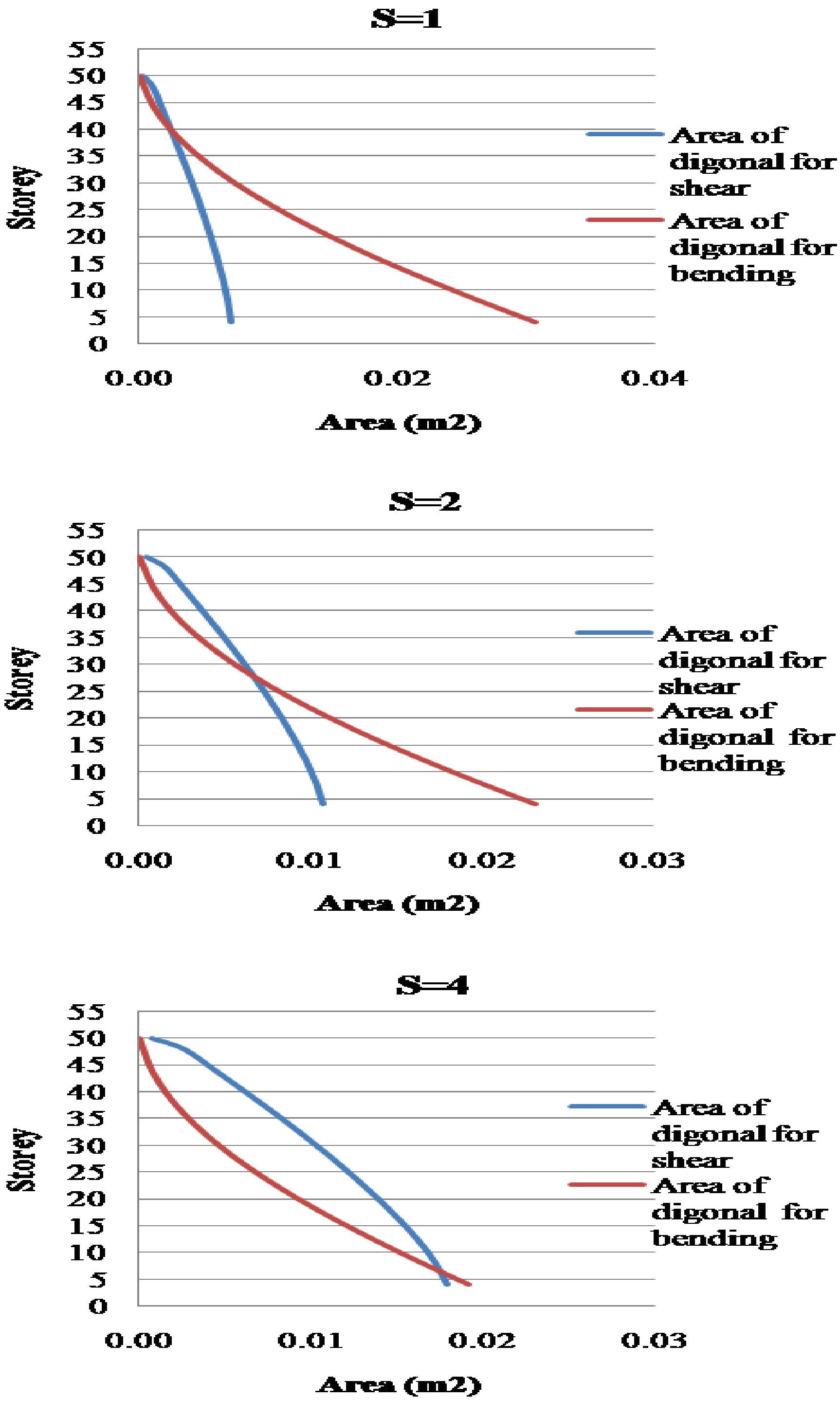


Figure B.3: Preliminary Member Sizing for the 50-Storey Diagrid Structure (S=1,2,4)

From the area of diagonal total steel required for diagonal columns are calculated for varying values of 'S' = 1,2,4 and is presented in Fig. B.4.

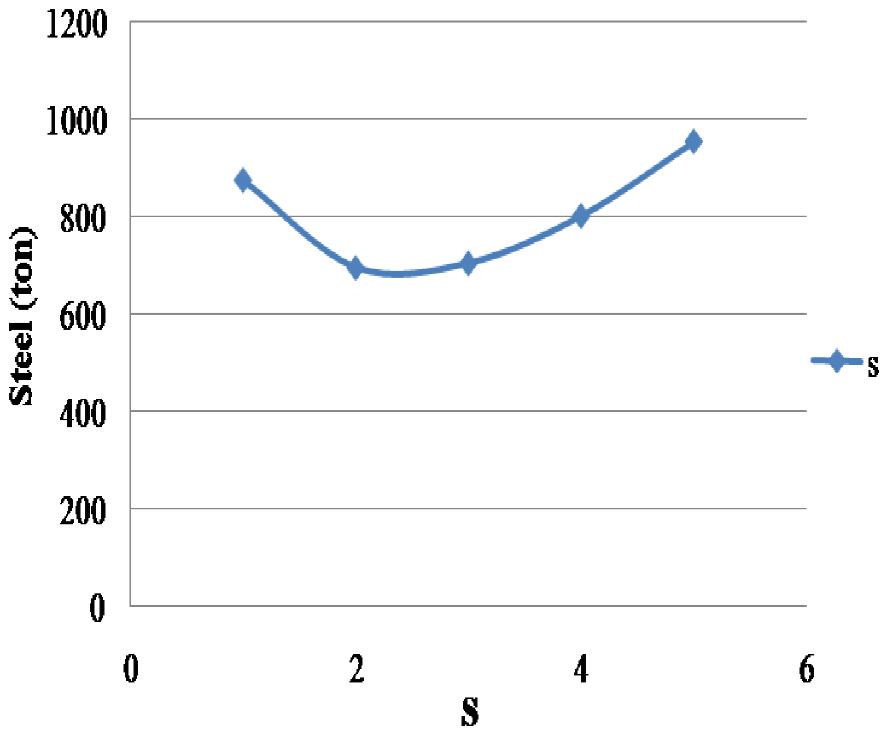


Figure B.4: Diagrid Steel Mass for the 50-Storey Structure

60 Storey Diagrid Structure with 67.4 degress

Storey shear and bending moment due to wind load at each module is calculated and presented in Table.B.3. The area of diagonal to resist shear and bending are also shown in Table.B.3 and Fig. B.5, for various values of 'S'= 1,3,5.

Table B.3: Preliminary Member Sizing for the 60-Storey Diagrid Structure (S=1,3,5)

Storey	V	M	S=1		S=3		S=5	
	(kN)	(kN-m)	A_{dw} (Shear) (m^2)	A_{df} (Bending) (m^2)	A_{dw} (Shear) (m^2)	A_{df} (Bending) (m^2)	A_{dw} (Shear) (m^2)	A_{df} (Bending) (m^2)
57th-60th	485	2218	**0.0007**	0.0001	**0.0014**	0.00011	**0.00215**	0.0001
53rd-56th	1024	12070	**0.0015**	0.0009	**0.0030**	0.00061	**0.00453**	0.0005
49th-52nd	1548	29604	**0.0022**	0.0022	**0.0045**	0.0015	**0.0068**	0.0013
45th-48th	2059	54619	0.0030	**0.0041**	**0.0060**	0.0027	**0.0091**	0.0025
41st-44th	2556	86917	0.0037	**0.0066**	**0.0075**	0.0044	**0.0113**	0.0039
37th-40th	3037	126288	0.0044	**0.0096**	**0.0089**	0.0064	**0.0134**	0.0057
33rd-36th	3501	172496	0.0051	**0.0131**	**0.0103**	0.0087	**0.0154**	0.0078
29th-32nd	3949	225304	0.0058	**0.0171**	**0.0116**	0.0114	**0.0174**	0.0102
25th-28th	4377	284469	0.0064	**0.0216**	0.0129	**0.0144**	**0.0193**	0.0130
21st-24th	4777	349648	0.0070	**0.0266**	0.0140	**0.0177**	**0.0211**	0.0159
17th-20th	5150	420442	0.0075	**0.0320**	0.0151	**0.0213**	**0.0227**	0.0192
13th-16th	5496	496470	0.0081	**0.0378**	0.0162	**0.0252**	**0.0243**	0.0226
9th-12th	5804	577292	0.0085	**0.0439**	0.0171	**0.0293**	0.0256	**0.0263**
5th-8th	6061	662309	0.0089	**0.0504**	0.0178	**0.0336**	0.0268	**0.0302**
1st-4th	6245	750609	0.0092	**0.0571**	0.0184	**0.0381**	0.0276	**0.0342**

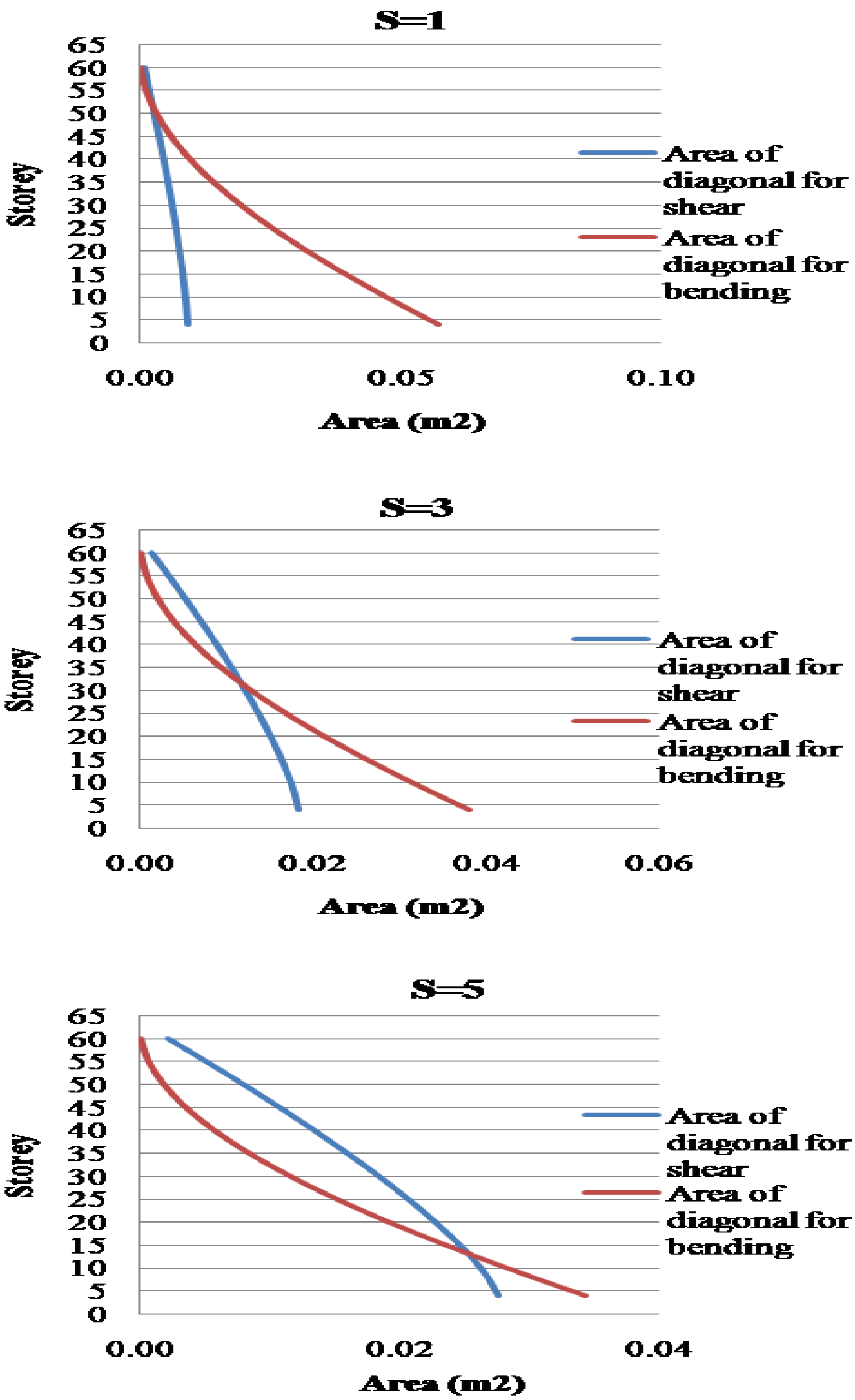


Figure B.5: Preliminary Member Sizing for the 60-Storey Diagrid Structure (S=1,3,5)

From the area of diagonal total steel required for diagonal columns are calculated for varying values of ‘S’ = 1,3,5 and is presented in Fig. B.6.

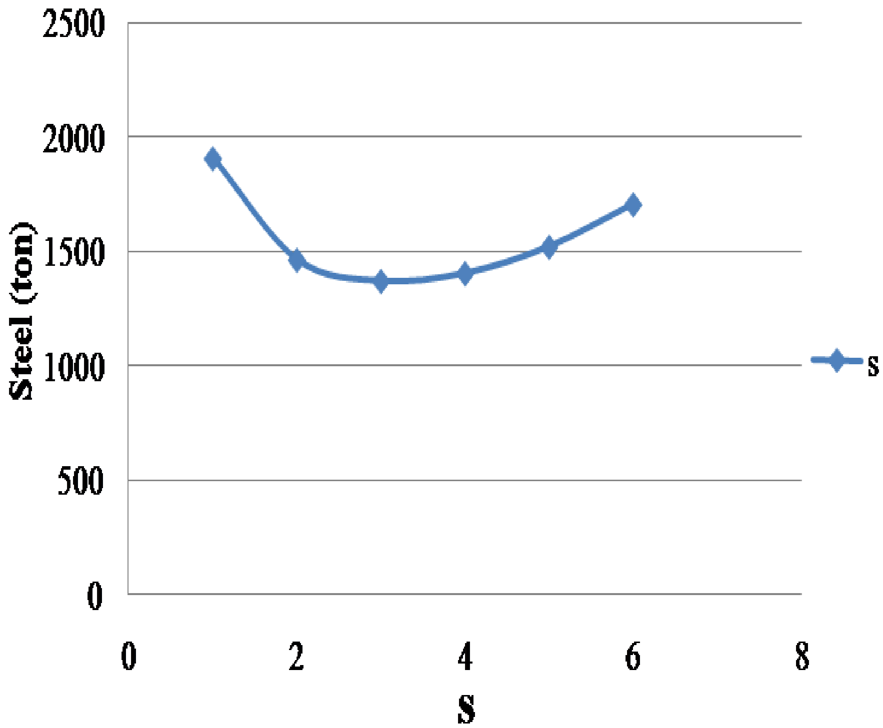


Figure B.6: Diagrid Steel Mass for the 60-Storey Structure

70 Storey Diagrid Structure with 67.4 degress

Storey shear and bending moment due to wind load at each module is calculated and presented in Table.B.4. The area of diagonal to resist shear and bending are also shown in Table.B.4 and Fig. B.7, for various values of 'S'= 1,4,8.

Table B.4: Preliminary Member Sizing for the 70-Storey Diagrid Structure (S=1,4,8)

Storey	V	M	S=1		S=4		S=8	
	(kN)	(kN-m)	A_{dw} (Shear) (m^2)	A_{df} (Bend ing) (m^2)	A_{dw} (Shear) (m^2)	A_{df} (Bend ing) (m^2)	A_{dw} (Shear) (m^2)	A_{df} (Bend ing) (m^2)
69th-70th	227	285	**0.0003**	0.00003	**0.00084**	0.00002	**0.00151**	0.00001
65th-68th	811	6654	**0.0012**	0.00059	**0.00299**	0.00037	**0.00538**	0.00033
61st-64th	1385	21380	**0.0020**	0.0019	**0.00510**	0.00119	**0.00919**	0.00107
57th-60th	1947	44306	0.0028	**0.0039**	**0.00718**	0.00246	**0.01292**	0.00221
53rd-56th	2497	75270	0.0036	**0.0066**	**0.00921**	0.00418	**0.01657**	0.00376
49th-52nd	3033	114084	0.0044	**0.0101**	**0.01118**	0.00633	**0.02013**	0.00570
45th-48th	3555	160543	0.0052	**0.0142**	**0.01311**	0.00891	**0.02359**	0.008021
41st-44th	4063	214444	0.0059	**0.0190**	**0.01498**	0.01191	**0.02696**	0.01072
37th-40th	4555	275574	0.0067	**0.0244**	**0.01679**	0.01530	**0.03022**	0.01377
33rd-36th	5029	343690	0.0074	**0.0305**	0.01854	**0.01908**	**0.03337**	0.01717
29th-32nd	5487	418550	0.0080	**0.0371**	0.02022	**0.02324**	**0.03640**	0.02091
25th-28th	5924	499907	0.0087	**0.0444**	0.02183	**0.02775**	**0.03930**	0.02498
21st-24th	6333	587409	0.0093	**0.0521**	0.02334	**0.03261**	**0.04201**	0.02935
17th-20th	6714	680650	0.0099	**0.0604**	0.02475	**0.03779**	**0.04454**	0.03401
13th-16th	7068	779240	0.0104	**0.0692**	0.02605	**0.04326**	**0.04689**	0.03894
9th-12th	7383	882728	0.0108	**0.0784**	0.02721	**0.04901**	**0.04898**	0.04411
5th-8th	7645	990503	0.0112	**0.0879**	0.02818	**0.05499**	**0.05072**	0.04949
1st-4th	7833	1101633	0.0115	**0.0978**	0.02887	**0.06116**	0.05197	**0.05505**

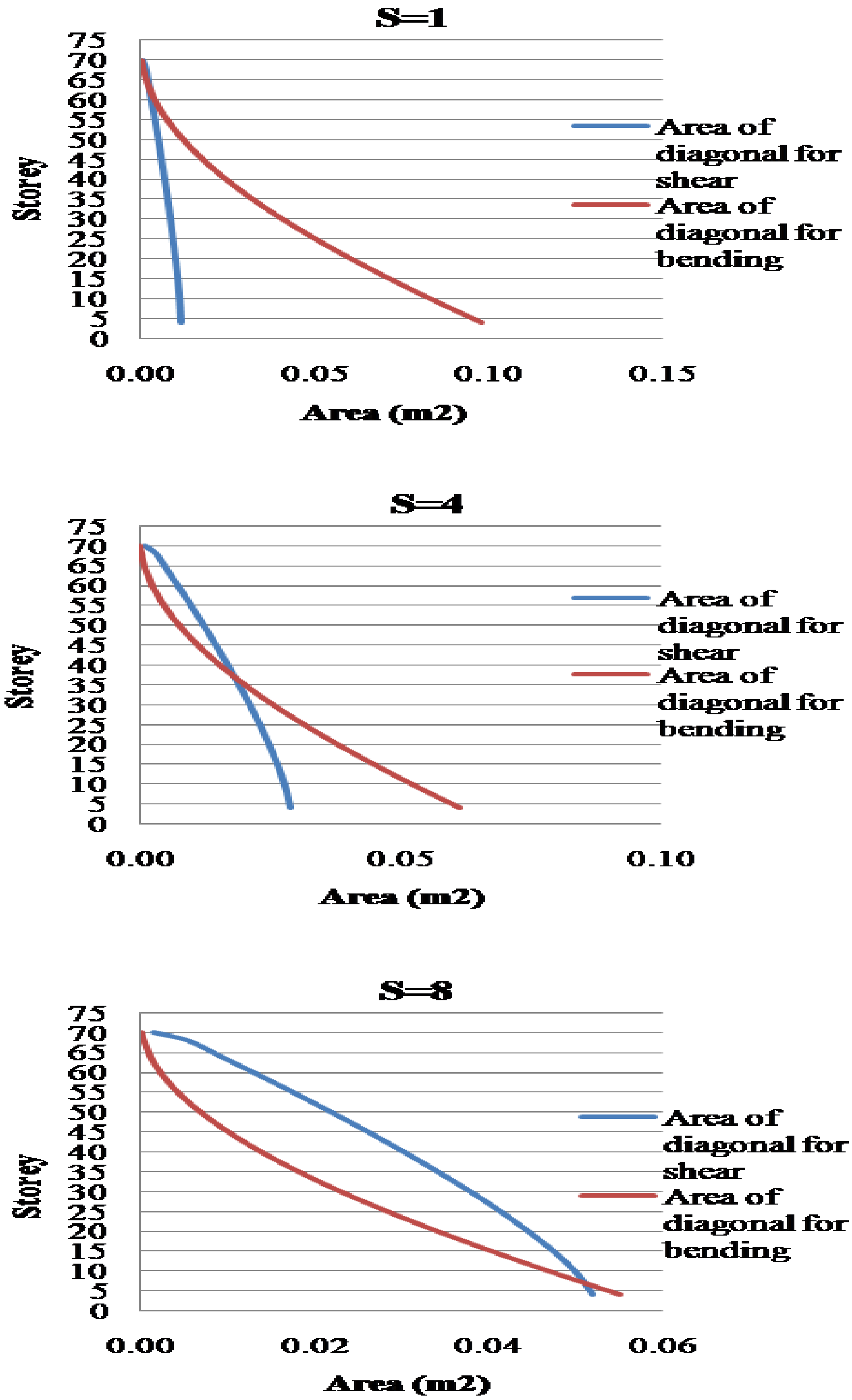


Figure B.7: Preliminary Member Sizing for the 70-Storey Diagrid Structure (S=1,4,8)

From the area of diagonal total steel required for diagonal columns are calculated for varying values of ‘S’ = 1,4,8 and is presented in Fig. B.8.

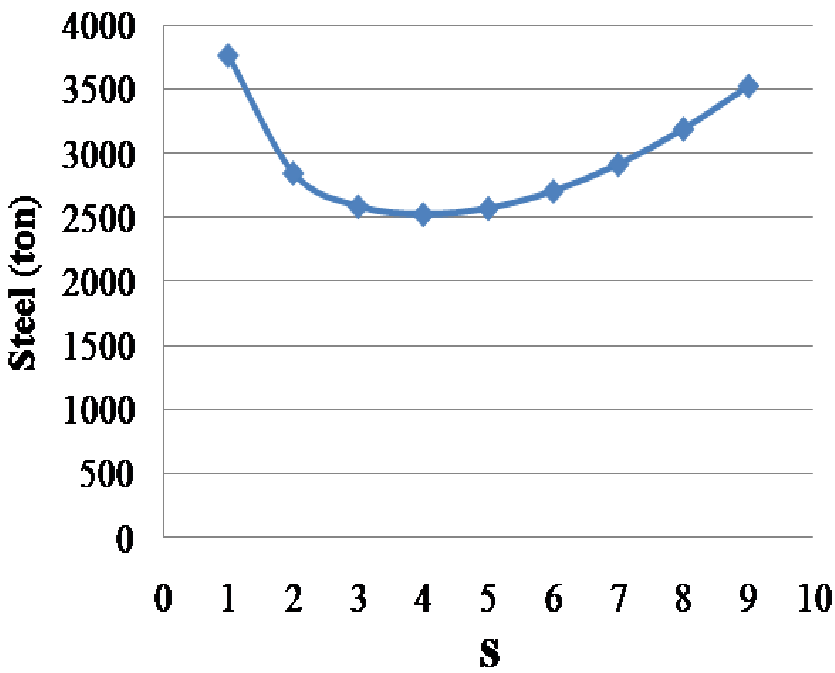


Figure B.8: Diagrid Steel Mass for the 70-Storey Structure

80 Storey Diagrid Structure with 67.4 degress

Storey shear and bending moment due to wind load at each module is calculated and presented in Table.B.5. The area of diagonal to resist shear and bending are also shown in Table.B.5 and Fig. B.9, for various values of 'S'= 1,5,9.

Table B.5: Preliminary Member Sizing for the 80-Storey Diagrid Structure (S=1,5,9)

Storey	V	M	S=1		S=5		S=9	
	(kN)	(kN-m)	A_{dw} (Shear) (m^2)	A_{df} (Bending) (m^2)	A_{dw} (Shear) (m^2)	A_{df} (Bending) (m^2)	A_{dw} (Shear) (m^2)	A_{df} (Bending) (m^2)
77th-80th	542	2471	**0.0008**	0.0002	**0.0024**	0.00015	**0.0040**	0.0001
73rd-76th	1148	13484	**0.0016**	0.0013	**0.0050**	0.00082	**0.0084**	0.0007
69th-72nd	1746	33186	0.0025	**0.0033**	**0.0077**	0.00202	**0.0128**	0.0018
65th-68th	2335	61456	0.0034	**0.0062**	**0.0103**	0.00374	**0.0172**	0.0034
61st-64th	2912	98138	0.0042	**0.0099**	**0.0128**	0.00598	**0.0214**	0.0055
57th-60th	3478	143071	0.0051	**0.0145**	**0.0153**	0.00871	**0.0256**	0.0080
53rd-56th	4032	196095	0.0059	**0.0199**	**0.0178**	0.01194	**0.0297**	0.0110
49th-52nd	4571	257020	0.0067	**0.0260**	**0.0202**	0.01566	**0.0337**	0.0145
45th-48th	5096	325639	0.0075	**0.0330**	**0.0225**	0.01984	**0.0375**	0.0183
41st-44th	5608	401748	0.0082	**0.0407**	**0.0248**	0.02447	**0.0413**	0.0226
37th-40th	6102	485132	0.0090	**0.0492**	0.0269	**0.0295**	**0.0449**	0.0273
33rd-36th	6580	575548	0.0097	**0.0584**	0.0291	**0.0350**	**0.0485**	0.0324
29th-32nd	7040	672752	0.0103	**0.0683**	0.0311	**0.0409**	**0.0519**	0.0379
25th-28th	7480	776494	0.0110	**0.0788**	0.0330	**0.0473**	**0.05514**	0.0437
21st-24th	7892	886420	0.0116	**0.0899**	0.0349	**0.0539**	**0.05817**	0.0499
17th-20th	8275	1002124	0.0122	**0.1017**	0.0366	**0.0610**	**0.06100**	0.0565
13th-16th	8631	1123209	0.0127	**0.1140**	0.03817	**0.06842**	**0.06362**	0.0633
9th-12th	8948	1249225	0.0131	**0.1268**	0.03958	**0.07609**	0.06596	**0.0704**
5th-8th	9212	1379556	0.0135	**0.1400**	0.04074	**0.08403**	0.06791	**0.0778**
1st-4th	9402	1513263	0.0138	**0.1536**	0.04158	**0.09218**	0.06930	**0.0853**

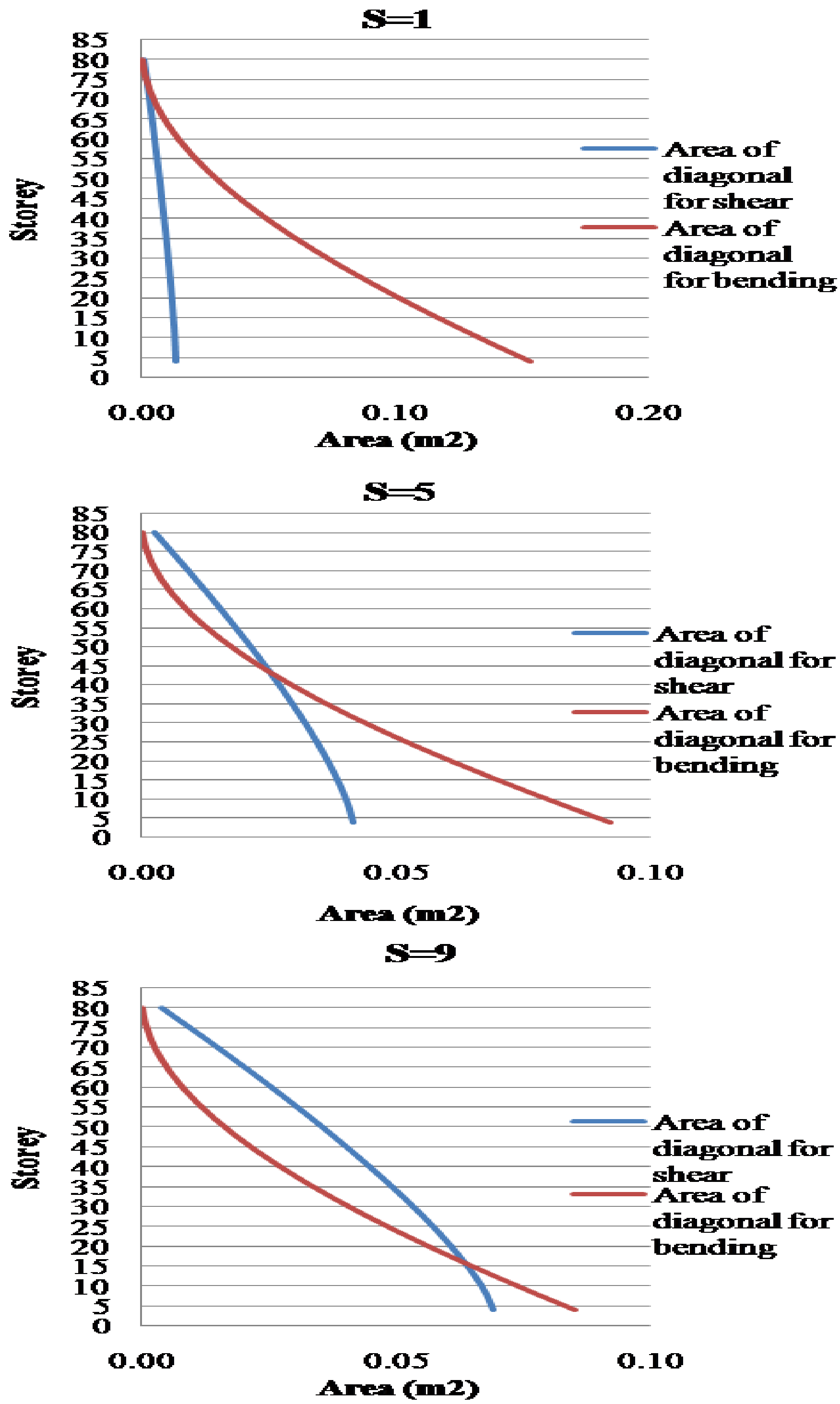


Figure B.9: Preliminary Member Sizing for the 80-Storey Diagrid Structure (S=1,5,9)

From the area of diagonal total steel required for diagonal columns are calculated for varying values of 'S' = 1,5,9 and is presented in Fig. B.10.

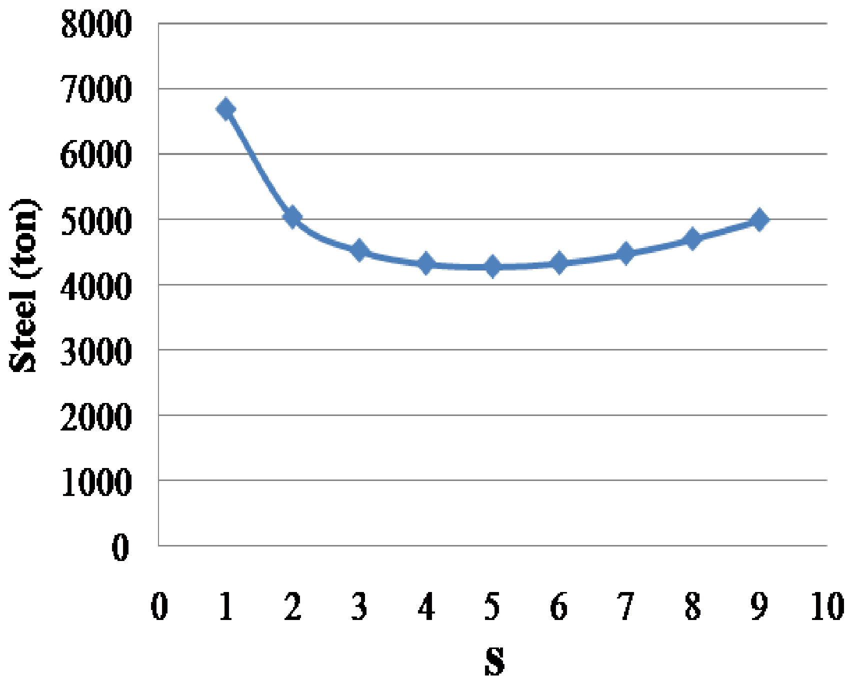


Figure B.10: Diagrid Steel Mass for the 80-Storey Structure

Appendix C

List of Paper Published / Communicated

List of Paper Published:

- Jani Khushbu D., "Comparison of Approximate and Exact Analysis of Diagrid Structural System for High Rise Steel buildings", 4^{th} National Civil Engineering Student's Symposium - AAKAAR, Department of Civil Engineering, IIT Bombay, 3-4 March 2012. (Won first prize in PG category)
- Jani Khushbu D. and Patel Paresh V., "Optimal Angle of Diagrid Structures for High Rise Buildings", 2^{nd} International Conference on ADVANCES IN MECHANICAL, MANUFACTURING AND BUILDING SCIENCES (ICAMB), VIT University, Vellore, India, 9-11 January 2012. (Paper Accepted)
- Jani Khushbu D. and Patel Paresh V., "Diagrid Structural System for High Rise Buildings", 26^{th} Indian Engineering Congress (IEC), Bangalore, India, 16-18 December 2011.
- Jani Khushbu D., "Diagrid Structural System for a Tall building ", A National Symposium - CONTECH'11, Department of Civil Engineering, Nirma University, Ahmedabad, 16-17 September 2011. (Won first prize in PG category)

List of Communicated:

- Jani Khushbu D. and Patel Paresh V., “Optimal Design of Diagrid Structures for High Rise Steel Buildings”, 8^{th} Biennial Conference (SEC), SVNIT, Surat, India, 19-21 December 2012. (Abstract Communicated)

- Jani Khushbu D. and Patel Paresh V., “Analysis and Design of Diagrid Structural System for High Rise Steel Buildings”, 3^{rd} International Conference (NUiCONE), Nirma University, Ahmedabad, India, 6-8 December 2012. (Abstract Communicated)